underground waste management and artificial recharge

volume 1

Preprints of papers presented at the Second International Symposium on Underground Waste Management and Artificial Recharge, New Orleans, Louisiana, September 26–30, 1973

Sponsored by:

The American Association of Petroleum Geologists
United States Geological Survey
International Association of Hydrological Sciences

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Edited by Jules Braunstein

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PREFACE

The two volumes of this work are dedicated to the technology and the maturing arts of underground waste management and artificial recharge.

At this time it may be predicted that both arts will occupy an increasingly important role in our way of life, preserving the ecology and environment of the world in which we live.

Only through communication, such as is made possible by this symposium, can the problems, with all their ramifications, be presented and appreciated--and understanding of them thereby increased. May the symposium serve its purpose well.

A house is never completed until the roof is in place. I trust that this symposium, the second "shingle," will be a sturdy and enduring one.

I express my grateful thanks to all committees and chairmen for their fine efforts.

LESLIE BOWLING, *General Chairman*
New Orleans, Louisiana
June 1973

SECOND INTERNATIONAL SYMPOSIUM ON
UNDERGROUND WASTE MANAGEMENT AND ARTIFICIAL RECHARGE

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FOREWORD

The two volumes of this publication represent the third in a continuing series of AAPG publications devoted to the subject of underground waste management. They were preceded by *Memoir* 10, *Subsurface Disposal in Geologic Basins,* and *Memoir* 18, the proceedings of the First Symposium on Underground Waste Management and Environmental Implications.

The first symposium on Underground Waste Management was sponsored jointly by The American Association of Petroleum Geologists and the United States Geological Survey. To these, this meeting has added the sponsorship of the International Association of Hydrological Sciences and the cooperation of numerous technical and scientific associations.

It was the aim of the steering committee to distribute at this meeting a single preprint volume containing the full text of each paper presented at the Symposium. The twin problems of getting all of the manuscripts in hand in time for careful editing in advance of printing, plus the size limits imposed by the printing process employed, made that goal unachievable. Therefore, we have adopted the expedient of printing in this first volume all of the papers that came in close to our announced deadline, and we are holding the late arrivals for a second volume.

The relation between petroleum geology and underground waste management is not readily apparent. Thus the question is frequently asked: "Why is The American Association of Petroleum Geologists involved in the Underground Waste Management Symposium?" The key words in the answer, of course, are "underground," and "geologists." What we are dealing with here is not the underground world of caves, but the subsurface as the habitat of hydrocarbons and associated water. It is a fitting, social enterprise to extend the petroleum geologists' expertise for discovering, and directing the exploitation of, hydrocarbon reservoirs toward the increasingly important matter of discovering geologic conditions suitable for the underground disposal of waste and for the artificial recharge of underground sources of water.

I am indebted to R. W. Boebel and F. A. Kohout for their aid in expediting the flow of manuscripts from the authors. Peggy Rice, AAPG Book Editor, has functioned, in effect, as co-editor of this volume. No one but me can appreciate the magnitude of her contribution to its production. I also thank Deborah Zikmund, Carol Thompson, and Ann Mayes, of the AAPG editorial staff, for their considerable help.

JULES BRAUNSTEIN, *Technical Program Editor*
New Orleans, Louisiana
June 1973

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STATE OF THE ART--AN OVERVIEW

LEGAL AND INSTITUTIONAL CONSIDERATIONS OF DEEP-WELL WASTE DISPOSAL[1]

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ABSTRACT Deep-well injection of wastes is subject to two levels of legal and institutional constraints. The first consists of regulatory procedures established by state and federal legislation. Waste injection has traditionally been regulated by the states through use of a variety of statutory constructions and administrative organizations. Federal control over subsurface disposal has essentially been limited to radioactive wastes, but influence currently is being extended into the general area of underground waste management. The apparent intent of the Federal Water Pollution Control Act Amendments of 1972 is the subjugation of state regulatory procedures to

[1]Manuscript received, June 14, 1973. The research upon which this paper is based is being supported by the National Science Foundation under Grant GI-34815.

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federal standards, with actual administration of controls ultimately to remain with the states.

The second level of constraints consists of the property rights of adjacent landowners. These adjacent rights are important because injected wastes do not respect property boundaries and therefore may produce conflict with certain aspects of property ownership.

The most obvious type of infringement involves injurious contamination of property interests by the injected waste. A more indirect case of contamination may involve the pressure-induced migration of naturally occurring pollutants such as mineralized water. Another potential type of pressure-related interference with property consists of structural damage from seismic activity initiated by injection. In some jurisdictions, even the unauthorized occupation of underlying space without measurable damage to the landowner may constitute a violation of property rights. The courts in most states have not been confronted with all these issues, but the party adversely affected by injection generally will be able to invoke a variety of legal actions, including nuisance, negligence, and trespass. In addition, some states accept the concept of strict liability regarding hazardous activities and the escape of deleterious substances, with the result that the injured party is relieved of the requirement of proving fault.

INTRODUCTION

Over the last few years the increased use of injection wells for the disposal of liquid industrial wastes has brought added attention to the legal and institutional controls applicable to this disposal method. These controls have differed to some extent from those pertaining to discharge of wastes into surface waters as the injection concept involves the confinement of waste materials in isolated geologic strata rather than discharge into the usable environment. One of the primary differences has been that waste-treatment requirements have not been as stringent in the case of disposal by injection, a fact which is viewed by critics of the technique as a loophole in environmental quality-control efforts. On the other hand, proponents of injection maintain that use of a lesser degree of treatment is justified by the fact that the waste is removed from the usable environment. According to the latter viewpoint, use of subsurface space as a storage zone for waste materials constitutes an economic resource which should not be ignored in cases where higher quality uses are not feasible.

The need for legal controls has generally been recognized, however, because of the potential for the injected material to escape from confinement and the capacity of the injection process to produce other undesirable results. Existing legal and institutional controls applicable to deep-well injection are of two distinct types. The first consists of regulatory controls administered by the federal and state governments. The second level of constraints consists of the common-law rights of adjacent landowners and other persons concerning protection from the harmful consequences of the injection operation.

GOVERNMENTAL REGULATION

State Controls

Governmental regulation of deep-well disposal has traditionally been a state responsibility because the federal government has been relatively inactive in the area of groundwater quality control. One of the most conspicuous characteristics of state regulatory programs has been their variability, even with regard to the acceptance of the deep-well waste-disposal concept. A number of states accept injection as a feasible alternative or last resort for waste disposal, while certain others are philosophically opposed to it. Among those that accept the concept, considerable variation is exhibited with respect to the form of the controls, the administrative organization for their implementation, and the actual substance of the controls.

With regard to the form of the statutory controls, a limited number of states have adopted specific disposal-well statutes (Texas Water Code, sec. 22.001 et seq; Michigan Statutes Annotated). Some states utilize legislation created for purposes other than the control of injection wells. For example, North Carolina has a water-well statute applicable to disposal wells (General Statutes of North Carolina). In Ohio, a part of oil and gas law applies to the injection of industrial wastes (Ohio Revenue Code, sec. 1509.01 et seq). A number of states regulate subsurface disposal through use of the pollution-control statute applicable to surface water.

In a few cases, the language of the pollution-control statute specifically encompasses disposal wells (West Virginia Code; Annotated Code of Maryland; Arizona Revised Statutes Annotated; Colorado Revised Statutes), whereas in others no explicit reference is made. Many state pollution-control laws can reasonably be interpreted to apply to disposal-well operations without specific inclusion as they cover discharges of wastes to both surface and groundwaters. A question may arise as to whether discharge of wastes to subsurface saline waters comes within the jurisdiction of pollution-control statutes which define "pollution" in terms of actions having adverse effects on other uses, inasmuch as such waters may have no other existing uses. However, the potential contamination of usable water in other strata if the waste should migrate from the disposal zone would likely be sufficient to support the claim that injection constitutes an activity within the jurisdiction of pollution-control legislation.

In a number of states, governmental organization for the administration of statutory controls places final regulatory responsibility with a single agency. In certain other states, the authorization procedure requires the approval of two or more agencies. For example, the Texas Disposal Well Act vests permit-granting authority in the Water Quality Board, but the permit is conditioned on the certification by the Texas Railroad Commission that the proposed injection well will not endanger oil- and gas-bearing strata (Texas Water Code, sec. 20.015). In Ohio, issuance of an industrial waste-injection well permit by the Division of Oil and Gas requires the approval of the State Environmental Protection Agency, the State Geological Survey, and the Division of Mines if the

proposed well is located in a coal-bearing township (Ohio Revised Code).

Although certain elements of the actual state requirements imposed on injection-well operations are relatively uniform, there are substantive provisions which vary considerably among the states. One such variation involves the specification of the quality parameters which define waters from which waste discharges are to be prohibited. It is generally accepted that injection should be restricted to saline aquifers, but there is agreement that waters of low salinity may be potentially useful and should be protected. There is considerable variation among the states as to the upper limit on concentration used to designate the saline waters that are potentially useful. Standards established by the New York Department of Environmental Conservation classify waters having a chloride content greater than 250 mg/l or a total-dissolved-solids content of more than 1,000 mg/l as saline, but saline waters containing less than 1,000 mg/l chloride or less than 2,000 mg/l total solids are protected (New York codes). On the other hand, the Texas Water Quality Board considers to be potentially beneficial all water having a total-dissolved-mineral concentration between 3,000 and 10,000 mg/l (Texas Water Quality Board). Illinois also protects all water having a total - dissolved-solids content of less than 10,000 mg/l (Illinois Sanitary Water Board, 1968). The unwritten guidelines used in Alabama prohibit waste discharges into all groundwater less saline than seawater, which contains about 33,000 mg/l of dissolved solids (Personal commun., Roy M. Alverson, Chief, Water Resources Div., Alabama Geol. Survey, March 20, 1973). The determination of what quality of water to protect should logically be based on a careful evaluation of existing and projected water needs. In the absence of complete data of this type, a conservative philosophy would be expected to govern the determination, inasmuch as it involves a possibly irreversible commitment of an important natural resource.

Another substantial variation in state regulatory controls is among preliminary information requirements. Requirements of the various states include information concerning geology, topography, wells and other excavations within a specified radius (usually about 2 mi), water and mineral resources, agriculture, fish and wildlife, industrial development, population densities, culture, and other factors. Whereas some states have comprehensive regulations requiring essentially the whole gamut of possible information (Colorado Department of Health, 1970), others have relatively limited requirements (West Virginia Department of Natural Resources). Part of the apparent variation is due to the fact that requirements in existence in certain cases may not be embodied in formal regulations, but it is likely that a significant part of the variation is real.

Requirements for monitoring the injection process constitute a third aspect in which state regulations differ considerably. Injection rates and pressures and the quality of injected waste must usually be monitored at the injection well, but there has been no uniformity with respect to requirements for separate monitor wells for the determination of waste migration within the disposal zone and into other strata. A consistent policy with respect to use of monitor wells has not always been applied, even within individual states--such wells are utilized in some cases and not in others. Some of the variations, of course, can be attributed to differing conditions presented by individual injection sites. For example, the close proximity of an injection well to an aquifer serving as an important source of water supply may create a need for monitor wells which does not exist in the absence of such resources. Likewise, the presence of a fault in the vicinity of injection may require the installation of monitor wells to detect movement of the waste toward the fault, or may require monitoring of freshwater aquifers near the fault to determine possible leakage. The presence of active faults may also require seismic monitoring for detection of any activity resulting from the injection operation.

Although flexibility in these and other aspects of state regulatory procedures is essential to efficient operation, it is likely that a greater amount of uniformity in state control procedures would be possible and, probably, desirable. The most effective controls applicable to a given situation in one state should also be the most effective with respect to an identical or similar situation in another state. The issue of the uniformity of criteria utilized for the control of disposal wells is of special interest where interstate aquifers are involved. The existence of less restrictive requirements in one state may have the potential of negating or compromising standards in effect in another. For example, the efforts of one state to protect brackish water of a certain salinity could be nullified if an adjacent state allowed waste injection into interstate aquifers containing such water. This and other potential problems of an interstate nature will require cooperative state action, and will be likely to necessitate greater uniformity in state regulatory procedures. Much work remains to be done with regard to standardization of controls and the accompanying considerations of the effectiveness of

various controls.

Federal Controls

One of the principal exceptions to state control over subsurface disposal has been the case of radioactive wastes. Methods utilized for the underground disposal of this type of waste, such as its emplacement in salt cavities or the injection of cement slurries into impermeable formations, differ from the traditional injection process used for liquid industrial wastes. The disposal of such materials has been regulated by the Atomic Energy Commission (AEC) pursuant to the Atomic Energy Act of 1954 (U.S. Code Annotated, 1954). This Act preempted state control by giving AEC exclusive authority to regulate radiation hazards associated with nuclear materials. An amendment enacted in 1959 provides for agreements with the states for transfer of certain regulatory functions, but a limitation on such agreements provides for retention of AEC authority with respect to regulation of the disposal of materials which the agency determines should not be disposed of without a federal license (U.S. Code Annotated, 1959). A potential limitation on AEC authority to regulate radioactive-waste disposal arises from a governmental reorganization plan in 1970 creating the Environmental Protection Agency (EPA). This plan transferred to EPA the responsibility for establishing environmental standards with respect to radiation (U.S. Government Reorganization Plan No. 3, 1970). Exercise of this standard-setting authority would seem to impinge upon the regulation of waste disposal by AEC.

Federal influence with respect to subsurface waste disposal is expanding considerably with the implementation of the Federal Water Pollution Control Act Amendments of 1972 (U.S. Code Annotated, 1972). Although the terms of the Amendments do not directly encompass disposal wells, regulations proposed by EPA for their implementation (Code of Federal Regulations, 1972) will subject certain disposal wells to federal control. These regulations provide for the imposition of EPA terms and conditions on wells used for the disposal of pollutants which are part of a waste-disposal program having elements that require an EPA permit under the National Pollutant Discharge Elimination System (NPDES). The injection of brines produced during petroleum extraction and the injection of substances to facilitate oil or gas production are exempt from federal control since these materials are not classified as pollutants under the Amendments, provided that such operations are subject to state regulation (U.S. Code Annotated, 1972, sec. 502[6]).

Perhaps of even greater significance is the fact that provisions specifying the conditions for the approval of state permit programs to be operated in place of NPDES establish a basis for the imposition of federal standards on state regulatory programs for disposal wells. Existence of "adequate" authority for the state to issue permits for the disposal of pollutants into wells is one of the conditions which must be met before the state permit program to regulate the discharge of pollutants into navigable water will be approved (U.S. Code Annotated, 1972, sec. 402[b][1][D]).

COMMON-LAW CONSTRAINTS

The second category of legal and institutional constraints referred to previously consists of the rights of individuals to take legal action in the event the injection operation infringes upon their legally protected interests. Encompassed here is the law of torts, which provides remedies for a variety of civil wrongs. Of primary concern in tort law is the distribution of losses between injured parties and those responsible for the injury in accordance with some concept of social justice. Tort law also provides the remedy of injunctive relief under certain conditions, but the role of the injunction is likely to be diminished in an area subject to administrative regulation.

In the case of injection wells, administrative controls are intended to prevent injury to other parties, but, in the event that injury does occur, the injured party can seek compensation under one of several tort theories, including nuisance, negligence, strict liability, and trespass. In general, nuisance encompasses actions which interfere with the enjoyment of property and is based to a large extent on the inherent nature of the objectionable activity. Negligence, on the other hand, applies to actions causing injury as a result of failure to employ a reasonable standard of care in conducting a given activity. The concept of strict liability is usually associated with activities having a relatively high potential for injury even when conducted with the best of care, and serves to transfer losses automatically to the party in control of the activity without regard to actual fault. The concept of trespass is generally used to protect exclusive property rights and therefore is applicable in cases of unauthorized entry onto land (Prosser, 1971).

Since the choice of a legal basis of action is somewhat dependent on the nature of the injury, it is necessary to give separate consideration to the types of injury that are likely to occur. The most probable detrimental consequence of injection is the contamination or destruction

of natural resources. The underlying concept of deep-well disposal is confinement of the waste material to strata of low utility, but there is always some possibility of the existence of undiscovered or currently unrecognized resources within the disposal zone. And, of course, permanent confinement may not be achieved. The waste may escape into other strata by such avenues as faults, abandoned wells or other excavations, or the injection well itself. Another potential category of injection-related injury includes instability problems arising from subsurface pressure alterations or other interference with the structural integrity of underlying formations. A third area of possible application of tort law encompasses situations in which injected materials invade the underlying space of adjacent landowners without measurable injury. The principal issue in such cases is not compensation for actual damages but rather the protection of an exclusive interest in property.

It should be noted that only a very limited number of court cases concerning damages associated with deep-well waste disposal has been decided. Most of the existing cases concern legal controversies arising from the injection of salt water for disposal purposes or in connection with secondary-recovery methods of petroleum production. These cases have addressed some of the potential problems associated with injection wells but provide no guidance with respect to others. It should be noted that law governing secondary-recovery operations is strongly influenced by public policy considerations associated with its necessity to the petroleum industry, thereby greatly reducing the transferability of such law to industrial waste-disposal operations. The injection of oil-field brines strictly for disposal purposes is somewhat distinguishable from the injection of industrial wastes because the brine is a naturally occurring substance being returned to an environment similar to that in which it originated. Recognition of this fact is contained in the FWPCA Amendments of 1972 in which oil-field brines are exempted from the definition of pollutant. These limitations on the general applicability of decisions regarding oil-field waste-disposal practices necessitate reliance on decisions in related areas in the determination of legal principles that will serve to resolve rights conflicts between individuals. The use of analogy always has certain inherent weaknesses, but more definitive law does not exist.

Subsurface Pollution

There is a substantial body of law concerning the escape of deleterious substances in general that gives an indication as to how conflicts concerning pollution damages from injected wastes might be resolved. All the theories of tort liability have been invoked in pollution cases, but the majority appear to rely on nuisance and negligence.

Nuisance has been a standard basis for recovery in situations involving pollution which renders a water supply unfit for use (Gulf Oil Corp. v. Hughes, 1962; Panther Coal Co. v. Looney, 1945; Masten v. Texas Co., 1927; Love v. Nashville Agricultural and Normal Institute, 1922). Since nuisance depends on the result of the defendant's actions and not on the actions themselves, the essential elements in an allegation of nuisance are proof of actual damages and the establishment of a causal connection between the activity in question and the injury. Since direct proof of a causal relationship between an alleged source of subsurface pollution and the injury is difficult to obtain, the courts will often accept proof by inference based on circumstantial evidence. Factors usually given consideration by the courts in determining causation include (1) proximity of the suspected source to the pollution, (2) the existence of other possible sources, (3) the relationship in time between the pollution and some act of the defendant, and (4) the capability of the suspected source for causing the pollution in question. The combination of factors necessary to prove causation by inference varies from case to case depending on the circumstances surrounding each situation. Contamination by the injected waste itself would likely establish a direct causal relationship, but proof of causation in injection-related pollution cases is likely to be complicated by the fact that contamination may be caused by naturally occuring fluids which migrate in response to subsurface pressure increases. The migration of resident fluids has been a significant problem in the Port Huron, Michigan, area where industrial waste injection in Ontario, Canada, was apparently the cause of seepage of salt water, oil, and natural gas from abandoned oil wells. The situation was further complicated by the international aspect of the problem, but the fact that the pollution could not be directly connected with the injected waste was significant. It was not until a chemical analysis of the seepage indicated the presence of the injected substance that the Ontario officials took affirmative action to phase out the injection operations responsible.

Negligence is also a frequently utilized basis for court actions concerning pollution problems, but the person seeking relief is under an increased burden of proof in this situation. In addition to proving damages and causation, the plaintiff must prove the acts or omissions

constituting the negligence. Because of the difficulties of proving negligence where the activity in question is located on property entirely within the defendant's control, some courts have accpeted general proof of negligence in place of the designation of a precise negligent act (Sinclair Refining Co. v. Bennett, 1941; Texas Co. v. Giddings, 1912). Another element of proof in negligence cases is the concept of foreseeability. The principal concern here is whether the defendant could reasonably have anticipated injury as a result of his actions. The absence of a reasonable anticipation of injury in connection with lawful uses of property has been employed to deny the right of recovery for a variety of injuries, including groundwater pollution. The doctrine has been applied to such activities as the burial of animal carcasses (Long v. Louisville and Nashville R. R. Co., 1908) and the location of privies (Davis v. Atkins, 1896), but apparently not to more extensive waste disposal operations.

In contrast to this group of cases where the absence of negligence has shielded the defendant from liability, another group of decisions has imposed strict liability for pollution damages without regard to negligence. This doctrine of strict liability has been applied to pollution cases primarily where potentially harmful substances are brought onto property and subsequently escape, causing injury to adjacent landowners. The doctrine has been applied to the escape of salt water in several states. In some cases, the courts have imposed strict liability on the basis of a common-law concept (Berry v. Shell Petroleum Co., 1934) first enunciated in an 1866 English case (Fletcher v. Rylands, 1865), whereas in others the basis has been a statutory enactment or administrative regulation (Gulf Oil Corp. v. Alexander, 1956). This doctrine may become significant with respect to future pollution problems which may be caused by industrial waste injection. Many of the wastes involved can be described as hazardous, and the injection process may well be viewed as an unnatural use of land, for which responsibility for accidental losses should be borne by the operator even in the absence of fault. It should be noted that some 30 states have now accepted the strict liability concept in certain areas, and the number appears to be increasing (Prosser, 1971).

Structural Instability

In addition to the destruction or damage of natural resources, injection has the capacity for producing more catastrophic types of damage. The possibility of injection-induced seismic activity has been recognized since a correlation was noted between injection at the Rocky Mountain Arsenal well near Denver, Colorado, and numerous earthquakes in the area. Law regarding man-made earthquakes is essentially nonexistent, but the potential extent of economic loss for which the injector may be liable is staggering. Of course, the proof of causation would likely be a considerable burden in such a case. Most geographical areas have some potential for seismic activity in the absence of interference from man, a fact which would raise the question of whether the injector located near the site of an earthquake was actually responsible or was only an innocent bystander.

Another possible type of injury might result from collapse of the land surface as the result of the destruction of underlying formations. This phenomenon might have the greatest probability of occurrence where acidic wastes are discharged into carbonate rock, resulting in the formation of solution channels and cavities. As in the case of seismic activity produced by injection, legal precedent is inadequate for the formulation of conclusive principles of law to govern the situation. The mechanism of damage in this case is less complicated, making a determination of causation less difficult should damage occur in this manner. It should be noted that the likelihood of damage from surface collapse as a result of injection appears small with respect to deep wells, but the potential would probably increase as depth decreases.

Subsurface Trespass

Assuming that an injected waste is effectively confined to the disposal zone so that damages are not inflicted on other parties, legal confrontations are still possible where the waste crosses property boundaries and infringes upon underlying space of adjacent landowners. The likelihood of this occurrence depends on such factors as location of the injection well with respect to property boundaries, injection rates, and characteristics of the injection zone. The customary basis for an action to protect the exclusive interest in property is trespass. Although the concept of subsurface trespass has been recognized in cases involving the extraction of underlying minerals (North Jellico Coal Co. v. Helton, 1920), the courts generally have refused to uphold the trespass concept in cases involving unauthorized entry into subsurface space in the absence of injury.

In a 1950 case decided by the Supreme Court of Oklahoma (West Edmond

Salt Water Disposal Ass'n. v. Rosecrans, 1950), the plaintiff sought to enjoin injection of salt water on adjacent property and to obtain damages for alleged trespass in connection with previous infringement of subsurface property rights. The plaintiff was requesting recovery of profits accruing to the defendant because of the alleged unauthorized use of plaintiff's land; physical damages to the land, although no specific damages were set out; and punitive damages for the oppressive disregard of plaintiff's property rights. The defendant admitted liability for any actual damages resulting from the injection but denied that damages had occurred inasmuch as the injection zone was saturated with salt water prior to the initiation of injection. The court concurred with the contention of the defendant that liability should be limited to actual damages.

Regarding the allegation of trespass, a principal consideration of the court was whether the salt water remained the property of the defendant upon its escape to the property of others. Had ownership remained with the injector, storage beneath adjoining land apparently would have constituted trespass, but the court held that ownership and control were lost upon escape and that consequently there was no trespass. In reaching this conclusion concerning loss of possession, the court compared the salt water with natural groundwater and petroleum, which are not necessarily fixed in position beneath one proprietor's land but are subject to migration and change of ownership. The court specifically noted that the migration of the injected fluid under plaintiff's land constituted only a displacement of a similar resident fluid. The applicability of this decision to a case involving an industrial waste not closely comparable to a natural fluid may therefore be somewhat questionable. However, a key issue in the case was the absence of actual injury to the plaintiff. A change in the nature of the injected fluid may not be significant provided the waste is confined to a stratum so that no damage is produced.

A similar decision regarding the subsurface trespass issue had been handed down by the Supreme Court of Kentucky in 1934 in a case involving the underground storage of natural gas (Hammonds v. Central Kentucky Natural Gas Co., 1934). The court in that case also held that the injector was not guilty of trespass since he lost possession of the gas once it escaped to the plaintiff's property. Although the defendant gas company in this case was not found guilty of trespass, the loss of ownership of the injected gas must have left the company with mixed emotions about the decision. Other courts (Lone Star Gas Co. v. Murchison, 1962; White v. New

York State Natural Gas Corp., 1960) have since refused to follow the Kentucky court with respect to the loss-of-possession issue where natural gas is involved, and this and other aspects of such underground storage operations are now regulated by statute in many states.

Another potentially relevant decision with regard to the right to control use of subsurface space is a 1931 New York case (Boehringer v. Montaldo, 1931). The case did not involve injection but arose from a dispute between the buyer and seller of property with regard to an undisclosed sewer line located 150 ft below the surface. The court held that the sewer was not an encumbrance on the basis of the view that a landowner's rights are restricted to a depth of "useful ownership." The concept of indefinite ownership upward and downward was rejected as an unacceptable principle of law.

Representative of an opposing point of view is a 1936 Kentucky case (Edwards v. Lee's Administrator, 1936) which supports the right of the landowner to exercise exclusive control over subsurface space which he cannot put to use himself. The owner of land underlain by a portion of a cave was awarded damages from an adjacent owner, on whose land the entrance was located, who made commercial use of the cave. This decision was reached in spite of the fact that the plaintiff had no means of access to the cave located 360 ft beneath his land. The primary consideration appears to have been the fact that the defendant made an economic use of space theoretically owned by another, therefore incurring liability for a portion of the profits accruing from such use.

The author of a well-know treatise on tort law (Prosser, 1971) criticizes this holding as bad law and points to the New York sewer-line case referred to previously as the more enlightened view, but it does not appear that the issue of ownership of underground space has been the subject of enough court decisions to establish a trend of opinion effectively. Of possible relevance is the fact that the concept of limiting exclusive property rights to that space which can be put to effective use has been accepted with respect to overlying space in recognition of the needs of aviation. This restriction of exclusive property rights has been necessitated by an activity generally recognized as being in the public interest. Since the feasible uses of subsurface space have been more limited, the concept of public control has not been applied to the same extent. However, expanding utilization of subsurface space for such uses as underground storage and waste disposal may require consideration of property-rights limitations in the downward direction. A restriction

of ownership of subsurface space appears to be analogous to the restriction of overlying rights, provided that freedom of use of such space is generally held to be in the public interest. Although the concept of subsurface injection of wastes is given limited acceptance at present, there appears to be no general agreement that widespread use of the underground for waste disposal is in the public interest.

In the event that the property rights of the landowner are held to encompass exclusive control over use of subsurface space, the waste injector desiring to utilize strata underlying the property of another would have to acquire rights of a nature similar to those obtained for the underground storage of natural gas. In some jurisdictions, the acquisition of a subsurface lease may have the effect of reducing the possibility of liability for accidental damages to the lessor's property resulting from injection. For example, in some states, the holder of a mineral lease is not liable for damages to the lessor without proof of specific acts of negligence, whereas strict liability is imposed for damages off the leased premises (Holbrook v. Continental Oil Co., 1955; Phoenix v. Graham, 1953).

CONCLUSION

The legal and institutional aspects of deep-well waste disposal are in the developmental stage of their evolution. There is as yet no general philosophical acceptance of deep injection as an environmentally sound method of waste disposal. The policies and regulatory procedures adopted by the states are highly variable. Implementation of the FWPCA Amendments of 1972 should result in a greater degree of uniformity, but the full impact of federal influence will not be known for some time. Several of the unanswered questions concerning subsurface disposal are related to uncertainties in the institution of property rights. Certain of these issues may be resolved by legislation, whereas others may have to await the actions of the courts as potential problems become realities. It is not clear at this time what direction the development of the legal and institutional aspects of deep-well disposal will take, but it is certain that the situation will not remain static.

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REGIONAL HISTORIES--RECHARGE AND DISPOSAL

DESIGN AND OPERATION OF LAND TREATMENT SYSTEMS FOR MINIMUM CONTAMINATION OF GROUNDWATER[1]

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ABSTRACT The increasing interest in land treatment systems for sewage effluent and other liquid wastes, as well as some solid wastes, poses a threat to the quality of the native groundwater even though the waste water itself undergoes a marked improvement in quality as it moves through the ground and becomes "renovated" water. To avoid large-scale spread of the renovated water into the groundwater basin, the renovated water should be collected again at some point by wells (deep aquifers) or drains (shallow aquifers) for reuse or release into the surface water. For the Salt River Valley, the effective transmissibility of the aquifer for recharge was evaluated from a pilot project and then used in the design of a full-scale system. This effective transmissibility was less than the aquifer transmissibility.

INTRODUCTION

The "no-discharge" policy of the Federal Water Pollution Control Act Amendment of 1972 will undoubtedly cause a stepped up interest in land disposal of liquid wastes such as conventionally treated sewage effluent, processing-plant wastes, animal wastes, etc. Although the soil mantle is an effective filter system that causes considerable improvement in the quality of the waste water as it moves down to the groundwater (McGauhey and Krone, 1967; Kardos, 1967; Bouwer et al., 1972; Bouwer, 1973), the quality of the resulting "renovated" water usually will not be as good as that of the native groundwater. Thus, the degree and spread of contamination of the existing groundwater resources by

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renovated waste water must generally be minimized.

One way to minimize contamination of existing groundwater is to apply only small amounts of waste water per unit area and grow a crop or other vegetation on the disposal field. If, for example, secondary sewage effluent is applied to vegetated land at a rate of about 1 in./week, the total nitrogen load is not much more than the amount removed from the soil by crop uptake and subsequent harvest (Kardos, 1967). Other elements, such as phosphorus, metals, etc., are also taken up by the crop, which reduces their rate of accumulation in the soil. Thus, the effluent water that seeps beyond the root zone and reaches the groundwater as renovated water is of fairly good quality. In humid areas, dilution of the renovated water by rainfall may be a significant factor.

The disadvantage of these so-called "low-rate" systems is that they require large land areas (at 1 in./week, about 260 acres are required per 1 million gal/day of waste water). Where large volumes of waste water are to be disposed of, the social obstacles associated with large disposal areas may be insurmountable. Thus, it is tempting to apply more waste water per unit area, particularly if permeable soils are available that can take the waste water at high infiltration rates. With such "high-rate" systems, waste water may be applied at rates of several feet to several yards per week. Such application considerably reduces the land area required, but the impact on the groundwater will be greater than with low-rate systems. To minimize the effect on the groundwater, high-rate systems should be designed and operated (a) to obtain the best-quality renovated water (particularly as regards the nitrogen content), and (b) to restrict the spread of renovated water into the native groundwater.

For both low- and high-rate systems, the waste water is applied to the land in intermittent fashion, rotating infiltration periods with dry or "resting" periods to allow recovery of the infiltration rates and entry of oxygen into the soil (Bouwer et al., 1972; Bouwer, 1973). The waste water may be applied to the land with sprinklers or, if the topography permits, with basins or furrows (Kardos, 1967; Bouwer et al., 1972; Bouwer, 1973).

MAXIMIZING NITROGEN REMOVAL

If sewage effluent is applied to land with a high-rate system, the nitrogen load may be much greater than the few hundred pounds that can

be removed per acre per year from the soil by growing and harvesting a crop (at an application rate of 1 ft/day, the nitrogen load may be 25,000 lb/acre per year!). The only process whereby nitrogen can be removed from the soil in quantities much greater than that which crops can take out is denitrification. This is a microbiological process whereby nitrate in the soil is reduced mainly to free nitrogen gas, which returns to the atmosphere. The process requires anaerobic conditions and the availability of organic carbon as an energy source for the denitrifying bacteria. Thus, where the nitrogen load exceeds the amount that can be removed by a crop, the system should be designed and managed to stimulate denitrification. How this should be accomplished depends on the form of the nitrogen in the waste water and on the amount of organic carbon available in the water or in the soil. If the nitrogen occurs as nitrate in the waste water and there is sufficient organic carbon, denitrification in the soil can be promoted by continuing the wastewater application sufficiently long to cause oxygen depletion in the soil.

For conventionally treated sewage effluents, the nitrogen is mostly in the ammonium form. If this effluent is applied frequently in small amounts, the upper portion of the soil profile will be sufficiently aerobic for essentially complete conversion of the nitrogen to the nitrate form (Bouwer et al., 1972; Bouwer, 1973; Lance and Whisler, 1972; Lance et al., in press). At the same time, the organic carbon level in the waste water, which is generally low for secondary effluent, will also become completely oxidized. Thus, little or no denitrification can be expected as the nitrates move down with the water to anaerobic zones, because there is no organic carbon available for the denitrifying bacteria. The nitrogen will then remain in the highly mobile nitrate form, which can result in nitrate levels in the groundwater that exceed the maximum permissible limit for drinking water. If the organic carbon level in the waste water is high, however, such as in certain liquid animal wastes, sufficient organic carbon can be left after the waste water passes through the aerobic zone to stimulate active denitrification farther down (Erickson et al., 1971).

To maximize denitrification in soils receiving secondary sewage effluent in which the nitrogen is mostly in the ammonium form and the organic carbon levels are usually fairly low, the effluent should be continuously applied for a sufficiently long period to cause oxygen depletion in the soil. The ammonium is then no longer converted to nitrate, and it can be adsorbed by the clay and organic matter in the soil. Before this cation-exchange complex is saturated with ammonium, the application of waste water should be stopped so that the soil can drain and dry. Oxygen entering the soil will then cause the adsorbed ammonium to be nitrified, after which denitrification occurs in (micro) anaerobic zones. If waste water is then applied again, the nitrate-enriched capillary water mixes with the incoming waste water which contains organic carbon, and denitrification can occur when anaerobic conditions are reached.

Thus, whereas short, frequent flooding (2 days wet, 3 days dry, for example) of infiltration basins with secondary sewage effluent gave essentially complete conversion of the nitrogen to nitrate, flooding periods of 2-3 weeks alternated with drying periods of about the same length gave approximately 30 percent removal of nitrogen in a pilot high-rate land filtration system west of Phoenix, Arizona (Bouwer et al., 1972; Bouwer, 1973; Lance and Whisler, 1972). This amounted to a total nitrogen removal of about 8,000 lb/acre per year, which is much more than can be removed by a crop. Additional research may show how the systems should be managed to obtain even greater removal of nitrogen by denitrification. This may require more careful manipulation of application rates, adding organic carbon to the effluent or to the soil, growing certain crops and incorporating the crop residue into the soil, additional treatment of those portions of the renovated water that contain most of the nitrate (including recycling through the infiltration system), etc.

RESTRICTING SPREAD OF RENOVATED WATER

To minimize the spread of renovated waste water into a groundwater basin, the renovated water should be taken out of the aquifer at some distance from the place where it reaches the groundwater. This may happen naturally if the groundwater drains to a stream or lake (Fig. 1). If the renovated water does not leave the aquifer in a natural manner, it should be collected by drains (for shallow aquifers) or wells (for deep aquifers) to limit its spread into the aquifer. After collection (and additional treatment if necessary), the renovated water may be used for irrigation, recreation (including lakes), industrial, and perhaps municipal purposes, or it may be discharged into surface water. With such a system, a portion of the aquifer is essentially used as a natural filter.

The proper distance between the points where the waste water enters the soil and where it leaves the aquifer as renovated water depends on the type of waste water, the desired quality of the renovated water, and the nature of the soil and aquifer materials. For granular soils and aquifers, underground travel distances of several hundred feet and underground detention times of several weeks may be sufficient to yield a renovated sewage effluent of suitable quality. The more time and distance allowed for underground travel, the better will be the quality of the renovated water, at least to a certain limit. Most of the quality improvement, however, takes place in the first 1 or 2 ft of the soil profile.

Deep Aquifers

When the aquifer is unconfined and relatively deep, a "closed" wastewater-renovation system can be obtained by concentrating the areas where the waste water is applied to the land in two parallel "infiltration strips" (Fig. 2). The renovated water can then be pumped from wells midway between the strips. Other possibilities are a single infiltration strip with wells on both sides, or a central infiltration area surrounded by a ring of wells (Fig. 3). The wells in the two systems of Figure 3 will pump a mixture of native groundwater and renovated waste water. This may be desirable if the use of the pumped water requires dilution of the renovated water anyway. However, if the wells should pump renovated water only, the additional native groundwater also pumped increases the pumping costs and constitutes an extra draft on the native groundwater.

The design and operation of a wastewater-renovation system consisting of two parallel infiltration strips and wells midway between the strips (Fig. 2) should be based on the following three criteria:

1. The water table below the infiltration strips should not rise to field surface, where it can restrict the infiltration rates. The water table preferably should not come closer to field surface than a distance of about 4 ft. This depth enables rapid drainage of the soil profile, and thus entry of oxygen, when infiltration periods are rotated with dry or resting periods.

2. All waste water that has infiltrated should be pumped as renovated water from the wells. No renovated water should move into the aquifer outside the system of infiltration areas and wells.

3. The renovated water should have traveled the proper time and distance underground when it reaches the wells.

In order to investigate whether a certain design meets these three criteria, the underground flow system must be predicted. This prediction will also yield an estimate of the pumping lift in the wells.

The prediction of the underground flow system for renovation systems such as those in Figure 3 requires knowledge of the rate of entry of waste water into the soil and of the hydraulic properties of the aquifer. The infiltration rates may be evaluated by local experimentation to see what hydraulic loading rate can be maintained, regardless of whether the waste water is applied with sprinklers, basins, or furrows. The main hydraulic property to be evaluated for the aquifer is the effective transmissibility for groundwater recharge, which will govern the flow system under and near the infiltration strips.

The effective transmissibility for groundwater recharge is less than the total transmissibility of the aquifer, particularly for relatively deep, unconfined aquifers, because recharge flow systems are characterized by an upper active zone and a lower passive zone (Bouwer, 1965, 1970). The effective transmissibility for recharge depends on the width of the infiltration strip. It increases essentially linearly with width until it has become equal to the total transmissibility of the aquifer. Once the underground flow has become mainly horizontal, as it does in the vicinity of the wells (Fig. 3), the total transmissibility of the aquifer can be used to analyze the rest of the flow system (Bouwer, 1970). If the wells do not completely penetrate the aquifer, the appropriate correction factors should be applied to the total transmissibility.

A good way to evaluate the effective transmissibility of an aquifer for groundwater recharge is to determine the response of groundwater levels to infiltration, as may be done in an experimental recharge project. This was done for the Flushing Meadows Project in the Salt River bed west of Phoenix, Arizona, where renovation of secondary sewage effluent by land application is studied with six parallel recharge basins covering a block measuring 220 x 700 ft (Bouwer et al., 1972; Bouwer, 1973). Two observation wells, one 30 ft deep and the other 100 ft deep, were installed in the center of this block. The response of the water levels in these wells to infiltration was simulated on an electric analog, which then indicated the hydraulic conductivity of the aquifer in vertical and horizontal directions (Bouwer, 1970). The resulting values agreed with data obtained from direct permeability measurements at the seven observation wells in the project (Bouwer, 1970).

With known directional permeability components of the aquifer, the theoretical shape of the groundwater mound was evaluated by electric analog. The Dupuit-Forchheimer theory was then applied to this mound to obtain the effective transmissibility of the aquifer for the recharge flow system, which was only 11 percent of the total transmissibility (Bouwer, 1970). This effective transmissibility, corrected for the

width of the infiltration strip, was used in analog analyses of flow systems for the prototype system (Fig. 3) to predict the shape of the water table and to construct a network of streamlines and equipotentials (Bouwer, 1970). When a certain porosity of the aquifer material was assumed, the macroscopic velocities of the water from one equipotential to the next could be determined for each stream tube, which in turn yielded estimates of the total underground travel time of the renovated water (Bouwer, 1970). The procedure was applied to various designs so that the optimum layout of infiltration areas and wells could be selected. Similar procedures can be applied to the design of other high-rate, closed, waste-water renovation systems.

Shallow Aquifers

If the water table and the impermeable layer are relatively close to field surface, wells may not be effective and the renovated water can be collected better by open or closed drains. The system can consist of two parallel strips where the waste water is applied to the soil with a drain midway between the strips (Fig. 4A), or of a series of infiltration strips and drains (Fig. 4B). Since infiltration periods are usually rotated with drying periods, short underground travel distances and detention times can be avoided in the system of Figure 4B by closing the drains below the strips receiving waste water and collecting the renovated water with the drains below the drying strips. These drains are closed and the other drains opened when infiltration and drying periods are rotated (Fig. 4C).

The water table in the systems of Figure 4 should not rise so high that it reaches the soil surface in the infiltration areas and reduces the infiltration rates. The shape of the water table in these systems can be calculated with drainage theory (Bouwer and van Schilfgaarde, 1963). By using the Dupuit-Forchheimer assumption of horizontal flow and by assuming a uniform infiltration rate, the following equation can be derived for the system below the infiltration area (Fig. 5).

$$IX = -K \frac{H_c + H_e}{2} \frac{dH}{dX}, \qquad (1)$$

where

I = infiltration rate (distance/time)
X = horizontal distance

H = vertical distance between water table and impermeable layer

K = hydraulic conductivity of soil (distance/time).

The term H_c refers to H at the outer edge of the infiltration strip for Figure 4A and at the center of the strip for Figure 4B, while H_e refers to the water table beneath the edge of the infiltration area near the drains. The horizontal distance, X, is measured from the outer edge of the infiltration strip for the system of Figure 4A, and from the center of the infiltration strip for the system of Figure 4B. Integrating Equation 1 between X = 0 and X = W (Fig. 5) yields

$$H_e^2 = H_c^2 - \frac{IW^2}{K}. \tag{2}$$

The flow from the edge of the infiltration strip to the drain can be described by the equation

$$IW = \frac{K}{L}\frac{(H_e + H_d)}{2}(H_e - H_d), \tag{3}$$

where H_d is H at the drain and L is the distance between the drain and the edge of the infiltration strip (Fig. 5). Equation 3 can be rearranged to yield

$$H_d^2 = H_e^2 - \frac{2ILW}{K}, \tag{4}$$

which, after combining with Equation 2, gives

$$H_c^2 = H_d^2 + \frac{IW}{K}(W + 2L). \tag{5}$$

The term W refers to the longest horizontal distance of travel for the water beneath the infiltration strip. Thus, W is the entire width of the infiltration strip for the system in Figure 4A, and one half the width of the strip for the system in Figure 4B.

If the drain is running free, H_d will be equal to the height of the center of the drain above the impermeable layer. However, if a back-pressure is maintained in the drain (as is sometimes done to exclude air and to avoid deposits of iron or manganese oxides in the drain), H_d is the height of the drain above the impermeable layer, plus the back-pressure head.

When H_d, I, and K are known, the value of H_c can be calculated for various combinations of W and L. Thus, the optimum combination of W and L whereby H_c does not exceed a preselected value can be evaluated. If the waste water is applied to the soil in infiltration basins and the groundwater table is so high that it coincides with the water surface in the basins, Equation 5 can be used to calculate the average infiltration rate in the basin.

Equation 5 applies to relatively shallow systems. Where the impermeable layer is at sufficient depth to render the horizontal-flow theory invalid, equivalent depths of the impermeable layer should be used, as is done in the design of agricultural drainage systems (Bouwer and van Schilfgaarde, 1963).

SUMMARY

Land application of sewage or other waste water has the least impact on groundwater when only small amounts are applied. For large wastewater flows, however, the large land requirements for such low-rate systems are a disadvantage. The land area can be reduced by applying the waste water at higher rates, particularly if permeable soils are available, but the impact on the groundwater is greater. To minimize this impact, the high-rate system should be managed to remove as much of the pollutants (particularly nitrogen) as possible from the waste water as it seeps through the soil, and to restrict the spread of the renovated waste water in the groundwater basin. Nitrogen removal can be maximized by stimulating denitrification in the soil. The spread of renovated water in the groundwater can be controlled by intercepting the flow of renovated water with wells or drains for reuse or discharge into surface water.

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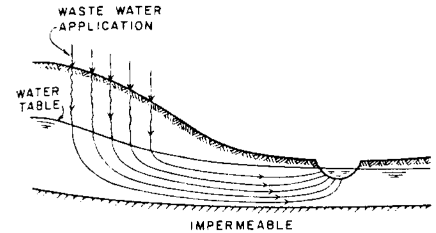

FIG. 1--Renovated waste water draining naturally into surface water.

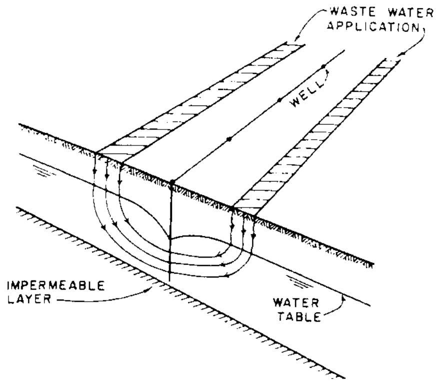

FIG. 2--Schematic diagram of two parallel strips (hatched areas) for applying waste water, and wells midway between the strips for pumping renovated water.

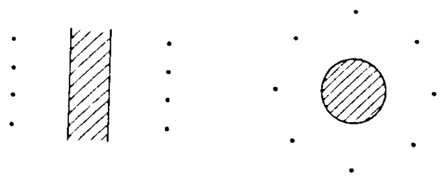

FIG. 3--Long infiltration strips (hatched area) with wells on both sides (left) and circular infiltration area surrounded by wells (right).

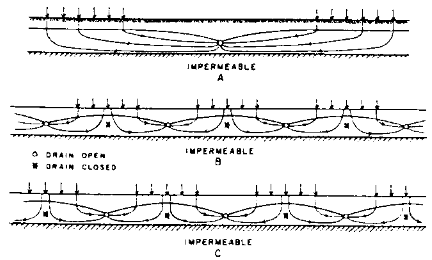

FIG. 4--Two parallel infiltration strips with drain midway between strips (A) and continuous system of infiltration strips and drains with alternate infiltration and drying (B and C).

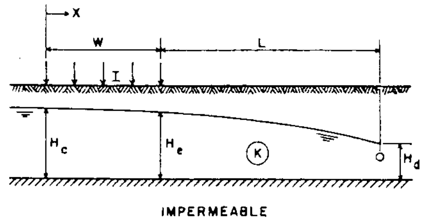

FIG. 5--Geometry and symbols for parallel infiltration strip and drain.

ARTIFICIAL RECHARGE IN UNITED KINGDOM WITH SPECIAL REFERENCE TO LONDON BASIN[1]

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ABSTRACT In England and Wales, about one third of all public water supplies are taken from natural groundwater. In many places groundwater levels have been lowered extensively, causing saline intrusion and other problems. In recent years there has been much increased interest in the development potential of groundwater, with the result that some schemes for the combined use of naturally recharged underground resources and rivers are under construction and others are under investigation.

Artificial recharge is not yet used, but it is considered to have major potential both for underground storage to supplement surface storage and for the partial purification of polluted surface water where suitable aquifers crop out at the surface. The Board's artificial-recharge program includes hydrogeologic, engineering, and economic mathematical-model studies and field experiments in most of the techniques of artificial recharge.

One important example, the London basin, is taken as an illustration. A hydrogeologic study has been carried out to assess the potential for recharge beneath London. The aquifers are the Chalk overlain by the lower London Tertiary strata, which comprise the fine-grained Thanet Sands and the sands, gravels, and clays of the Woolwich and Reading Beds. These strata form an asymmetric syncline with an axis striking east-west through London's center. The Chalk crops out in the Chilterns of Oxfordshire and Buckinghamshire in the north, and the North Downs of Kent and Surrey in the south. Over the last 170 years, groundwater levels have fallen, in some areas more than 250 ft, creating a storage volume exceeding 200 billion gal--about 5 times the total surface storage available in the Thames basin. Three areas were identified where hydrogeologic conditions are suitable for recharge.

During the hydrogeologic study, an electric analog was constructed to assist in proving the transmissivity and storativity maps calculated from pumping test data obtained during the last 100 years. The model highlighted problems of saline intrusion from the Thames and has been used to illustrate the effectiveness of proposed control measures.

More recently an engineering and economic investigation has been undertaken using 2 main techniques: (1) digital groundwater models of the selected recharge areas, and (2) a digital simulation, using the 84 years' records of daily flows in the Thames. This work has shown that additional yields of more than 70 million gal/day could be made available at low cost without the need of further surface storage.

To substantiate this work further, 2 field experiments have been carried out. One involved the recharge of an existing Chalk well and adit system; in the other, water was injected into a pair of new wells, one open only to the Chalk and one open only to the Thanet Sands. There was a comprehensive program of recharge and water-quality sampling and analysis.

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INTRODUCTION

The Water Resources Board's research and development program, carried out either by the Board's staff or under their control at an annual cost of about £1 million, includes such subjects as water resources system simulation, instrumentation, mathematical modeling of groundwater systems, geophysics, combined use of surface and groundwater, ecologic problems, and artificial recharge. This paper is concerned with artificial recharge of underground strata and the way in which the technique has been investigated and is being applied in one particular situation, the London basin.

In England and Wales about one third of all public water supplies are taken from natural groundwater resources. Extraction (abstraction) from aquifers in many areas is already at rates equal to natural recharge. In some cases groundwater levels have been lowered, leading to reduced surface stream flow and also to the abandonment of wells with reduced yields. There are also problems of saline intrusion from the sea, or of mineralized groundwater from contiguous strata. There are some areas, however, where groundwater resources have remained unused. In recent years, there has been much increased interest in the development potential of groundwater, with the result that some schemes for the combined use of naturally recharged underground water and surface water are now under construction and others are under investigation.

Although artificial recharge is not yet used in the United Kingdom, it is considered to have major potential both for underground storage to supplement surface storage and for partial purification of polluted surface water where suitable aquifers crop out at the surface. The Board's artificial-recharge program includes hydrogeologic, engineering, and economic mathematical-model studies and field experiments in several artificial-recharge techniques. In this paper this approach is illustrated by the example of the London basin, where the work carried out has provided information to help the planner assess the role of artificial recharge along with other water-resource possibilities.

The principal water authority, the Metropolitan Water Board, supplies 1,750 x 10^3 m^3/day of which 1,350 x 10^3 m^3/day comes from the River Thames (Fig. 1) and its existing surface storage of 164 x 10^6 m^3; 90 x 10^3 m^3/day comes from the River Lee in north London, which has a total storage of 50 x 10^6 m^3; 110 x 10^3 m^3/day is taken from the groundwater in the Lee Valley and the remaining 200 x 10^3 m^3/day comes from other groundwater sources in the London basin. It is estimated that demands will rise to 2,100 x 10^3 m^3/day by the end of this century (Great Britain, Water Resources Board, 1966). Existing resources in the Thames and Lee catchments are considered inadequate to meet these requirements, and the development of both new surface sources and increased groundwater use is under construction or consideration.

Groundwater levels under and near London have been severely depleted over the last 170 years, providing a large storage potential, and there have been some successful artificial-recharge experiments by the Metropolitan Water Board (Boniface, 1959). It was therefore decided to carry out desk studies of artificial-recharge potential, supported as much as

possible by field experiments, in order to provide the basis for decision making.

HYDROGEOLOGIC STUDY

The region examined in the study (Great Britain, Water Resources Board, 1972a, b) covers about 6,000 km^2 of the central part of the London basin with a population of over 10 million. The extent of the region and its main features are shown in Figure 1. The region is drained by the River Thames and some of its tributaries, the most important of which is the River Lee. As indicated in the geologic map and cross sections of Figures 2 and 3, the London basin is an asymmetric syncline in Cretaceous and Tertiary deposits; its axis trends approximately east-west through London's center. The Chalk crops out in the Chiltern Hills of Oxfordshire and Buckinghamshire in the north, and the North Downs of Kent and Surrey in the south. In the intervening region, the Chalk is overlain by Tertiary deposits which compose a thick clay, the London Clay, which lies on a series of sands and clays collectively referred to as the "Lower London Tertiaries." The lowermost deposits of this series, referred to as the "Basal Sands," are the fine-grained sands of the Thanet Beds, which are overlain by the sands, gravels, and clays of the Woolwich, Reading, and Blackheath Beds.

The Chalk attains a maximum thickness of about 240 m. Although the overlying "Basal Sands" are up to 40 m thick in the east of the region, they thin progressively toward the west and are mainly absent in Berkshire.

The "Basal Sands" and the Chalk are the principal aquifers of the London basin. Groundwater development began in the early part of the 18th century and increased until there was a maximum extraction of about 227 x 10^3 m^3/day. In recent years this rate has declined to about 185 x 10^3 m^3/day, which appears to be in equilibrium with inflow from the Chalk outcrops. With increasing extraction of groundwater, natural recharge from the Chiltern Hills and North Downs was inadequate to maintain the initially high groundwater levels, with the result that groundwater levels fell in some areas by as much as 76 m. At present the "Lower London Tertiaries" have been dewatered over an area of about 906 km^2 and the Chalk over 660 km^2, the corresponding reductions in storage being 864 million m^3 and 164 million m^3, respectively, a total of 1,028 million m^3.

Two important consequences of this extensive lowering of groundwater levels, apart from encouraging both public and private users to seek other supplies, was to reduce spring and stream flows at the outcrop and to induce saline intrusion from the River Thames estuary into the Chalk.

In assessing the feasibility of artificial recharge in the London basin it was thought essential to have a knowledge both of the variation, transmissivity, and storage properties of the Chalk and of the thickness and nature of the "Lower London Tertiaries" overlying the Chalk, as well as to know the performance of existing wells.

The hydraulic properties of the Chalk are reasonably well known because of the many wells and boreholes that have been used to exploit it. The mass of the rock, a fine-grained fissured white limestone, has a relatively low permeability, and the average hydraulic conductivity is in the range of 5×10^{-4} to 1.25×10^{-2} m/day. The high permeability of the aquifer as a whole is due to the porosity associated with the fissure system. High transmissivity values, commonly 1,100 m^2/day, occur where the Chalk is unconfined, particularly along the river valleys and in association with dry valleys in the Chalk. Lower values, generally less than 75 m^2/day, are found along the interfluves. The transmissivity generally decreases toward the central synclinal axis and, indeed, the transmissivity pattern reflects many of the structural features of the basin. High transmissivity values are associated with dome structures and with anticlinal axes; low transmissivities occur in synclinal areas (Ineson, 1962). It is thought that the development of fissures in the Chalk is the combined result of tectonic disturbance, reduction in confining pressure due to erosion, and enlargement by solution in groundwater flow.

Although the volume of dewatered "Basal Sands" which could be used for artificial recharge greatly exceeds that of the Chalk, their hydraulic properties are not as well known as those of the Chalk. This is probably due to the difficulties encountered in drilling through these unconsolidated water-charged sands, which were lined with solid casing during well drilling. As a consequence, no hydraulic properties of the sands were determined from pumping tests. However, laboratory tests have shown that the Thanet Beds are uniform fine sands with an average hydraulic conductivity of 2.5 m/day. A representative average value for the Woolwich and Reading Beds is about 20 m/day.

The "Basal Sands" are thought to be in good hydraulic connection with the Chalk. Therefore, it should be possible to combine the relatively large storage in the sands and the relatively high transmissivity in the Chalk in any artificial-recharge development. There is no clear indication from the hydrogeologic study whether the sands should be recharged through the Chalk or vice versa.

The mean yield from a 460-mm-diameter well penetrating the confined aquifer, excluding those areas of particularly low permeability, is estimated to be about 0.6×10^3 m^3/day, with a drawdown of 15 m. However, yields well in excess of 5×10^3 m^3/day are sometimes obtained where extraction takes place at two or more larger wells interconnected with adits, or where there are adits radiating from a single well.

Artificial recharge of the London basin was first practiced experimentally by the East London Water Works Company in 1890, but no detailed records were kept. The possibility of supplementing water resources by recharge was given no further practical attention until the 1950's, when the Metropolitan Water Board conducted experiments in the Lee Valley (Boniface, 1959). These experiments demonstrated that water could be recharged into the Chalk at rates up to 45×10^3 m^3/day through a group of four wells normally used for extraction.

The recent hydrogeologic studies (Great Britain, Water Resources Board, 1972a, b) suggested that the ideal location for artificial recharge should include as many of the following features as possible: (1) the "Basal Sands" should be greater than 9 m thick; (2) the transmissivity of the Chalk should exceed 150 m^2/day; (3) the Basal Sands should be completely, and the Chalk partially, dewatered. These conditions were found only over very limited areas, but by some relaxation two areas of good potential were identified. Storage may also be developed by dewatering, and one suitable area was found on the Chalk outcrop. The main conclusions of the hydrogeologic study of the potential for artificial recharge were as follows:

1. The hydrogeologic conditions are particularly favorable for artificial recharge in the confined area in two regions, the Lee Valley and the Leyton-Dagenham area. Potential underground-storage volume in the Lee Valley is at least 91×10^6 m^3 and possibly as much as 205×10^6 m^3. In the Leyton-Dagenham area, the storage potential is 114×10^6 m^3 (see Fig. 2).

2. Recharge of the Chalk at the outcrop should be feasible in the Wandle and Ravensbourne catchments south of the Thames. Storage for 36×10^6 m^3 could possibly be created by lowering groundwater levels on a regional scale.

The principal source of water for recharge would be the River Thames. There are other possible sources including other rivers, transfers from other basins, and sewage effluent, but the resources of the Thames are sufficient to meet the demand for water for any regional recharge scheme for some time to come.

ELECTRIC-ANALOG MODEL STUDIES

An electric-analog model study was undertaken (Great Britain, Water Resources Board, 1973) because it was thought that this type of model could simulate the groundwater system with sufficient accuracy to allow the effects of recharge to be evaluated. Although the complexity of the groundwater regime and the extent of the area to be covered subsequently indicated that a numerical approach would have been more appropriate, the study was valuable in four ways. First, the understanding of the system was greatly improved by the detailed work undertaken to investigate changes in groundwater levels and flows between the early 19th century and the present. During this work the natural groundwater flow through the London basin was established. Second, it was possible to investigate the possible consequences of artificial recharge in two areas. Third, the effectiveness of artificial recharge was assessed as a means of preventing further saline intrusion from the Thames estuary and, last, the appropriateness analog models for that type of study were evaluated.

A steady-state resistor network analog was first constructed, covering an area of 3,600 km^2 with a mesh spacing of 2 km, resulting in about 900 nodes. The area covered allowed peripheral groundwater levels which had remained essentially constant over the last 100 years to be represented by constant potentials used at the model boundary. The scaling factors for the model were 15.22 m/volt over the modeled range from 80 m below sea level to 80 m above sea level; transmissivity was modeled at 200 x 10^6 $m/^2$day/amp using resistors of 2.7 k-ohm to 270 k-ohm to cover the range from about 15 m^2/day to over 1,500 m^2/day.

The model was calibrated using two distinct methods. First groundwater levels were simulated by iterative adjustment of the resistor network leading to a transmissivity distribution in the London basin. Second, transmissivities were calculated using the available yield/drawdown relationships from wells within the basin and the resulting transmissivity map. Using the map, resistor values were fixed and the model provided a solution for natural groundwater potentials in the area under

steady-state conditions. Although it was realized that measured water levels were most likely to give an accurate result, there were some instances where the flow conditions were known to differ from those indicated, and the use of transmissivities was then thought to be a better way of establishing the correct conditions. This part of the study concluded that although a steady-state model gives a good appreciation of conditions which would ultimately occur under a given recharge and extraction regime, it gives no indication of the changes with time which occur within the aquifer, and a non-steady-state model would be preferable.

ECONOMIC AND ENGINEERING DESK STUDIES

When there was sufficient hydrogeologic evidence to suggest that artificial recharge should be considered further, the broad technical problems involved in using the technique were studied and, later, the costs were estimated. The objective of the study was to enable comparisons to be made between artificial recharge and other resource development. It was recognized at the outset that extensive recharge of the London basin could only be accomplished by means of wells, owing to the high degree of urbanization and the geologic conditions. The study progressed in three stages. First, the possible sources of water were examined and their yield assessed in combination with the existing surface storage and underground storage in the London basin. Second, groundwater models were used to assess the behavior of the aquifer under different recharge regimes and then to design well fields. Finally, the first two stages were combined in possible prototype developments which were cost estimated.

Sources and Yield Assessment

Although there are a number of possible river sources in the London basin, by far the most important is the River Thames. The mean daily flow of the Thames at Teddington Weir, the tidal limit, is 5,780 x 10^3 m^3/day; the flow-duration curve at Teddington for the years 1883-1964 is shown in Figure 4. The highest recorded flow was 68,000 x 10^3 m^3/day in 1884 and the lowest was recorded in 1922, when flows were below 909 x 10^3 m^3/day for 22 days. Water is extracted above Teddington Weir principally by the Metropolitan Water Board, which is licensed to take an average of 1,350 x 10^3 m^3/day. There is, however, a statutory minimum flow of 773 x 10^3 m^3/day which has to be maintained at the Weir.

Artificial recharge in the London basin can be evaluated only as part of a larger system in which both surface and groundwater resources are used together in the most efficient way. The system studied was considered to comprise two pumped storage reservoirs, one a surface reservoir and the other an underground reservoir to which severe constraints on the rates of filling and emptying applied. The yield assessment was made by a digital simulation using the mean daily natural flows at Teddington Weir for the 84 years from 1883 to 1967. The Thames simulation model, shown diagrammatically in Figure 5, contains six basic variables as follows.

1. Demand (D) is the water-supply demand on the whole system and it may be drawn directly from the Thames, surface-water storage, or underground storage.

2. Surface-Water Storage (S) is considered as a single unit, although in practice there would be several separate reservoirs. Storage is refilled by pumping from the Thames when flow allows and is emptied to meet the water-supply demand.

3. Pumping Capacity (P) is the size of the pumps used to fill surface reservoirs. Again only a single unit is assumed although there would be several in practice.

4. Underground Storage (GWS) is the volume of the groundwater reservoir being used. It is filled from the Thames and emptied to meet the water-supply demand.

5. Groundwater-Recharge Rate (R) is the maximum rate of recharge to underground storage.

6. Groundwater-Extraction Rate (A) is the maximum rate at which groundwater is extracted and used to meet demand.

Before running any simulation it is necessary to specify the operating rules and, because of the large numbers of variables already present within the system, they are simplified. Although it is recognized that such an approach probably will not give an estimate of the optimum yield, it is considered to be sufficiently precise for desk-study purposes. When particular schemes are being designed, the rules can be refined to meet a particular objective. The operating rules considered were as follows.

1. During periods when river flow is above the immediate demand (D) plus the minimum acceptable flow at Teddington (MAF) water is first pumped to the surface-water storage at up to the maximum rate (P) if there is spare capacity. If there is residual river flow available or

if surface reservoirs are at full capacity, water is used to fill underground storage at rates up to the maximum recharge rate (R). When river flow is less than D + MAF, the deficiency is met first by pumping from underground storage at up to the maximum groundwater-extraction rate; then, if this is insufficient, surface storage is used.

2. This rule is similar to rule 1 in all respects except that any excess river water is pumped first to underground storage and then to the surface-water reservoirs if storage is available.

3. This rule is also similar to rule 1 except that from mid-September to the end of January of the following year, demands are met from underground storage in the first instance only if the surface reservoirs are less than half full.

Subsequent simulations showed that there was little difference between the results obtained from operating rules 1 and 2. Operating rule 3 did give somewhat increased yields from the system, but only at the expense of more extraction wells. For the purposes of this paper only rule 1 will be considered further. Changes in the value of P between present capacity of 5,460 x 10^3 m^3/day and projected capacity of 7,750 x 10^3 m^3/day made only marginal differences to the system yield. As all simulations included P within this range, the variable is ignored in the presentation of the results.

The computer simulation program considered each mean daily flow at Teddington for the 84-year period of record and allocated river water to meet the demand or to fill surface or underground storage in accordance with the operating rule. The program output the dates at the beginning and end of a recharge period and the state of surface and underground storage at those times. The dates on which storage was full or empty were also recorded. Empty surface-water storage defines failure, and the program printed the deficit so that severity of failure could be assessed. The criterion of reliability was selected as a failure rate of twice during a 100-year period. The limitations of this definition of failure have been recognized (Jamieson and Sexton, 1972), but the approach was considered adequate for the recharge study.

A typical relationship between the yield from the combined surface-water and groundwater systems and the maximum groundwater-extraction rate for a range of recharge rates is shown in Figure 6. The surface and groundwater storage capacities are taken as 260 x 10^6 m^3 and 226 x 10^6 m^3 respectively. By setting underground storage to zero, the yield from the surface storage alone has been calculated. By subtracting this from the overall yield of the combined scheme, the additional yield resulting from the introduction of underground storage has been calculated and is also given in Figure 6.

For a given recharge rate, the yield initially increases with increase in the groundwater-extraction capacity until a maximum is reached, after which further increase in groundwater-extraction capacity causes the yield to decline. The maximum yield is obtained when the groundwater-extraction rate is such that underground and surface storage are emptied at the same time. To the left of the peak groundwater is used at an insufficient rate, allowing surface reservoirs to fail first. To the right of the peak, groundwater is pumped too heavily between critical droughts and is depleted before the surface reservoirs, causing increased demand on them; this condition also reduces the overall yield of the system.

Curves similar to those in Figure 6 were plotted for a wide range of surface and underground storage values. The resulting peak groundwater yields and recharge rates were used to plot the relations with surface-water storage, groundwater storage, and recharge rate. In turn, these relations were used to determine a range of yields for the main potential recharge areas in the London basin, thus providing the basic information for cost comparisons. Figure 7 shows typical relationships between the groundwater component of yield and surface-water storage volumes for an available underground storage of 226×10^6 m^3 and a range of recharge rates. The simulation also determines the period of time during which water is being recharged into and extracted from underground storage.

Groundwater Models of Recharge Schemes

Regional Models--Previous artificial recharge experiments in the London basin were limited. However, the hydrogeologic study gave a sufficient understanding of the aquifer properties and the distribution of groundwater levels and extractions to allow operational predictions to be made using the mathematical theory of groundwater flow in the manner described by Oakes and Wilkinson (1972). The resulting digital simulation models, of more limited extent than the analog models, covered the Lee Valley and Leyton-Dagenham region, a total area of 540 km^2 subdivided into a 1-km^2 grid. The objectives of the model were to determine: (1) the rise and fall in groundwater levels during the operation of selected schemes and the relationship between well layout and the recov-

erable quantity of water, (2) the recharge and extraction capacity of existing public supply wells under different conditions, (3) the effects of artificial recharge on inflow from the Chalk outcrops and from the Thames estuary, and (4) the rate of advance of saline water under present conditions and the effectiveness of artificial recharge in reversing saline intrusion. The complex aquifer system was simplified for the initial analyses in two main ways. First, the groundwater was assumed to be unconfined even though locally raised water levels would ultimately make this untrue. Second, the aquifer was treated as a single layer with a single set of properties.

During the period 1960-1965 groundwater levels in the area modeled were generally static, showing that a balance existed between groundwater extraction and inflow from both the Thames estuary and the Chalk outcrops. This steady-state situation was first modeled to determine whether the transmissivity values were similar to those obtained from the hydrogeologic study. Although the pattern was similar, modeled transmissivities were generally much higher. They probably reflect more closely the properties of the large wells and adit systems which could not be included previously.

After a transmissivity pattern compatible with the 1965 groundwater levels had been established, an attempt was made to simulate the earlier Metropolitan Water Board recharge experiments (Boniface, 1959). Figure 8 shows that the predicted and actual levels were in fairly close agreement, thus giving some confidence in the modeling process.

From the large number of possible recharge and extraction cycles, a few were chosen and the model was run, adjusting as necessary the numbers and distribution of recharge and extraction wells to maintain groundwater levels that were neither too near ground level nor below groundwater levels in 1965—that is, the level below which existing extraction wells would be affected. However, the model predicted that dewatering the aquifer in excess of 1965 conditions was necessary, particularly with cycles having long recharge periods, in order to recover all the recharged water. This was necessary because recharged water spread away from the wells and became less accessible and because raised groundwater levels reduced inflow from the Chalk outcrop, causing increased surface-stream flow. This would possibly become available for surface storage. Because of the need to keep groundwater levels at least as high as those in 1965, a number of models were run to determine the amount by which recharge should exceed extraction and to establish the effective loss of water to the recharge system. The range was found to be between 25 and 35 percent, and an average loss of 30 percent was assumed in the economic analysis.

Well-Field Models--The regional models give only average water levels, owing to artificial recharge, and it is necessary to look in detail at particular areas in order to design new well fields. The region was subdivided on the basis of topography and aquifer properties and a uniform spacing of recharge and extraction wells was disposed over each area. It was then assumed that the well field was very large and that recharge and extraction rates for each well are the same throughout each region. Thus, it was necessary to analyze only a single well with an impermeable boundary at the radius of influence.

Two recharge and extraction cycles were chosen from the infinite number available from the Thames simulation model: one where the total available storage was refilled at a constant rate over 4 years and emptied at a constant rate in the following year, the other where available storage was filled in 5 months and emptied in the following 7 months at constant rates. Well-field spacings for other regimes were interpolated between those extremes. The model was then used to determine the spacing of wells in an area where raising well-water levels above ground surface and lowering groundwater levels below those in 1965 were avoided.

Possible Prototype Developments and Costs

The engineering feasibility was assessed separately for each area, taking into account both the groundwater modeling and Thames simulation results and the physical features. In one example, that of the Lee Valley, it was assumed that water would be taken from the River Thames through an existing 2.5-m-diameter tunnel (see Fig. 1) which has a capacity of 450×10^3 m^3/day, including spare capacity of up to 225×10^3 m^3/day. A well recharge scheme would be introduced in stages beginning at 45×10^3 m^3/day, with a yield of at least 90×10^3 m^3/day, rising to a recharge rate as high as 225×10^3 m^3/day giving a yield of about 450×10^3 m^3/day. The existing distribution system and wells would be used wherever possible, but there would be a need to construct some new boreholes.

The capital and operating costs of artificial recharging in the potential areas were estimated using 1972 cost rates currently in use by the Water Resources Board (Appendix I) and taking into account river

intakes, pumps and pumphouses, pipelines, treatment works, recharge and extraction wells, observation wells and ancillary works. In common with other Water Resources Board studies, distribution to the consumer was not included. For each area the schemes were designed using values of storage and yield from the Thames simulation model and recharge/extraction well numbers and spacings from the groundwater modeling. Typical results are illustrated in Figure 9, in which unit capital costs for artificial recharge are related to yield, surface storage, and recharge rate for a given underground storage in Lee Valley, assuming that (a) 100 percent and (b) 70 percent of the recharged water is recovered.

FIELD EXPERIMENTS

As the Lee Valley had been shown by the desk studies to be the most potential area for artificial recharge, and as some successful experiments had already been carried out by the Metropolitan Water Board, it was decided to carry out two further pilot schemes at different sites in the same area. The objectives were: (1) to determine the rate at which potable water could be recharged into the Chalk and "Basal Sands" (a) through an existing Chalk well and adit system, or (b) through two new wells, either separately or together, one open only into the "Basal Sands" and the other open only into the Chalk; (2) to determine the manner and causes of any change in recharge performance with time; (3) to examine the extent and causes of any chemical changes in the aquifer and groundwater and to investigate any trends; and (4) to examine the quality of water extracted following recharge.

At one site, known as Ridge Avenue, in the Middle Lee Valley (Fig. 10), two 600-mm-diameter recharge wells were constructed side by side early in 1972. One well 60 m deep is arranged to recharge the Chalk only, and the other, 30 m deep, is arranged to recharge the "Basal Sands" only. The groundwater table is 45.5 m below surface. There are eleven 150-mm-diameter observation wells, eight into the Chalk and three into the "Basal Sands." After an initial pumping test, the Chalk well was treated with ten tons of hydrochloric acid and then repumped. The yield improved from 340 m^3/day with a drawdown of 7.4 m to 545 m^3/day with only 1.8 m drawdown. The calculated transmissivity improved from about 90 m^2/day to 225 m^2/day and the storage coefficient was about 0.5. Treated Thames water was used to recharge the Chalk well at rates up to 4.5×10^3 m^3/day and Table 1 shows the results of one experiment during 1972.

The results showed that over a period of 75 days a recharge rate of 4.5×10^3 m^3/day was maintained without substantial change in groundwater level. Although the highest levels were 2 m above the base of the "Basal Sands," there was no recharge into them and efforts to recharge these strata directly have not yet been satisfactory; it is considered that recharge of the two aquifers through the Chalk only would be most successful. Chemical analysis of the groundwater and recharge water during the experiment showed that the good-quality groundwater within the command of the observation wells was replaced by Thames-derived water. There were no indications of quality problems.

At the other site, Figure 10, known as Ponders End, an existing 4-m-diameter 63-m-deep Chalk well with over 700 m of adits in the Chalk was recharged at rates up to 4.5×10^3 m^3/day. The natural groundwater table is 31 m below ground level, about 4 m above the interface between the Chalk and the "Basal Sands." There are three 150-mm-diameter observation wells, two of which were drilled into the "Basal Sands" and one into the Chalk. Table 2 shows the quantitative effects of recharging at up to 4.5×10^3 m^3/day during 1972. Borehole No. 5 is the Chalk observation well and Nos. 1 and 6 penetrate only the "Basal Sands."

As far as quantitative results at Ponders End are concerned, there is little doubt that, as at Ridge Avenue, rates in excess of 5×10^3 m^3/day can be achieved. But there are doubts about some quality aspects and these, which are still under investigation, may be outlined as follows.

Natural groundwater level at Ponders End is in the "Basal Sands." Analyses during recharge have shown that, as water levels were raised, increasing quantities of iron (up to 200 mg/l) and of sulfate ion (up to 3,000 mg/l) were taken into solution in the two observation boreholes within the "Basal Sands." This change is accompanied by a low pH value of 5.5, compared with about pH 7.5 in the Chalk water, and the dissolved oxygen content of the water is lowered from about 5 mg/l to zero. The investigation of these phenomena includes a long-term pumping test of the Chalk well to determine the overall changes in groundwater quality, laboratory examination of cored samples of the sands, and bacteriological determinations. In the future additional recharge and extraction cycles are planned to establish the degree to which these effects may attenuate.

INITIAL PROTOTYPE DEVELOPMENTS

Although the experiments are not yet completed, early results and the desk investigations are considered sufficiently encouraging to begin an initial prototype development. The proposed scheme (Fig. 10) comprises six existing Chalk wells and four new Chalk wells in an area of relatively high transmissivity along the River Lee. At present the Water Resources Board and the river authority, the Lee Conservancy, are preparing a proposal for presentation to the responsible government department. This plan includes refinement of the digital models in order to estimate more precisely the effects of different recharge regimes and the probable rates of recovery from this first scheme. The recharge rate and estimated yield of the scheme are 60×10^3 m^3/day and 110×10^3 m^3/day, respectively, and the estimated capital cost of the scheme is about £250,000.

CONCLUSIONS

1. The hydrogeologic study was a comprehensive assessment (from 170 years data) of aquifer storage and other hydraulic properties of the London basin aquifers. This study identified usable underground storage of between 205×10^6 m^3 and 319×10^6 m^3, or 1.0 to 1.5 times the total existing surface storage in the Thames and Lee basins.

2. Experiments carried out by the Metropolitan Water Board have demonstrated that artificial recharge through Chalk wells can be practiced for a period of at least ten years without noticeable deterioration in performance through clogging.

3. Although the electric-analog studies were later thought to have been of less value than numerical models, mainly because of the relative difficulty in modeling non-steady-state conditions, they were of considerable help in understanding aquifer behavior.

4. By combining the results of the surface-water simulation model of the Thames at different storage-capacity levels with the results of digital-model studies of the groundwater system, it has been shown that additional yields as high as 350×10^3 m^3/day can be obtained economically from the River Thames by using artificial recharge. This represents a 26 percent increase over the available yield of the present system.

5. There is some practical confidence in the digital-modeling technique because it was successfully used to simulate the Metropolitan

Water Board's artificial-recharge experiments.

6. The existing digital groundwater model needs refinement, to include more precise data for the two layers of the aquifer, before it can be used in the detailed design of artificial-recharge schemes. This work is in hand.

7. The early results of the Water Resources Board experiments at Ponders End and Ridge Avenue further indicate that recharge of the aquifers can be accomplished at satisfactory rates through both existing and new wells. Potential problems of groundwater quality remain to be solved.

8. The results of the studies have been sufficiently good to encourage the Lee Conservancy Catchment Board to propose a prototype development in which Thames-derived water would be artificially recharged at 60 x 10^3 m^3/day. The additional yield would be 110 x 10^3 m^3/day. The detailed design of this scheme is in hand.

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Table 1. Groundwater-Level Changes During Recharge of Chalk Well at Ridge Avenue During 1972 (m)

Recharge Rate 10^3 m^3/d	Recharge Period (days)	No. of days since recharge started	Recharge well	Chalk observation well number and distance from recharge well: 3 (5 m)	1 (10 m)	2 (10 m)	4 (15 m)	5 (30 m)	6 (45 m)
0.6	19	19	1.27	0.25	0.28	0.26	0.31	0.32	0.36
1.1	15	34	2.49	1.60	1.86	1.87	1.74	1.56	----
1.7	42	76	3.46	2.40	2.15	1.91	1.83	1.64	1.38
2.3	13	89	4.76	3.71	3.13	2.69	2.58	2.22	1.81
3.4	7	96	8.14	6.95	5.37	2.82	3.98	3.36	2.40
4.5	74	170	14.23	12.91	9.12	6.51	5.95	4.77	3.53
0	7	177	0.67	-0.26	-0.18	-0.14	-0.06	0.05	0.15
0	19	196	0.22	-0.76	-0.62	-0.61	-0.49	-0.37	-0.20

Table 2. Groundwater-Level Changes During Recharge of Well at Ponders End During 1972 (m)

Recharge Rate $10^3\ m^3/d$	Recharge Period (days)	No. of days since recharge started	Well No., distance from recharge well and aquifer monitored			
			Recharge well	Basal Sands 1 (15 m)	Basal Sands 6 (90 m)	Chalk 5 (130 m)
1.7	33	33	3.65	1.09	1.92	2.17
2.4	64	97	5.48	2.51	2.99	4.14
3.5	14	111	8.17	----	----	5.43
4.5	81	192	14.69	10.59	10.65	10.81
0	12	204	3.61	8.04	8.10	9.23

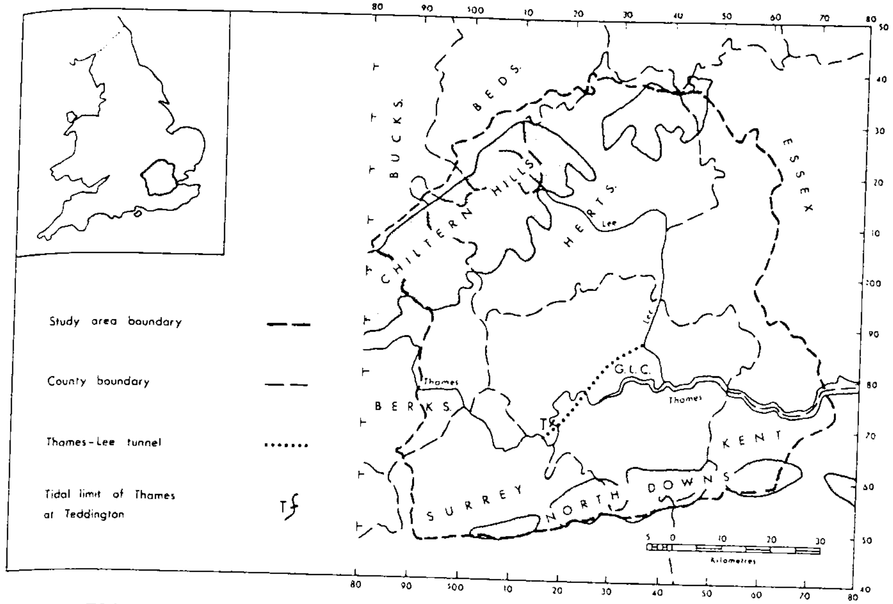

FIG. 1--Extent of London basin and its principal features.

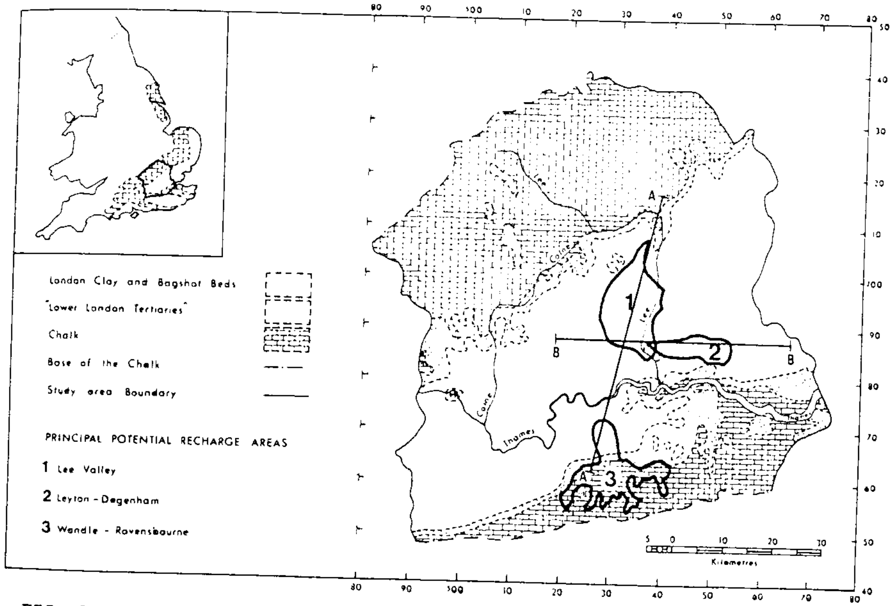

FIG. 2--Geology of London basin and principal potential recharge areas.

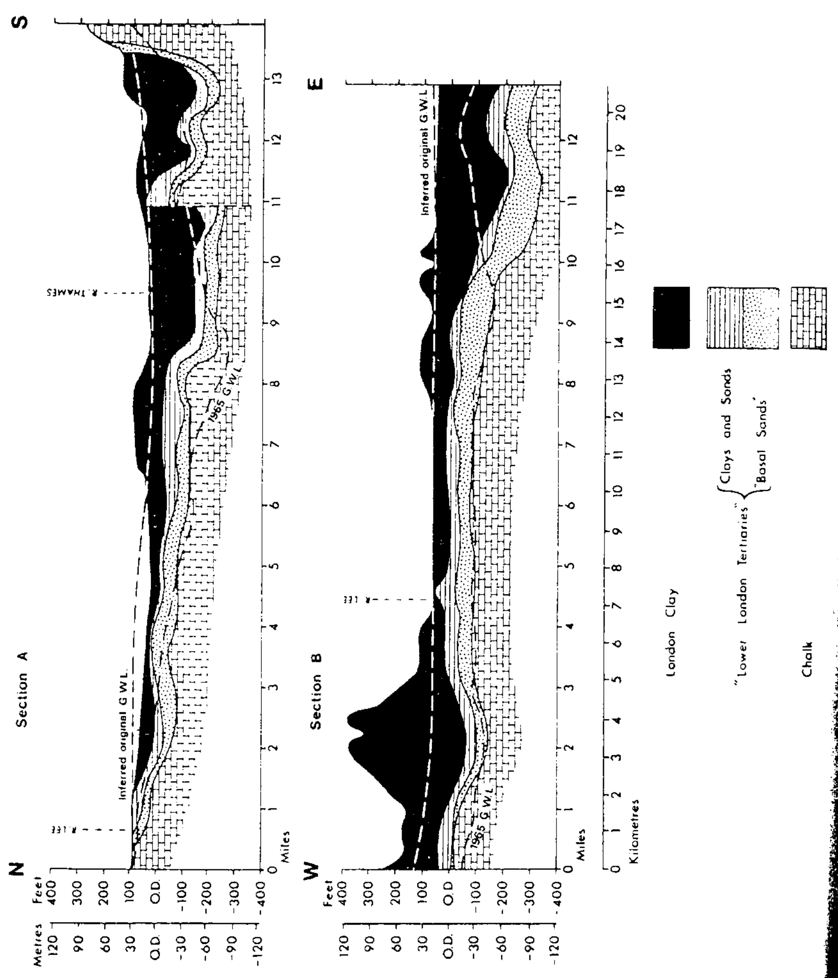

FIG. 3--Geologic cross sections through London Basin; lines of sections are shown in Fig. 2.

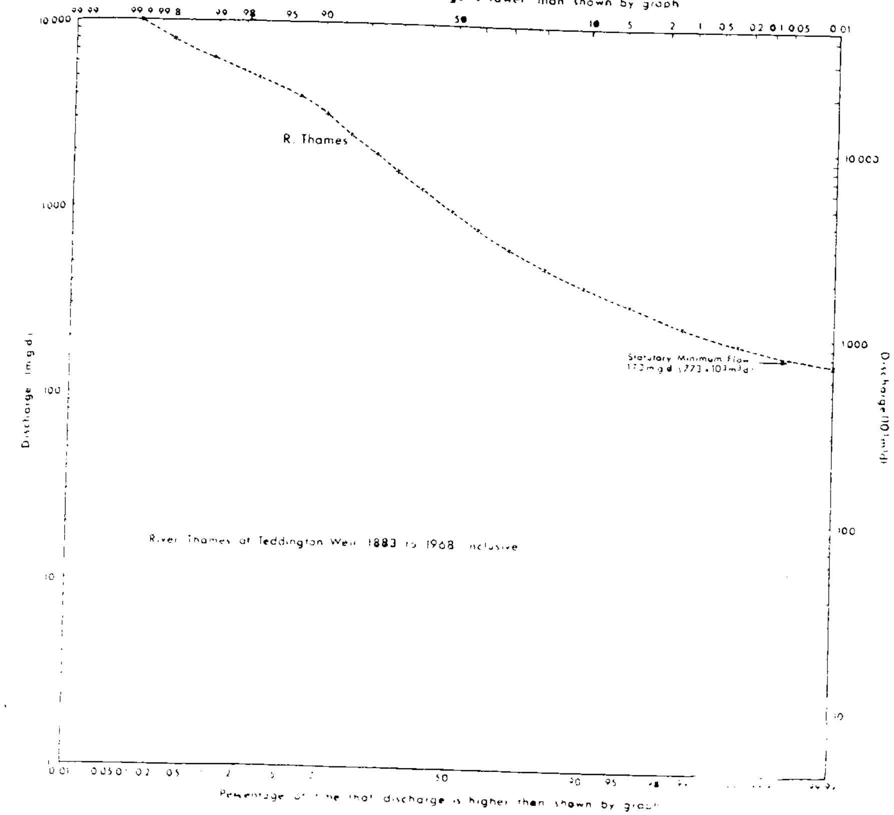

FIG. 4--Discharge frequency curve of River Thames at Teddington Weir.

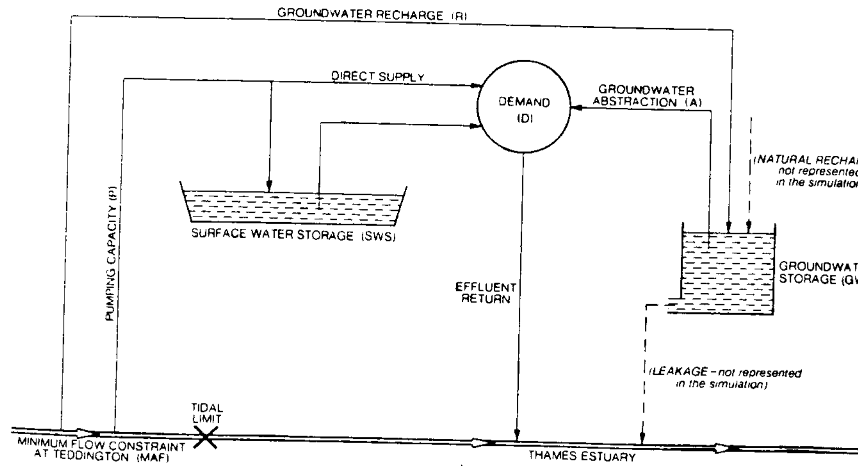

FIG. 5--Components represented in underground and surface-water storage simulation model of River Thames.

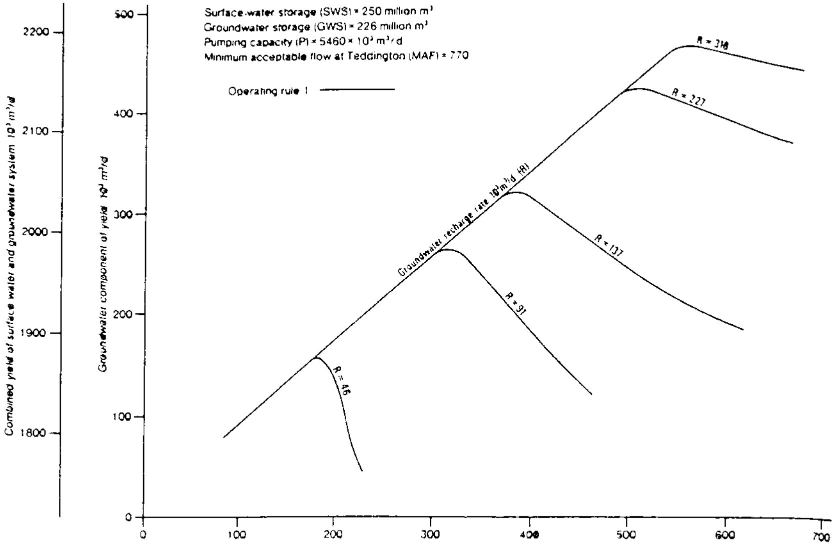

FIG. 6--Typical relationships between yield and groundwater exstraction (abstraction) for a range of recharge rates derived from River Thames simulation study using operating rule 1.

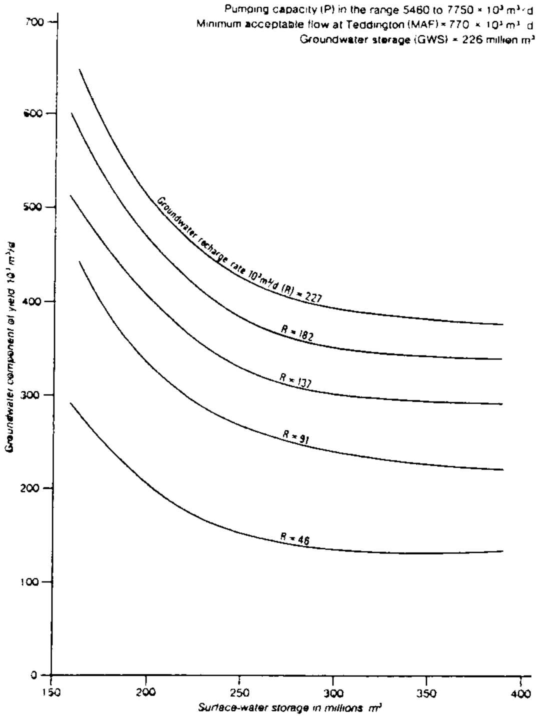

FIG. 7--Relationships between groundwater component of yield and surface-water storage for a groundwater storage of 226 x 10^6 m^3 and a range of recharge rates

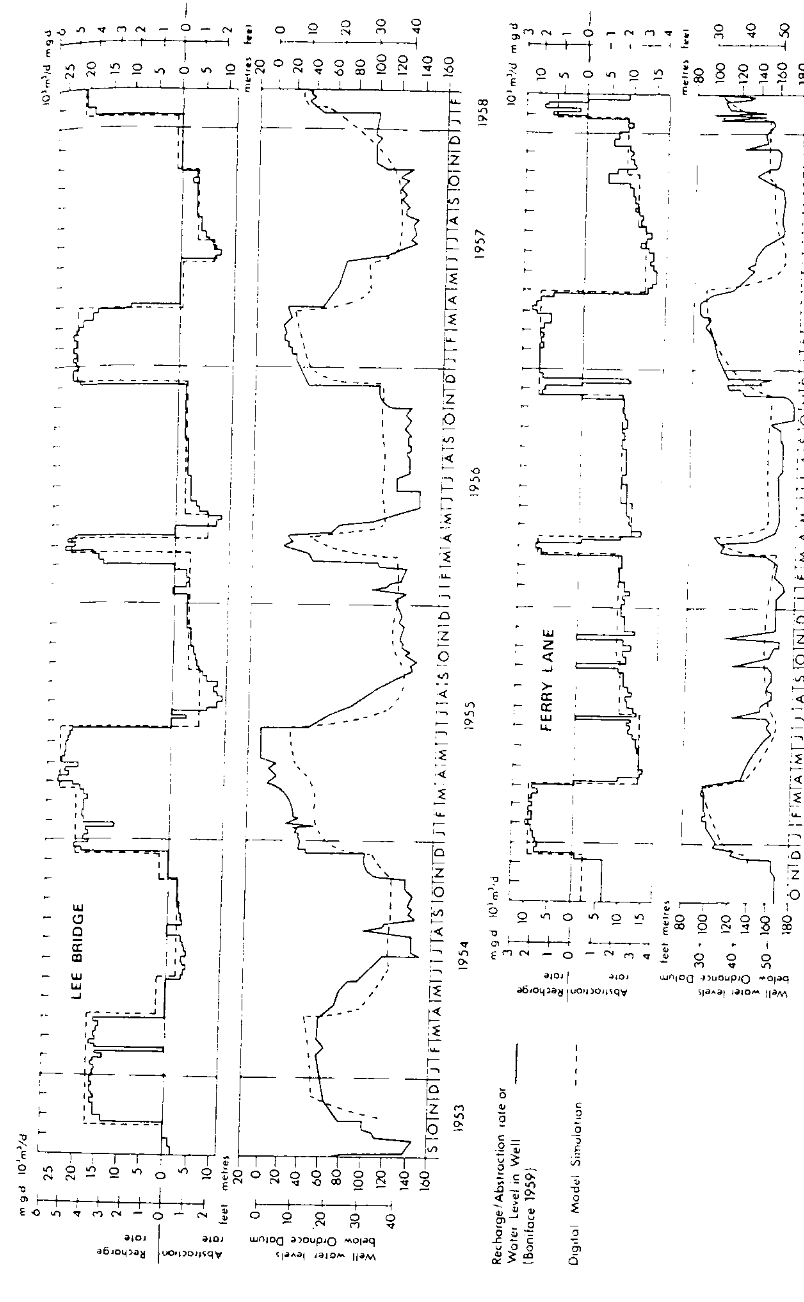

FIG. 8--Comparison of water level changes in Ferry Lane and Lee Bridge wells during the 1954-57 recharge of Lee Valley (Boniface, 1959) with a simulated recharge scheme using regional groundwater flow model.

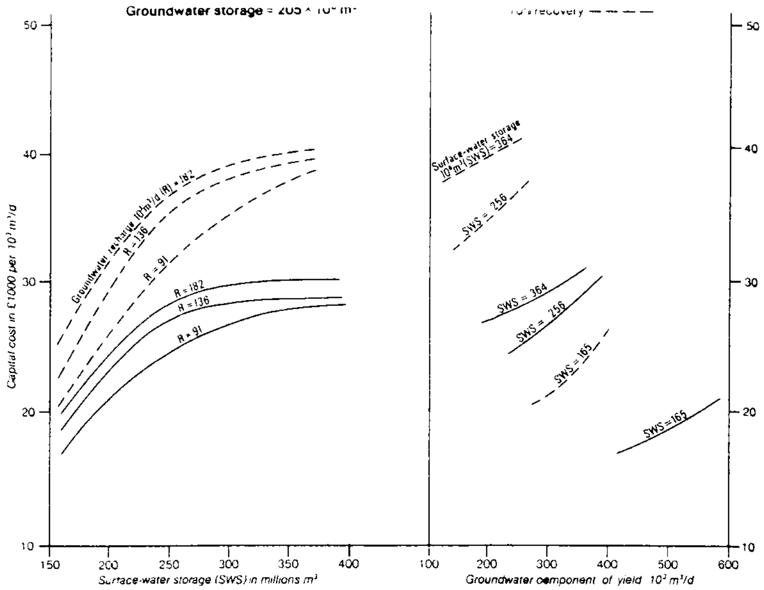

FIG. 9--Capital costs per unit of groundwater yield for Lee Valley.

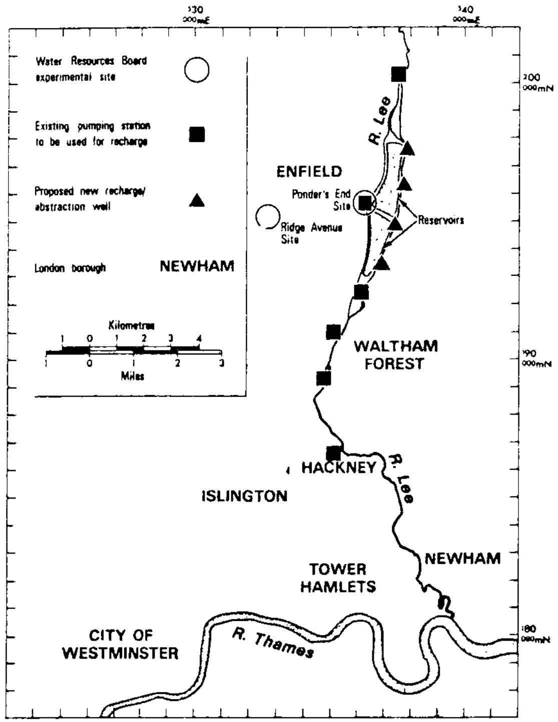

FIG. 10--Experimental recharge sites and location of wells for proposed type schemes in Lee Valley.

APPENDIX I

Cost Rates Used in Estimating For Artificial-Recharge Development in London Basin

Description	Unit	Rate £
River Intakes	10^3 m^3/d	220
Pump Houses		
a. up to 45 x 10^3 m^3/d	10^3 m^3/d	1,320
b. over 45 x 10^3 m^3/d	10^3 m^3/d	880
Pumps and associated equipment including 50% standby capacity and standby diesel generators		
a. up to 180 x 10^3 m^3/d	output kw	200
b. over 180 x 10^3 m^3/d	output kw	175
Pipelines		
a. 150 mm	km	6,000
b. 450 mm	km	24,000
c. 900 mm	km	60,000
d. 1,800 mm	km	161,000
Treatment Water		
a. up to 65 x 10^3 m^3/d	10^3 m^3/d	16,500
b. over 65 x 10^3 m^3/d	10^3 m^3/d	14,300
Service Reservoirs (24 hr storage)		
a. up to 18 x 10^3 m^3/d	10^3 m^3/d	12,000
b. over 18 x 10^3 m^3/d	10^3 m^3/d	10,000
Extraction/Recharge and observation wells 5 x 10^3 m^3/d capacity	--	30,000

UNDERGROUND WASTE DISPOSAL AND ARTIFICIAL RECHARGE IN JAPAN[1]

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ABSTRACT Artificial recharge, which is needed to control problems of land subsidence and saltwater intrusion, is in the experimental stage in Japan. Private companies at three sites--in Kanazawa, Tsuruga, and Yamagata--have had success with artificial-recharge wells. These sites are all on similar alluvial-fan deposits, but have quite different injection rates. This difference is attributed to differences in well structure and completion procedure, in particular, perforation.

Underground waste disposal is practiced with many complex and often dangerous fluid wastes in Japan, but the operations are secret. Two successful injection operations involve disposal of acid waste from a sulfur mine and thermal water from a geothermal electric generating plant.

INTRODUCTION

In Japan, artificial recharge is still in the experimental stage and has had few successful results. Land subsidence is still progressing all over Japan regardless of the regulation or prohibition of groundwater development. Saltwater intrusion is occurring along the sea coast. An artificial recharge program is considered to be necessary as the countermeasure for these problems (Fig. 1).

Ichiro Ozo conducted artificial-recharge work under the author's guidance in Kanazawa City. This experiment has had successful results, which are reported here. Summaries concerning special types of underground waste disposal in Japan, such as acidic-water injection, are also presented.

[1]Manuscript received, June 4, 1973.

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ARTIFICIAL RECHARGE IN KANAZAWA

Location

The artificial-recharge well is located in the yard of Marueki textile factory in Kanazawa City (Fig. 2). Kanazawa is the biggest city along the Japan Sea coast, and most of the urban area is situated on the Sai River alluvial fan. This area has thick snow in the winter. The drawdown of the water table in this district is remarkable and it is increasing, owing to the heavy development of groundwater and snow melt for industry. The city authority established two observation wells in the city area, shown in Figure 2. I. Ozo, president of Hokkoku Sakusen (drilling) Co., installed an injection well at his own expense, with the cooperation of the Marueki Co.

Facilities

The injection well, 80 m deep and 300 mm in diameter, was drilled on July 21, 1971. Slotted screens (each slot is 6 mm wide and 160 mm long) are set at the depths of 23-32, 38-44, 50-65, and 74-80 m. The geologic section mainly consists of gravel layers, intercalated with thin clay layers.

Used air-conditioning water was transferred to the recharge well by pipe. The original water temperature was 18°C and the temperature after filtration was 16°C. The result of chemical analysis is shown in Table 1. The injection water has a pH value of 7.2 and an rpH of 7.4. There are no significant differences between the injection water and the original water except in iron content. The problem of why the iron content increases after filtration has not been solved.

The general aspect of the operation facilities is shown in Figure 3. Air-conditioning water is polluted by fine dust in the air, necessitating the type of filtration equipment shown.

Preliminary Test

An aquifer test, which was made after drilling of the well, yielded a hydrologic constant of $K = 5.0 \times 10^{-2}$ cm/sec and $S = 0.1$. Step drawdown testing was carried out on July 28, 1971. The result of this test is summarized in Table 2. As there were many wells near the testing site, some interruptions in the injection tests occurred.

Several injection tests were made at different injection rates at the end of August, 1971. In these tests, disturbances by other operating wells were conspicuous.

Injection rates in this test varied from 0.5 m^3/minute (720 m^3/day) to 0.98 m^3/minute (1,410 m^3/day). A constant rate of buildup by injection was observed after 4-8 hours.

Operation

Waste water was transferred from the factory to the recharge well by a pipe extending into the water in the well through the filtration facilities.

Water quantities were observed daily through a venturi tube and the water level was measured by an automatic water-stage recorder. The water temperature was kept at an average of 16°C throughout the year. The water was injected at a rate ranging from 200 to 1,200 m^3/day and averaging about 800 m^3/day. The injection schedule is shown in Table 3. Average injection rate, average water level, water volume, and cumulative water volume are shown in Table 4 for every ten days throughout the injection periods. The injection rate was not constant, as it depended upon the available water supply. In an extreme case, air-conditioning water was unavailable from December 1, 1972, to January 7, 1973.

Although injection should be continued at a constant rate, it was necessary to clean filtration facilities about every 20 days. The resultant interruptions in injection are conspicuous in the hydrograph (Fig. 4).

The specific injection capacity of the recharge well was 340-545 m^3/day per 1 m buildup at the beginning of this operation, and 180-230 m^3/day at the end. After 9 months of operation, swabbing was needed to redevelop the recharge well. By this swabbing, the injection rate recovered to that of the initial stage. Consequently, about 100,000 m^3 of water was injected underground in 9 months. During this time surging did not occur and there was no need for swabbing.

Discussion

An average natural injection rate of 800 m^3/day over 9 months was considered a successful result in Japan. An injection rate for discharge was about 50 percent. Two step injection tests were run on September 1, 1971, and June 2, 1972. Between these two tests, specific injection capacity decreased by 50 percent (Table 5). After swabbing, specific pumping capacity and specific injection capacity recovered to the initial values. The decrease in injection capacity is considered to result from

the clogging of the screen and/or plugging of aquifers. We could not clarify its causes.

To compare this result to others, we will discuss some other operations. Wells of Toyobo Company have depths ranging from 50 m to 100 m, and are 400 mm in diameter. They are situated at the center of the Tsuruga alluvial fan, and have transmissibility capacity of 5,780 m^2/day ($K = 2.5 \times 10^{-1}$ cm/sec). The results of aquifer tests showed that the specific capacity of these wells was 1,130-1,500 m^3/day/m. The water, at a temperature of 38°C, was injected into the well at the rate of 2,400 m^3/day, and the resulting buildup in the well was 1.5 m. As specific injection capacity was 1,200 m^3/day/m, the injection rate was 100 percent. Five wells were operated continuously at the total rate of 7,000 m^3/day.

The Nippon Chikasui Kaihatsu Company well, 84 m deep and 300 mm in diameter, was constructed on the margin of the Yamagata alluvial fan for an artificial-recharge experiment. On this site, transmissibility was determined to be 310 m^2/day ($K = 1 \times 10^{-2}$ cm/sec). As specific capacity was 430-1,000 m^3/day/m and injection capacity was 80-60 m^3/day/m, the injection rate was 7-18 percent.

These three recharging sites, Kanazawa, Tsuruga, and Yamagata, are located in similar topographic and geologic settings--on alluvial fans with high permeability. In spite of their similar favorable conditions, they have quite different injection rates. The difference must result from differences in well structure and completion procedure, in particular, the perforations.

UNDERGROUND WASTE DISPOSAL IN JAPAN

Underground disposal is practiced with many types of fluid waste in Japan. The kinds of waste, both domestic and industrial, are very numerous and complex and are sometimes highly toxic or dangerous. We cannot describe the actual situation, as disposal operations are secret. However, we will discuss two types of waste injection which have been practiced successfully in Japan.

The first type of waste injection involves underground disposal of acidic drainage water from a sulfur mine. Nishiazuma sulfur mine is situated on the eastern part of Yamagata Prefecture. Spring water from this region, with a pH of 3.3 and 35 ppm of SO_4, and tunnel water from the sulfur mine, with a pH of 1.9 and 650 ppm of SO_4, were collected in a canal leading to an earth dam. From this dam, acidic water was diverted into a hole drilled in the bottom of a shallow square shaft.

About 150 holes, 35-60 m deep and 120 mm in diameter, were drilled into andesite underlain by sandstone. Through these holes, 0.15 m^3/sec (12,960 m^3/day) of acidic water was injected into the ground. The operation was conducted successfully for several years. Similar operations were conducted at Matsuo sulfur mine and Zao hot spring spa.

The second type of water injection involves thermal water from a geothermal electric generating plant. The purpose of disposal is to prevent the decrease of vapor pressure and to prevent thermal pollution. This operation is still in the experimental stage.

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Table 1. Chemical Analysis of Water, April 1973

	Injection water		Original water	
	ppm	epm	ppm	epm
Na	17.7	0.770	15.9	0.691
K	1.69	0.0432	1.24	0.0317
Ca	15.0	0.749	14.9	0.744
Mg	6.71	0.552	8.70	0.715
		2.11		2.18
Cl	21.5	0.607	21.4	0.604
SO_4	13.6	0.283	16.5	0.344
4.3Bx		0.891		0.912
		1.78		1.86
SiO_2	22.5		23.0	
Total Fe	0.38		0.13	

Table 2. Results of Step Drawdown Test

Volume of Discharge m^3/day	Initial Water Level m	Present Water Level m	s m	Specific Discharge m^3/day/m
212	16.325	16.770	0.445	476
737		17.375	1.05	701
1054		18.025	1.70	620
1441		18.725	2.40	600

Table 3. Injection Schedule, 1972-1973

Operation	Period	Injection rate m^3/day	Specific injection capacity m^3/day/m
Inj.	Sept. 15-Oct. 9	750-1,200	340-545
Stop	Oct. 10-Oct. 16	--	--
Inj.	Oct. 17-Nov. 6	750-1,000	280-380
Stop	Nov. 7-Jan. 8	(no water)	--
Inj.	Jan. 9-Feb. 8		
Stop	Feb. 9-Feb. 13		--
Inj.	Feb. 14-Feb. 22		
Stop	Feb. 23-Mar. 3		--
Inj.	Mar. 4-Mar. 28	200-1,000	
Stop	Mar. 29-Apr. 1	--	--
Inj.	Apr. 4-Apr. 28	500	190
Stop	Apr. 29-May 2	--	--
Inj.	May 13-June 1	750-900	180-230
Stop	June 10-June 30	(swabbing)	
Inj.	July 20-	800-1,000	

Table 4. Injection Volume and Water Level

Period 1972-1973		Injection rate m^3/day	Water level m	Total volume m	Cumulative total m^3
Sept.	11 - 20	974	14.55	3,895	3,895
	21 - 30	913	15.20	9,125	13,020
Oct.	1 - 10	830	15.34	5,810	18,830
	11 - 21	783	15.81	2,350	21,180
	21 - 31	1,388	16.08	8,330	29,510
Nov.	1 - 10	1,260	16.21	3,780	33,290
	11 - 20	0	16.79	0	33,290
	21 - 30	0	17.55	0	33,290
Dec.	1 - 10	0	17.19	0	33,290
	11 - 20	0	15.87	0	33,290
	21 - 31	0	15.80	0	33,290
Jan.	1 - 10	1,170	15.54	3,510	36,800
	11 - 20	900	16.80	3,600	40,400
	21 - 31	556	16.15	5,000	45,400
Feb.	1 - 10	855	16.19	4,275	49,675
	11 - 20	788	16.12	4,725	54,400
	21 - 28	525	17.55	1,050	55,450
Mar.	1 - 10	683		4,100	59,550
	11 - 20	622		3,735	63,285
	21 - 31	875		5,250	68,535
Apr.	1 - 10	407	16.47	1,220	69,755
	11 - 20	586	15.53	5,275	75,030
	21 - 30	265		795	75,825
May	1 - 10	1,013	16.47	6,075	81,900
	11 - 20	838		6,700	88,600
	21 - 31	854		5,125	93,725
June	1 - 10	639		2,555	96,275
	11 -				

Table 5. Injection Tests Run September 12 and January 2

	Injection Rate m^3/day	Buildup m	Specific Injection Capacity m^3/day/m
Sept. 12	538	0.80	666
	1008	1.67	604
	1325	2.42	548
	1656	4.40	376
Jan. 2	403	1.35	299
	662	2.90	228
	871	3.07	234
	1195	5.55	215

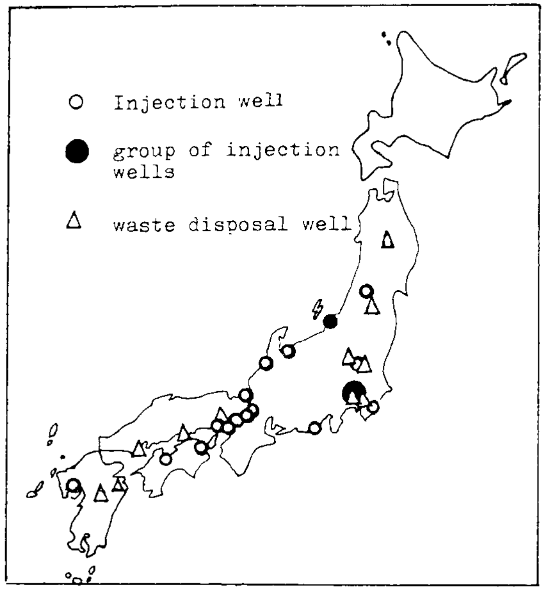

FIG. 1--Distribution of injection wells in Japan.

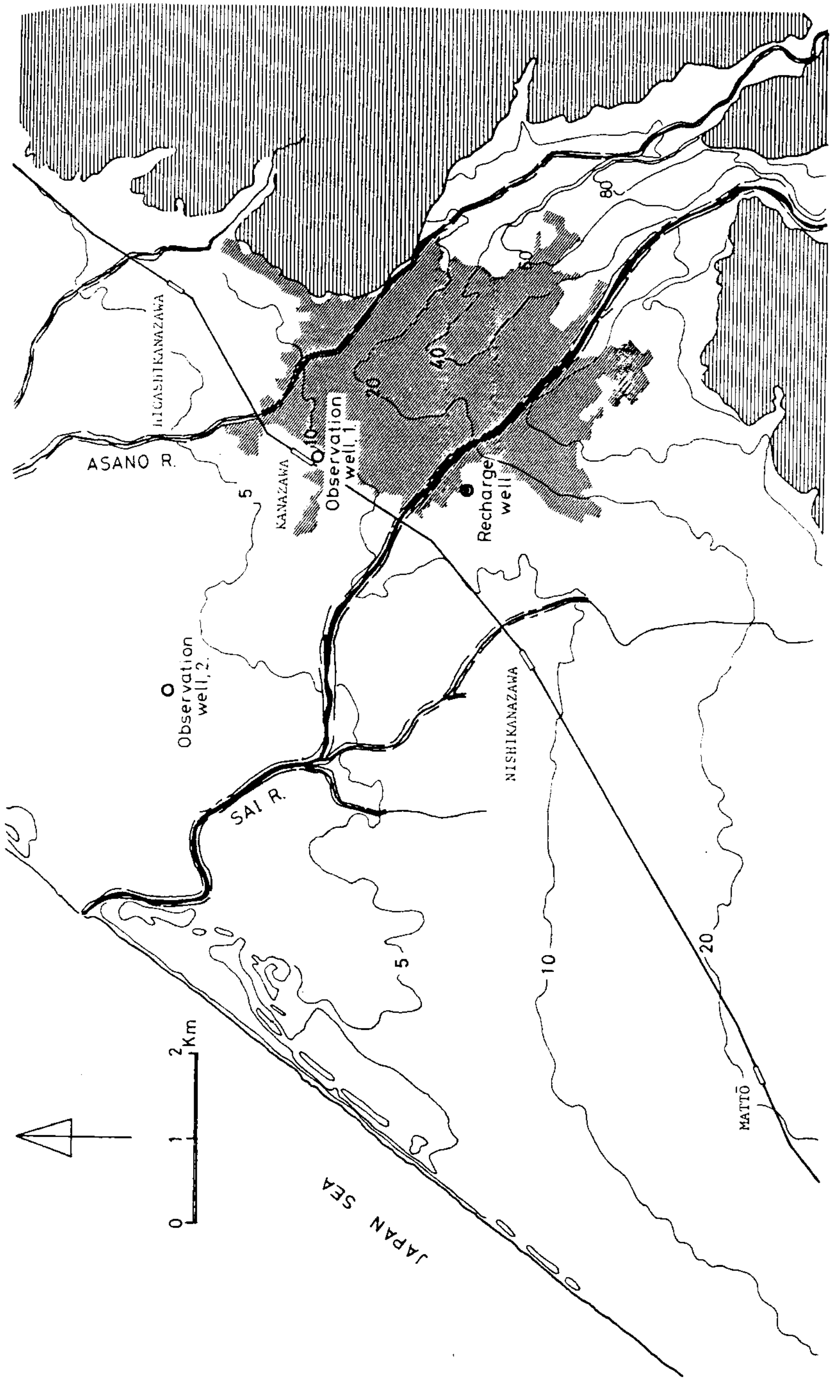

FIG. 2--Location map.

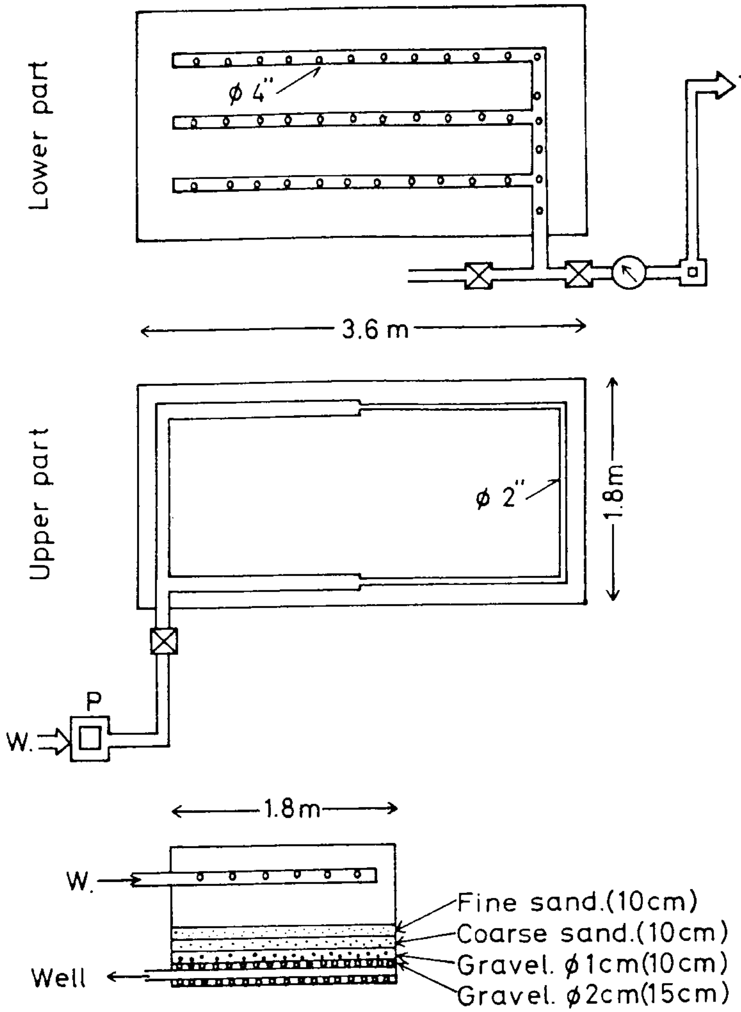

FIG. 3--Filtration facility.

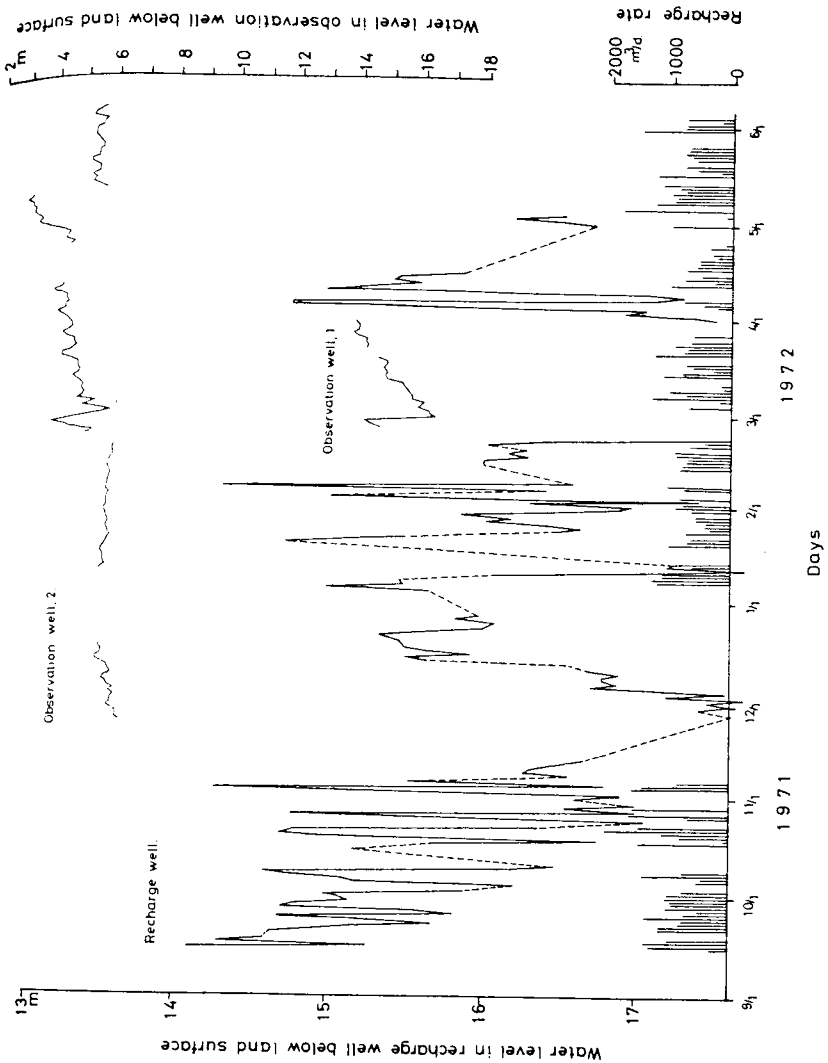

FIG. 4--Hydrograph.

SUBSURFACE DISPOSAL OF LIQUID INDUSTRIAL WASTES IN ALABAMA--A CURRENT STATUS REPORT[1]

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ABSTRACT Five subsurface disposal wells have been drilled and completed in Alabama. These are: Stauffer Chemical Co., Mobile County; Ciba-Geigy, Inc.--two wells, Washington County; U.S. Steel Corp., Jefferson County; and Reichhold Chemicals, Inc., Tuscaloosa County. The Geological Survey of Alabama has been directly involved in all four projects. The Survey serves as a consultant to the Alabama Water Improvement Commission, the state agency responsible for protection of surface and groundwater in Alabama, to the Stauffer and Ciba-Geigy projects, and as consultant and supervisor on the U.S. Steel Corporation and Reichhold Chemicals, Inc., projects. The Environmental Protection Agency provided some funding on the research aspects of the Reichhold Chemicals, Inc., disposal well. These projects were undertaken as a research effort to insure that the state agencies responsible are fully cognizant of all aspects of this method of waste disposal.

At present in Alabama, subsurface disposal is permissible for some types of wastes if the well is properly designed and completed in an appropriate geologic environment, if conventional methods of waste treatment have been evaluated and proved to be inadequate, and if an adequate monitoring system has been installed.

The Stauffer and Ciba-Geigy wells are in the Coastal Plain geologic province and the U. S. Steel and Reichhold Chemicals, Inc., wells are in Paleozoic sediments of the Black Warrior basin. The geology,

[1]Manuscript received, May 4, 1973.

[2]Geological Survey of Alabama.

[3]State Oil and Gas Board.

drilling, completion, and testing techniques are presented as a basis for decision making for approval or rejection of the proposed deep-well disposal projects by a regulatory agency.

INTRODUCTION

The transition of Alabama in the last ten years from a raw-material-producing state to a manufacturing state has placed an increasing burden on its water resources. In 1965, the State Legislature amended the Water Improvement Commission Act, thereby creating an agency to protect the surface-water and groundwater supplies of the state and removing the "grandfather" industries from exemption. The Geological Survey of Alabama is authorized by the Legislature to conduct studies on the occurrence and availability of surface water and groundwater. This responsibility has been expanded to consider water usage and the environmental impact of industrial and population evolution. The Survey serves as a consultant to the Water Improvement Commission in the fields of geology, hydrology, and geologic engineering.

As subsurface disposal of industrial waste became popular in the middle '60s, the Survey undertook research projects to determine all aspects of this method of waste disposal. The Survey has served as regulatory supervisor on three wells and directed the total program on two wells. In January of 1971, a general policy was adopted by the Alabama Water Improvement Commission regarding subsurface disposal in Alabama. The general policy approved this method of disposal, provided that all other methods are examined and that such geologic and hydrologic conditions exist that the environment is protected to the maximum extent possible.

Application for a subsurface disposal system, in the form of a feasibility study, is made to the Alabama Water Improvement Commission. The Geological Survey reviews the material presented and makes its own recommendation to the Commission. If the plan is approved, the applicant is allowed to drill and test the well. The applicant then applies for permission to utilize the well. Information gathered during drilling and testing is reviewed by the Survey, and further recommendations regarding use, requirements, and monitoring procedures are prescribed.

General Geology

For the purpose of this report the state is divided into three major geologic provinces. These are: (1) the Piedmont area in east-central Alabama; (2) the Paleozoic rocks in north Alabama; and (3) the Coastal Plain in south and southwest Alabama (Fig. 1).

The Stauffer and Ciba-Geigy wells are in the central Coastal Plain geologic province (Fig. 1). Most of the geologic structures found in Early Cretaceous or younger sediments in this basin are the result of movement of the underlying Louann Salt. Salt at depth responds as a plastic medium and will move into zones of weakness in response to sediment loading. Structures formed as positive features by salt swells or domes and as collapse-type features, such as grabens where salt was removed, are present in southwest Alabama (Fig. 2). A cross section through the coastal plain area exhibits sedimentary rock from Jurassic to late Tertiary age with disposal zones available by virtue of intergranular porosity in sandstones and secondary porosity in limestones and dolomites (Fig. 3).

The Coastal Plain of Alabama is the most important groundwater-producing area in Alabama. The consistent occurrence of water-bearing sand and gravel beds makes the completion of large-capacity wells quite common. Most outcropping sands in this area are considered to be good to excellent aquifers. Water well depths range from a few feet to 2,000 ft, and yields may be more than 1,000 gallons per minute (gpm).

The Citronelle, South Carlton and Tensaw Lake oil fields are located in the area of the Ciba-Geigy wells and the Stauffer well. Salt, limestone, and lignite are other mineral resources that are being developed or have potential as mineral deposits in the Coastal Plain.

The U.S. Steel Corporation and Reichhold Chemicals, Inc., wells are in the Paleozoic formations (Fig. 1). Geologically, the Reichhold plant site is located in the Black Warrior sedimentary basin. The Black Warrior basin is wedge-shaped and extends across Alabama and Mississippi and contains considerable thicknesses of siliciclastic and carbonate sediments (Fig. 2). The geologic structure in most of the Black Warrior basin is fairly simple; the formations generally dip to the south or southwest. A cross section through the Black Warrior basin exhibits sedimentary formations ranging from Cambrian to Pennsylvanian in age (Fig. 4). Disposal zones are chiefly in the Silurian, Ordovician, and Cambrian-Ordovician, where secondary porosity is developed in the form of fractures and cavernous vugs.

Water-well yields in the Paleozoic rocks vary considerably and, although topographic considerations are important in locating a successful water well, the controlling factor for a high-yield well is the

penetration of solution channels and cavities within limestone reservoirs. Yields range from a few to several thousand gallons per minute.

Mineral resources in the area of the U.S. Steel Corporation and Reichhold Chemicals, Inc., wells are primarily iron ore, coal, and limestone.

DISPOSAL WELLS IN ALABAMA

The Stauffer Chemical Company disposal well--This was the first industrial subsurface disposal system drilled and completed in Alabama. An initial feasibility study was completed in June 1968 and the well was drilled, completed, and tested by February 1969. Surface facilities were constructed from April to August 1969 and injection was begun in August 1969. The well is the only industrial disposal system in operation in Alabama, and has been in operation since August 1969 without interruption, except for a remedial acid treatment.

The well was drilled to a total depth of 4,300 ft and completed in a sand of the Wilcox Group from 3,400 to 3,480 ft (Fig. 5). This sand has a porosity of 34 percent and a permeability of 400 md. The well is equipped with 10 3/4-in. surface pipe set to a depth of 1,238 ft, centralized and cemented to the surface. The 7-in. long string was set to a depth of 4,216 ft, centralized and cemented to 1,250 ft. The long string was perforated from 3,400 to 3,480 ft. The 4 1/2-in. injection tubing was set to a depth of 3,400 ft, and a packer was set at the bottom of the tubing between the tubing and the 7-in. long string. The annular space is filled with a noncorrosive fluid and the pressure is monitored.

The waste is a result of chemical processing, and contains chloride and phosphorus. Its specific gravity ranges from 1.04 to 1.11.

Surface equipment consists of holding tanks, transfer pumps, filters, clean tanks, injection pumps, transfer and injection piping, and instrumentation for monitoring.

The Ciba-Geigy Corporation--An initial feasibility study for two subsurface disposal wells was completed in April 1969. Well No. 1 was drilled, completed, and tested by December 1970, and well No. 2 was drilled, completed, and tested by June 1971.

The No. 1 well was drilled to a total depth of 7,513 ft and completed in a sand of the Wilcox Group from 3,827 to 3,947 ft through a screen and gravel pack (Fig. 6). This sand has a porosity of 28 percent and a permeability of 300-500 md. The well is equipped with 16-in. surface pipe set to a depth of 1,063 ft, centralized and cemented to the surface. The 10 3/4-in. long string was set to a depth of 3,827 ft, centralized and cemented to the surface. The 4 1/2-in. injection tubing was set to a depth of 3,765 ft with a 180-ft screen liner attached below. The annular space is filled with diesel oil and the pressure is monitored.

A drill-stem test was run over the interval from 3,805 to 3,950 ft and 34 bbl of clean formation fluid were recovered. Nine sidewall cores were recovered from the interval 3,885 to 3,951 ft.

The well has a tested injection rate of 420 gpm at a surface pressure of 400 pounds per square inch (psi).

The No. 2 well was drilled to a total depth of 2,510 ft and completed in the top sand of the Wilcox Group from 2,000 to 2,296 ft through a screen and gravel pack (Fig. 6). This sand has a porosity of 32 percent and a permeability of 300-500 md. The well is equipped with 13 3/8-in. surface pipe set to a depth of 1,017 ft, centralized and cemented to the surface. The 9 5/8-in. long string was set to a depth of 2,000 ft, centralized and cemented to the surface. The 7-in. injection tubing was set to a depth of 1,885 ft with a 411-ft screen liner attached below. The annular space is filled with diesel oil and the pressure is monitored.

The well has a tested injection rate of 800 gpm at a surface pressure of 300 psi.

Two principal effluents will be injected into the wells; one is composed primarily of a by-product of dilute hydrochloric acid; the other, of organic and inorganic waste including diazinon, chlorobenzilate, and triazines.

Surface equipment consists of a settling pond, transfer pumps, filters, injection pumps, transfer and injection piping, and instrumentation for monitoring.

The U.S. Steel Corporation disposal well--This well was completed and tested by March 1970. The initial feasibility study was completed in October 1964. All operations were under the direct supervision of the Geological Survey of Alabama as a research project in the field of deep-well disposal of aqueous wastes. The project was financed by a research grant from U.S. Steel Corporation to the Alabama Geological Survey. All steps taken in the drilling and completion of the well were designed for the protection of surface and subsurface water resources and other natural resources of the area.

The well was drilled to a total depth of 6,060 ft and was completed from open hole between 4,415 and 6,060 ft (Fig. 7). The primary injection zone, 4,440-4,688 ft, is a fine-grained calcareous sandstone with fractu

porosity and permeability. The zone of secondary importance, 5,620-5,770 ft, is a dolomite whose porosity and permeability are provided by fractures and/or cavernous vugs. The upper zone has the capacity to accept a greater volume-rate at a lower pressure than the lower zone, and the well is so constructed that the lower zone would begin to take fluid if the upper zone failed to function properly.

Three cores were cut during drilling of the well; however, none of the zones cored had enough permeability to be suitable as a disposal zone. Four drill-stem tests were conducted, but formation fluid was not recovered; permeability was extremely low at the zones tested.

The injection zones were located by radioactive injection tests which were run over the interval from 2,870 to 6,060 ft.

The well is equipped with 10 3/4-in. surface pipe set to a depth of 1,112 ft, centralized and cemented to the surface. The 7 5/8-in. long string was set to a depth of 4,415 ft, centralized and cemented to 2,750 ft. The 2 7/8-in. injection tubing was set to a depth of 4,434 ft for injection tests, and a packer was set at the bottom of the tubing between the tubing and the 7 5/8-in. long string. The annular space is filled with a non-corrosive fluid and the pressure is monitored.

The well has a tested injection rate of 300 gpm at a surface pressure of 600 psi.

The pickle-liquor waste contains hydrochloric acid (HCl) or sulfuric acid (H_2SO_4).

Due to the high percentage of carbonate material in the sandstone and almost total carbonate composition of the dolomite, consideration must be given the reaction between the waste $FeCl_2$ + HCl or $FeSO_4$ + H_2SO_4 and the formation constituents. Injection of waste HCl effluent into a carbonate formation will result in a dissolution of the soluble parts of the rock and will cause an increase in permeability, thereby increasing the injection rate and/or reducing the pressure required. The reaction products will be water, carbon dioxide, and calcium chloride--the latter two being soluble in water. Adversely, the pH increase resulting from these reactions may cause the plugging of injection-zone pore space by formation of $Fe(OH)_3$ precipitate.

$$Fe^{+++} + 3(OH)^- = Fe(OH)_3 \text{ (gel precipitate)}$$

$$Fe^{++} + 2(OH)^- = Fe(OH)_2 \text{ (precipitate)}$$

In a sulfuric acid reaction with a carbonate formation, the calcium released will be reprecipitated as $CaSO_4$ (anhydrite) or $CaSO_4 \cdot 2H_2O$

(gypsum). The high iron content of the waste effluent (1.2 moles/l) coupled with the pH increase from mineral reactions will cause precipi tation of $Fe(OH)_3$ gel.

There is a considerable controversy among experts as to whether t precipitation resulting from acid-waste injection into a carbonate res voir will plug the pore space and reduce permeability. Fortunately, ex perience has shown that low strength acid waste may be injected into carbonate strata. Hammermill Paper Company at Erie, Pennsylvania, and Dupont at New Johnsonville, Tennessee, have successfully injected acid waste into carbonate strata.

From all indications, the porosity of the injection zone in the su ject well is fractured rather than intergranular. Such porosity should prove more favorable to acid injection because less surface area is available for reaction, and the "zone of reaction" should extend to a greater distance from the well bore. Considering that two extraneous precipitates are caused by the reaction of H_2SO_4 with a carbonate rock (compared with the HCl reaction), it must be stated that the HCl reacti would be more compatible with the salaquifer water and rock than the H_2SO_4 pickle-liquor reaction. However, if the H_2SO_4 pickle liquor were reduced in acid strength by dilution of 1 part acid to 1 part water or 1 part acid to 2 parts water, the well and receptor formation should function equally as well as with the HCl pickle liquor.

<u>The Reichhold Chemicals, Inc., waste-disposal well</u>--This well was completed and tested by August 1970. The initial feasibility study was completed in September 1968. All aspects of the project were under the direct supervision of the Geological Survey of Alabama as a research project in the field of deep-well disposal of aqueous wastes. The proje was financed by a research grant from the Southeast Regional Office of the Environmental Protection Agency in Atlanta, Georgia, and Reichhold Chemicals, Inc. The objectives of this phase of the project were to design and complete a well that would assure protection of surface and subsurface water resources and other natural resources of the area, and to locate a zone in the deep subsurface formations that would be receptive to the plant's waste effluent.

The well was drilled to a total depth of 8,098 ft and was complet from open hole between 5,902 and 8,098 ft (Fig. 8). Permeability exis in two main horizons in the well. The upper zone is from 6,620 to 6,6 ft and the lower zone is from 7,036 to 7,230 ft. Porosity and permeab exist in the upper zone by virtue of fractures and in the lower

zone by virtue of fractures and cavernous vugs. During drilling of the well a core was cut from 8,075 to 8,098 ft, but did not indicate the presence of enough porosity or permeability to provide a suitable disposal zone.

One drill-stem test, made at the interval from 6,986 to 7,466 ft, located the bottom injection zone. The upper injection zone was located by radioactive-injection tests which were run over the interval from 5,806 to 7,900 ft.

The well is equipped with 10 3/4-in. surface pipe set to a depth of 1,220 ft, centralized and cemented to the surface. The 7-in. long string was set to a depth of 5,902 ft, centralized and cemented to 4,205 ft. The 2 7/8-in. injection tubing was set to a depth of 5,902 ft for injection tests, and a packer was set at the bottom of the tubing between the tubing and the 7-in. long string. The annular space is filled with a non-corrosive fluid and the pressure is monitored.

The well has a tested injection rate of 600 gpm at a surface pressure of 700 psi.

As a result of chemical processing, wastes containing sodium sulfite and sulfate, phenol, caustic, and traces of organic compounds are produced.

Pre-injection treatment of waste will consist of a granular activated carbon system. Flow equalization, neutralization, and chemical clarification will be pretreatment steps ahead of the granular carbon system. The company's decision to use granular carbon was reached only after thorough feasibility studies of alternative methods such as biological treatment and chemical oxidation. Granular carbon treatment proved to be most economic and most effective in meeting effluent requirements, and the process is capable of removing over 90 percent of the organic compounds present in the waste stream.

SPECIAL TECHNIQUES

During the drilling of a disposal well, a geologist constantly examines cuttings and maintains a sample log. This information is used to determine zones of importance in which open-hole drill-stem tests should be made. The drill-stem tests provide information concerning formation pressures and producing rates, and a sample of the reservoir fluid. Electric-log techniques are used extensively to evaluate the formations penetrated and to determine potential injection horizons. The data collected while drilling the wells are utilized to determine the casing and cement program. Preliminary injection tests are conducted to establish rates and pressures that will be utilized in the design of surface equipment. A composite of this information, made for a graphical presentation, exhibits the results of drilling and testing a well.'

The most advanced logging techniques available were utilized in Alabama. These included the induction log, sonic log, density log, neutron porosity log, radioactive-tracer log, cement-bond log, and temperature log. Each of these logs is also recorded on magnetic tape and composited by computer in the form of a synergetic log, which is a continuous record of all formations penetrated, showing porosity, permeability, water saturation, and lithology. The cement-bond log is useful in determining the amount of cement fill-up and the degree of bonding between the casing and the cement, and the cement and the formation. Temperature logs and radioactive-tracer logs are run to determine the zone of entry of the injected fluid.

Monitoring Techniques

The extent of the monitoring program is determined by the geologic and hydrologic conditions for each well and by the type of waste. For example, areas of faulting would require a very stringent monitoring program, as would the disposal of toxic wastes.

Well-head pressure in the injection tubing--and that in the annulus between the injection tubing and the long string and in the annulus between the long string and the surface pipe--is monitored by continuous clock recorders (Fig. 9). Injection rates and cumulative volumes are also recorded on continuous recorders. The continuous pressure recorders are equipped with an alarm and automatic shut-down system in case of casing or tubing rupture or other malfunctions.

Composite samples of the wastes are taken and analyzed periodically. Temperature and density are recorded on a continuous basis and corrosion tags are analyzed periodically. Sample stations are set up on surface streams and water-supply wells are also sampled.

Seismic monitoring is required in some areas of Alabama to determine any possible effects that injection may have on tectonic activity. The system consists of seismometers placed around the well and connected by a telephone service to a seismic recorder at the Survey office. Continuous films (16 mm) are made of seismic activity in the area. These are transmitted to the U.S. Geological Survey National Center for Earthquake Research at Menlo Park, California, for analysis.

The use of monitor wells is dependent on the toxicity of the waste and the geologic and hydrologic conditions. The U.S. Steel well, for

example, will monitor the injection zone with a deep monitor well because the Birmingham area is faulted.

Extensive monitoring requirements may be required initially in an effort to gain knowledge and to give assurance that the environment is fully protected. As more knowledge is gained through these monitoring programs, a reduction in monitoring may be feasible.

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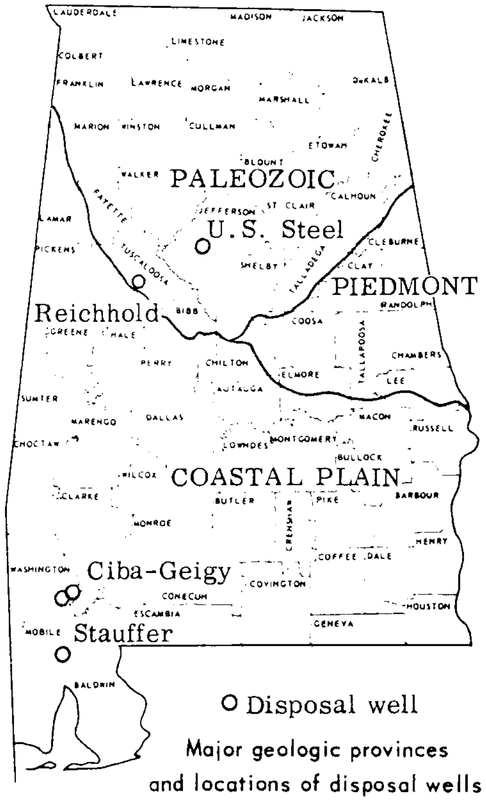

FIG. 1--Index map.

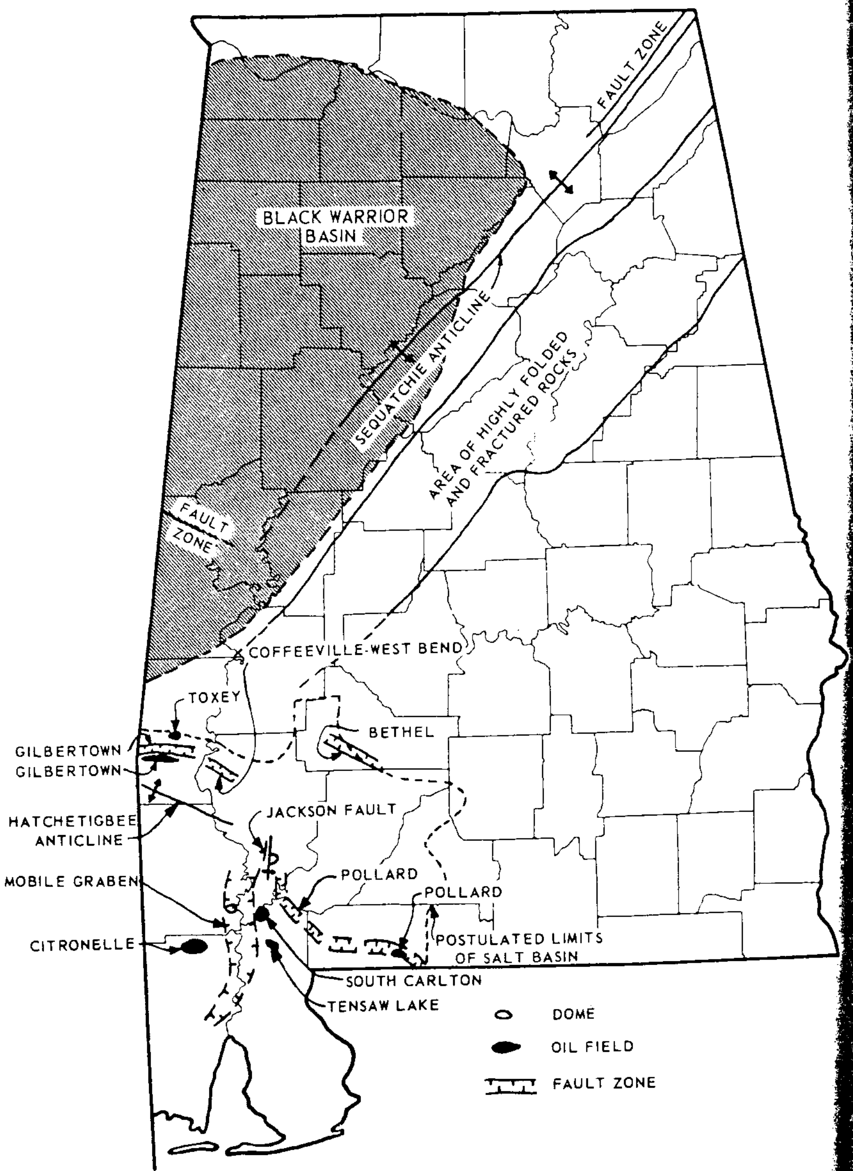

FIG. 2--Major structural features in Alabama.

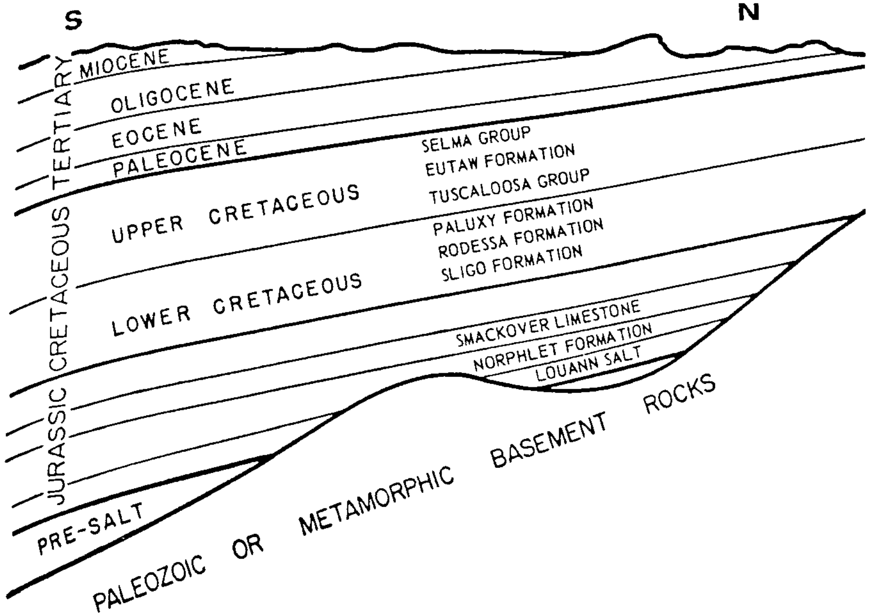

FIG. 3--Generalized cross section of south Alabama.

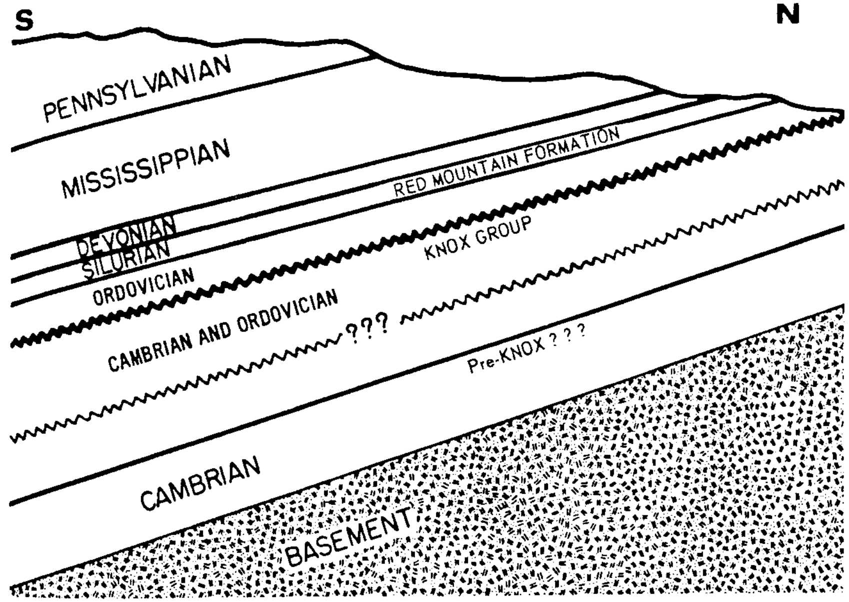

FIG. 4--Generalized cross section of north Alabama.

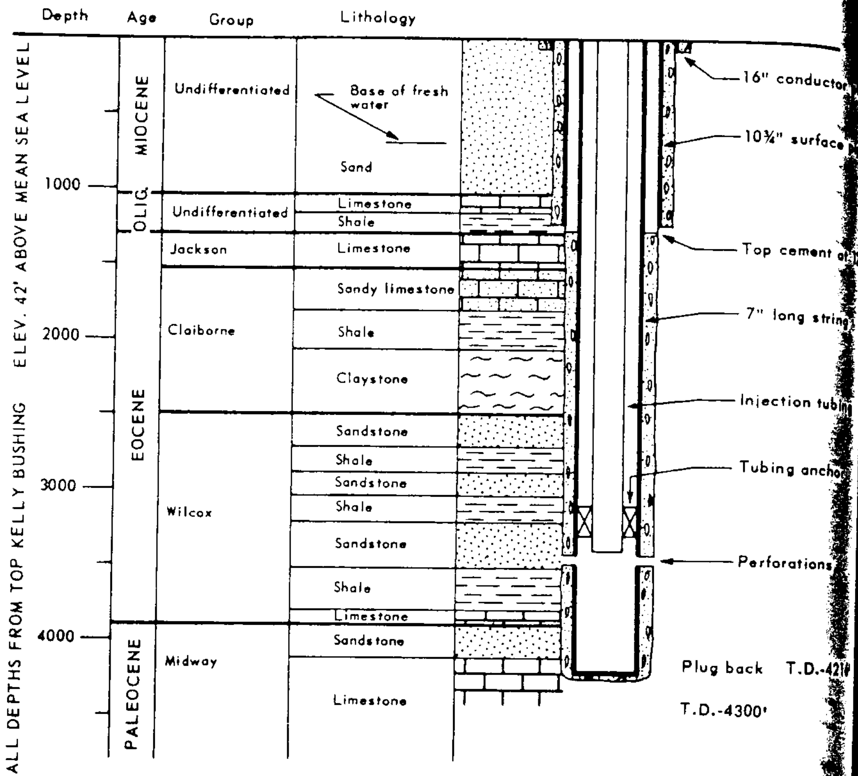

FIG. 5--Schematic cross section of Stauffer well.

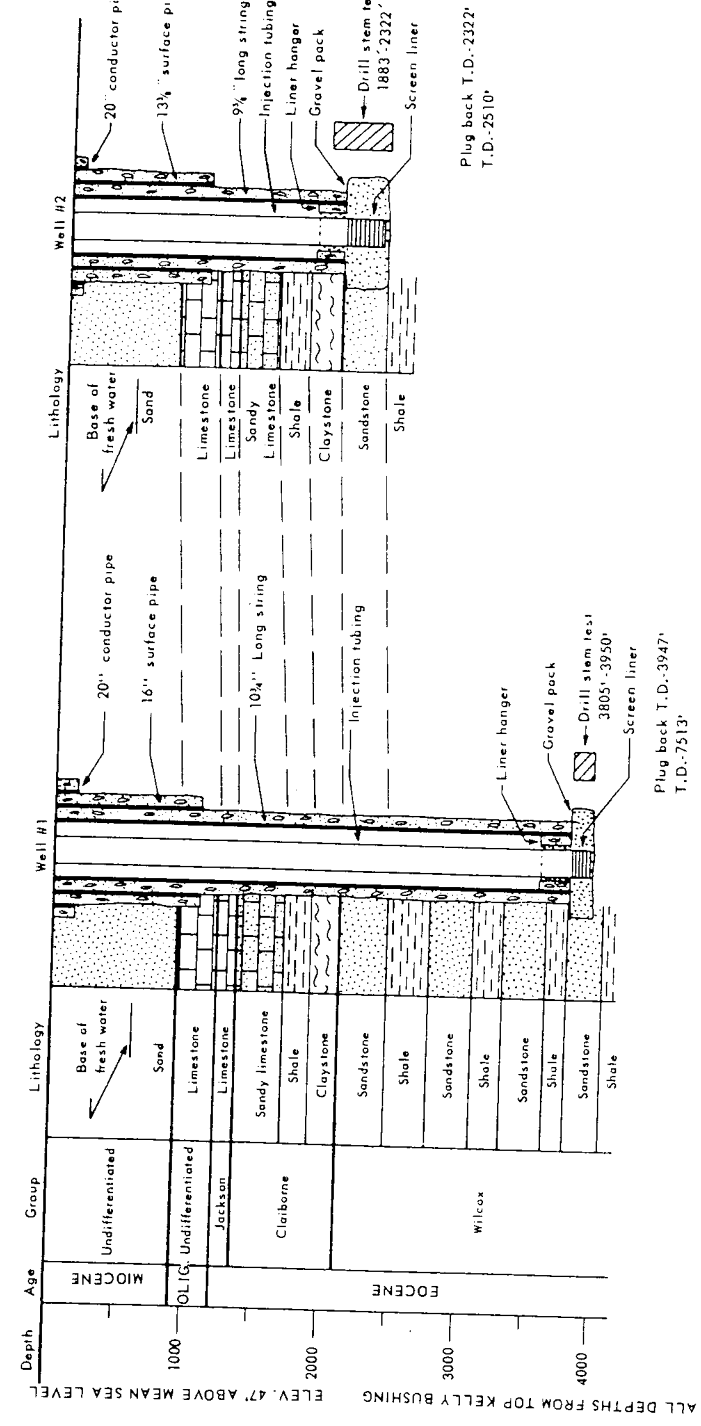

FIG. 6--Schematic cross section of Ciba-Geigy wells.

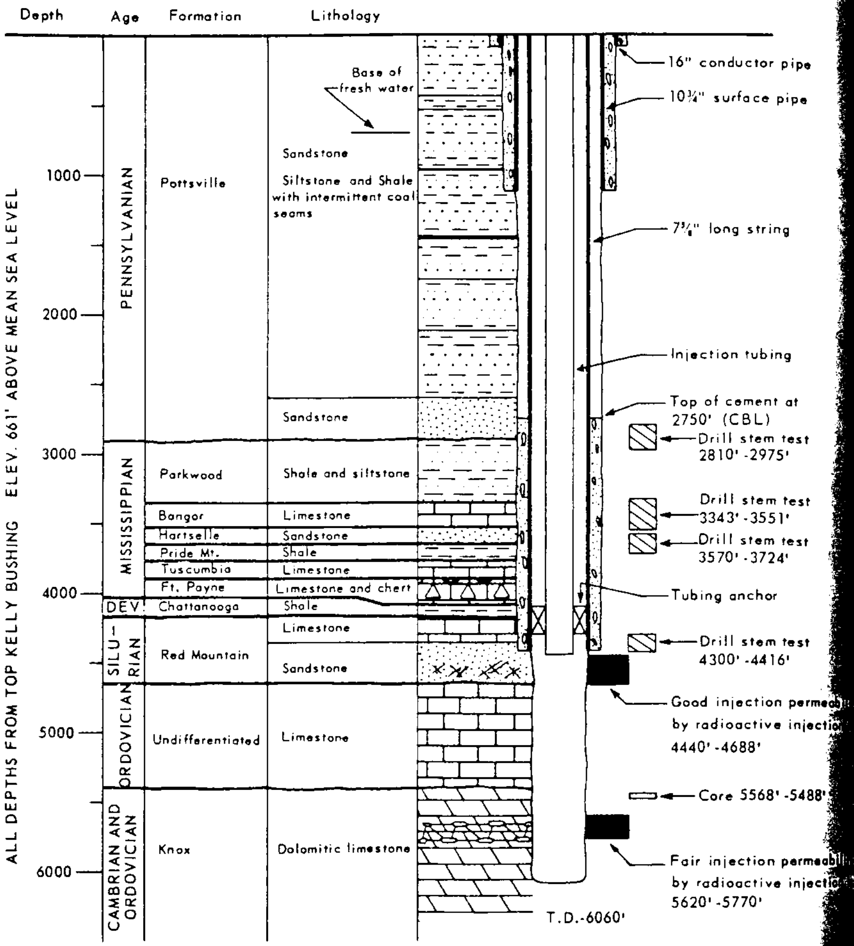

FIG. 7--Schematic cross section of U.S. Steel well.

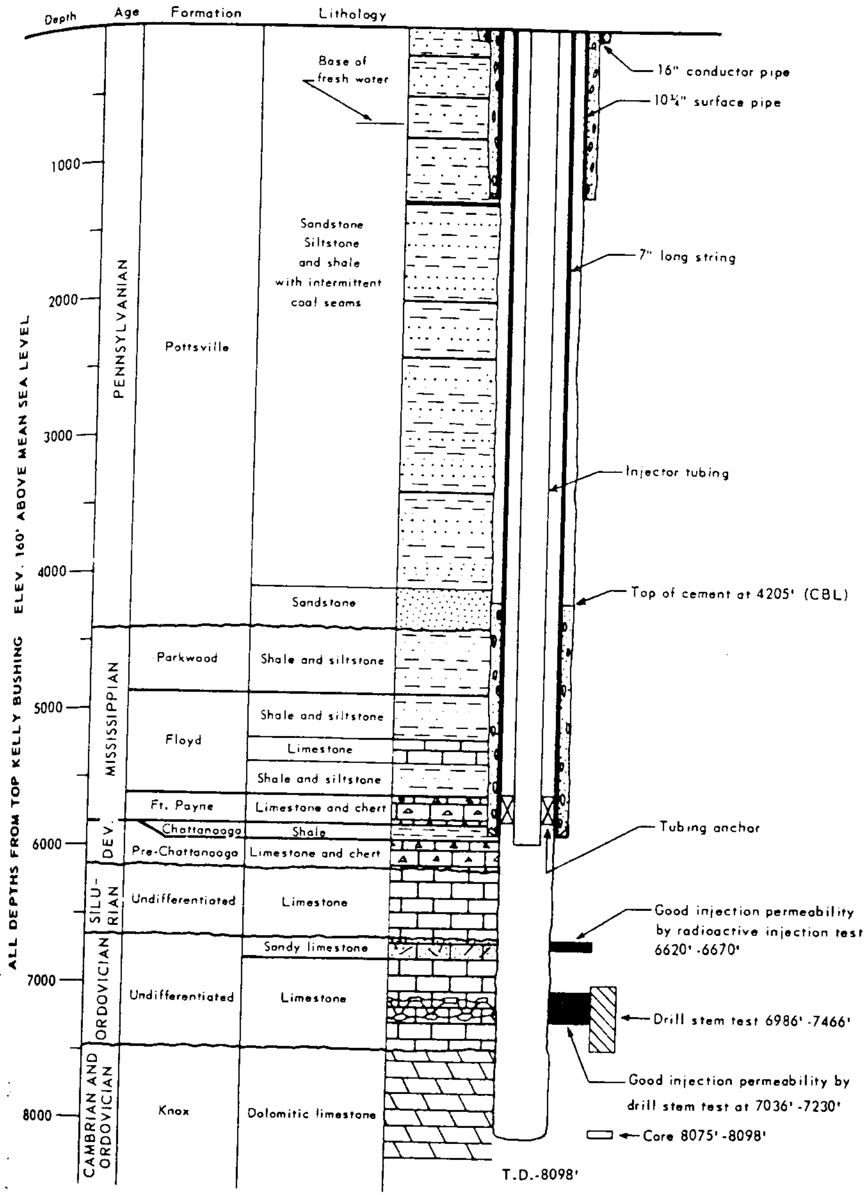

FIG. 8--Schematic cross section of Reichhold well.

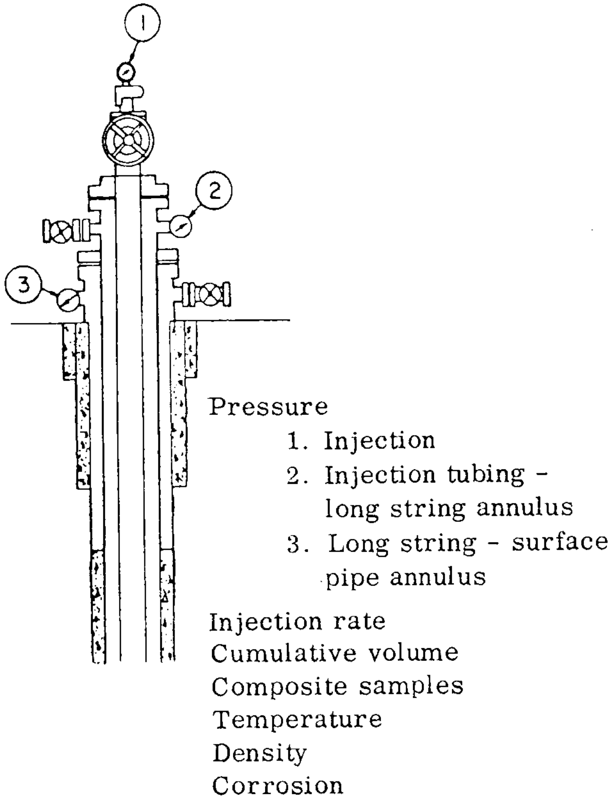

FIG. 9--Well-monitoring system.

EFFECTS OF WASTE PERCOLATION ON GROUNDWATER IN ALLUVIUM NEAR BARSTOW, CALIFORNIA[1]

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ABSTRACT Part of the alluvial aquifer along the Mojave River near Barstow, California, has been subjected to pollution from percolation of wastes and sewage from industrial and municipal sources for about 60 years. The pollution has forced the abandonment of several domestic wells because of taste, odor, and foaming, and it threatens the well field serving the U.S. Marine Corps Supply Center.

The stratification of chemical constituents found within the aquifer indicates that an old plume of degraded water (produced by percolation from sewage and waste-disposal facilities near Barstow) is moving near the base of the alluvial aquifer. Since 1910 this degraded plume has moved downgradient about 4 mi. A more recent overlying plume of degraded water is located near the downstream edge of the deeper plume. This overlying plume is produced by percolation from sewage-treatment facilities installed in 1968. Concentrations of detergent in groundwater beneath this site reflect the current use of readily biodegradable LAS-type detergents, in contrast to the ABS types that are found in the deeper zones of degradation.

The gradual increase in concentration of dissolved solids in the U.S. Marine Corps wells results in part from the use of treated sewage effluent on a local golf course. Areal and vertical mapping of the

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[2]U.S. Geological Survey.

The study was made in cooperation with the U.S. Marine Corps and the California Regional Water Quality Control Board, Lahontan Region. Their support is gratefully acknowledged.

degraded water indicates that the water supply at the Marine Corps Supply Center will be affected if no measures are taken to avoid further pollution.

A digital water-quality model of the aquifer aided in evaluating the effects of several alternative groundwater management practices. The model was first verified by comparison with existing hydrologic data and then was used to calculate the water-level and water-quality conditions that might occur in the aquifer within the next 20 years. Model results indicated that by 1991 present sewage-percolation practices would result in dissolved-solids concentrations exceeding 900 mg/l in the Marine Corps well field.

INTRODUCTION

In arid regions, groundwater in alluvial aquifers is commonly the only reliable source for man's water requirements. These aquifers have also constituted a convenient "out of sight--out of mind" medium for disposal of municipal and industrial wastes. This practice may effectively meet the immediate water-supply and disposal requirements but can produce a legacy of groundwater pollution that must be faced not only by future generations, but, in some cases, by the same officialdom responsible for the pollution.

Groundwater pollution in an alluvial aquifer near Barstow, California, is threatening the well field that serves the U.S. Marine Corps (USMC) Supply Center at Nebo. Mere knowledge that a groundwater-quality problem exists is not an adequate basis for implementing effective management practices to alleviate the problem. The chemical nature and the horizontal and vertical extent of the degraded plume, and the dynamic characteristics of the hydrologic system must be understood before corrective measures can be taken. Water-quality modeling techniques can then be employed to evaluate the efficacy of proposed management practices in terms of the effects of the practices on water levels and water quality within the aquifer. Such an evaluation can provide engineers with quantitative data for use in designing and operating facilities that will achieve the desired results with a minimum of uncertainty.

In June 1971 the U.S. Geological Survey began an investigation of groundwater degradation in the Barstow area, Mojave River basin, California. This area is in the Mojave desert about 96 mi northeast of Los Angeles and includes the reach of the Mojave River from above Barstow to below the USMC Supply Center at Nebo (Fig. 1).

The Mojave River at Barstow is dry except during periods of flooding caused by heavy precipitation on the mountainous areas 50 mi to the southwest. Precipitation at Barstow averages about 5 in./year and produces negligible groundwater recharge. Wells perforated in alluvial deposits of Holocene and Pleistocene age are the only dependable source of supply for the area's main water users (the city of Barstow and the Marine Corps Supply Center at Nebo). The quantity of groundwater in storage has been adequate to sustain local demands. The water supply for the city of Barstow is derived from wells in the younger alluvium north and west of the city, where the chemical quality of groundwater is generally good. The USMC supply wells are in the younger alluvial aquifer about 4 mi southeast of Barstow.

During recent years, the chemical quality of groundwater along the Mojave River, about 4 mi southeast of Barstow, has deteriorated significantly. A series of investigations by the California Department of Public Works (1952), California Department of Public Health and California Department of Water Resources (1960), and California Department of Public Health (1966, 1970) indicated that the degradation had resulted from local municipal and industrial waste disposal. The affected areas were delineated on the basis of taste, odor, and the presence of detergents. A comparison of the degraded areas in each of the above reports indicates that degraded groundwater is advancing downstream and could pose a threat to the well field at the USMC Supply Center. Should the degraded water appreciably affect the quality of water pumped at the USMC Supply Center, it would be necessary to obtain substantial quantities of water from other sources at considerable cost.

The objective of this study was to identify and describe the areas of degraded groundwater and the potential effect on the quality of water pumped from the USMC well field. This required identification of the chemical nature of the degraded water, with particular emphasis on its areal and vertical distribution and rate and direction of movement.

The scope of the study included (1) collecting hydrologic, geologic, and chemical data from published and unpublished sources; (2) augering enough test wells so that, when they were combined with existing irrigation and domestic wells, there would be a network of sampling points encompassing the areas of degraded groundwater; (3) collecting and analyzing water samples from selected wells; (4) performing aquifer-evaluation tests such as pumping tests and tracer-dilution tests in order to understand aquifer characteristics such as transmissivity, storage coefficient, porosity, and dispersivity; (5) adapting a water-quality model to the local hydrologic conditions and verifying the resulting model; and (6) using the water-quality model to evaluate the effects of several proposed groundwater management practices.

WELL-NUMBERING SYSTEM

Wells referred to in this report are numbered according to their location in the rectangular system for subdivision of public land. The number preceding the slash (for example, 9N/1W-13H1) indicates the township (T.9N.); the number following the slash indicates the range (R.1W.); the number following the hyphen indicates the section (sec. 13); the letter following the section number indicates the 40-acre subdivision. The final digit is a serial number for wells in each 40-acre subdivision. The area covered by this report lies east and west of the San Bernardino meridian and north of the San Bernardino base line.

GEOHYDROLOGIC SETTING

Groundwater Geology

In general, the surface geology of the area (Fig. 2) is characterized by outcrops in the highland areas of consolidated rocks of Tertiary and Quaternary age which yield very little water--less than 100 gallons per minute (gpm)--to wells. The lowland areas are characterized by unconsolidated alluvial deposits consisting of older fan deposits and older alluvium of Pleistocene age and younger fan deposits and younger alluvium of Holocene age. River-channel deposits are a part of the younger alluvium and compose much of the aquifer system studied in detail.

Data from more than 50 test wells augered during this study indicate that the river-channel deposits are highly permeable and consist of moderately well sorted, subrounded to subangular, fine to coarse sand and gravel. Wells in this unit typically yield from several hundred to more than 1,000 gpm. Clay is uncommon in the river-channel deposits and, where present, occurs in thin (0.5-1.5 ft) lenses. The hydraulic character of the younger alluvial deposits appears to approach that of a homogeneous and isotropic medium. Although stratification in sand layers may retard the vertical movement of groundwater, well logs and water-level data suggest that variations in vertical permeability are not of major consequence. Test wells were drilled to depths as great as 150 ft; the contact with the underlying older alluvial deposits is probably not

much greater than 150 ft below land surface.

The older alluvial deposits are poorly sorted and consist largely of boulders, gravel, sand, silt, and clay. The deposits are locally cemented and generally yield small to moderate quantities of water to wells. Water in this aquifer is typically high in dissolved solids, boron, and fluoride (Hughes and Patridge, 1973), but because of the low quantities of underflow recharge this water does not seem to be a significant source of degradation in the younger aquifer.

The Waterman fault trends northwestward across the Mojave River along the eastern side of the USMC Supply Center (Fig. 2), where its trace is buried by unfaulted surficial sediment. The difference in water levels across the fault was 45 ft in 1965 (Miller, 1969) and 30 ft in 1971 (Hughes and Patridge, 1973).

Groundwater Hydrology

Locality permeabilities of the younger and older alluvial aquifers, as calculated from pumping tests, are about 150 and 1.5 ft/day. Saturated thickness was estimated from logs of about 130 wells in the younger alluvium. Because of inadequate data on the full thickness of the older alluvium south of the Mojave River, a saturated thickness of 100 ft was assumed for the model study. Transmissivity (T), a parameter necessary in groundwater models, is taken as the product of the average permeability (P) times the saturated thickness of the aquifer (M). Transmissivity for the younger and older alluvial aquifers ranged from about 22,000 to 100 ft^2/day.

A barrier approximately coincident with the projected alinement of the Waterman fault restricts the movement of groundwater. Available data suggest that the barrier becomes increasingly effective with depth. To simulate the effects of the barrier on the movement of groundwater, a T of 200 ft^2/day was used.

The data used to determine the storage coefficient (S) for the aquifer were less reliable than the transmissivity. Storage coefficients were based on laboratory analyses of undisturbed core samples (Hardt, 1971) and data from an extensive aquifer test of a USMC supply well during the study. The results suggest that the storage coefficient is about 20 percent for the younger alluvial aquifer. A storage coefficient of 16 percent was assumed in areas where the younger and older aquifers locally intertongue.

In addition to those aquifer parameters discussed (T, S, P, and M), estimates of dispersivity and porosity were required. The coefficient of dispersion is essential to the water-quality model because it is a measure of the rate of dispersion or mixing of water of different chemical qualities which accompanies movement within the aquifer. Tracer-dilution tests and model computations indicated a dispersivity of about 200 ft and a porosity of about 40 percent. The porosity value of 40 percent compared favorably with the probable average porosity of lithologies described in drillers' logs of the younger alluvium.

Hydrologic Budget

The aquifers near Barstow receive recharge as underflow from the west and south, and from floodflow in the river. Contouring of groundwater levels (1972) indicates a gradient from west to east of 10-15 ft/mi in the reach upstream from the Waterman fault, and a somewhat steeper gradient downstream from the fault (Fig. 2). Long-term hydrographs indicate that water levels in the younger alluvial aquifer reflect the intermittent surface flow in the Mojave River. Steady declines in the groundwater level in some areas exceed 40 ft during dry periods when no surface flow occurs, and may be followed by as much as 50 ft of recovery during a year with ample floodflow in the Mojave River.

Groundwater development in the Barstow area has resulted in a complex system of recharge and discharge. The effect of this development has been to lower the water table (Fig. 2), thereby reducing the volume of groundwater in storage and reducing the rate of groundwater flow eastward.

The water budget for the Barstow area can be divided into the following categories: (1) Recharge by underflow, by surface water and sewage effluent, and by irrigation return; and (2) discharge by underflow and pumpage (Table 1). Each of these quantities is considered in the water-quality model.

Recharge by underflow is the subsurface inflow from the aquifers to the west of Barstow and from the much less permeable aquifer to the southeast of Barstow. Variations in this quantity of recharge represent changes in inflow due to changes in saturated thickness of the aquifers to the west of Barstow (Table 1). The aquifer to the southeast of Barstow is undeveloped and has undergone very little change in head.

Surface water is recharged to the area from streamflow in the Mojave River. Most of the recharge since 1946 occurred as a result of floods in 1952, 1958, and 1969 (Table 1). This was the major source of

recharge to the aquifer and it largely determined the distribution and length of the recharge-discharge pulses used in the model. The quantities of a surface-water recharge shown in Table 1 were uniformly distributed among the water-quality model nodes used for surface-water (Fig. 3). The concentration of surface-water recharge nodes northeast of the Waterman fault reflects the increased ability of the aquifer to accept recharge in this area. This increase in recharge potential is due to the large volume of unsaturated alluvium resulting from the drop in head across the Waterman fault.

Recharge from effluent occurs where there is deep percolation of sewage-effluent water. This recharge originates from two main sources in the Barstow areas--(1) the city and railroad sewage-treatment facilities (sites A, B, and C, Fig. 2), and (2) the USMC sewage-treatment facilities (site D, Fig. 2).

Discharge by underflow from the area occurs along the Mojave River on the northeast side of the Waterman fault. The large variations in this quantity shown in Table 1 are due to correspondingly large variations in the saturated thickness of the aquifer downstream from the fault.

The term "pumpage" may be used to represent either the net consumptive use of water extracted from the aquifer or the total quantity of water extracted from the aquifer. When the pumping well and the area of use are in close proximity, the quantity of extracted water that percolates and returns to the aquifer may be subtracted from the total extraction from the well to calculate the pumpage. However, this can be done only when the degraded chemical quality of the irrigation-return water is not a significant source of groundwater degradation. When groundwater degradation occurs, pumpage is the total well extraction, and the quantity of water returned to the aquifer is considered to be a separate recharge quantity called "irrigation-return recharge." This allows the irrigation-return recharge to be assigned a different chemical quality than that of the groundwater and enables the model to consider this recharge as a source of groundwater degradation.

Evaluation of the volumes of recharge and discharge (Table 1) indicates that for the 26-year period (1946-1971), the volume of groundwater in storage has been reduced by an estimated 13,000 acre-ft. The beneficial effects of surface-water recharge on groundwater in storage are illustrated by omitting the recharge from the 1969 floods from the change in storage computations; under these conditions the computed decline in storage was approximately 32,000 acre-ft in 26 years.

SOURCES OF GROUNDWATER DEGRADATION

Sources of impairment of groundwater quality in the Barstow area include (1) natural contamination, (2) domestic- and industrial-waste disposal, and (3) irrigation return.

Natural Contamination

Groundwater inflow from areas south of the river is generally high in dissolved solids, and for modeling purposes was assumed to average about 1,000 mg/l. Because the rate and volume of inflow associated with this source are comparatively low (Table 1), the influence of this source on the groundwater quality in the basin is minimal.

Domestic and Industrial Wastes

The city of Barstow has operated three sewage-treatment plants since 1938. Each facility has been constructed in the highly permeable younger alluvial deposits and has used this medium for percolation. Effluent from the 1938 plant (site A, Fig. 2) was discharged directly to the Mojave River bed (California Department of Public Health and California Department of Water Resources, 1960, p. 24). In 1953, a new sewage-treatment plant (site B, Fig. 2) was constructed about 0.5 mi east and downstream from the older, abandoned site. Disposal of effluent from this facility was by direct percolation from oxidation ponds. Some treated effluent was diverted for nearby irrigation of alfalfa. In late 1968, a new, larger capacity, treatment facility was built immediately upstream from the USMC Supply Center (site C, Fig. 2). At present (1973), the new facility provides primary treatment with mechanical aeration in one pond, and six additional oxidation ponds. Modifications in this plant to provide complete secondary treatment are now under construction.

Chemical analyses of waste from the city of Barstow's facilities indicate that the effluent has been high in detergents (as MBAS [methylene blue active substances]), with concentrations ranging from 0.48 to 16 mg/l. The average has been well above the limits set for discharge at this facility by the California Regional Water Quality Control Board. The Board's maximum limits for detergents (as MBAS) in the effluent that percolates to the groundwater basin has been, at different times, 0.5 mg/l and 1.0 mg/l. The concentration of phenols in the Barstow

effluent has been as high as 0.15 mg/l. The concentration of dissolved solids, chloride, and sodium has been marginal in comparison to their respective limits of 900, 120, and 160 mg/l.

Available records of industrial wastes at the Atchison, Topeka and Santa Fe Railway disposal site (site A, Fig. 2) suggest that discharge to the Mojave River began about 1910 with the construction of a drain system and installation of an oil trap in which some of the fuel oils and solvents in the discharge could be separated from the water.

A laundry was constructed in 1949 by the railroad to wash grease- and oil-saturated rags and clothing. This facility produced an effluent containing emulsified oil, grease, synthetic detergents, and a disinfectant with a high concentration of phenolic compounds.

Large volumes of wastes were also produced in the railroad diesel-repair shop by overflow during the filling of fuel tanks, in the cleaning of locomotives, and by drainage from the caustic cleaning vats. In addition, hexavalent chromium (Cr^{+6}) was used from about 1948 to 1952 as a rust inhibitor for radiators in diesel engines, and it was common practice to drain the coolants to the waste-disposal system (California Department of Public Health and California Department of Water Resources, 1960, p. 29).

During 1954 and 1958, the railroad constructed two dikes in the river (site A, Fig. 2) to hold treated effluent. The diked areas provided for physical separation of oil and sludge, but did not control percolation of synthetic detergents, chromium, emulsified oils, and phenolic compounds. The dikes were constructed of river-channel sand, which offered little resistance to floods. Both dikes were destroyed by floods in April 1958 but were reconstructed during the summer of 1958. During the period 1959-1968, the railroad changed detergent from ABS to LAS (nonbiodegradable to biodegradable synthetic detergents), and abandoned its laundry and caustic cleaning vats at the diesel shop, thereby eliminating two sources of phenolic compounds from the waste effluent.

In 1968 the railroad began treating wastes to comply with the standards established by the California Water Quality Control Board (resolution 66-18) and by the city of Barstow. Most railroad waste subsequently was exported to the new city waste-treatment facility (site C, Fig. 2). Unacceptable solid matter and some concentrated solutions reportedly are removed from the Barstow area by the railroad.

The chemical quality of waste from the facilities of the Atchison, Topeka and Santa Fe Railway has varied depending on the source and volume of discharge. These variations are indicated in part by pH values which range from very acidic, 3.9, to very basic, 12.4. Comparison of concentrations in wastes for detergents (0.35-64 mg/l), phenols (0.000-1.0 mg/l), oil and grease (0.04-4,000 mg/l), chlorides (34-496 mg/l), boron (0.06-46 mg/l), sodium (80-1,200 mg/l), dissolved solids (311-2,700 mg/l), total chromium (0.00-4.81), and hexavalent chromium (0.000-11 mg/l) with California Water Quality Control Board requirements (1966) indicates that concentrations far above published standards have been available for percolation to the groundwater system.

Facilities for treatment and disposal of industrial and domestic waste at the USMC Supply Center were built in 1942 (site E, Fig. 2) and were subsequently modified and expanded in 1952 and 1957. The present system provides primary treatment and some degree of secondary treatment. The treated effluent is either percolated to the Mojave River or is used as irrigation water for a local golf course (site D, Fig. 2). Some concentrated industrial-waste products are transferred to the USMC annex at Yermo (Fig. 1) for disposal.

Chemical analyses of waste discharged by the USMC Supply Center indicate that the concentrations of oil and grease and of phenols in the waste effluent have been as high as 79 mg/l and 0.06 mg/l, respectively. As a result of influent industrial wastes, pH values in the effluent have ranged from 6.5 to slightly above 9.0.

Irrigation Return

The high rate of evapotranspiration in the Barstow area requires that large quantities of water be applied to crops. Alfalfa, the major crop in this area, transpires about 3 ft of water per year (Meyer and Horn, 1955, p. 197) when about 5 to 7 ft per year of water is applied for irrigation. Allowing for consumptive use, leaching of minerals from the soils, and the solution of fertilizers, the concentration of dissolved solids in irrigation-return water could be at least double the original concentration. Assuming that the applied irrigation water in the northeastern part of secs. 4 and 10, T.9N., R.1W. (Fig. 2) contained 350-450 mg/l of dissolved solids, the irrigation-return water could contain more than 700 mg/l of dissolved solids.

The annual volume of irrigation-return water (Table 1) has been nearly equal to annual underflow into the Barstow area from the west; it is greater than the average annual volume of domestic and industrial waste which percolated to the river from the city treatment facilities

prior to 1952; and it is only slightly less than the average annual volume associated with the facilities operated from 1962 to 1968.

Effluent from the wastewater treatment facilities at the USMC Supply Center (site E, Fig. 2) was used for irrigating a golf course (site D, Fig. 2) between 1959 and 1972. The annual volume of effluent used varied only slightly and totaled about 4,000 acre-ft or approximately 300 acre-ft/year. Most of the treated water was applied during the summer. Chemical analyses of the treated water indicated a concentration of dissolved solids of about 1,000 mg/l. The concentrations of most chemical constituents were within the generally accepted limits for irrigation. Because of the high consumptive use and the presence of sandy soils at the golf course, about half of the applied water returned to the aquifer. Therefore, the concentration of dissolved solids in the return water is estimated to be about 2,000 mg/l.

Effluent from the city of Barstow treatment facility (site B, Fig. 2) was used for irrigation of alfalfa at a site about 0.5 mi downstream. That part of the effluent not required for irrigation was discharged by the city of Barstow to the Mojave River (California Department of Public Health and California Department of Water Resources, 1960, p. 25-26). This source of groundwater recharge is included with effluent-recharge data in Table 1. The concentration of dissolved solids in the effluent recharged near site B (Fig. 2) was adjusted in the model to compensate for the additional concentration of dissolved solids resulting from irrigation and consumptive use.

Estimates were made of the average concentration of dissolved solids from each source of recharge in the project area. These data, in addition to the approximate location of each source of recharge and discharge, are shown in Figure 3.

DISTRIBUTION AND EVALUATION OF CHEMICAL SUBSTANCES AFFECTING GROUNDWATER DEGRADATION

Percolation of waste effluents from industrial and municipal sources, plus irrigation return, has seriously affected the quality of the groundwater in the younger alluvial aquifer east of Barstow. The extent of the degraded water has been identified both areally and vertically by the concentration of (1) dissolved solids, (2) detergents (as MBAS), and (3) dissolved organic carbon.

The vertical sections used to illustrate the distribution of selected chemical substances in groundwater are based on analyses of

samples collected from March 27 to April 7, 1972. The patterns of dis-
tribution were drawn on the basis of chemical analyses of water from
wells which are either on or near the trace of the vertical sections
and reflect hydrologic stresses resulting from recharge and discharge
in the aquifer.

The patterns of distribution of chemical substances suggest a cer-
tain uniformity of occurrence and flow within the hydrologic system.
However, monthly sampling of three test wells, 9N/1W-9H5, 9H7, and 10,
indicates that the distribution of chemical quality of groundwater is
changing with time, particularly in the deeper zone. Such changes in
groundwater quality are to be expected because the system is dynamic;
the quality and quantity of past waste discharge has varied. Figure 4
illustrates the changes in chemical quality of groundwater in these wells
with time.

Some similarities in the distribution of general chemical quality
apparent in each of the sections illustrated in Figure 5. Along section
A-A' there is a general increase in constituent concentration with depth
upstream from well 9N/1W-9B1. This gradation with depth has resulted
from recharge by large volumes of good-quality flood flow that have be
superimposed on the poorer quality water which percolated from sites A
and B (Fig. 2).

Distribution of chemical substances in groundwater beneath the
present city of Barstow treatment facility (site C, Fig. 2) reflects
the quality, quantity, and timing of sewage percolation. Estimates by
the California Department of Public Health (1970, p. 7) of the rate of
waste percolation from the city oxidation ponds indicate that percola-
tion from pond 2 is approximately 10 times that of the other ponds. The
greater percolation rate at pond 2 results from the use of six lagoons
adjacent to pond 2. The two irregularities in the shallow plume of de-
graded water shown in Figure 5 reflect this distribution of recharge.
The first irregularity is located beneath pond 2 and results from waste
percolation from the lagoons; the other is in the vicinity of ponds 6
and 7.

The hydrologic effects on the aquifer system caused by the city's
waste discharge is somewhat similar to the effects caused by flood re-
charge. This discharge and concomitant recharge to the aquifer has
depressed or displaced underlying groundwater of different quality.
Section A-A' (Fig. 5) suggests that a body of better quality ground-
has been isolated between the deeper, older degraded groundwater and

the younger plume produced by current waste percolation. The quality of groundwater in the deep plume worsens with depth, and the quality of groundwater in the shallow plume becomes better with depth. Chemical analyses of groundwater from shallow wells adjacent to the present oxidation ponds are very similar to chemical analyses of waste effluent in each of the ponds.

Groundwater recharge resulting from about 15 years of use of treated sewage to irrigate the USMC golf course has produced a body of degraded groundwater that reflects both the quality and quantity of the effluent used. The vertical distribution of water quality beneath the golf course is similar to that described beneath the city's treatment facility (Fig. 5) with the exception of the absence of the deeper, older plume of poorer quality water. The quality of groundwater beneath the golf course is much poorer at a depth of 50 ft than at a depth of 100 ft below land surface. The only apparent source for this poor-quality groundwater is the irriga-tion-return water that has been concentrated by evapotranspiration and by domestic and industrial use.

Chemical and hydrologic data indicate that the distribution of water quality in the vicinity of the golf course is influenced by the Waterman fault and by extensive pumping at the USMC Supply Center. Because the Waterman fault tends to retard groundwater flow, drawdown of the water table caused by pumping at the USMC Supply Center is exaggerated and has reversed the natural groundwater gradient. This reversal in gradient has resulted in groundwater flow from areas of poor-quality water in the vicinity of the golf course toward the USMC Supply Center wells (Fig. 5, section B-B').

Dissolved Solids

In the Barstow area the concentration of dissolved solids in ground-water ranges from less than 500 to more than 2,000 mg/l (Hughes and Patridge, 1973). Available data suggest that the concentration of dis-solved solids in flood flows that recharge the aquifer is about 150 mg/l (Miller, 1969, p. 24). Very few wells have been drilled south of the river in the older, more consolidated deposits; therefore, chemical data on groundwater inflow from these areas are sparse. Analyses of groundwater samples from wells 9N/1W-15Q1, 15Q2, and 27D1 (Fig. 2) and 9N/1E-19J1-5 (not shown on Fig. 2) indicate that dissolved solids south of the river are generally high, varying directly with well depth. The concentrations of dissolved solids in water from wells 9N/1E-19J1 and J5, which are 250 and 660 ft deep, respectively, range from 800 mg/l in the shallow well to 2,300 mg/l in the deep well. Water from other wells south of the river (9N/1W-15Q1, 15Q2, and 27D1) contains less than 800 mg/l of dissolved solids. For modeling purposes, an average of 1,000 mg/l of dissolved solids has been assumed for groundwater entering the area from the south. The effect of this inflow on the quality of water in the river-channel deposits appears to be minor in comparison to that of the quantity and quality of municipal and industrial waste disposal. Volume of inflow from the south has been estimated to be 120 acre-ft/year (Table 1). Inflow from the consolidated rocks to the north is probably negligible.

Available data suggest that the greatest impact on quality of groundwater in the project area has resulted from the large volumes of groundwater used for domestic, industrial, and agricultural purposes. Water used for these purposes has increased dissolved solids in groundwater by (1) addition of solid or liquid concentrates, such as detergents, fertilizers, rust inhibitors, and oil and grease; and (2) evapotranspiration associated with agriculture and with exposure in waste-treatment facilities. Dissolved solids in industrial and municipal wastes have averaged more than 1,000 mg/l, with some effluent from the railroad facilities exceeding 2,500 mg/l of dissolved solids.

The distribution of dissolved solids in groundwater in the Barstow area is illustrated in Figure 5. The concentration of dissolved solids in water from well 9N/1W-10J4 is very similar to waste water in the city oxidation ponds. During 1970, dissolved solids in these ponds averaged more than 950 mg/l. In 1972 dissolved solids in groundwater from well 9N/1W-10J4 were approximately 1,000 mg/l. This well is about 500 ft from the ponds.

The adverse effects on groundwater from use of treated and reclaimed sewage on the golf course are indicated by the distribution of dissolved solids in Figure 5, section B-B'. Wells 9N/1W-11P1, 11Q1, and 11R1 are approximately 50 ft deep, and indicate a very high concentration of dissolved solids (and several other chemical constituents including chloride, sulfate, calcium, sodium, and magnesium) in groundwater when compared with adjacent deeper wells 9N/1W-11P2, 11Q2, and 11R2, which are about 100 ft deep. Chemical analyses of water from the shallow wells showed dissolved-solids concentrations of 1,140, 1,520, and 1,160 mg/l in contrast to 500, 439, and 389 mg/l for the deeper wells (Hughes and Patridge, 1973). The distribution of dissolved solids in section B-B' appears to be influenced by pumping at the USMC Supply Center and

by the barrier effects of the Waterman fault.

Detergents

Synthetic detergents were first used in the United States about 1946 and now constitute the principal washing compounds for industrial and domestic purposes. These compounds contribute to water-quality problems when they percolate to groundwater, usually as part of the effluent from industrial and domestic waste sources. Problems associated with treatment and disposal of water that contains detergents were partly solved in 1964 by the development of the biodegradable LAS (linear alkylate sulfonate) and by restrictions placed on the use of the nonbiodegradable ABS (alkyl benzene sulfonate). Experiments conducted on the biodegradation potential of LAS and ABS (Halvorson and Ishaque, 1969, p. 571-576) indicated that under proper aerobic conditions, LAS would biodegrade by 97.5 percent in 120 hours at 25°C, whereas ABS remained unaffected. Under similar aerobic conditions the rate of biodegradation of LAS at a temperature of 10°C decreased by a factor of 2.5, and at 2± 0.1°C no biodegradation of LAS was observed over a period of 12 days. Similar experiments for anaerobic conditions showed that neither LAS nor ABS was appreciably biodegraded at 25°C. Results of these experiments strongly suggest that the concentration of ABS detergents in domestic and industrial waste in the Barstow area during the period 1946-1965 biodegraded very little prior to percolation into the younger alluvial deposits. Since 1965, the amount of biodegradation of detergents probably has been largely dependent on water temperatures in the treatment plant. Winter temperatures in Barstow average 10°C±, and night-time temperatures fall below 2°C (National Weather Service, 1972). During summer months, temperatures probably are sufficiently high to permit maximum biodegradation.

Synthetic detergents are not indigenous to groundwater and can therefore be used to detect degraded and polluted groundwater. The area generally affected by detergent concentrations of 0.2 mg/l in the groundwater extends from the approximate location of the city of Barstow's 1953-1968 waste-disposal facility to the vicinity of pond 7 at the present treatment facility (Fig. 6). Sections A-A' and C-C' (Fig. 5) illustrate the vertical distribution of detergents in groundwater. The higher concentrations of detergents occur at depth. Samples analyzed from this deep zone during 1972 have fluctuated in concentration from 1.3 to 3.6 mg/l (well 9N/1W-9H7, Fig. 4) and probably reflect pre-1964 waste effluent that was high in ABS-type detergents. The lower concentration of detergents in groundwater samples from shallow wells 9N/1W-9H5 and 10J (Fig. 5) are interpreted to be a result of more complete biodegradation associated with the current use of LAS. Detergent concentrations in both groundwater zones equal or exceed the limits (0.5 mg/l) recommended by the U.S. Public Health Service (1962) for drinking water.

During this investigation only trace quantities of synthetic detergents were found in water samples from the test wells augured adjacent the USMC golf course and from the wells at the USMC Supply Center. Apparently the detergents in the effluent used to water the golf course have been consumed either by biodegradation, by adsorption, or by the grass.

Dissolved Organic Carbon

Dissolved organic carbon (DOC) is a quantitative parameter, which measures that part of total organic carbon in water which passes through a 0.45-micron silver membrane filter (Malcolm and Leenheer, 1972). Dissolved organic carbon can therefore reflect the presence of many individual organic chemical compounds soluble in water, such as synthetic detergents, phenolic compounds, oil and grease, herbicides, humic and fulvic acids, and pesticides (oral commun., L. A. Eccles, 1972). As illustrated in Figure 6, DOC analyses of groundwater have been used to identify areas affected by municipal and industrial waste disposal and irrigation return.

A comparison of the areas degraded by detergents and by DOC (Fig. 6) indicates a lack of detergents but appreciable quantities of DOC in groundwater upstream from well 10N/1W-32R2. The concentration of detergents in this half-mile reach may have been diminished by the large volume of flood recharge that occurred in 1969. More likely, the lack of detergents represents the downstream movement of the last waste discharged by the railroad in 1968. The distance involved and rate of movement are compatible with this 4-year time span. Those organic constituents that have a greater potential for adsorption onto the sediment may not have been affected appreciably by the flood recharge, and the organic compounds that appeared in the water samples may be the result of desorption. Thus, there appears to be a continuous source of DOC from material adsorbed onto the sediment. The concentration of oil and grease, detergents (as MBAS), and chemical oxygen demand (COD) from groundwater in wells 10N/1W-32N1, N2, and N3 are low, and do not account for the organic compounds represented by the high DOC values of 4.6, 5.1, and 5.9 mg/l.

Location of the downstream nose of the plume defined by DOC (Fig. 6) suggests that degraded groundwater has advanced downstream sufficiently to be influenced by the barrier effects of the Waterman fault and by pumping at the USMC Supply Center. Groundwater samples from wells 9N/1W-11M1, M2, P1, P2, 12N1, and 12N3 contained 1.3-2.4 mg/l of DOC. The highest concentrations of DOC in groundwater in the project area were from wells in the vicinity of the city of Barstow's oxidation pond 7 (Fig. 6). Water samples from wells 9N/1W-10Q5 and 10J4 contained 6.3 mg/l of DOC. Both wells are shallow--58 and 25 ft deep.

The vertical distribution of DOC in groundwater in the project area is illustrated in sections A-A' and C-C', Figure 5. A comparison of these DOC sections with similar sections for dissolved solids and detergents indicates that flood recharge in the areas upstream from the present city oxidation ponds has had little effect on the concentrations of DOC. This uniformity of concentration of DOC with depth is not readily understood from available data, but may be related to adsorption and desorption.

EFFECT OF PUMPAGE ON WATER QUALITY AT USMC SUPPLY CENTER

Since 1957 the concentration of dissolved solids in groundwater pumped at the USMC Supply Center has steadily increased from about 350 mg/l to more than 600 mg/l. The concern about the increase in dissolved solids and the appearance of some undesirable organic compounds (for example, phenols) is justified because poor-quality groundwater is moving downstream in the direction of this well field. The extent of degradation caused by the poor-quality groundwater near the golf course and near the Barstow treatment facility (site C, Fig. 2) depends largely on the area affected by pumping from wells at the USMC Supply Center.

An aquifer test was conducted at the USMC Supply Center in March 1972 to define the approximate area influenced by pumping, and thereby the possible source or sources affecting the quality of water on the base. USMC supply wells 9N/1W-14A2, 14B3, 13E1, and 13E2 were pumped at full capacity for 96 hours, discharging a total of about 34 acre-ft. Response of groundwater levels to the discharge was measured in the pumping wells and in 17 nonpumping observation wells. The combined discharge from the four wells is comparable to peak water-supply demands at the USMC Supply Center during the summer. Test results were used to delineate an area of drawdown equal to or greater than 0.25 ft as shown in Figure 7. The barrier effect of the Waterman fault on groundwater reduced the water-level declines downstream from the fault and tended to increase declines in the direction of the golf course

Although the aquifer at the USMC Supply Center is capable of supplying large quantities of water, the groundwater gradients that were produced by pumping induced recharge from the direction of the golf course and the Barstow waste-treatment facility (site C, Fig. 2). Therefore, unless some changes are made to alter groundwater flow paths, the water pumped at the USMC Supply Center can be expected in future years to increase in dissolved solids, chloride, and those compounds indicated by dissolved organic carbon (DOC). Because of the proximity of the USMC golf course, degraded groundwater from this source will appear first in the USMC supply wells.

APPLICATION OF WATER-QUALITY MODELING TECHNIQUES

Adaptation and Verification

A digital water-quality model of the aquifers in the project area was used to evaluate the hydrologic effects of various methods of managing the groundwater system. The model is two-dimensional and assumes complete vertical mixing of a conservative chemical constituent (for example, dissolved solids) in the water-table aquifer. An alternating-direction implicit iterative mathematical procedure is used to calculate the head configuration (Pinder and Bredehoeft, 1968) at each node point in the constant-interval rectangular array (Fig. 3). The method of characteristics proposed by Garder, Peaceman, and Pozzi (1964) with tensorial dispersion modifications by Reddell and Sunada (1970) was used to develop the computer program for the water-quality model (Bredehoeft and Pinder, 1973). The model calculates the chemical and hydraulic response of an aquifer to varying rates, durations, and chemical quality of recharge and withdrawal.

The parameters used in the water-quality model include: saturated thickness and transmissivity; storage coefficient; porosity; dispersivity; distribution, rate, and chemical quality of recharge (Fig. 3, Table 1); rate and distribution of discharge (Fig. 3, Table 1); and water-level and water-quality configurations for the start of the simulation period (Fig. 2). These parameters were determined by detailed study of the geologic, hydraulic, and water-quality data of the area as presented in work by Miller (1969), Koehler (1970), Hardt (1971), and Hughes and Patridge (1973).

The model was used to calculate water-level and water-quality conditions for the period 1945-1971. These model-generated data were then compared with historic water-level and water-quality data for the same period in order to verify the model calculations.

Field data indicate that there are only minor head differences with depth in the younger alluvial aquifer; however, significant vertical stratification of water quality occurs within the aquifer. Due to aquifer heterogeneities, wells may not derive water uniformly throughout the perforated interval. As a result, the groundwater-quality data are probably more representative of water quality in a particular zone than of the average water quality throughout the saturated thickness of the aquifer. This was found to be the case in the model verification, in that the model-generated water-level data have a high correlation with the field water-level data, whereas the model-generated water-quality data have a lower correlation with field water-quality data.

The correlation between model data and field data was considered to be adequate for verification, with the understanding that the model-generated water-quality data probably do not represent the quality of water that could be pumped from a particular well but rather the average water quality in a node area.

In the course of verification, the model calculates the change in water quality that occurred between 1946 and 1969 in the historic plume of degraded water below the Barstow sewage-treatment facility (site B, Fig. 2). As shown in Figure 8, the area of the degraded plume with concentrations in excess of 400 mg/l dissolved solids gradually increased from 1946 through 1968. During 1969 large quantities of good quality (150 mg/l dissolved solids) surface-water recharge occurred as the result of floods in the Mojave River. In the model this recharge decreased the size of the 1969 plume of degraded water. The location of the contour along the southwest edge of each of the plumes is approximate because the exact location is obscured by the effects of poor-quality inflow from the aquifer to the south.

Before the model was used to project future conditions in the aquifer, it was necessary to revise the recharge, discharge, and water-quality data to take into consideration the increasing water demands of a growing population. The population of the city of Barstow was assumed to increase at a uniform rate from 18,000 in 1970 to 45,000 in 1985 (Inerfield and Montgomery, 1971, p. VX-2) and to reach 58,000 by 1992. The resulting percolation from the lower Barstow sewage ponds would total about 5,300 acre-ft/year by 1992. Because of the limited quantities of groundwater in storage within the model area, it was assumed that the future increase aquifers out of the model area or by the importation of water. With the exception of the USMC supply wells, the 1971 pumping (rates and location) was maintained throughout the 20-year projection period from 1972 to 1992. The Marine Corps water demand and quantity of sewage-effluent recharge were assumed to increase at a rate of about 2 percent per year, reaching about 2,000 acre-ft/year and 750 acre-ft/year by 1992.

Because the future occurrence of flood flow in the Mojave River cannot be predicted accurately, it was assumed that the future surface-water recharge would occur at the same magnitude and at about the same frequency as did the historic recharge between 1947 and 1969. Thus the flood-flow recharge that occurred in 1952, 1958, and 1969 was projected to recur in 1977, 1983, and 1992. The modified recharge-discharge data were then broken into pulses similar to those shown in Table 1 in order to simulate conditions between 1972 and 1992.

By assuming that the magnitude and frequency of surface-water recharge would not change significantly in future years, the effects of the following two potential changes in the quantity of future groundwater recharge are not considered--(1) a recently completed flood-control dam on the Mojave River about 50 mi upstream from Barstow might affect the river-flow regime enough to alter the quantity of surface-water recharge that the study area would receive; and (2) the quantity of surface-water recharge is partly controlled by the space available between the water table and the land surface. Thus, if future water-level conditions differ markedly from historic conditions, the same magnitude of flood that occurred historically could produce a different quantity of recharge under future water-table conditions. No attempt was made to consider either of these conditions in the model projections because the quantitative effects could not be determined.

The concentrations of dissolved solids in the irrigation-return recharge and the underflow recharge from west of Barstow were increased to 750 and 550 mg/l, respectively, to simulate future water-quality better (Fig. 3). The chemical quality of other water recharging the aquifer was not altered from that which occurred under historic conditions. The resulting model projections represent average head and water-quality conditions that could be expected to occur in the future if the model parameters correctly describe the future stresses on the aquifer. The estimated 1972 dissolved-solids configuration shown in Figure 9 is included as a basis for comparison between present and future conditions.

Model Projections

The first set of conditions to be evaluated was labeled "run 55" and shows the changes that would occur in the aquifer if present operational practices are continued at the Barstow sewage-treatment facility (site C, Fig. 2), the USMC golf course (site D, Fig. 2), and the sewage-treatment facility (site E, Fig. 2). The projections take into consideration the prior water-level and water-quality conditions in the model area.

The model indicates that by 1981 the two plumes of degraded water that were near site C and sites D and E in 1972 (Fig. 9) have enlarged and merged, forming a single plume. The zone of water of better quality to the south of sites E and D in 1972 has, by 1981, markedly decreased in quality.

The degradation of groundwater quality continues through 1991 (Fig. 10), and water in excess of 900 mg/l extends into most of the USMC supply wells and below the Waterman fault. In general, a deterioration in water quality occurs throughout the aquifer downgradient from the northeast corner of sec. 9, T.9N., R.1W. Upstream from California Highway 15, the model indicates a general improvement in the quality of the degraded plume associated with the cessation of recharge at the Barstow sewage-treatment facility in 1958 (site B). It is important to note, however, that the elimination of a source of groundwater degradation does not eliminate the plume of degraded groundwater, for remnants of the model-calculated plume are present 24 years after the disposal site was abandoned in 1968. This result is due partly to reduced groundwater velocities caused by the recharge mound produced at the Barstow waste-treatment facility (site C). As a result, dilution by surface-water recharge is the main mechanism by which the degraded plume is dissipated.

The 1972-1991 water-level change map (Fig. 11) indicates that moderate water-level declines will occur downstream for the Waterman fault and north of Highway 15. Water-level rises of as much as 20 ft are produced by the increasing quantities of effluent recharge at the Barstow waste-treatment facility (site C, Fig. 2). This source of poor-quality recharge has the beneficial effect of inhibiting water-level declines in the central part of the model area.

The cessation of all effluent recharge in the vicinity of the USMC Supply Center is perhaps the most obvious technique which could be used to combat further groundwater degradation. In order to evaluate the hydrologic effects of this technique, model run 58A was made to show the effects of halting the percolation of all sewage effluent by lining the ponds at sites C and E and piping the effluent out of the study area. It was further assumed that all irrigation would cease at the USMC golf course. The quantity of recharge thus eliminated from the model ranged from 2,800-ft/year in 1972 to 6,250 acre-ft/year in 1992.

By 1991 this practice produced model-generated water-level declines of as much as 70 ft near the USMC supply wells and between 45 and 50 ft in the aquifer upgradient from site C. The large quantities of surface-water recharge in 1922 (analogous to the 1969 recharge quantities) produced 10 to 15 ft of water-level recovery in the area and did not significantly alter the water-level conditions.

Water-level declines of 70 ft near the USMC supply wells would have the detrimental effect of dewatering the highly permeable, uppermost part of the aquifer, thereby greatly reducing the yield of these wells and increasing the pumping costs. Thus, any potentially beneficial effects on water quality would be offset by detrimental hydraulic effects, and this management practice would not be entirely desirable unless an additional source of water was available to supplement the requirements at the USMC Supply Center.

Another method of combating the spread of degraded groundwater involved the installation of a line of protective-pumping wells between the Barstow waste-treatment facility (site C, Fig. 2) and the USMC supply wells. The protective-pumping wells could be pumped at a constant rate of 4,000 acre-ft/year to meet the cooling requirements at an existing industrial facility located out of the study area. Recharge originating from the USMC golf course (site D, Fig. 2) and sewage-treatment facility (site E, Fig. 2) would be eliminated by relocating the golf course and utilizing the Barstow waste-treatment facility for disposal of USMC raw sewage. Percolation would be allowed to continue at the Barstow facility (site C, Fig. 2).

The model-generated head conditions (run 60) for this management practice were similar to those of run 58A in the vicinity of the USMC well field, but showed less head decline upstream from the Barstow waste-treatment facility (site C, Fig. 2). Again, the potential beneficial effect on water quality is overshadowed by detrimental water-level declines in the vicinity of the USMC well field.

As an alternative to the above conditions, model run 61 was made assuming that the protective-pumping wells would be pumped at a reduced rate of 2,000 acre-ft/year.

The model indicated that from 1972 to 1987 (Fig. 12) the protective pumping wells could effectively retard the spread of degraded groundwater. However, by 1991 the rate of percolation from the Barstow sewage ponds exceeds the protective-pumping rate by about 3,600 acre-ft/year, and water in excess of 900 mg/l would begin to move past the line of protective-pumping wells.

Under these conditions, the 1972-1991 water-level changes in the area are modest. Thus the establishment of a line of protective-pumping wells could be an effective deterrent to the groundwater degradation near the USMC Supply Center. If the rate of pumping from the wells were gradually increased in response to the increasing rate of percolation from the Barstow treatment facility, the effectiveness of the barrier could probably be extended beyond 1991.

In view of the prior model run which used a barrier pumping rate of 4,000 acre-ft/year, increasing or decreasing the barrier pumping rate could produce either a more effective water-quality barrier with greater pumping lifts and lower well yields in the USMC well field, or a less effective water-quality barrier with smaller pumping lifts and higher well yields in the USMC well field. A compromise between acceptable limits of water quality and water quantity must be made at the local level before the pumping barrier can be used to best advantage.

CONCLUSIONS

A body of degraded groundwater caused by municipal and industrial discharge has been mapped both areally and vertically in the Barstow area. This study has indicated that several plumes of degraded groundwater are moving downgradient towards the USMC well field. One plume, near the base of the alluvial aquifer, is probably the result of percolation from abandoned upstream waste-disposal sites. A more recent overlying plume occurs near the downstream edge of the deeper plume and has been produced by percolation from sewage-treatment facilities installed in 1968. A third plume, in the vicinity of the USMC golf course, is moving towards the USMC well field in response to pumpage from these wells. Higher concentrations of dissolved constituents in this third plume are the result of the use of reclaimed industrial and domestic effluent for irrigation on the golf course.

A digital water-quality model aided in evaluating the hydrologic effects of several possible management practices designed to alleviate water-quality problems near the USMC Supply Center. The model and other data indicate that a steady deterioration in water quality will occur the USMC well field unless remedial measures are taken. Elimination of percolation from the USMC golf course and from waste-treatment facilities in the area was undesirable, owing to excessive head declines produced the USMC well field. A line of pumping wells between the waste-treatment facilities and the USMC well field could retard the degradation in the well field if proper barrier design and pumping rates are maintained.

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Table 1. Water Budget (acre-ft; Numbers in parentheses, acre-ft/year)

Period	Duration (years)	Recharge: Underflow from West	Recharge: Underflow from Southeast	Recharge: Surface water	Recharge: Effluent City and railroad	Recharge: Effluent USMC	Recharge: Irrigation return	Discharge: Underflow to east	Discharge: Pumpage	Change in storage for period
1946-1951	6	6,600 (1,100)	720 (120)	2,100 (350)	3,300 (550)	2,460 (410)	540 (90)	-3,540 (-590)	-22,440 (-3,740)	-10,260
1952	1	1,100	120	7,370	640	410	320	-630	-3,140	6,190
1953-1957	5	5,500 (1,100)	600 (120)	0 (0)	3,750 (750)	2,300 (460)	3,750 (750)	-1,800 (-360)	-24,300 (-4,860)	-10,200
1958	1	1,100	120	9,900	860	610	1,200	-490	-7,080	6,220
1959-1968	10	8,000 (800)	1,200 (120)	2,600 (260)	12,200 (1,220)	3,500 (350)	[1]13,600 ([1]1,360)	-4,200 (-420)	-55,600 (-5,560)	-18,700
1969	1	800	120	20,280	1,690	380	[1]1,230	-700	-4,680	19,120
1970-1971	2	1,600 (800)	240 (120)	0 (0)	3,700 (1,850)	960 (480)	[1]2,840 ([1]1,420)	-840 (-420)	-13,960 (-6,980)	-5,460
Period total	26	24,700	3,120	42,250	26,140	10,620	23,480	-12,200	-131,200	-13,090

[1]Includes 150 acre-ft/year irrigation return from USMC golf course.

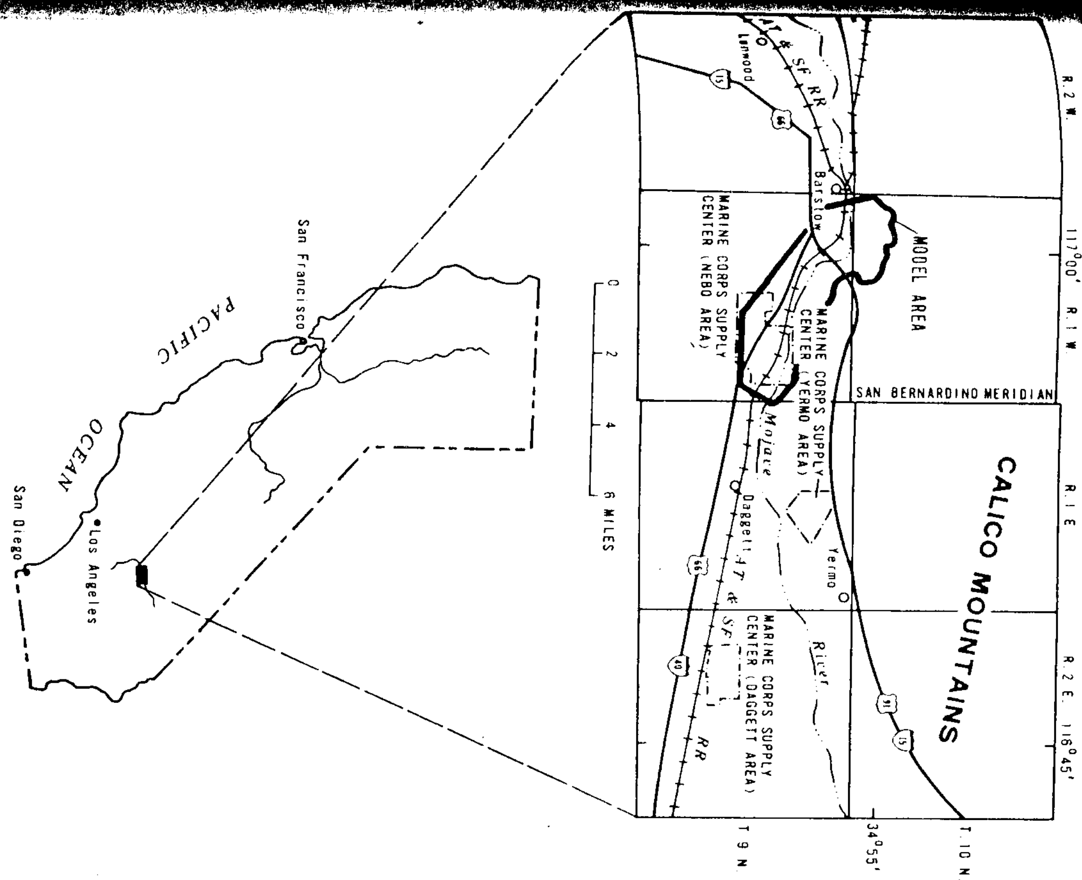
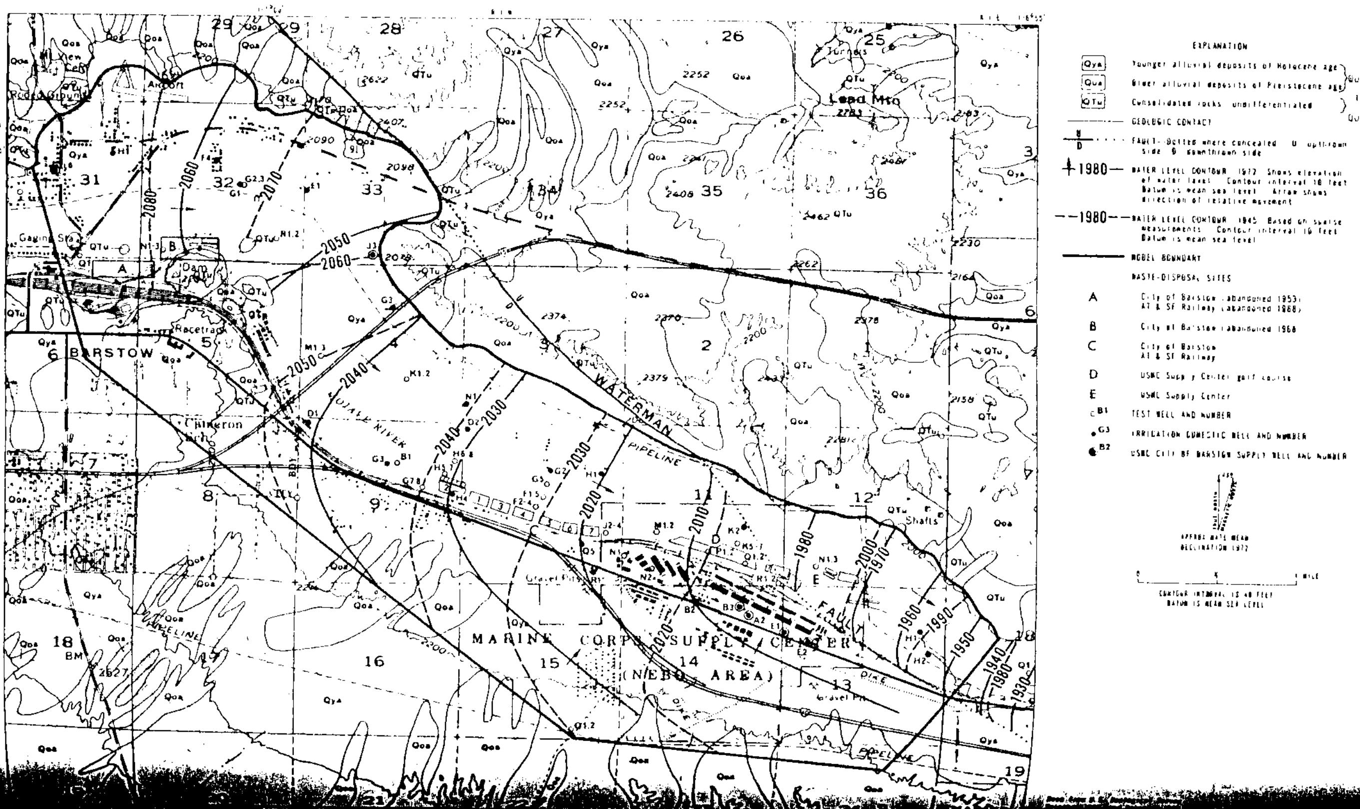


FIG. 1--Study area.



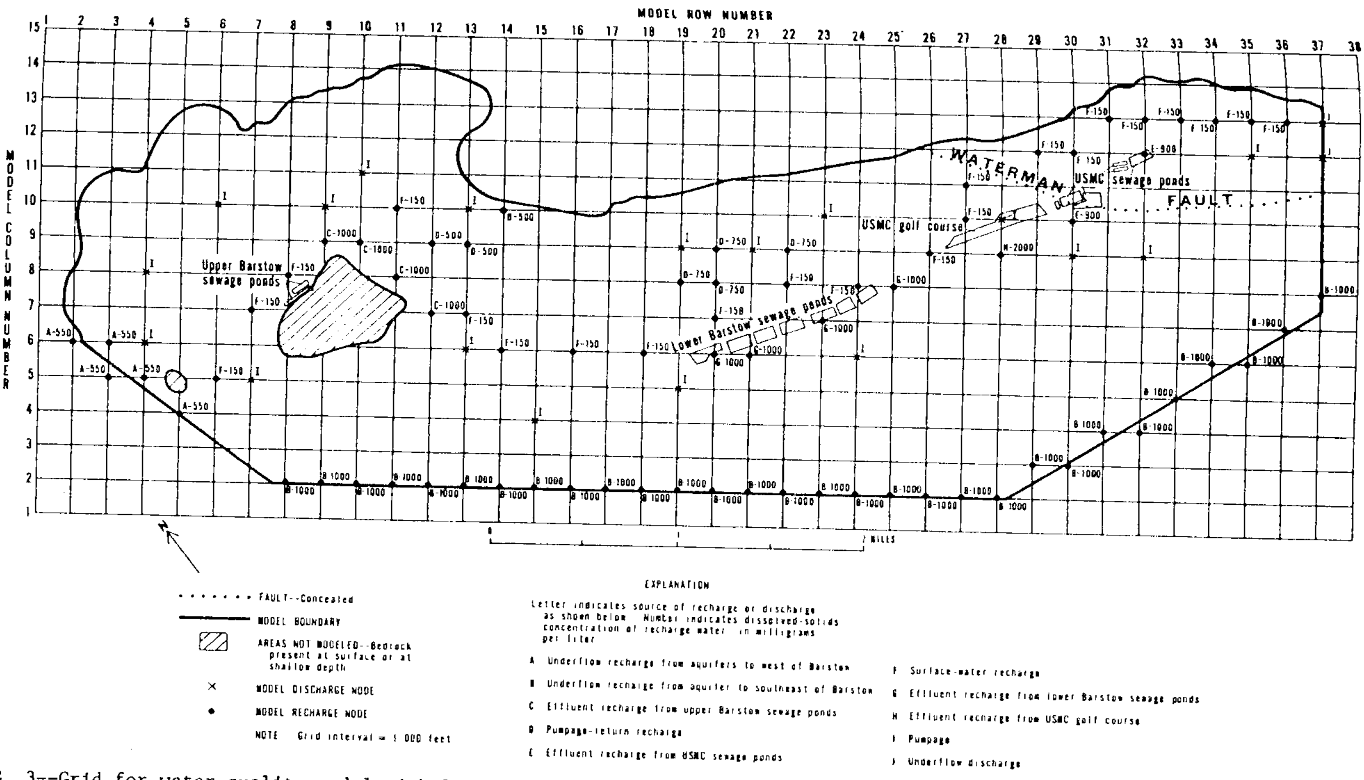


FIG. 3--Grid for water-quality model with location of points of recharge and discharge. Waste-disposal sites as identified in Figure 2.

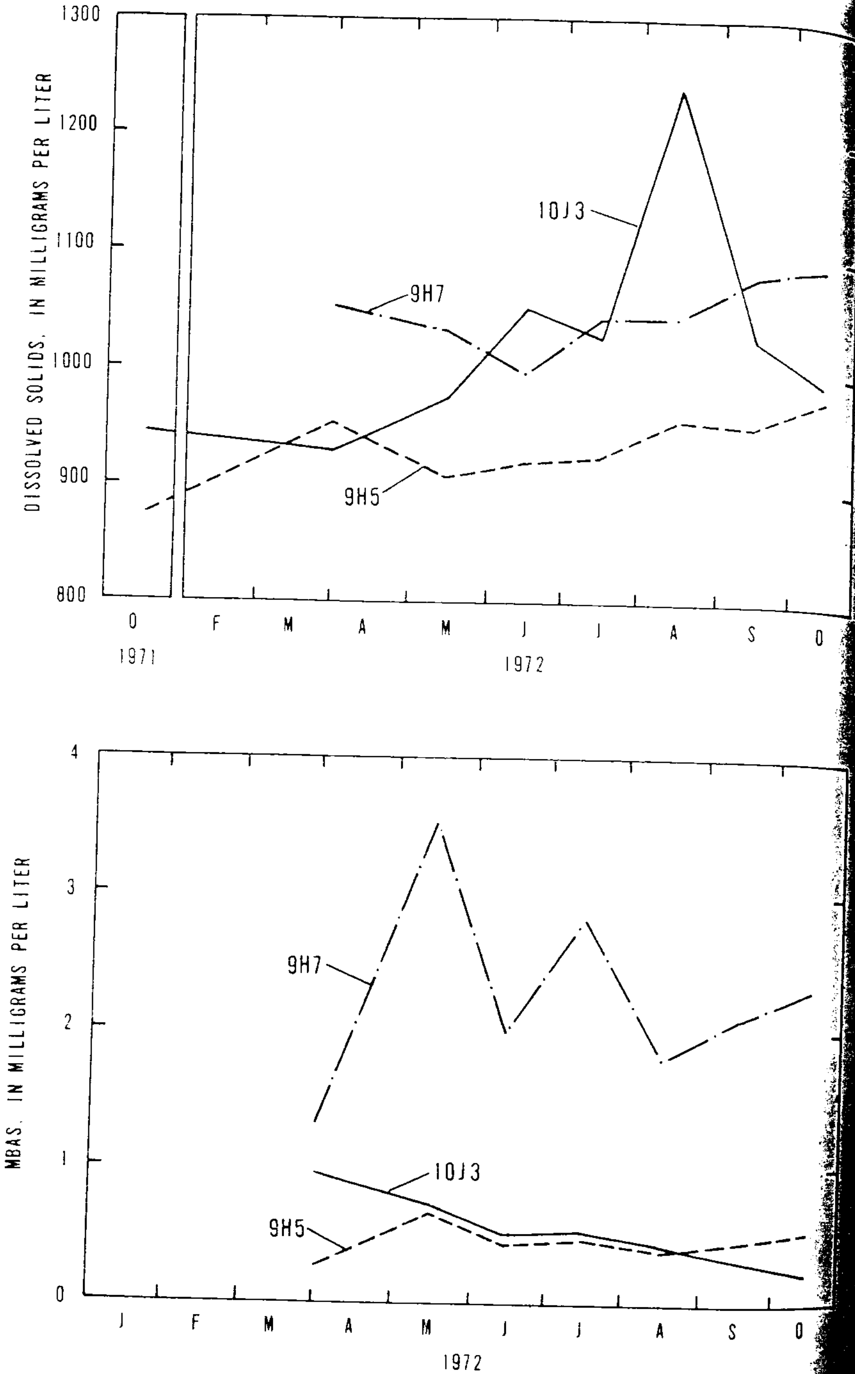


FIG. 4--Change in concentration of selected chemical constituents in wells 9N/1W-9H5, 9H7, and 10J3.

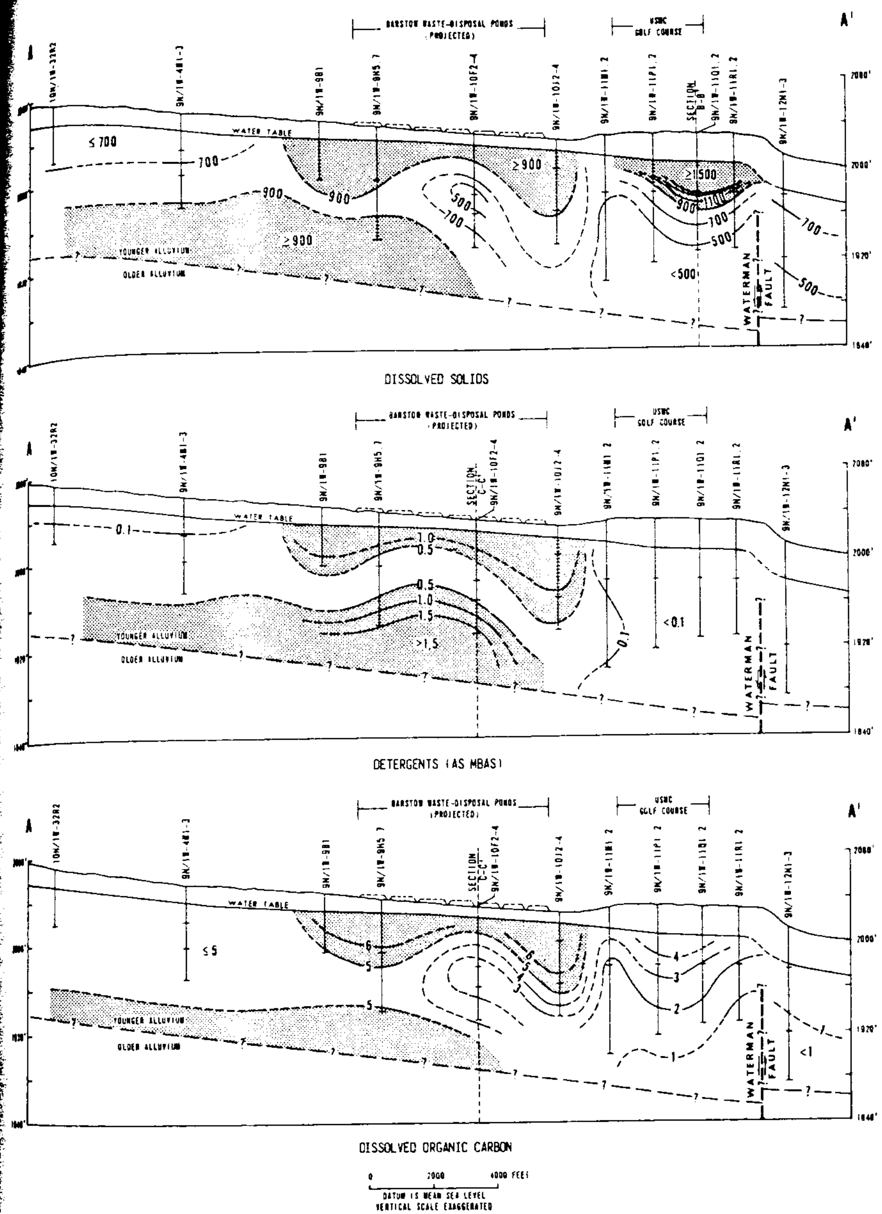


FIG. 5--Vertical distribution of selected chemical constituents (continued on next page).

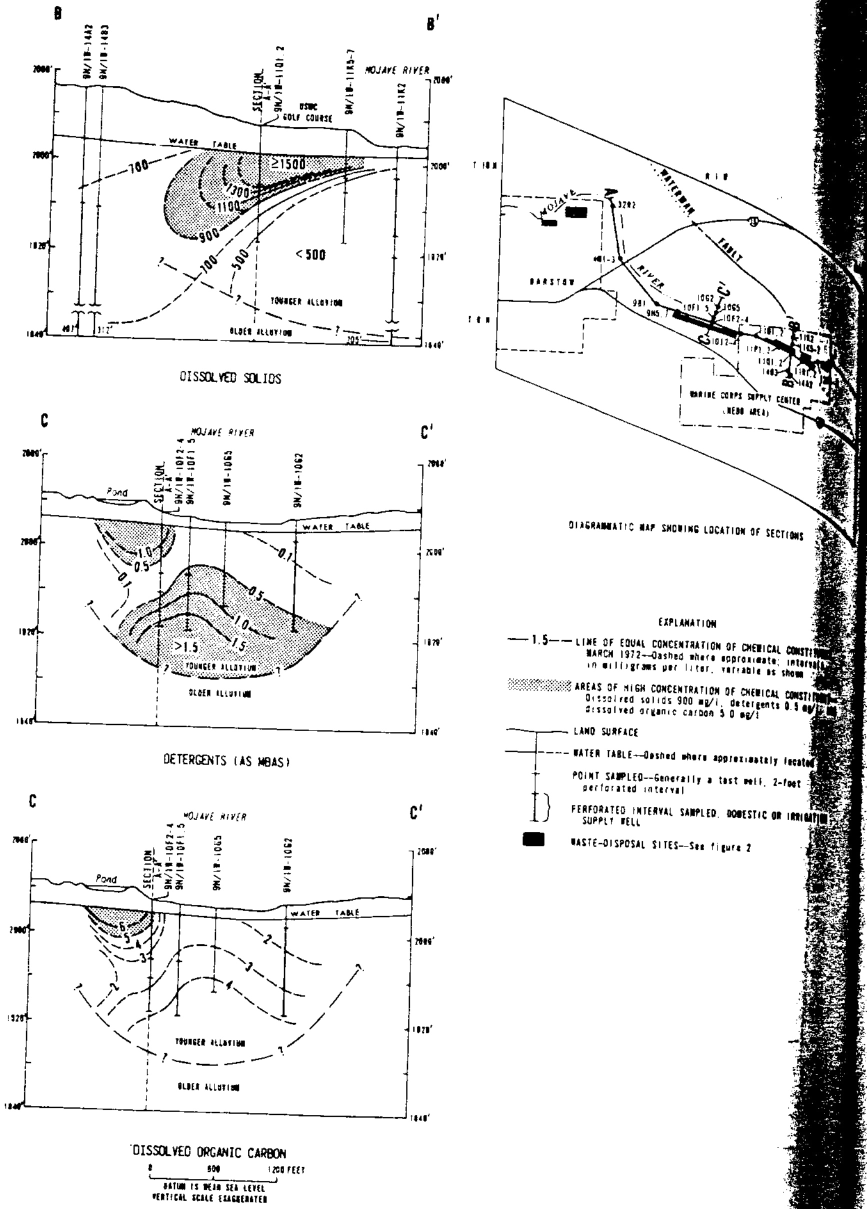


FIG. 5--Continued.

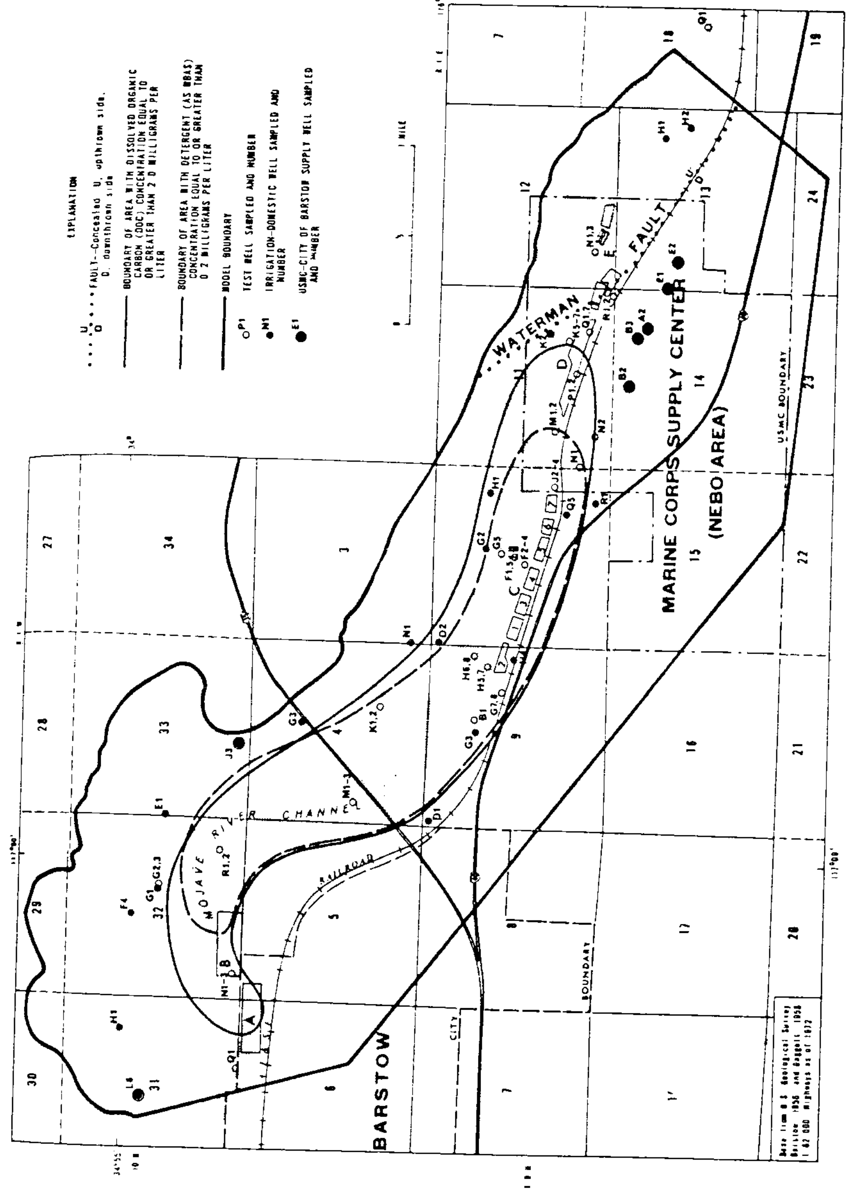
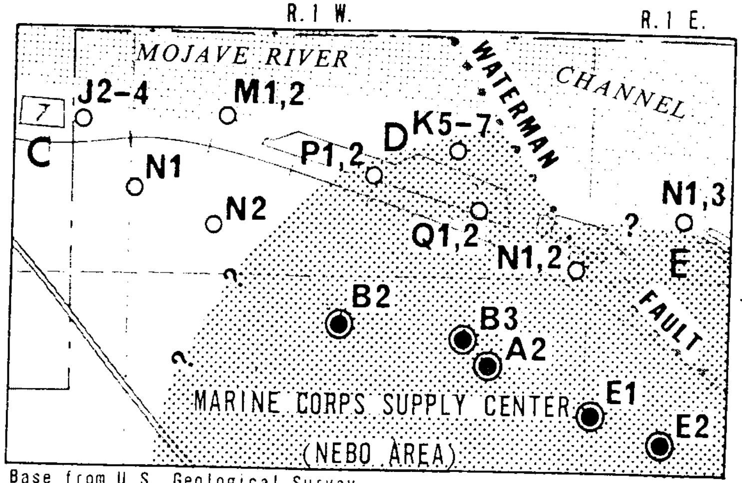


FIG. 6--Areal distribution of dissolved organic carbon (DOC) and detergents (as MBAS).



Base from U.S. Geological Survey
Barstow, 1956, and Daggett, 1956,
1:62,500.

0 ½ 1 MILE

EXPLANATION

• • • • FAULT--Concealed

AREA AFFECTED BY DRAWDOWN IN WELL EQUAL TO OR GREATER THAN 0.25 FOOT--Queried where doubtful

A2 USMC SUPPLY WELL AND NUMBER

N1 OBSERVATION WELL AND NUMBER

FIG. 7--Area affected by a 96-hour aquifer test at U.S. Marine Corps Supply Center.

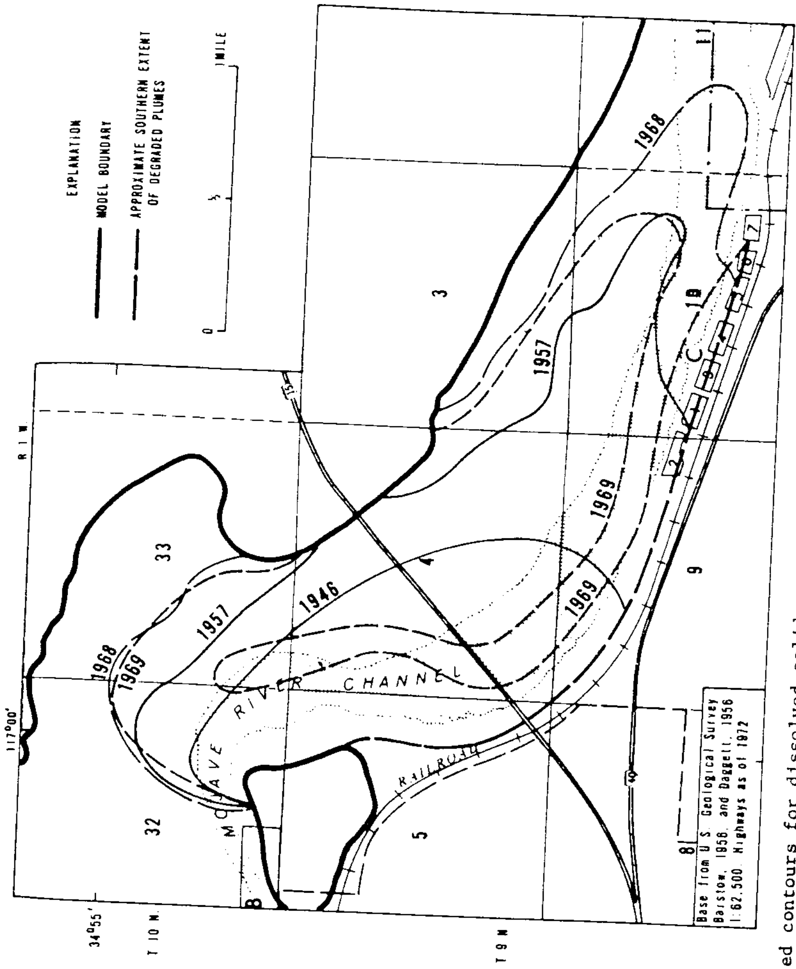
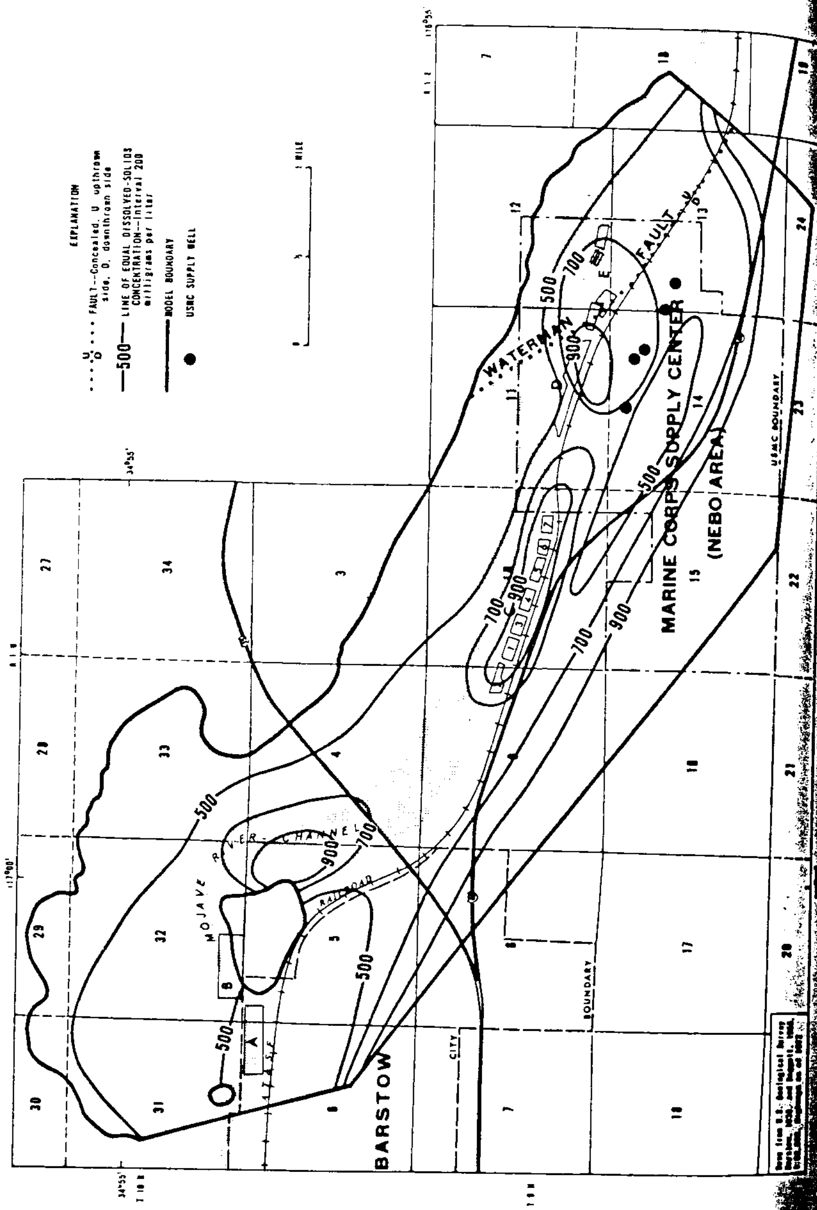


FIG. 8--Model-generated contours for dissolved-solids concentration of 400 mg/l in plume of degraded water below waste-treatment facility (site B) between 1946 and 1969.



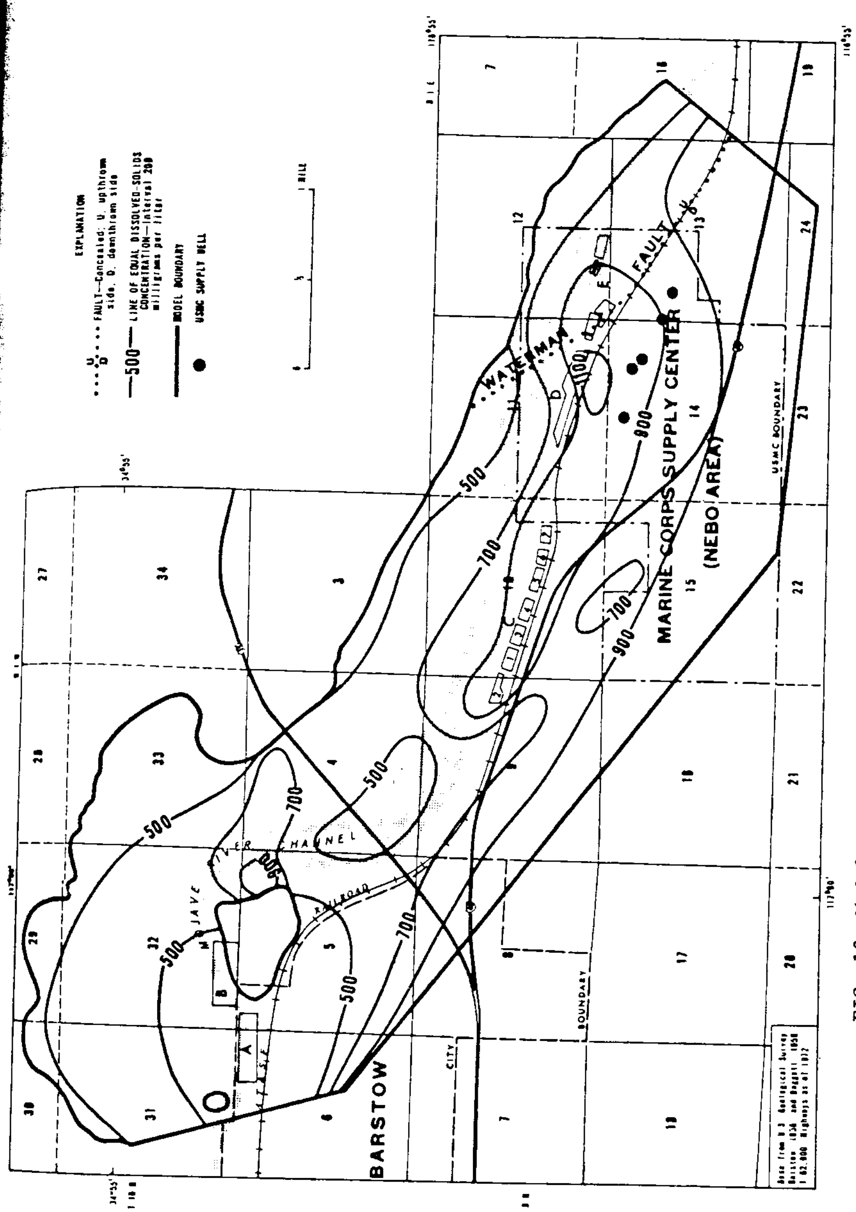
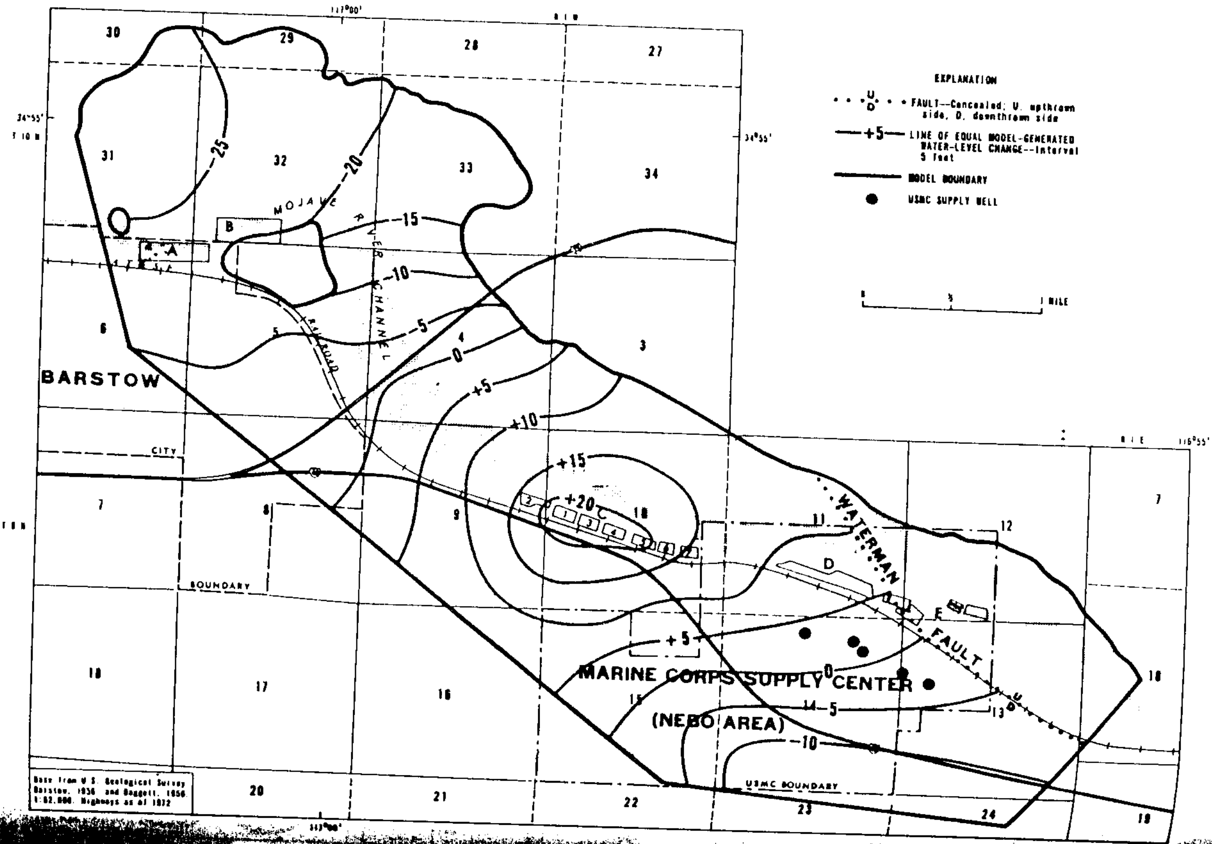


FIG. 10--Model-generated 1991 dissolved-solids concentration, run 55.



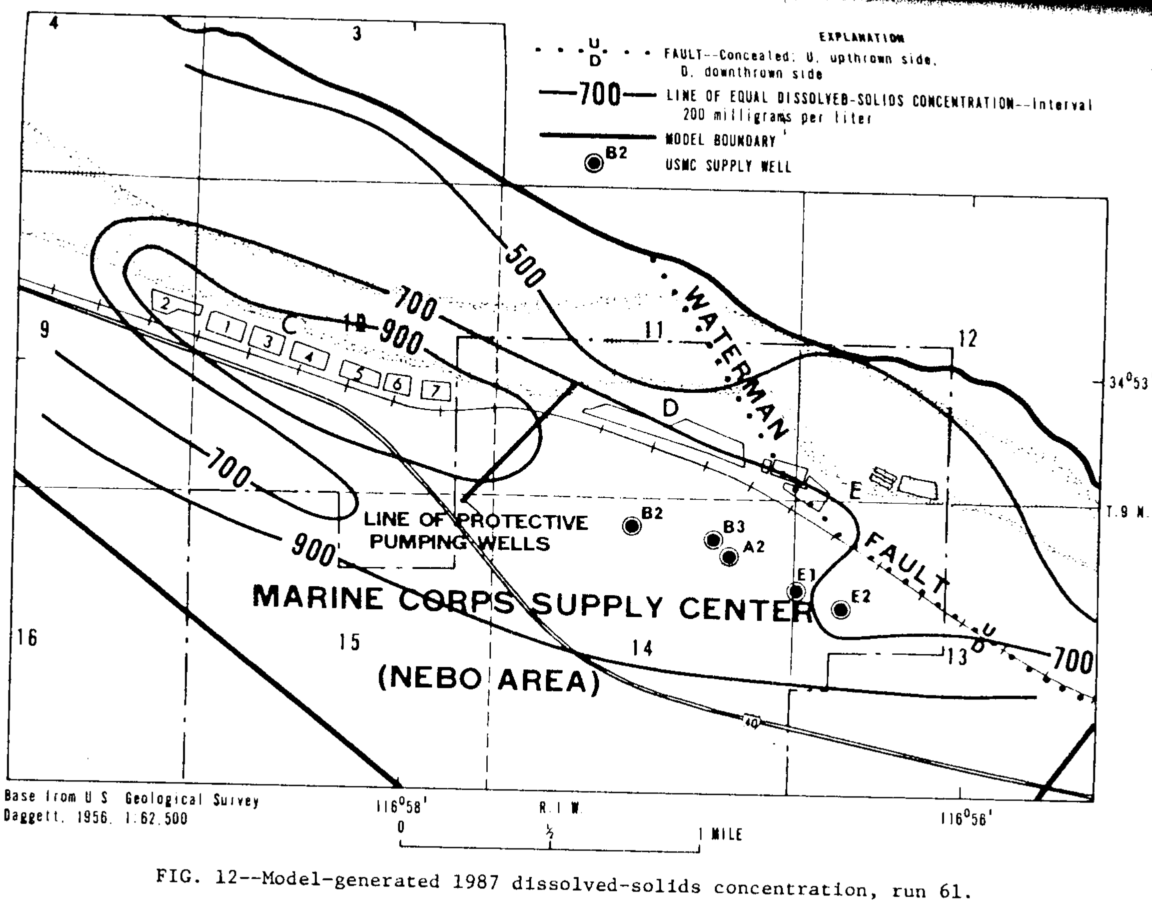


FIG. 12--Model-generated 1987 dissolved-solids concentration, run 61.

CONCEPTS AND INVESTIGATIONS

EDP AS AN AID FOR DECISION MAKING IN SUBSURFACE INJECTION OF LIQUID WASTES[1]

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ABSTRACT Subsurface-disposal feasibility was evaluated on a reconnaissance basis over a 1,785-sq mi area in southwest West Virginia. Data from 3,000 to 4,000 drillers' logs, 150 geophysical logs, and 1,033 hydraulic-fracturing records were transferred to computer cards by two full-time geologists and a key-punch operator over an 8-month period.

Computer-plotter-drawn maps were produced of maximum depth of fresh water, minimum depth of salt water, structure, thickness, fracture-pressure gradients, storage capacity, and oil- and gas-well locations for approximately 16 formations.

The use of electronic data processing (EDP) allowed the accommodation of much greater amounts of basic data, a more voluminous and diversified series of maps, and a more thorough reconnaissance study than would have been possible otherwise. In addition, EDP will serve in rapid evaluation of future disposal-well permit applications in the study area.

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[2]West Virginia Geological Survey.

The writers acknowledge the support of Robert B. Erwin, Director and State Geologist, West Virginia Geological and Economic Survey, who initiated the project, and the U.S. Bureau of Mines, which financed the project under USBM Grant Agreement No. G0122075.

Mary C. Behling and Richard A. Drabish, the principal research assistants for the project, spent many hours "translating" geologic data in conventional form and of variable quality into a form that could be processed by a computer. Others who assisted with the coding and data acquisition were S. M. Brock, Jr., J. B. Foster, J. A. Quagliotti, and R. J. Watts. Louanna Stenger key-punched the data.

The instructive comments and criticisms of an early draft of the paper by R. A. Brant, J. G. Craig, R. L. Dodd, M. T. Heald, T. E. Huzzey, W. K. Overbey, Jr., D. G. Patchen, Neilson Rudd, R. J. Watts, and B. M. Wilmoth were very helpful. J. A. Carte edited the final draft for the Survey and Barbara Wilson typed the manuscript.

Total computer costs for all phases of the project combined were than $3,500. In comparison, the addition of several full-time staff bers would have been necessary to accomplish conventional, noncompute generated results in the same time period.

Although EDP in no way makes up for inadequacies in basic data, is it a substitute for sound geologic judgment and experience, it can properly used, greatly facilitate the decision-making process which fa governmental agencies dealing with subsurface liquid-waste-injection posals.

INTRODUCTION

During its 75-year existence, the West Virginia Geological Survey has become the central storehouse for state geologic and mineral-resour data. Whereas the scientific assessment of subsurface waste injection lies within the charter responsibilities of the Survey, in 1969 the Survey became by act of law an official advisor to the Water Resources Division of the West Virginia Department of Natural Resources, which the state regulatory agency for subsurface injection of liquid wastes.

The Survey undertook a pilot study in 1972 to determine the feasi bility of subsurface injection of industrial liquid wastes in the stat The study, financed principally by a grant from the U.S. Bureau of Mi had three basic objectives:

1. The general assessment of subsurface injection of liquid waste in the pilot study area (Fig. 1) with regard to available reservoirs (Figs. 2, 3), their storage potential, and the protection of groundwat and other mineral resources.

2. The development of information sources and systems to provide adequate data for decision making regarding specific existing disposal wells and applications for permits for proposed wells.

3. Assessment of the adequacy, applicability, limitations, and u fulness of the Survey's data base, regarding West Virginia's policy regulatory procedures for subsurface waste disposal.

Electronic data processing and map generation were chosen as th means of information assimilation and output for the study. This pa is a description of the EDP operation.

The data base used in this study included previously compiled surface geology, 3,000-4,000 drillers' logs, 150 geophysical logs, 1,033 tabulated hydraulic-fracturing reports. With reference to th base, EDP has several advantages. Using it, people can work efficien

as a team to analyze subsurface data, discuss problem areas, and minimize individual bias. Several people can work simultaneously to assimilate data to be posted on one map or several maps without interference and without losing team efficiency. Subsurface geologic data are compiled well by well for the entire section--not well by well, horizon by horizon. Thus, only one "pass" through the data is necessary to compile all drilling information on all formations. Maps can be generated at will for one or all formations. Repetitive manual computations on raw data are not necessary. Editing of data is greatly facilitated. Data can be corrected, changed, expanded, or deleted at will; thus, data are current. A "current" or "edited" map or tabulation can be produced with minimal effort. No recompilation of data is necessary as with manual methods. Very large volumes of data can be used effectively.

COST

Computer time and/or system rental or purchase cost can appear staggering to an administrator without prior experience with EDP operations. Computer time equal to a man-year of manual operations costs only about $2. However, one cannot purchase 2, 5, or 10 years of output. Administrative restraint in curbing runaway objectives for the project is the best control. Costs can easily spiral beyond the value of immediate goals for the project.

As an example, computer costs for this project thus far total less than $3,500. This cost includes both developmental and utilization expenses for computer methods of data storage and retrieval for three data types on different storage media, data editing, data subset generation, data transformation and repetitive computations, and map making.

Development alone of any point listed above could have exceeded the total expenditures to date. With the exception of the computer-mapping routines, every program written and every computer-use system was kept as rudimentary as possible. The objective was geologic information, not computer products. Computer "overkill" was strictly avoided.

Two computers were used: an IBM 360/75 at the West Virginia University Computer Center, Morgantown, and an IBM 360/65 at the USGS Computer Center, Washington, D.C. The data base consisted of 28,000 cards that were also stored on magnetic tape and disc at the WVU Computer Center. From this data base, subsets totaling 41,000 cards were generated at WVU for map making. Data subsets were relayed to the USGS computer via a DATA 100 model 70 batch terminal and acoustic coupler at a 1,200-bps rate.

Total cards read into both computers was 280,000 for all jobs, lines of output totaled 370,000, and central-processor time totaled 4.642 hours. Plotting was done in Washington, D.C., using the CALCOMP GPCP contouring program and 30-in. drum plotter. Plotting time is estimated at 40 hours. Eight programs were written for data manipulation at the WVU Computer Center.

PERSONNEL

This project requires personnel who are geologists first and foremost. Computing and statistical skills are incidentals for personnel, as compared with the need for stratigraphic interpretation, knowledge of geophysical logging, hydraulic fracturing, etc.

Some computer knowledge is necessary for a member of such a project. He must be able to seek and converse with people highly versed in computer skills when high levels of skill or experience are necessary. In addition, he must be aware of the time frame for a specific computer application. Will it take a week, a month, or 6 months?

Perhaps the greatest personnel requirement is that of character or personality. Large volumes of geologic data had to be transcribed for coding and key-punching. Such work is frankly boring, even insulting, to some geologists. However, it must be done by competent geologists aware of the stratigraphy and structure, able to recognize mistakes, and able to interpret data correctly and to make a "firm" judgment on the "vague" geologic data. The ability to pursue computer problems (computer knowledge notwithstanding) until they are solved is also a prerequisite. Finally, the ability and persistence to edit faithfully reams of computer output are a necessity.

Personnel for this project consisted of three full-time employees. Two were geologists who coded data. They had no computer contact or responsibility other than entering data on appropriate forms and editing output. The third member acted as project coordinator. He wrote the necessary programs, consulted specialists when necessary, and in general oversaw scheduling and operations. Cooperative consultants at the computer centers greatly aided the operation. Requisite computer experience for project personnel is reduced if consultants are available. Our consultants were contacted at the set-up stage for the tape and disc systems and when the computer-mapping routines were first used. They were again contacted when data volumes grew so large as to create systems problems.

LOGISTICS

Large volumes of data were assimilated. Storage of the reams of output and boxes of cards is usually a very minor problem, but utilization of such output is not. Repetitive editing of paper and card output can be a nightmare. From the outset our data were planned in five forms.

Raw data existed in reports and logs and in tabular form. These were coded by geologists who initialed the coding sheets. Any question of interpretation could thus be traced back to the source. Coded sheets were keypunched on cards and verified to minimize errors. Card data were then read onto disc files. Repetitive editing and additions to the data necessitated direct-access (disc) files. Edited data were then read onto tapes, and the tapes were used exclusively in data manipulation.

Such a system employs the speed, low cost, and high-volume storage of tape. The disc permits ready editing of data item by item and makes it possible to add data and to update the tape. The disc also provides data security in case of loss or expiration of borrowed tapes or tape failure. The cards were our only input means to the disc file. The use of tapes and discs greatly speeded operations and lowered costs. Savings of $30 per job were typical when tapes were used instead of cards.

Computer-drawn maps posed several problems. A cooperative project with the U.S. Geological Survey permitted us to use the computing facilities of their Washington, D.C., computer center to generate maps. Cards were the sole input method via the DATA 100 batch terminal. Jobs averaged about 2,000 cards. By remote transmission, input took 15 minutes. Retrieval of data required 30 minutes per map to output to the line printer. The output was checked and, if acceptable, a request to have the map plotted was telephoned to the USGS Computer Center. The completed map or maps were then sent to us by air mail. Turn-around time from job input to receipt of a map averaged about 10 days. Costs per map ranged from $16 to $25.

Hardware requirements for this project were met by the West Virginia University Computer Center and a federal FTS telephone terminal, which were both located about 2 mi from our building. Transportation of data cards and output between these locations and turn-around time hampered the project greatly. However, hardware purchase or rental was not justifiable for this project alone.

COMPUTER EFFICIENCY VERSUS GEOLOGIC NEEDS

Optimum computer success and efficiency necessitate precisely defined goals. If possible, every type of output desired should be planned programmed from the beginning. Unnecessary data should be eliminat and data to be used should be simplified and compacted. However, ge needs often involve not knowing all paths leading to final goals. Co quently, to uncover useful paths to final geologic goals, the geolog must be willing to sacrifice some computer efficiency.

In our initial computerization, no data were eliminated for thr reasons: (1) machine editing of data is far faster than manual sorti prior to coding; (2) "superfluous" data can be deleted by the compute in data manipulations at a very minor cost; and (3) data must be in th system before they can be utilized.

Data originally thought "superfluous" to our project, but which had computerized, became pertinent as the project proceeded. We found source documents critically lacking in some type of information well the project was under way. For example, petroleum-production records included no data necessary for calculation of reservoir properties. Furthermore, hydraulic-fracturing records contained no information on actual bottomhole pressures or sustained bottomhole treating pressure and sufficient data for more than an estimate of these parameters were not available.

With regard to data simplification and compaction, again some sac in computer efficiency is justifiable. Commonly, coding forms are mad as compact as possible with as few blank spaces as possible, and fiel sizes (the number of spaces in which to record data such as well dept etc.) are made at the assumed minimum space necessary. An abnormally large but valid number may not fit in a field so restricted. No addi- tional information or codes may be added if sufficient space is not present. Recompilation may be necessary to include data that must be inserted within a record. This is easily done in the computer, but codi sheets and cards then become obsolete. Record lengths are commonly fi at a set size or number of cards. The data-storage system should be fl ible enough for record expansion. It is sound practice from a geologi viewpoint to include everything possible or reasonable in the data an provide ample, even slightly excessive, space in coding forms and da storage media.

Coding of oil- and gas-well data in general followed the points dis cussed. As many data as possible about each well were included. Info tion from more than 4,000 wells was recorded and checked by two geolo in approximately 7 months.

EXAMPLES OF DATA OUTPUT

Figure 4 is an example of a structure-contour map. The "busy look" --i.e., crenulated contours and numerous minor closed structures--is a function of the density of data control. Programming instructions could smooth the map as desired to produce a "hand-drawn" look, with subsequent loss of accuracy. Topography superimposed on the structure-contour maps would be lost. Although not illustrated, isopach maps would similarly loose topography, and lithotope geometry also would be lost. Forty such maps were made initially of potential disposal reservoirs. Cost for additional maps would be the $16-$25 computer fee plus personnel salaries for staff members.

Figures 5 and 6 are computer-plotter-generated contour maps of formation breakdown pressures. Compilation of the 1,033 hydraulic-fracturing records, the source data, took approximately 2 months. Sufficient engineering data were not present for accurate use of all fracturing information. Machine editing sorted data before computation, thus eliminating slow manual sorting.

Figures 7 and 8 are examples of maps of storage capacity in porosity-feet. These data were compiled from geophysical logs by the U.S. Bureau of Mines over a 4-month period. Coding of the data took approximately 2 weeks. Sparsity of data produces nebulous computer-plotter-generated maps such as Figure 7 and 8. Areas shown with no data are questionable continuations of surrounding trends.

Figures 9 and 10 are maps of the maximum depth to fresh water and the minimum depth to salt water in the study area. This information was gathered from the same well records used for geologic information.

For all the operations cited above, data assimilation was by far the most laborious process. It was the principal cost for the project in salaries for interpretation and coding, for key-punching, and for loading into the computer and editing. Actual manipulation and map making by computer were minor expenses and efforts.

Meaningful cost comparison between manual and EDP operations is difficult. Although the principal project expense was in data assimilation into computer-usable form, similar data reduction would be necessary for a strictly manual operation. A manual operation would necessitate a separate compilation of all data for every map, tabulation, or calculation. A reconnaissance-type project such as this one, dealing with large volumes of data of limited utility, would not have been manually feasible in the same time period using even a fraction of the data analyzed by computer.

SUMMARY

EDP techniques are fast and inexpensive in a reconnaissance-type study for subsurface waste injection. The ability to assimilate vast amounts of data with ease and clarity is an indispensable aid to decis[illegible] making in subsurface injection of liquid wastes.

Maximum bottomhole injection pressures in West Virginia have been set at 0.8 psi per foot of depth. EDP has made possible examination of all available hydraulic-fracturing data in the pilot study area, and thi[illegible] somewhat arbitrary figure has been shown to be unsafe in certain areas and overly restrictive in others. Future decisions on injection-pressur[illegible] limits for specific areas should be guided by this information.

Estimation of the minimum areal spread of injected liquid wastes can now be made for certain areas based on actual storage-capacity maps made from the compilation of geophysical logs. Decisions about waste encroach[illegible] ment into undesirable areas are facilitated.

In the assessment of subsurface disposal, EDP has made possible a synthesis of data sources. We now know, essentially, the total of our data base, the areal coverage of this data formation by formation, and the areas in which we lack data.

EDP techniques are also helpful when dealing with specific disposal wells or proposed wells. The West Virginia Geological Survey has approxi[illegible] mately 2 weeks in which to evaluate a proposed disposal well and formula[illegible] and submit recommendations. EDP makes possible a thorough data search in this time period and greatly facilities the decision-making process.

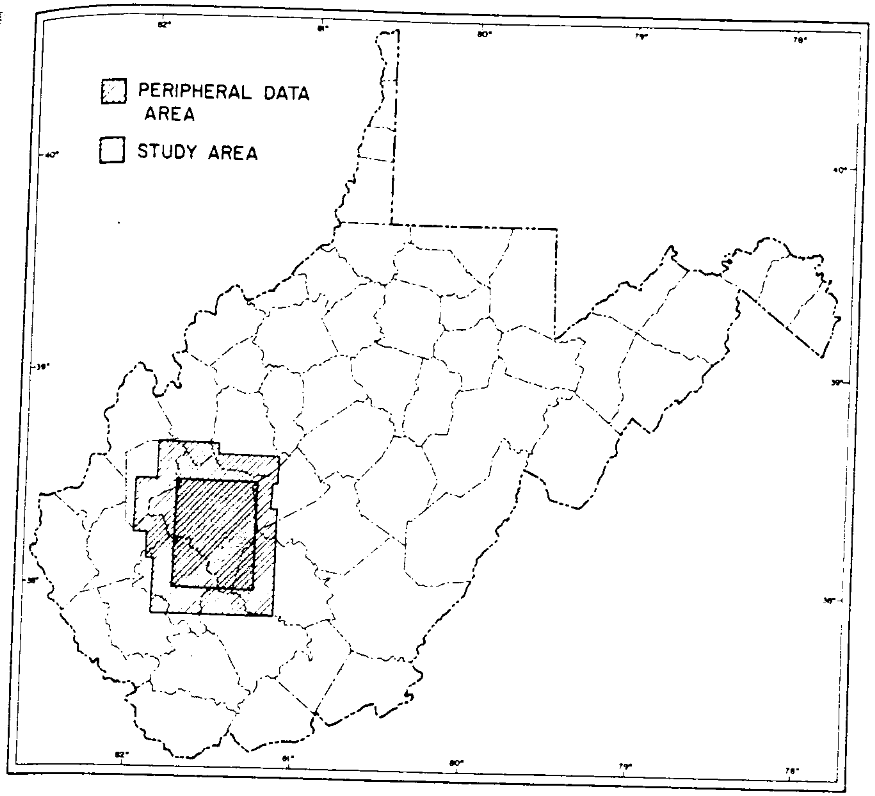


FIG. 1--Study area.

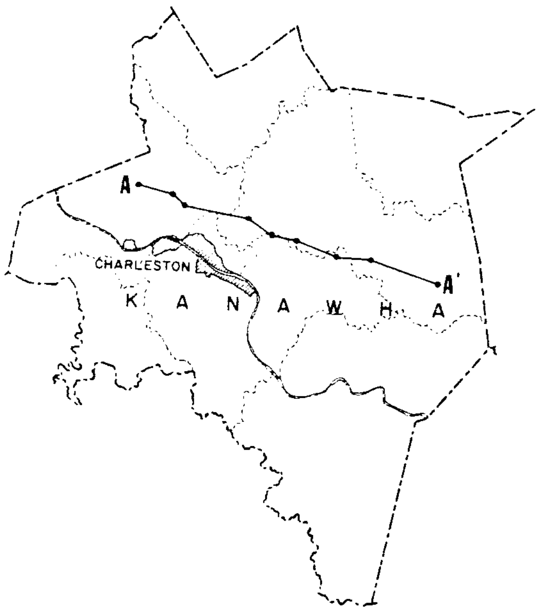
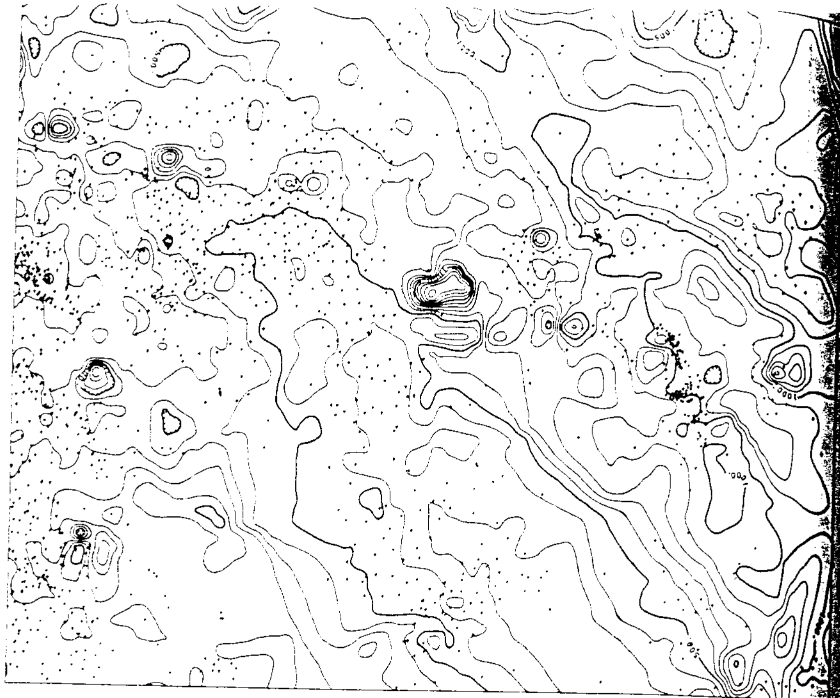
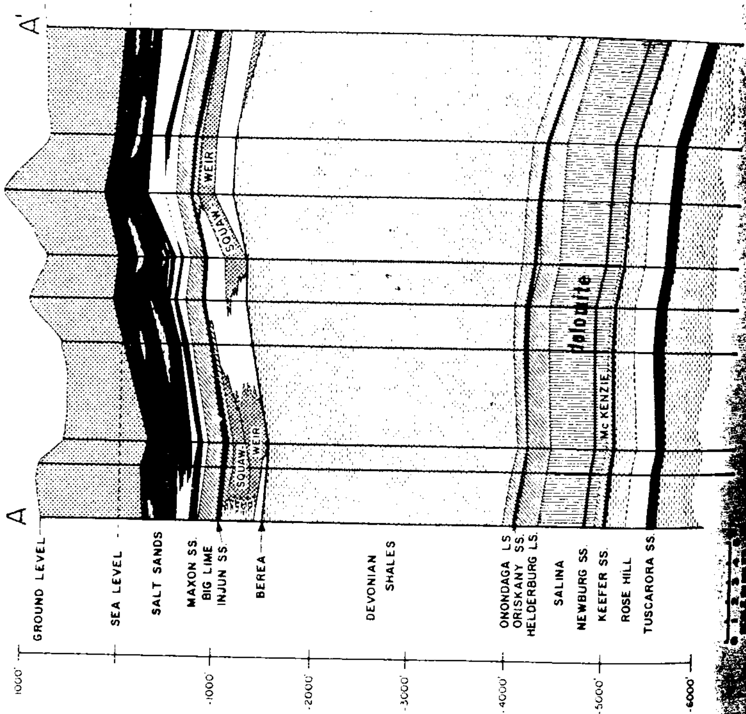


FIG. 2--Location of cross section A-A'.





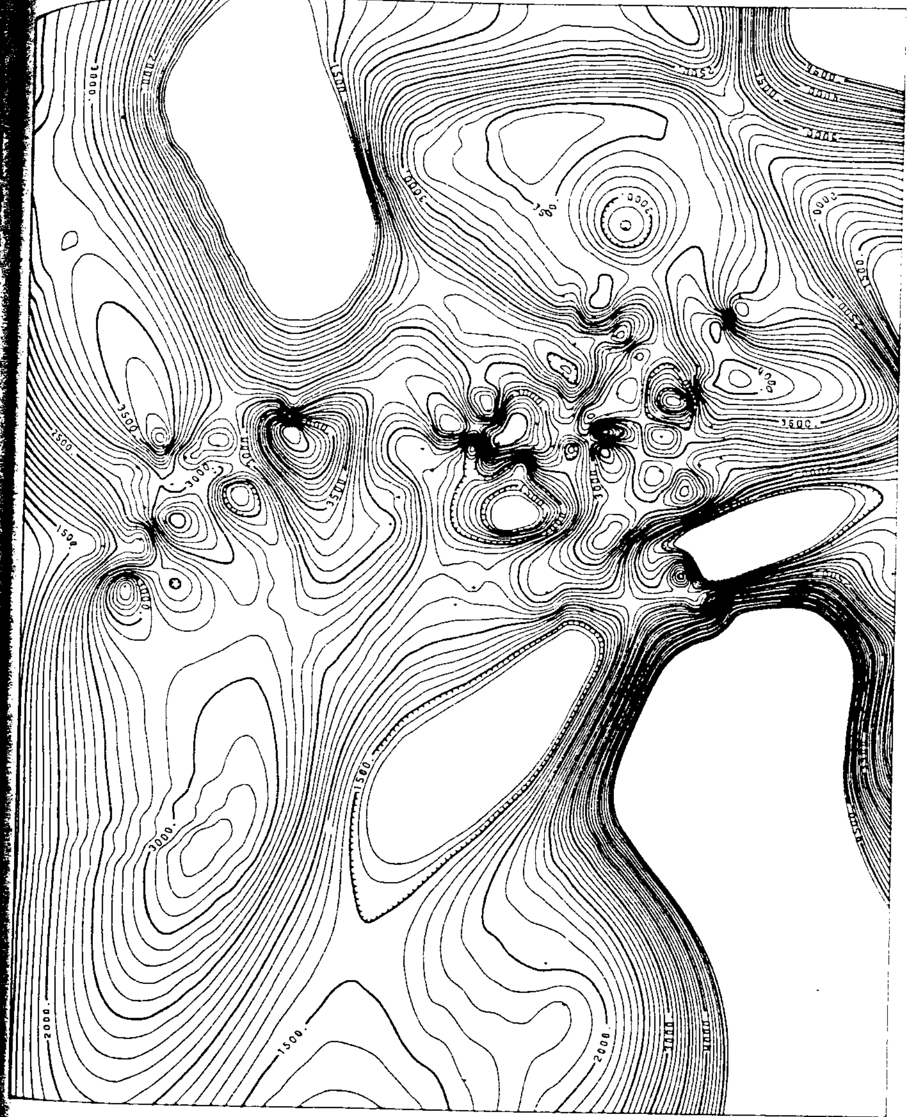


FIG. 5--Injun breakdown pressure.

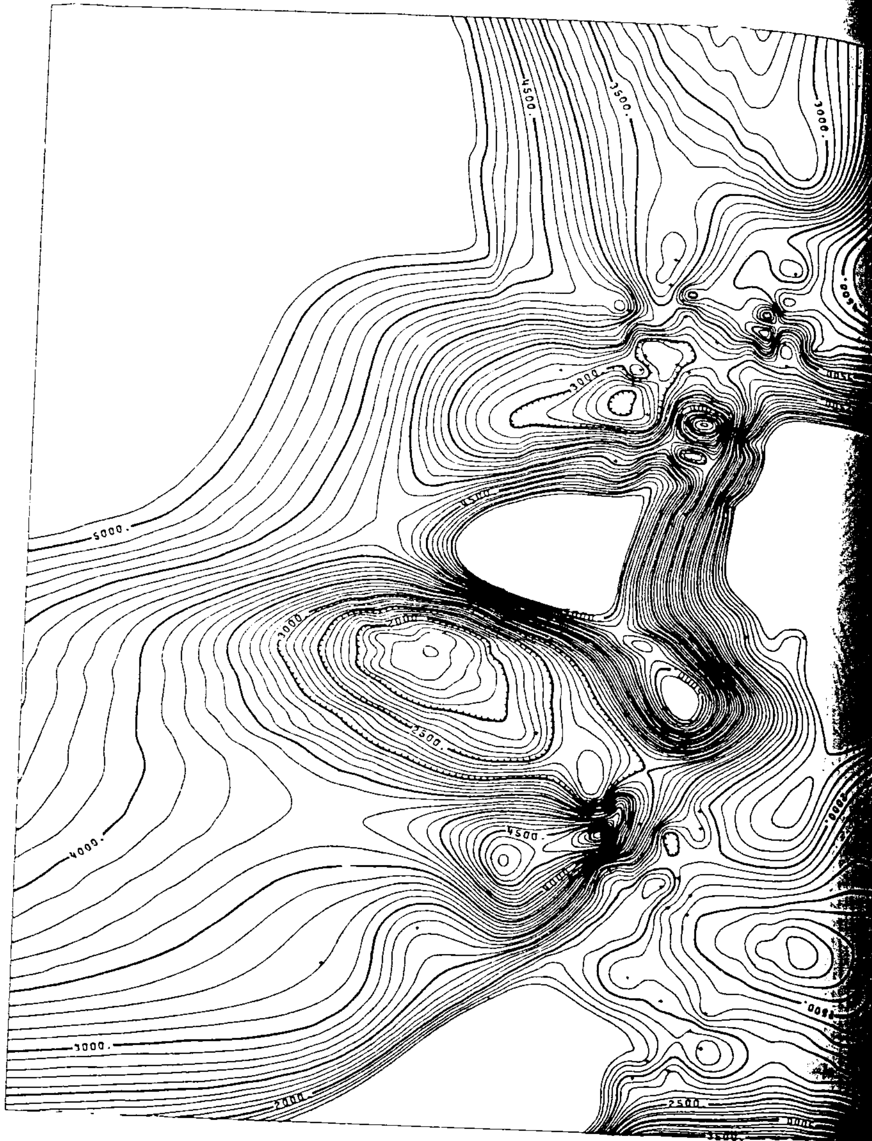


FIG. 6--Weir breakdown pressure.

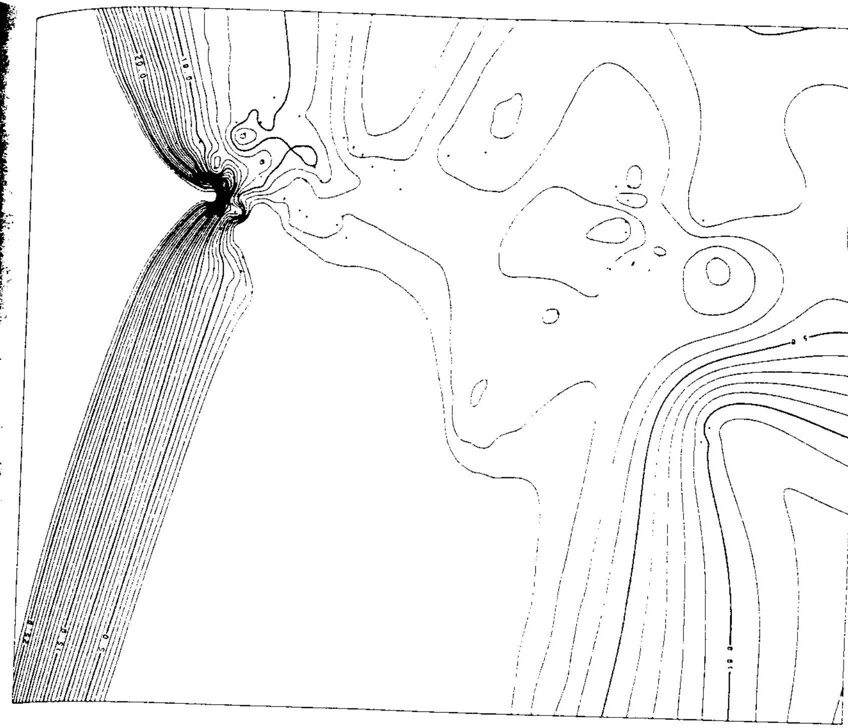


FIG. 8--Weir storage capacity in porosity-feet.

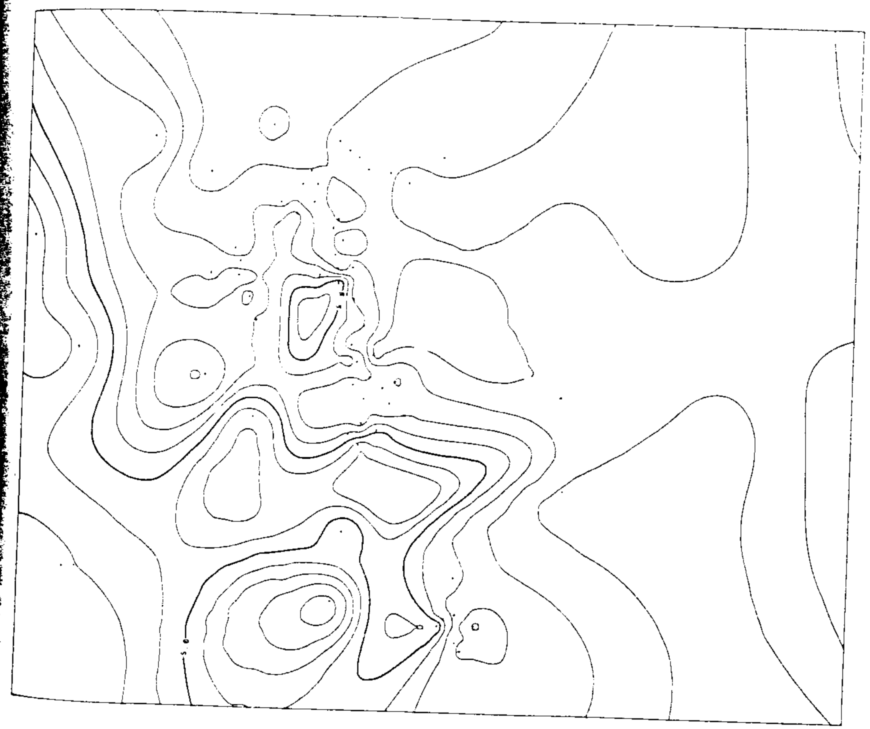
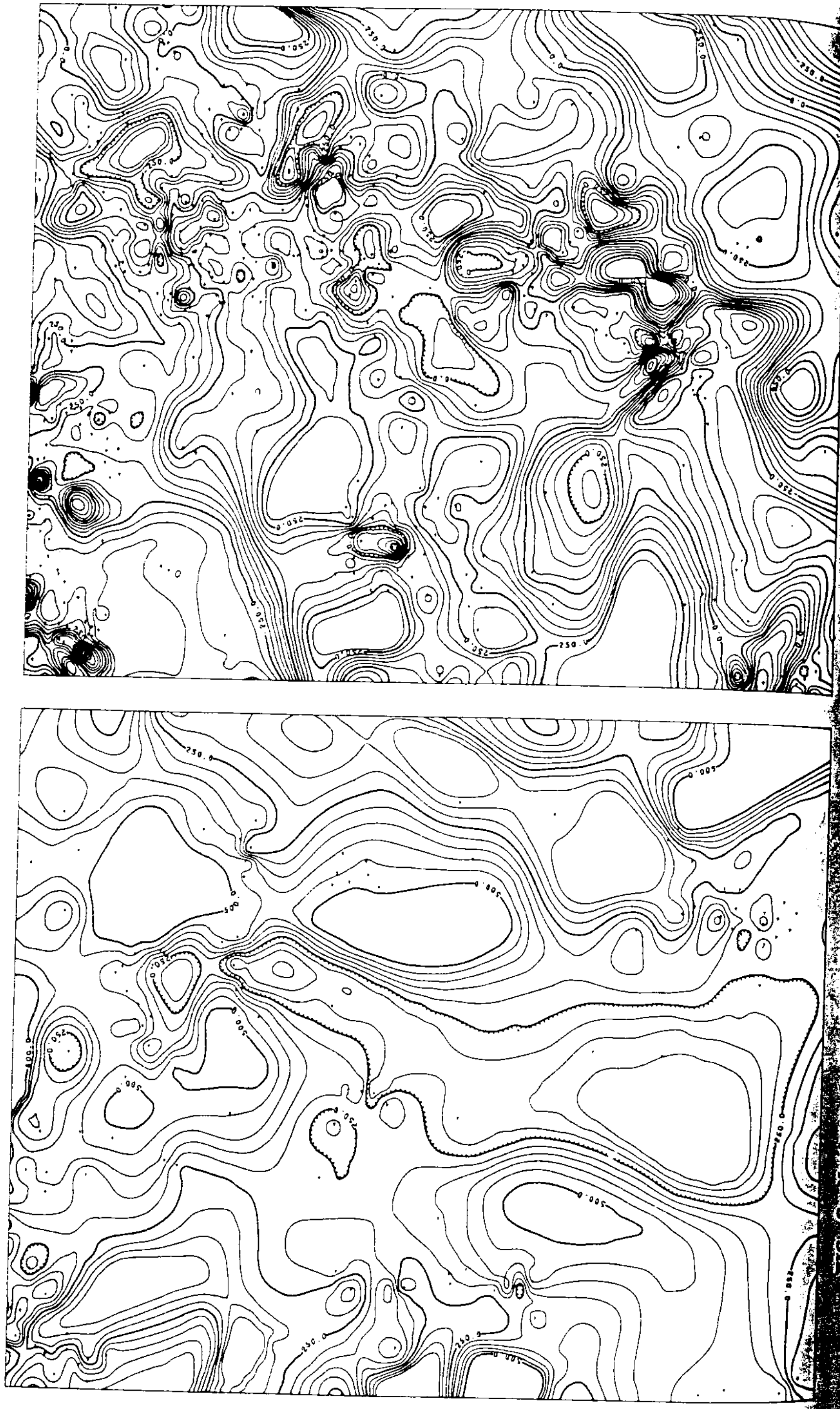


FIG. 7--Injun storage capacity in porosity-feet.



ROLE OF BOREHOLE GEOPHYSICS IN UNDERGROUND WASTE STORAGE AND ARTIFICIAL RECHARGE[1]

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ABSTRACT The optimum utilization of underground space for the emplacement and storage of waste and surface water can be achieved through an understanding of the geohydrologic environment. The emplacement of liquid waste and the artificial recharge of aquifers generally requires the drilling of exploration, injection, and monitoring wells. Some geophysical logs are run on almost all wells drilled for deep disposal. Even more information could be obtained, however, by utilization of all available logging techniques. To date, geophysical well logging has not been applied widely to artificial-recharge projects; however, borehole geophysics is being used by the U.S. Geological Survey to study geohydrologic parameters related to recharging the Ogallala Formation on the High Plains of Texas.

Geophysical well logs provide preinjection data necessary for the selection of environments for liquid-waste or water storage. Logs provide data on the location, thickness, and lateral continuity of storage zones and confining beds, percent and distribution of total or effective porosity, and the relative magnitude of permeability. Intergranular and fracture porosity can be discriminated by cross-plotting acoustic-velocity and neutron or gamma-gamma logs. The distribution and orientation of preinjection fractures can be determined by acoustic televiewer logs. Logs provide data on the chemical quality of the native fluids and the mineralogy of the aquifer, which are necessary to predict chemical reactions with injected fluids. The temperature and conductivity of the interstitial fluids may be measured directly and their specific gravity and viscosity may be calculated from log data.

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[2]U.S. Geological Survey.

Because aquifers overlying injection zones can be polluted by improper well construction or well failure, geophysical well logs should be used to guide the design, construction, and maintenance of injection and monitoring wells. It is important to answer questions such as: are the casing strings and screens properly installed; are they plugged or corroded; does the grout fill the annular space and is it properly bonded to the casing; and, are there leaks through the casing, between pipe strings, or through the annulus?

After waste injection or artificial recharge has started, logs provide in-situ measurements of changes in the system. We have found that an increase or decrease in porosity caused by solution or precipitation in pore spaces or cavities may be detected. This type of information not only explains changes in well efficiency but provides the basis for selecting remedial treatment. Accidental hydraulic fracturing caused by drilling or injection can be the reason for vertical leakage through confining beds, and acoustic televiewer logs can locate these fractures. The distribution and velocity of injected water and the location of chemical or thermal pollution may be determined by means of temperature logs. We have used temperature logs to map the horizontal and vertical distribution of injected fluids. Diurnal thermal changes in injected water provide the basis for measuring the velocity of flow and its change with time.

INTRODUCTION

Borehole geophysics includes all in-hole wire-line measuring techniques. Geophysical well logs are continuous analog or digital records of a parameter that can be interpreted in terms of the lithology, geometry, resistivity, formation factor, bulk density, porosity, permeability, moisture content, or specific yield of water-bearing rocks. Logs are also an aid in defining the source, movement, and chemical and physical characteristics of groundwater and injected fluids. The purpose of this report is to relate logging techniques that are commonly used in petroleum exploration to the problems associated with the underground storage of liquid waste and the artificial recharge of aquifers. Because some of the problems and particularly the history of application of borehole geophysics are different for waste storage and artificial recharge, they are treated separately in this report. The section on waste storage is based partly on information from published reports, whereas the section on artificial recharge is based almost entirely on a U.S. Geological

Survey research project studying the Ogallala Formation of the southern High Plains of Texas.

LOGGING AS APPLIED TO WASTE STORAGE

There are three basic requirements for the successful waste-injection system: (1) the disposal zone must be porous, permeable, and of sufficient volume to accept and contain the effluent for the required time at safe injection pressures. (2) The disposal zone must be bounded above and below by relatively impermeable confining beds to prevent the contamination of groundwater supplies or other natural resources. (3) Injected waste, interstitial formation water, and minerals in the disposal zone must be chemically and physically compatible so that the effectiveness of the disposal system will not be diminished by chemical and physical changes. A further important requirement is that injection and monitoring wells must be completed so that they will not short-circuit the confining beds. Borehole geophysics has been used to provide data relating to all of these important factors, and the utilization of well logging to solve problems related to underground fluid injection will undoubtedly continue to increase.

Published reports on the use of borehole geophysics for the location and the design of industrial waste-disposal wells are few. In contrast, reports of radioactive-waste disposal prepared under the auspices of the U.S. Atomic Energy Commission provide many data on the use of borehole geophysics. Many such disposal studies have shown that the extrapolation of laboratory values to the field will lead to errors in the prediction of the behavior of waste in the subsurface. In-situ measurements can help bridge this data gap and will also provide information on changes in the environment caused by waste storage. Ives and Eddy (1968) report that a number of states require the filing or submission of electric logs or well logs with applications for deep-well waste-disposal permits. We anticipate that this practice will become universal. Recommendations by Ives and Eddy include the submission to the regulatory agency of a "suite of mechanical logs" and temperature and bond logs necessary to verify the cement job. Warner (1969) recommends that "complete logging and testing of wells intended for injection should be required." A suggested completion report on waste-injection wells includes a section for listing the logs run including gamma-ray, neutron, temperature, caliper, cement-bond, resistivity, spontaneous-potential, and others. Warner also included a table summarizing the information needed in the evaluation of subsurface disposal sites and methods available for obtaining that information, lists the following categories of information that can be derived from various types of geophysical well logs: porosity, permeability, geologic formations intersected by the drill hole, thickness and character of the disposal zone, temperature of the formation, and amount of flow into various intervals. The report further recommends periodic checking of casing and tubing for corrosion and other defects. Inspection of this kind can be accomplished by periodic logging.

This report briefly summarizes the categories of information required to solve waste-injection problems and the ways in which various types of logs might provide this information. Because of space limitations we cannot describe the logging techniques in detail. For more detailed information the reader is referred to the standard text books (Lynch, 1962; Pirson, 1963) on borehole geophysics and the publications referenced. A very useful annotated bibliography presents information on subsurface waste-disposal by means of wells (Rima et al., 1971).

Lithology and Stratigraphic Correlation

Potential disposal zones and confining beds can be identified and the lateral continuity of both can be determined with the aid of a number of geophysical logs. These include natural gamma-ray, neutron, gamma-gamma, spontaneous-potential, caliper, and various types of acoustic and resistivity logs. The ultimate goal of an investigation of a potential disposal site is to predict the movement of waste. An understanding of the lithology and the geometry of various stratigraphic units is essential to understanding waste movement, but no logging technique will permit exact predictions. The dispersion coefficient, which is necessary to calculate the distribution of waste in the underground, is related to the non-homogeneity of the aquifer and confining beds. The relative degree of dispersion can be estimated from logs, but at the present time it is not possible to obtain a dispersion coefficient from the interpretation of geophysical logs.

P. H. Jones (1961a, b) describes an excellent example of the use of borehole geophysics in the understanding of a waste-disposal environment. He used electric, gamma-ray, caliper, fluid resistivity, temperature, and flowmeter logs to describe the geology and hydrology of the very complex basalt aquifer systems at the National Reactor Testing Station in Idaho (NRTS). Prior to the use of logging it had not been possible to correlate the basalt flows and interflow sediments in the Snake River

plain. By use of a combination of caliper and natural gamma-ray logs, Jones was able to identify and correlate interflow sediments, select the most permeable aquifers, and construct cross sections of a very complex system (Figs. 1, 2). He used temperature logs and fluid-resistivity logs to trace the movement of waste water in the basalt sequence. Flowmeter logs were used to determine the vertical movement of water in a multi-aquifer well. Well logs were essential to the drawing of structure-contour and aquifer-thickness maps and to an understanding of the distribution of wastes injected in the environment.

Marsh (1968) shows a simple but more typical example of the use of electric logs in evaluating a disposal-well system. Resistivity logs show the freshwater aquifers, the underlying saltwater aquifer, and the aquitard, and illustrate how the screened interval was selected on the basis of logs. In order to maximize lithologic data obtained from geophysical logs, it is recommended that several different types of logs be run, each measuring a different aspect of lithology. Logs should always be correlated to the nearest well where cores or sample data are available, because log response is not unique and background information in a new area is essential to correct interpretation.

Porosity

Porosity data must be available in order to evaluate a potential waste-storage environment, not only because of its relation to the underground volume required, but because of the effect on fluid velocity and dispersion. It is important to know the type of porosity--whether it is effective or total, intergranular or fracture--and the distribution of pore spaces. The effect of well development and injection operations on effective porosity can also be interpreted from logs. Porosity may be interpreted from neutron, gamma-gamma, resistivity, or acoustic-velocity logs. It is suggested that at least two types of these logs be run and cross-plotting be used in order to obtain the most accurate values for porosity. Furthermore, data from laboratory analyses of cores will permit more accurate calibration of logs. Pickett (1968) describes the combined interpretation of acoustic-velocity and neutron logs to determine reservoir porosity in the now-famous Rocky Mountain Arsenal disposal well. He pointed out that porosity was obviously important in the calculation of reservoir volume and area, and estimated a maximum porosity of 6 percent from the logs. Although 70 ft of the well was open to injection, no injectivity profiles were made to measure the actual thickness of the disposal zone at the well or to detect movement up the annulus; therefore thickness of the disposal reservoir is questionable.

Permeability

Although permeability of the reservoir to the injected waste is one of the most important parameters in investigating a potential disposal site, no geophysical log measures permeability directly. We have used injectivity profiling with radioactive tracers to provide information on the relative magnitude of permeability in a potential disposal environment (Keys, 1967). In the same way, flowmeter logs made during pumping or injection of water indicate the most permeable zones based on their calculated rate of fluid acceptance. The calculation of water-injection profiles from temperature logs is described by Nowak (1953). More recently Cocanower et al. (1969) describe how the digital recording and computer interpretation of temperature logs can be used to provide precise data on zones of water injection. Our own research on the use of temperature logs to estimate permeability during artificial recharge is also applicable to waste-disposal investigations.

The velocity of waste movement is an important parameter related to permeability. The point-dilution method may be used to estimate groundwater velocity and hydraulic conductivity (Lewis et al., 1966). Inoue (1967) employed sodium chloride solution as a tracer and an in-hole packer unit with built-in electrodes for measurement of changes in concentration. The equation for estimating groundwater velocity under existing head conditions is as follows:

$$V_{gw} = \frac{\pi d}{8t} \ln \frac{c_o}{c} ,$$

where d = well diameter, c_o = initial and c = final tracer concentration, and t = time. Inoue found that the point-dilution method provided velocity values that agree fairly well with calculations based on Darcy's Law.

Logging can provide data on groundwater velocity between wells if detectable tracers are injected in one borehole and searched for in a second borehole. Although tritium is the most nearly perfect tracer, it is only detectable by sampling and laboratory analysis. Gamma-emitting radioisotope tracers are preferable because they can be measured through casing, allowing the determination of vertical variations of velocity.

Biershenk (1969) describes the interrelations among permeability, porosity, and velocity as follows:

$$P_f = \frac{7.48 \phi V}{I} ,$$

where P_f = the field coefficient of permeability in gal/day per ft^2, ϕ = effective porosity, V = groundwater velocity in ft/day and I = hydraulic gradient in feet per foot.

Fracturing

Fracturing is a major problem in waste injection because of the likelihood that it will occur during either drilling or injection operations. Accidental fracturing during drilling, or fracturing due to excess injection pressure, may allow the migration of waste beyond the boundaries of the storage zone. Intentional hydraulic fracturing may be used to increase the permeability, or as a means of emplacing waste in grout (deLaguna, 1970). The importance of fracturing to waste injection can be seen from the fact that fracturing often occurs at 0.6 to 0.7 times the weight of the overburden and sometimes can occur at a pressure as low as 0.5 times the weight of the overburden, or approximately 1 $lb/in.^2$ per foot of depth.

Siple (1964) pointed out that acoustic-velocity and caliper logs were used to determine the distribution of fractures in crystalline rock being considered for waste storage under the AEC Savannah River Plant, South Carolina. It was not possible, however, to determine the water-transmitting properties of these zones by geophysical logs, and tracers were used to measure velocity between wells (Marine, 1966). Injectivity profiles with radioactive tracers were later used to determine the relative magnitude of permeability of the various fracture zones in a single well (Keys, 1967).

It is important to locate and measure the orientation of both pre-injection and post-injection fractures in order to determine the state of stress in the rocks and to predict leakage through fractures in confining rocks. Prior to the invention of the acoustic televiewer, the orientation and location of fractures were determined by inflating a special type of packer at the suspected fractured interval and obtaining an impression of the fractures. A magnetically oriented borehole television camera can also be used if there is clear water in the hole. The televiewer provides a continuous, magnetically oriented record of fractures and other openings regardless of whether the borehole is filled with clear water, liquid waste, brine, crude oil, or drilling mud (Zemanek et al., 1969). Zemanek et al. (1970) cited an example in which a vertical fracture was mapped for more than 300 ft by means of an acoustic televiewer log (Fig. 3). This fracture passed through both sand and shale and is thought to have been induced by drilling. The fracture was not evident on gamma-ray, caliper, acoustic-velocity, or resistivity logs. A fracture of this type might prevent isolation of waste-storage zones from aquifers producing fresh water.

The empirical relation between the velocity of acoustic waves and porosity has been utilized for some time. Carroll (1966) demonstrated the relation between acoustic velocities from logs, Young's Modulus, and the ultimate strength of rocks. Myung and Helander (1972) showed that Young's Modulus and shear modulus are both related to compressional-wave and shear-wave velocities. Young's Modulus is the parameter which is the most convenient and is frequently used in the field to investigate the state of stress within rocks. Periodic acoustic logging during waste injection permits the measurement of changes in elastic properties which are related to the fracturing of the rocks.

Pressure fracturing of relatively flat-lying shales has been used as a means of injecting radioactive wastes in grout. In one test of this procedure, radioactive tracers were added to the injected fluid so that gamma-ray logs could be used to locate the fractures in the injection and observation wells (deLaguna, 1972). In uncased holes, the acoustic televiewer would be a more effective means of mapping the fractures produced by injection.

Water Chemistry and Monitoring

Geophysical well logging provides techniques for measuring the quality of formation waters prior to injection and for monitoring changes in that quality as wastes migrate. Olmsted (1962) described the use of fluid-resistivity and temperature logs to map the distribution of groundwater types and to establish flow patterns and the distribution of waste as related to recharge areas and injection points at the National Reactor Testing Station. He also pointed out the important effect of differences in fluid density and viscosity on the paths and rates of wastewater flow.

Warm water from a disposal well may initially rise through the aquifer system because low density related to elevated temperature tends to offset high density caused by dissolved solids. As the waste water cools it may subsequently sink. Temperature is one of the most important

borehole parameters to measure in waste-storage projects because of its relation to chemical activity and to the viscosity effect on groundwater velocity. As an example, at a given hydraulic gradient, fresh water moves 23.6 percent faster at 67°F than at 49°F.

The thermal history of the Rocky Mountain Arsenal well is of interest because of the possibility that thermal stress was a triggering mechanism contributing to the seismic activity (Hoover and Dietrich, 1969). Temperature logs were used for correcting fluid density in this well, and for interpreting changes in transmissivity related to temperature. A temperature log of the Rocky Mountain Arsenal well made in 1968 showed a 25°C temperature inversion in the injection horizon from cold wastes injected in the period 1962-1966. Pumping the well removed this colder water and brought warmer water, which had reached equilibrium with the rocks, back to the vicinity of the bore. Such temperature changes produce thermal stress and may cause microfracturing of the rock matrix.

At the National Reactor Testing Station (NRTS), temperature and fluid-resistivity logs were used to map the distribution of waste from a deep disposal well (P. H. Jones, 1961b; Figs. 4, 5). Neutron moisture logs and gamma-ray logs were used to detect radioactive waste and monitor its downward and lateral movement and decay (Morris and Teasdale, 1964). A combination of the two logs distinguished meteoric water from waste water derived from a disposal pond. We have also used gamma-ray spectrometry in boreholes to identify the radioisotopes present in liquid wastes. Investigations with a radioactive tracer injector at the NRTS showed upward and downward flow in a single monitoring well, and demonstrated that changes in the in-hole flow system were caused by changes in the rate of waste injection or pumping in nearby wells (Barraclough et al., 1965). Information on borehole flow is most important in multi-aquifer wells. Without knowledge of in-hole flow, water samples may be taken from stagnant zones or at depth intervals that do not represent water in the adjacent aquifer.

Activation analysis in boreholes has been demonstrated as a qualitative technique for identifying several elements (Keys and Boulogne, 1969; Tanner et al., 1972). At the present time, borehole activation analysis is semi-quantitative at best. However, it does permit the monitoring of changes in the chemical composition of formation fluids through casing and may be an important technique for detecting non-radioactive tracers outside the casing. Pulse-neutron activation techniques permit the identification of short half-life daughter products and may permit the measurement of water flow outside the casing without the addition of a tracer (Wichmann, 1971).

The chemical effects of injected wastes on the disposal zone can be quite spectacular. Caliper logs of a 100-ft interval in a disposal well in Florida showed that the hole diameter increased from a pre-injection average size of less than 16 in. in 1966 to an average size of more than 30 in. in 1971 (Fig. 6). Other intervals of limestone penetrated by the same hole showed an increase in hole diameter of 1 to 5 in. (Black, Crow and Eidsness, Inc., 1972). This dramatic change was caused by the solution of limestone by acid wastes injected from 1969 to 1971. Reactions of this type are exothermic and a measurable amount of heat is produced. Heat due to acid solution may be detected behind the casing by temperature logging, if the borehole fluid is permitted to adjust to the temperature outside the casing.

Baetsle and Souffriau (1967) described a technique of injecting a chemical barrier into an aquifer in order to prevent the spread of an accidental discharge of waste. They used gamma-ray logs of tracer distribution before and after injection of the barrier in order to describe the heterogeneity of the aquifer and the effectiveness of the barrier. Probes to log Eh and pH and specific ions have been used experimentally at shallow depths and may eventually provide another technique for monitoring waste distribution in place. Where it is not possible to detect waste in situ with a logging probe, samplers that open and close in response to a DC voltage from the surface are available to operate on standard logging cable.

Well Construction

The extraneous effect of well construction on geophysical logs is an important factor to be considered in log interpretation (Keys and MacCary, 1971). Hole diameter, casing, cement, mud cake, and invasion may affect log accuracy. Where logging is to be an important part of a predisposal investigation, or where periodically run logs are to be used for monitoring changes in a disposal environment, wells should be designed for the maximum effectiveness of the logs.

A number of types of logs is also available for providing information related to the proper construction and performance of disposal wells. A direction or deviation log may be necessary to establish the degree to which the well has wandered from true vertical. Caliper logs are used to establish the volume of cement that might be required for grouting

operations, and gamma-gamma and temperature logs may be used to determine the position of grout behind the casing (Keys, 1963). Cement-bond logs are essential to establish the degree of bonding between the casing and annular cement and between the cement and formation. However, cement-bond logs may not provide useful data in casing 16 in. in diameter or larger (Ross Sproul, 1973, written commun.). Furthermore, cement-bond logs do not provide positive evidence that leaks through channels in the annular cement cannot take place. Radioactive-tracer techniques or temperature-logging techniques may be the only way of detecting annular leaks. Talbot (1972) described a disposal well where uphole annular leakage was suspected from improved injection performance. High-resolution temperature logs were used to show that leakage was not taking place.

The casing-collar locator can be used to confirm the depth of casing, tubing, screens, and perforations. The acoustic televiewer will also depict holes in the casing, and casing-corrosion logs are available commercially. The solution-mining of cavities in thick salt beds is a technique for producing underground space for fluid storage. The seis-caliper, an acoustic reflection device, can be used to provide measurements of such cavities and has a range of approximately 500 ft in brine (Caldwell and Strabala, 1970).

Neutron, gamma-gamma, and natural gamma-ray logs may be used to locate zones where aquifer development or plugging has taken place (Norris, 1972; Keys and MacCary, 1971). Morris et al. (1965) described the recompletion of a waste-disposal well at the National Reactor Testing Station based on geophysical logs. The waste-disposal well was originally drilled to a depth of 1,275 ft with slot perforations from 1,182 to 1,267 ft (Fig. 7). Specific capacity of the well was not adequate for the disposal anticipated and specific capacity decreased with time. Trace ejector log B showed that about 30 percent of the water injected at about 100 gallons per minute (gpm) left the well through the annulus between the 6- and 8-in. casing. The casing was then gun-perforated between 935 and 1,070 ft, and improvement in specific capacity was noted. The well was bridged at 1,005 ft and trace ejector log C showed part of the water leaving through the bottom of the hole. On the basis of caliper, gamma-gamma, and natural gamma-ray logs it was decided to gun-perforate the casing interval from 512 to 697 ft. Trace ejector log E showed that this zone took practically all the water injected at 500 gpm and that head rise was insignificant. Trace ejector log D was run during a period of no injection and a natural downward flow of about 25 gpm was measured.

There are many facets to the investigation of artificial recharge for which borehole geophysics can be utilized. Problems associated with recharge in many other areas are similar to those encountered in the Ogallala Formation of the southern High Plains of Texas and New Mexico. In this region attempts are made to recharge sediment-laden water into sand and gravel aquifer in the Ogallala Formation so that the water can be recovered economically for agricultural use. Although much of the discussion of investigations using borehole geophysics is applicable to all recharge problems, the examples are specifically taken from our experience (Keys and Brown, 1971). This research effort is currently in progress and many of the results reported here are preliminary at this time.

Although geophysical logs are important in the analysis of recharge through basins or injection wells, the emphasis in this paper is on the injection-well technique, both because of the extensive use that we made of geophysical logs in analyzing hydrogeologic conditions in injection wells and because there is a wider range of geophysical logging techniques applicable to injection-well recharge.

During recharge of the Ogallala Formation, a large amount of sediment or particulate matter is generally injected, which clogs the aquifer and ultimately decreases the specific capacity of the well. Further, injection of water with a chemical composition different from that of the native groundwater may result in chemical changes that affect the recharge or discharge capacity of the well and thereby change the volume of water that may be stored in or retrieved from the aquifer. The pore space in the formation may be filled with particulate matter, channels between grains in detrital sediments may be blocked, or there may simply be an increase in friction from movement of particulate matter. By various techniques of borehole geophysics we have been able to observe how plugging of the aquifer takes place, and to derive a better understanding of the problems and possible solutions to plugging. It has long been known that clogging of an aquifer usually takes place during artificial recharge and it has been assumed that some type of well redevelopment (heavy pumping, surging, or development with dry ice or compressed air) will remove a major part of the injected sediment and restore the well essentially to its original condition. In some instances this has apparently been a successful technique; in others it has seemed to fail. By using borehole geophysical logs, we have been able to identify the

degree and location of zones of redevelopment and, through a better understanding of the process, hope to be able to increase the specific yield of recharge wells.

Because recharge involves placing an exotic water into an underground environment, quality changes in the water are likely to take place. In addition, the injected water may flow down gradient from the point of recharge. Native water moving through the system may then partly displace and mix with recharged water so that water pumped from the well will be a mixture of the two. Chemical changes in the formation may result in dispersion and expansion of clay, flocculation of particulate matter in the formation, or precipitation of chemical particles. In each case, the permeability and porosity of the formation are reduced and the effectiveness of artificial recharge diminished. Thus, it is necessary to determine, in addition to the nature of plugging, the nature of chemical changes that take place and the areal distribution of recharge water that occurs during injection and pumping.

We have drilled, cased, and backfilled observation holes to obtain the best possible geophysical logs as an aid in interpretation of changes that occur in the recharge system. At each field test a small-diameter hole is first drilled from the surface to the bottom of the aquifer. In this hole caliper logs are run to establish borehole diameter, gamma-ray logs for general lithology, and neutron logs for lithology, porosity below the water table, and moisture content above the water table. A hole is then drilled to obtain continuous 3-in.-diameter cores through each of the lithologic zones selected from the geophysical logs of the pilot test hole. Following completion of the core hole, a similar suite of geophysical logs is run in the core hole. Composite interpretation of logs and core data is used to determine the depth and location of additional observation and logging wells in the recharge area. At most test sites in the Ogallala Formation we have cased these wells with 2-in. steel pipe which is plugged at the bottom. Four or five of these wells, which penetrate the entire thickness of the aquifer, are drilled at distances ranging from 5 to 150 ft from the recharge well. The wells are used for gamma-gamma, gamma-ray, neutron, and temperature logging. Caliper logs are made prior to setting casing, and the annular space between the hole and the casing is carefully backfilled. Chemical and biological sampling wells are drilled at similar distances and screened in the most permeable zones selected from logs. Because we are looking for changes that take place in the environment as a result of recharge, it is imperative that the space between the wall of the drill hole and the casing be backfilled in such a way that major changes will not take place in the annulus. Thus, if the backfill of an observation hole bridges and later collapses during recharge tests, the change seen in the geophysical logs does not represent a change in the formation due to recharge. Similarly, if the backfill absorbs moisture or transmits water at a rate greater or less than the formation does, geophysical logs will be incorrectly interpreted. The backfill technique is also designed to minimize log response to the annulus and to maximize formation response. To minimize backfill changes in early tests, topsoil was sieved and poured slowly down the annular space while the casing was being shaken. It was assumed that the backfill was uniformly placed in the annular space. Subsequent tests indicated that in many observation wells bridging of the backfill still took place, resulting in anomalies on neutron and gamma-gamma logs. More recently a pressure grout of expanding cement has been used for backfill. Although the cement must cure for a long period of time, possibly several months, to reach a stable moisture content, the character of the fill in the annular space is constant. When the cement is stabilized, logs provide an accurate measurement of changes in formation parameters except temperature, which may take longer to attain equilibrium.

There is a temptation to install continuous screens for sampling and logging in observation wells, particularly in an aquifer that is considered uniform and homogeneous. Unfortunately, long screened sections permit vertical flow through the wells, resulting in interflow between permeable zones of the aquifer. To avoid this complication, we have screened no more than one 3-ft interval of the aquifer in any single well.

The parameters which are most significant to recharge and which might be determined from logging include lithology, porosity, permeability, and water chemistry and movement. In order to describe these parameters, as well as to describe the changes that take place as a result of recharge, we have utilized several borehole geophysical logging methods and composite log interpretation techniques.

Lithology

In most aquifers, thin lithologic units have great significance relative to the volume of water that will be accepted through recharge or yielded by pumping. The identification and correlation of the most permeable units within an aquifer is one of the principal uses for geophysical logging. In the Ogallala Formation we regularly record logs at

a vertical scale of 10 ft to 1 in., with log width expanded to 10 in. Commercial logs are usually run at a lower sensitivity in order to reduce the number of off-scale deflections. At some test sites we have used two horizontal scales so as to amplify lithologic differences or changes due to recharge. Ideally a log should permit correlation of rock types, based on differences in mineral composition, grain distribution, sorting, density, and porosity, both in consolidated and unconsolidated systems. Identification of a rock unit is much more readily accomplished in consolidated aquifer systems than in unconsolidated systems. Because of lateral continuity, the consolidated units generally show a unique log "signature" which may be identified from hole to hole. In alluvial or glacial formations, the lithologic heterogeneity is so great and the lateral continuity so restricted that it is difficult to correlate from one area to another on the basis of a log signature. Within a restricted area or depth interval, alluvial or glacial formations usually are derived from the same source area and thus may provide a unique response on several types of logs.

The natural gamma-ray log permits the correlation of lithology based on the variations in occurrence of natural radioisotopes. It is particularly useful where the source of a formation or horizon is unique, such as the separation of two sedimentary units derived from different igneous source areas. A typical example is found in the High Plains, where clays are characteristically more radioactive than sands. In contrast, caliche zones in the same section may exhibit high or low radioactivity and thus are difficult to identify from natural gamma-ray logs alone. In the alluvial Ogallala aquifer system, natural gamma-ray logs are probably the most useful for the identification of lithology in all types of holes--cased or open, water-filled or dry. As with all logs, background information on lithology must be available to guide interpretation in each new section or area.

A calibrated neutron log is a measure of moisture above the water table and porosity below. Because clays characteristically have high porosity, it is useful as a lithologic log to distinguish between clay and sand in unconsolidated aquifers. Above the water table in the Ogallala Formation, the neutron log indicates clays and silts, because these have a high retained moisture content resulting from perching of natural recharge or from high residual moisture from the long-term decline of the water table. In conjunction with the natural gamma-ray log, the neutron log can be used to identify better the lithology of the section where either log by itself might be misleading (Fig. 8).

Resistivity logs have limited value in artificial-recharge studies of the Ogallala because they can be used only where there is sufficient saturated thickness penetrated by an uncased hole. Although log response to clays and shales in relation to sands is commonly distinct, the interpretation of the log is best based on samples from the hole or from some surrounding area and on correlation of the log with other types of borehole geophysical logs.

Auger holes in unconsolidated zones above the water table in the Ogallala Formation are uniform in diameter, and variations of more than 1 in. in a 4-in.-diameter borehole are uncommon. On the other hand, rotary drill holes may have very large washouts, extending outward more than 10 in. When holes are rotary drilled in unconsolidated formations, the caliper log is more useful for interpretation of lithology, because it shows a sharp contrast between friable or soft zones, which wash out as a result of circulation of drilling fluids, and competent zones, where the drill leaves a smooth hole of uniform diameter. The caliper log is also useful to indicate those zones where problems with backfill may be encountered, as where clays expand into the hole and reduce the diameter of the borehole to less than the bit size, or where collapsing soft sands may bridge against a casing and prevent a uniform backfill.

Porosity

Porosity is one of the most important parameters for artificial recharge that can be measured directly by geophysical logs. The distribution of pore space in the aquifer system, and permeability, which is sometimes related, are the most significant parameters controlling artificial recharge. A measurement of effective porosity indicates the amount of water that can be recharged to and pumped from an aquifer. The shape and size of the pores, and whether the porosity is intergranular or secondary, affect the capacity of the aquifer to receive water. Porosity can be determined from neutron, gamma-gamma, and acoustic-velocity logs. Gamma-gamma logs have been the most useful in estimating porosity and detecting changes in porosity as a result of recharge. In a test of a recharge well 15 mi northwest of Lubbock, Texas, water containing approximately 700 mg/l of suspended sediment was injected into the Ogallala Formation at a rate of 100 gpm for approximately 23 hours. Gamma-gamma logs were made in observation holes at distances of 3, 6, and 12 ft from

the injection well. In order to minimize the effect of borehole diameter and backfill, the logs were made in two parallel holes, 2 ft apart, by lowering a source in one hole while a detector was lowered at the same rate and at the same depth in the other hole. Changes in porosity resulted from recharge and from redevelopment of the well following recharge (Fig. 9). In the most permeable zone, as determined by temperature logging, between 90 and 110 ft, an increase in bulk density shown on the gamma-gamma log was interpreted as indicating a decrease in porosity below the pre-recharge water table. The permeable zone at a depth of about 100 ft exhibited the lowest radioactivity on the natural gamma-ray log. We presume that a large part of the injected sediment was deposited in that interval. A decrease in bulk density, interpreted as an increase in porosity, was measured for the segment of the formation above the original water table where the formation was developed by pumping of the well after recharge (Fig. 9). Redevelopment of the formation by pumping did not show any change in bulk density at the observation wells 6 ft from the injection well. Apparently the injected sediment that was deposited 6 ft from the recharge well was not recovered by pumping.

The porosity of siliciclastic aquifers consists mainly of intergranular pores that are saturated with water. Bulk resistivity is proportional to the formation-water resistivity and inversely proportional to porosity. Not only sand and sandstone, but also porous limestone and dolomite having uniformly distributed porosity may behave electrically as siliciclastic rocks. The determination of actual porosity from multiple-electrode resistivity logs must be based on analysis of core samples or field tests from the zones being investigated. Zones where changes in porosity have taken place as a result of recharge or redevelopment of a well can be readily identified without the necessity for other types of analysis if fluid resistivity is known.

Resistivity logs are usually run in an open hole immediately after drilling and thus would not be suitable for long-term monitoring of recharge into an aquifer because the wells are eventually cased. However, at several sites we have successfully used multiple-electrode resistivity logs in well bores lined with plastic well screen. In a field test a hole was drilled and logged using 4-, 16-, 32-, and 64-in. normal logs. After the initial logging, a series of 20-ft lengths of plastic well screen was placed in the hole. The hole was logged again using the same sensitivity adjustments. By suppressing zero resistivity it was possible to log the screened part of the observation well and obtain normal curves with the same character as those run in the open hole, except for deflections caused by the joints between screens. Periodic resistivity logging through plastic screens permits measurement of changes in porosity and fluid quality that take place as a result of recharge. Porosity changes may be caused by the injection or removal of particulate matter or by chemical changes that result in precipitation or solution in the aquifer. As noted previously, the installation of continuous screen in an observation well, particularly during injection, permits water to move vertically through the screened section and thus short circuit the normal flow pattern. Therefore, it is desirable to install a full-length packer in a screened observation well in order to insure that natural conditions are maintained except when actually logging. This has been done successfully by installing a length of thin rubber hose the entire depth of the screened section and inflating it with water to achieve a tight seal against the screen.

Multiple-electrode resistivity logging was done before and after recharge in an observation hole finished with plastic screen at the Dunlap test site, near Wolfforth, Texas (Fig. 10). The 16-in. normal curve showed a small decrease in resistivity that may be due to an increase in porosity or in the size of the annulus around the screen. The consistent increase in resistivity on the post-recharge 64-in. normal curve is probably due to the lower content of dissolved solids in the recharge water.

The neutron log indicates moisture content above the water table and porosity below. We have used it successfully below the water table to identify zones of high porosity. Using it in combination with gamma-gamma and multiple-electrode resistivity logs, we have been able to identify zones which will accept recharge water. The neutron log can also be used to identify zones which are plugged by deposition of sediment where this sediment displaces water from the formation. We have regularly run both gamma-gamma and neutron logs to show changes in porosity that have taken place as a result of injection. Figure 11 shows that changes in porosity and moisture content took place as a result of injection recharge.

A neutron moisture probe of the type commonly used for soil studies provides information on a relatively small volume of the formation. A comparison of logs produced by a neutron moisture probe and by a longer spaced neutron porosity probe commonly used for logging oil wells is shown in Figure 12. The effect of hole diameter and particularly the

effect of high residual moisture in the borehole as a result of drilling is much greater on the moisture log. Thus, the neutron porosity log more accurately depicts the moisture content or porosity of the formation. The water content of the drilling mud may be such a high percentage of the total volume investigated by a neutron moisture probe that even saturated and unsaturated zones in the formation cannot be distinguished (Fig. 13). A gamma-gamma log may be similarly used to detect changes in moisture or porosity but it tends to be more affected by extraneous borehole parameters.

Permeability

Permeability is the most important parameter that can be measured for any hydrologic investigation and, of course, is particularly important for artificial recharge. The multiple-electrode resistivity log can be analyzed to determine the formation factor (F), calculated as the relation between the fluid resistivity and the resistivity of the aquifer, saturated with groundwater (D. L. Brown, 1971). Studies by Croft (1971) indicated an approximate relation between the F factor with a given average porosity and the permeability of an aquifer system. Geophysical logs that respond to lithology give a clue to permeability in that they indicate the presence of clean sands or highly fractured consolidated rocks. In a rotary-drilled hole the caliper log can be used to estimate mud-cake thickness, which is related to permeability as well as drilling parameters.

In an artificial-recharge system, probably the most successful method of estimating permeability is through the use of tracers. Chemical or radioactive tracers can be placed in the recharge water and identified by various sampling and borehole-logging techniques (see p. 9). Radioactive tracers are very effective but are not suitable in areas where water may be used for domestic purposes. We have investigated the in-hole activation and detection of non-radioactive ions introduced as tracers. These tracers can be activated and identified through casing, but to date the technique has not proved practical in recharge experiments because of the high concentration of tracer ions required.

Above the water table, neutron logs can be used in tracing the expansion of the moisture front as a mound forms around an injection well and gradually migrates radially outward. Similarly we have used neutron logs to determine the rate of downward movement of a moisture front below a basin-spreading operation. The neutron moisture log has been similarly used in several artificial-recharge studies (Jones and Schneider, 1969). Figure 14 shows the movement of a moisture front in a spreading-recharge experiment in Lubbock, Texas. Perching of the moisture front in the 50-55-ft interval indicates a low-permeability zone through caliche near the observation well. Although the moisture logs indicated no more water moved down in the vicinity of well S-7, water moved through fractures elsewhere, causing the water table to rise.

A temperature-logging technique was used in a well 20 mi northwest of Lubbock, Texas, to locate zones through which recharged water moved (Schneider et al., 1971). This consisted of logging temperature changes in water-filled, cased, and capped observation wells. Subsequent use of this technique in the southern High Plains has shown it to be a highly effective method for determining the relative magnitude of permeability of various zones in an aquifer. Figure 15 illustrates how this technique defined a series of zones of different permeability in the Ogallala aquifer.

Characteristically, the injection water in the southern High Plains is obtained from shallow lakes which have a temperature different from that of groundwater. Diurnal fluctuations of water temperature in these lakes impose daily pulses on the initial thermal anomaly caused by recharge. Continuous injection of the recharge water transmits diurnal thermal pulses through the aquifer where they can be identified in the capped water-filled observation wells by a high-resolution temperature-logging probe. A recharge well near Hereford, Texas, was recharged at 600 gpm for approximately 10 days. The injection well was cased from the surface to 219 ft and casing perforated from 123 to 219 ft. During injection, water levels rose from 150 to 60 ft below land surface. Under this head, water moved out through the perforations in the well, into the permeable zones in the formation. Temperature logs made in observation wells at several distances from the recharge well showed the arrival of the warmer injected water. Repetitive logs showed the arrival of the diurnal thermal fluctuations of the input water and permitted a computation of the approximate transit time of recharge water from the recharge well to the successive observation wells. It was also possible to plot changes in transit time or velocity during the life of the test. Figure 16 shows a sequence of temperature logs made in a single observation well during the test; Figure 17 shows the transit time of the diurnal temperature pulses at a depth of 160 ft.

SUMMARY--THE LOGGING PROGRAM

The decision to make geophysical logs and the selection of logging techniques should be based on the information required. In addition to references cited in this report two text books on the various logging techniques as applied to petroleum investigations are useful (Lynch, 1962; Pirson, 1963). Before the start of an investigative drilling program, a basic decision should be made as to whether logs should be made by a large petroleum-oriented service company, by a smaller service company oriented toward water-well logging, or with logging equipment purchased for the project. The cost of logging is an important factor in making the decision as to who will do the logging. Commercial service logging can vary from a few hundred to many thousands of dollars for a single well, but the cost of logging is only a small fraction of the cost of a recharge or waste-management system. The cost of logging will vary with the distance from the nearest service company office, the depth of the well, and the types of logs to be run.

If the logs are to be made by a petroleum-oriented service company the Reference Handbook of Selected Formation Evaluation Tools by the Lafayette Chapter of Society of Professional Well Log Analysts (1972) is a very useful reference. This publication lists the trade names for logs made by the various service companies, the purposes, operating principles, uses, advantages, and limitations of the various logs as well as services that can be recorded simultaneously. Logging results in common borehole fluids, dimensions, pressure and temperature ratings, and publications on each type of log are also listed.

It is essential that someone qualified in the interpretation of geophysical logs be employed to analyze the logs, and it should be recognized that log analysis is a developing science. Interpretation still depends to a large degree on experience in each geohydrologic environment rather than on well-established equations.

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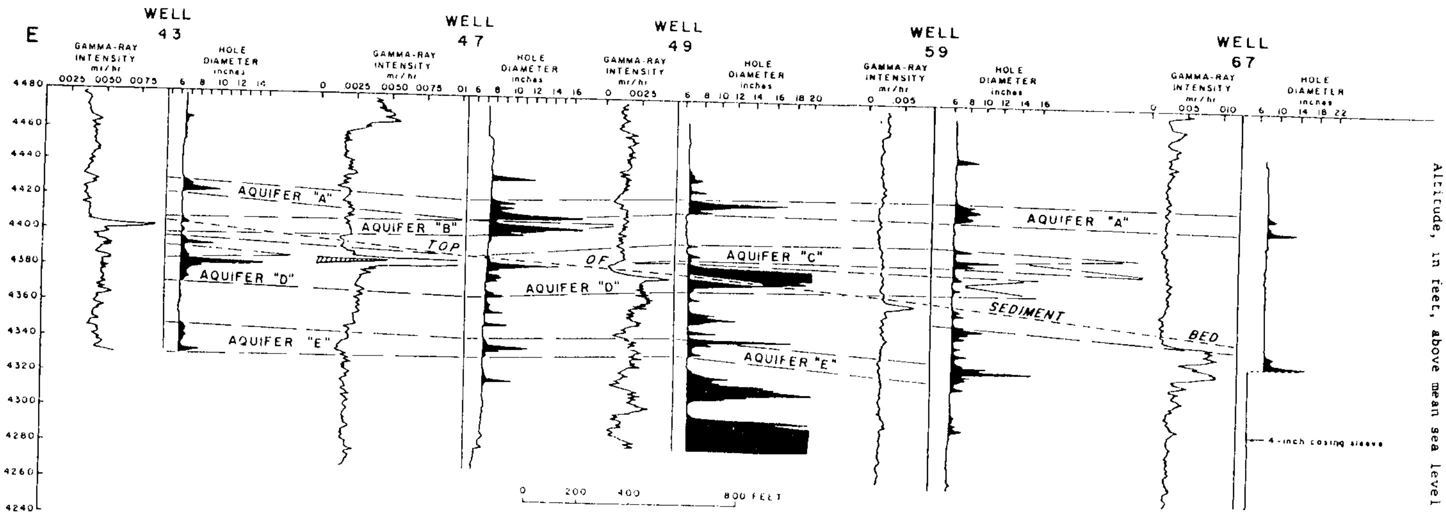

FIG. 1--Cross section through part of National Reactor Testing Station based on gamma-ray and caliper logs showing condi
below water table. From P. H. Jones (1961b).

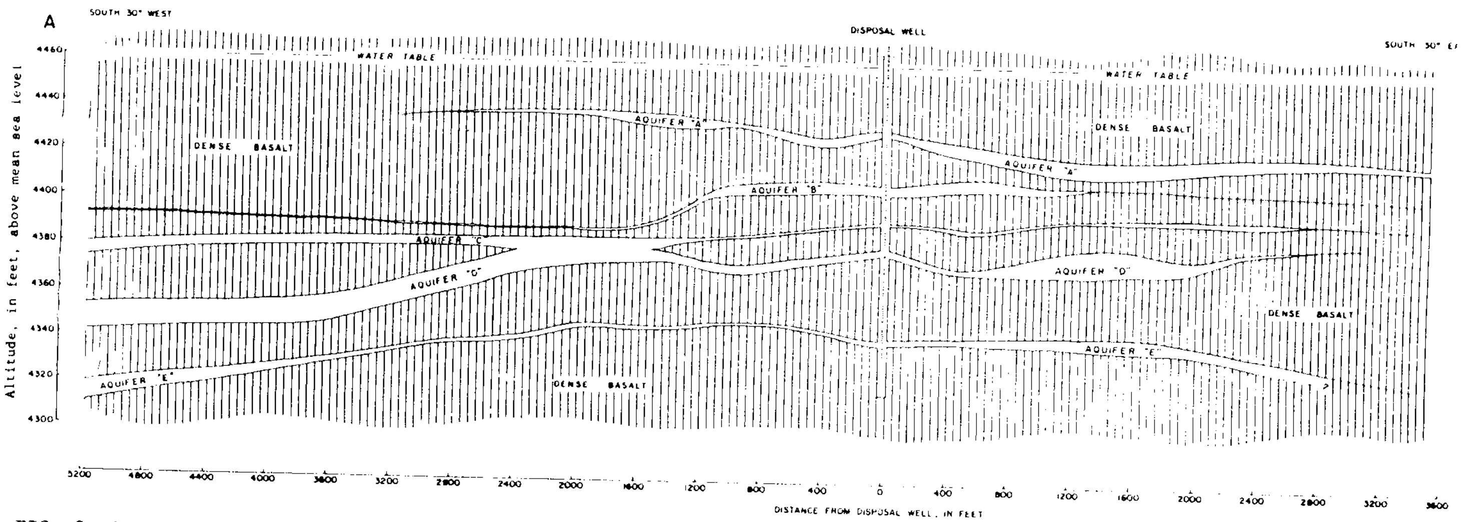

FIG. 2--Structure and thickness of aquifers "A" to "E" based on geophysical logs, National Reactor Testing Station, Idaho
From P. H. Jones (1961b).

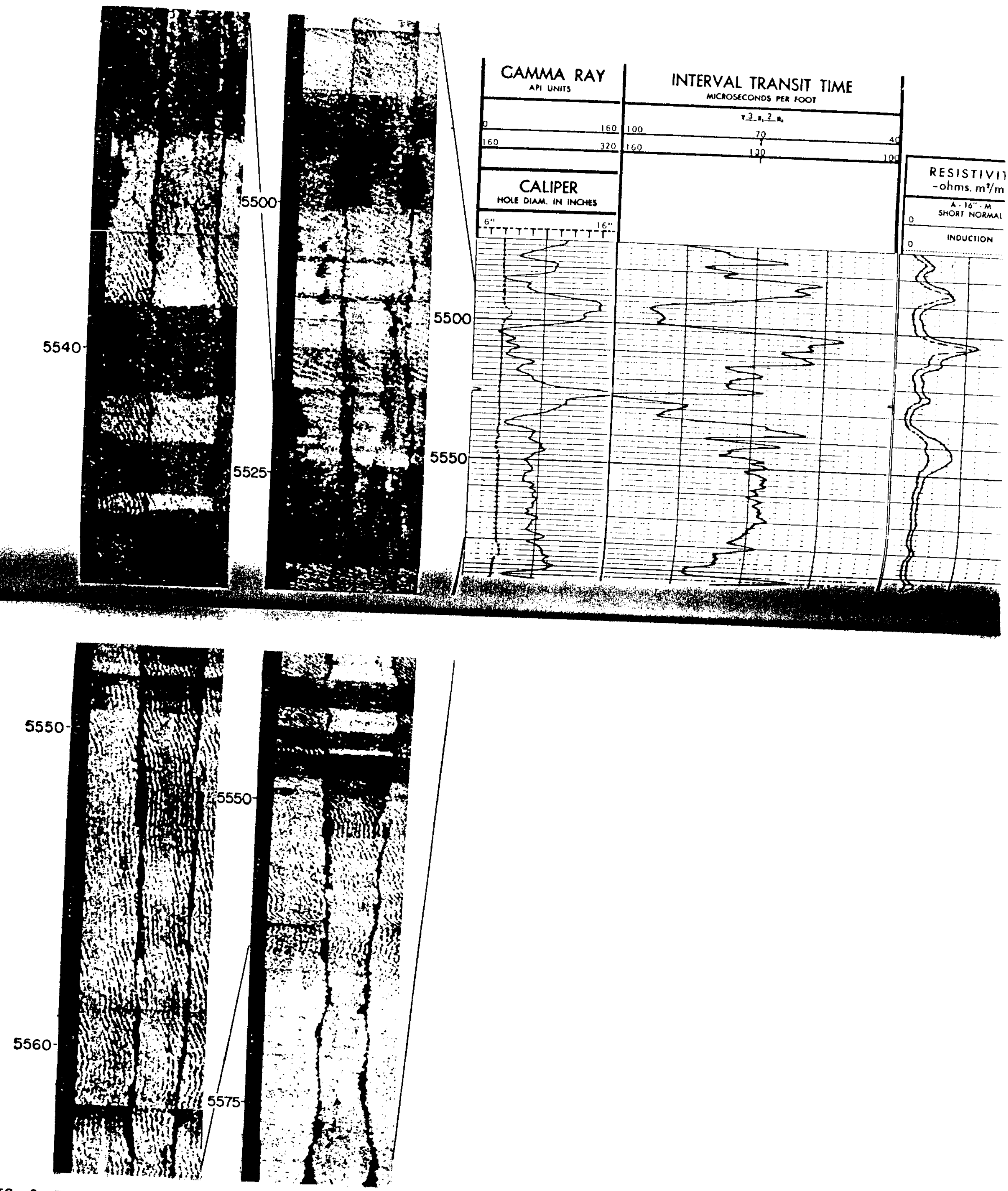

FIG. 3--Drilling-induced vertical fractures in sandstones and shales of western Oklahoma. From Zemanek et al. (1970)

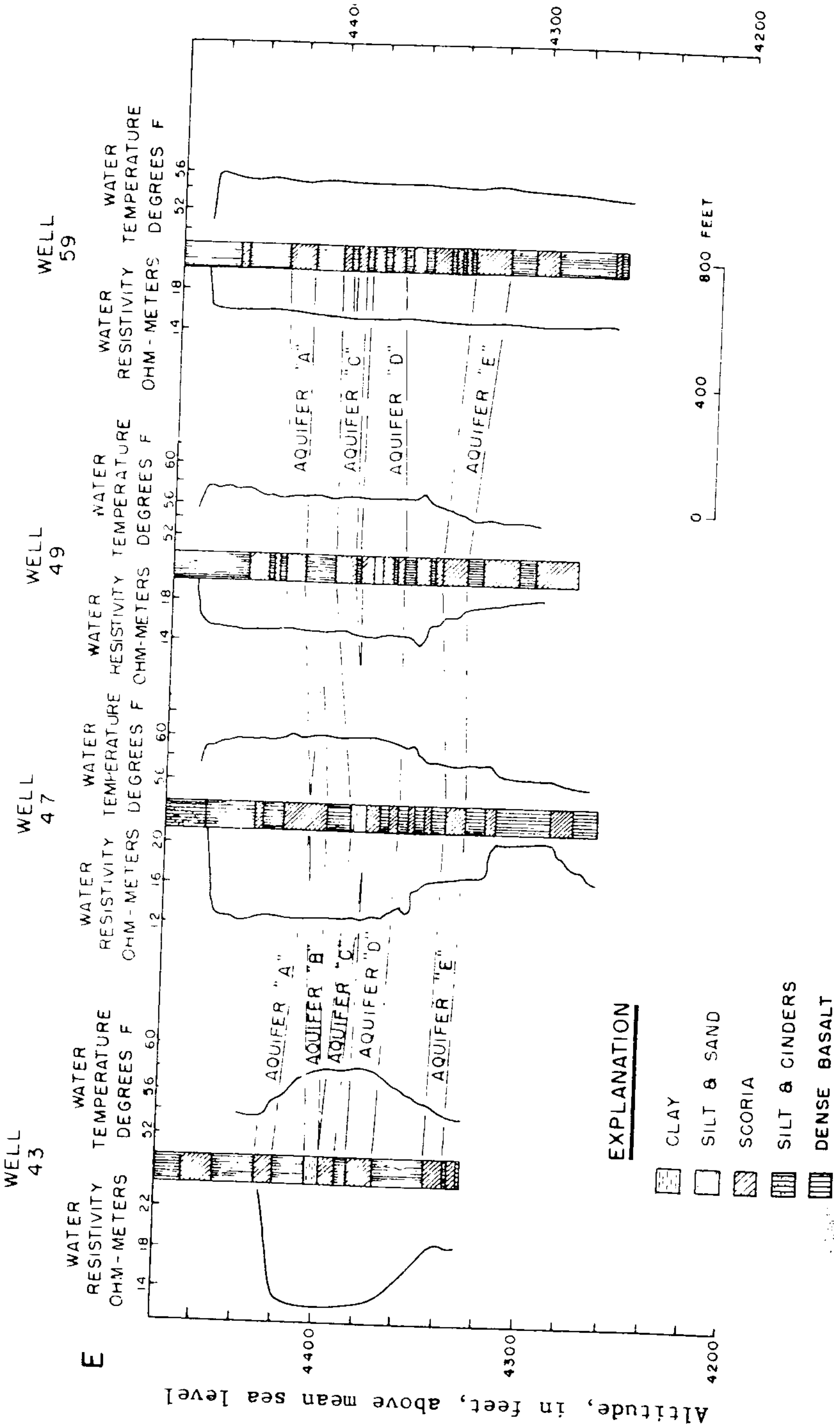

FIG. 4—Cross section through part of National Reactor Testing Station showing decrease in temperature and increase in resistivity of water with distance from disposal well which is nearest to well 43. From P. H. [illegible]

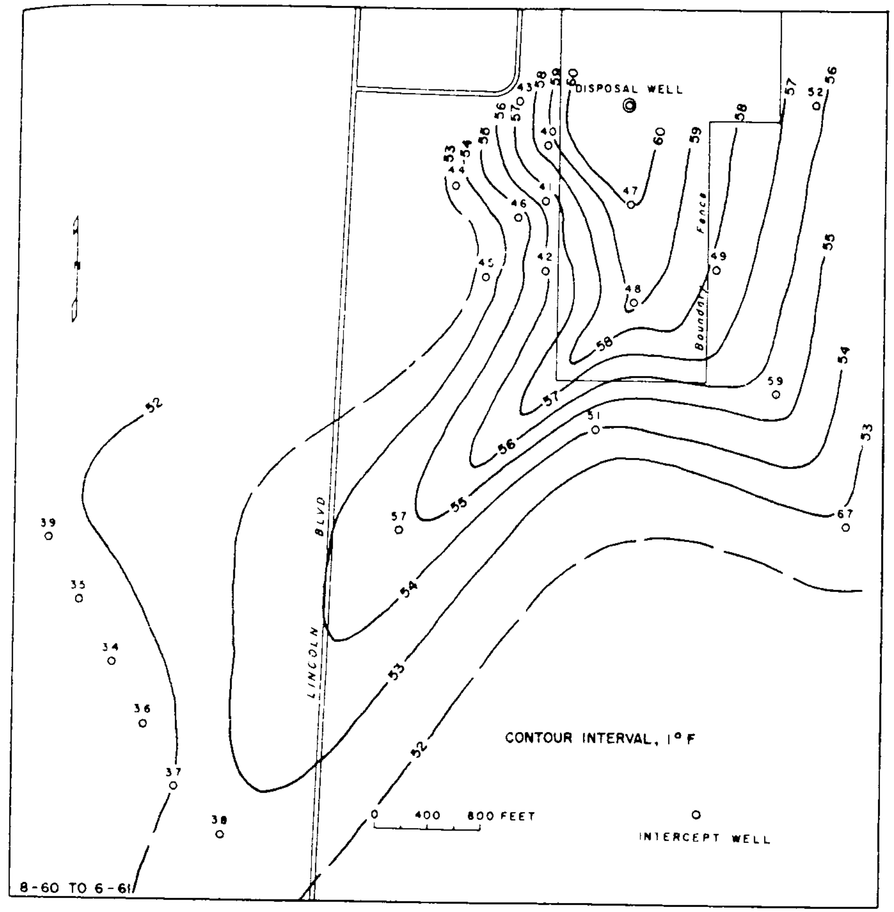

FIG. 5--Maximum temperature of water between depths of 455 and 655 ft based on geophysical logs of monitoring wells near a disposal well, National Reactor Testing Station, Idaho. From P. H. Jones (1961b).

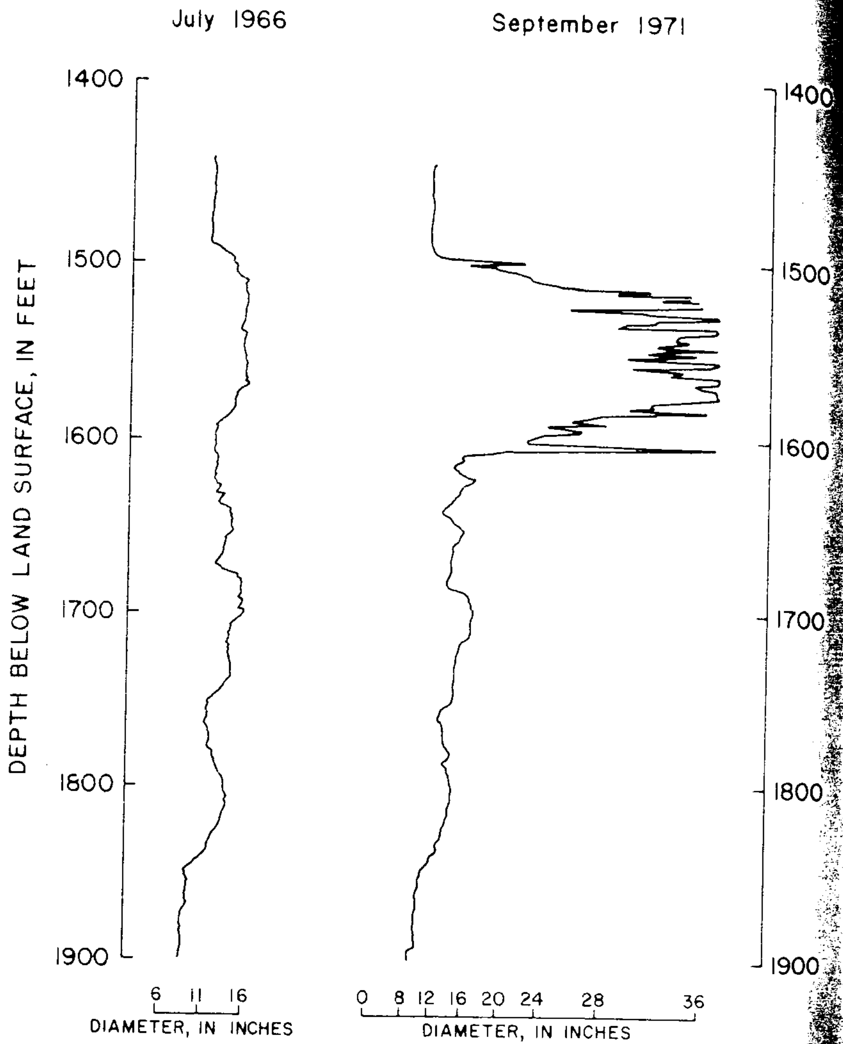

FIG. 6--Preinjection and postinjection caliper logs. Increase in hole diameter caused by solution of limestone by acid wastes. From Black, Crow, and Eidsness, Inc. (1972).

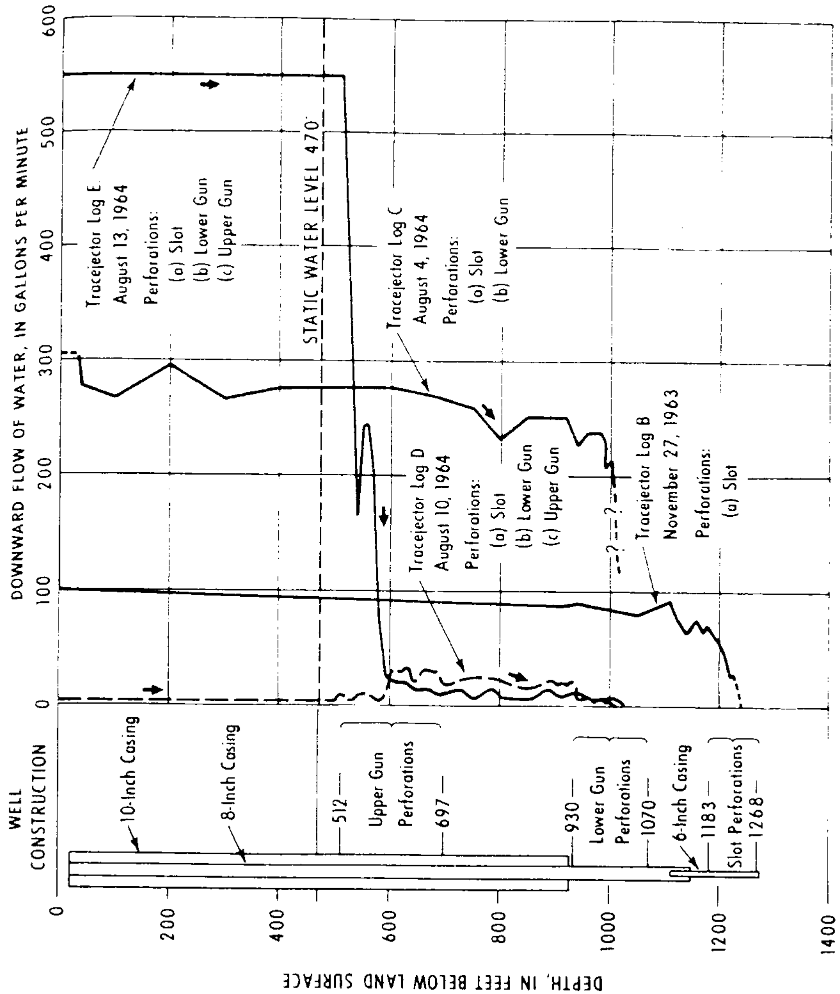

FIG. 7--Radioactive trace ejector logs of a waste disposal well, National Reactor Testing Station, Idaho. From Morris et al. (1965).

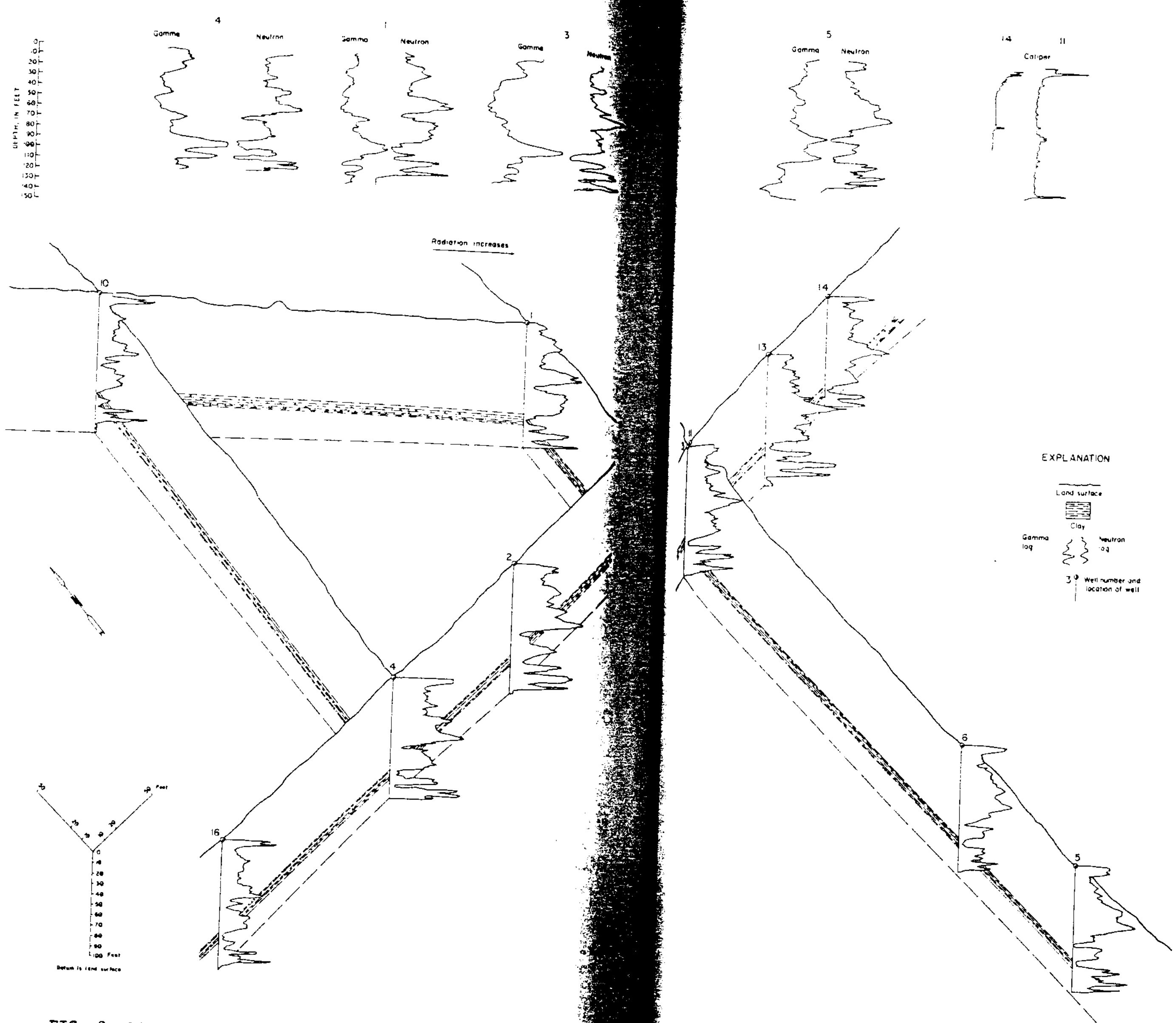

FIG. 8--Lithologic correlation in Ogallala Formation near Lubbock, Texas, based on neutron and gamma logs.

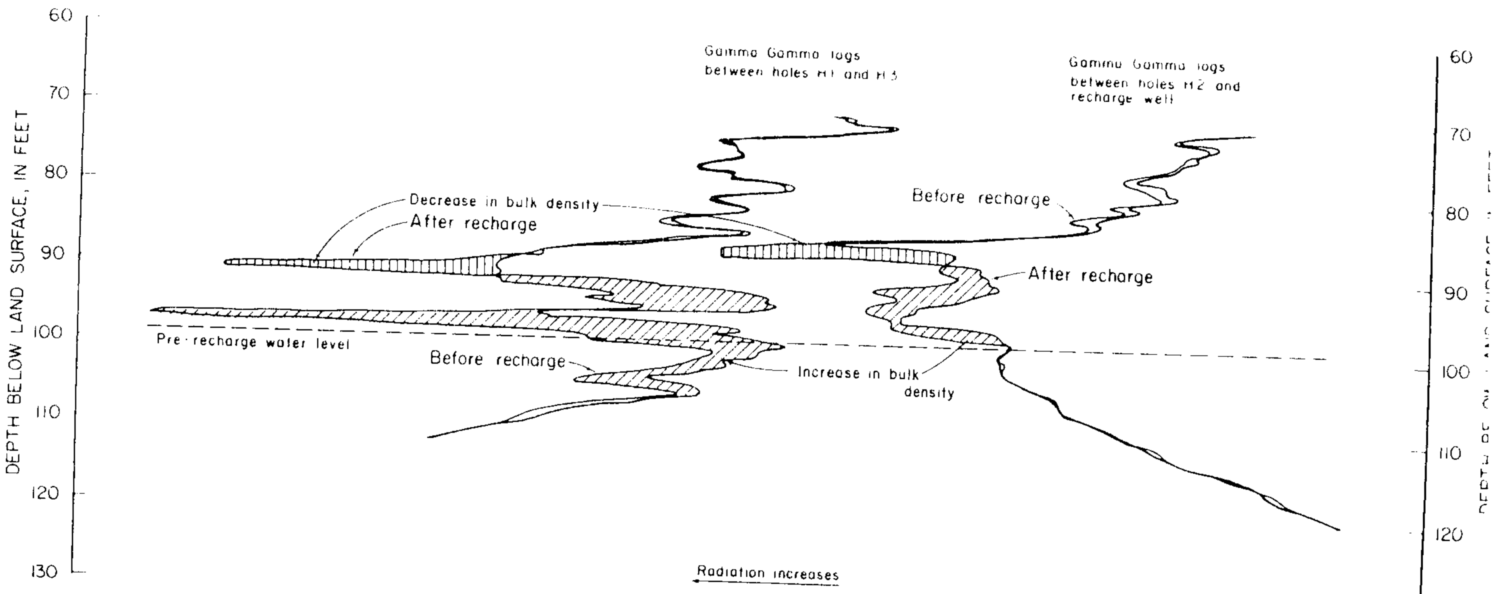

FIG. 9--Gamma-gamma logs made between two observation wells 6 ft from recharge well and between recharge well and observation well 2 ft away. Logs made after 23 hours of continuous recharge at 100 gpm and 1 hour of redevelopment.

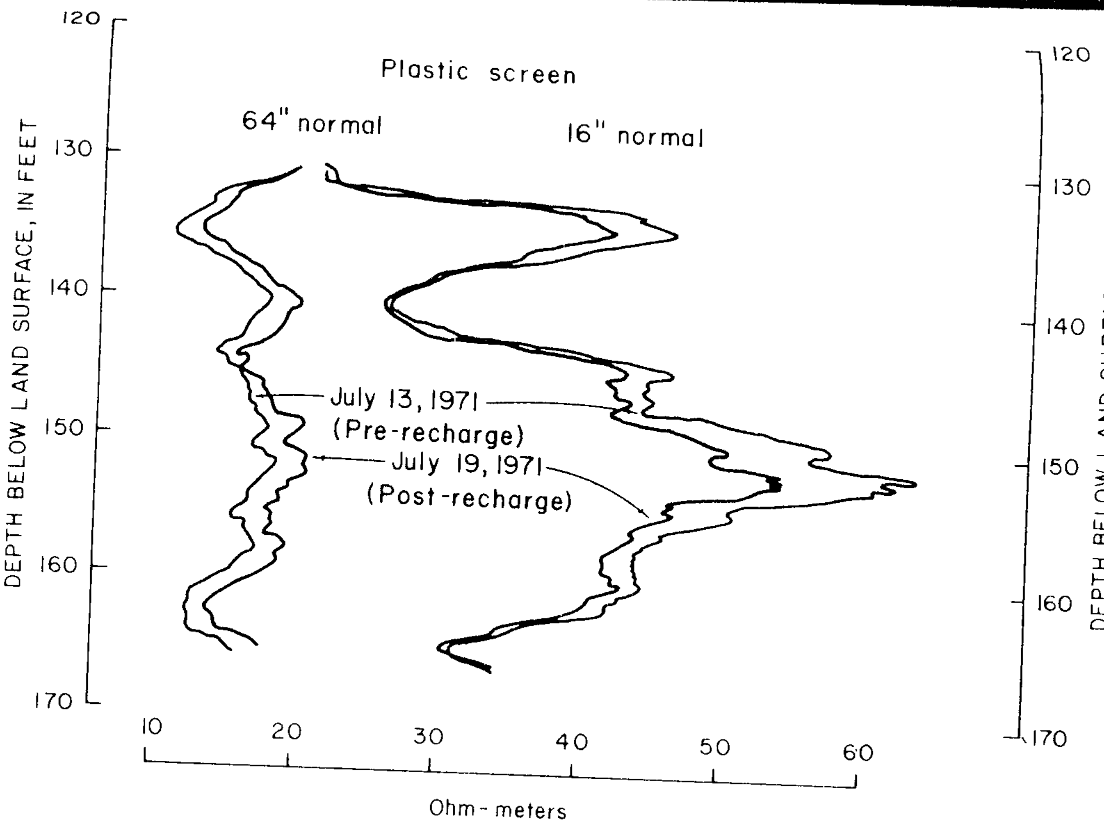

FIG. 10--Electric resistivity logs made through plastic screen in well Du-3 near Wolfforth, Texas. Increased resistivity on 64-in. normal curve after recharge due to lower dissolved solids in recharge water. Decreased resistivity after recharge on 16-in. normal curve is probably due to development in annulus around screen.

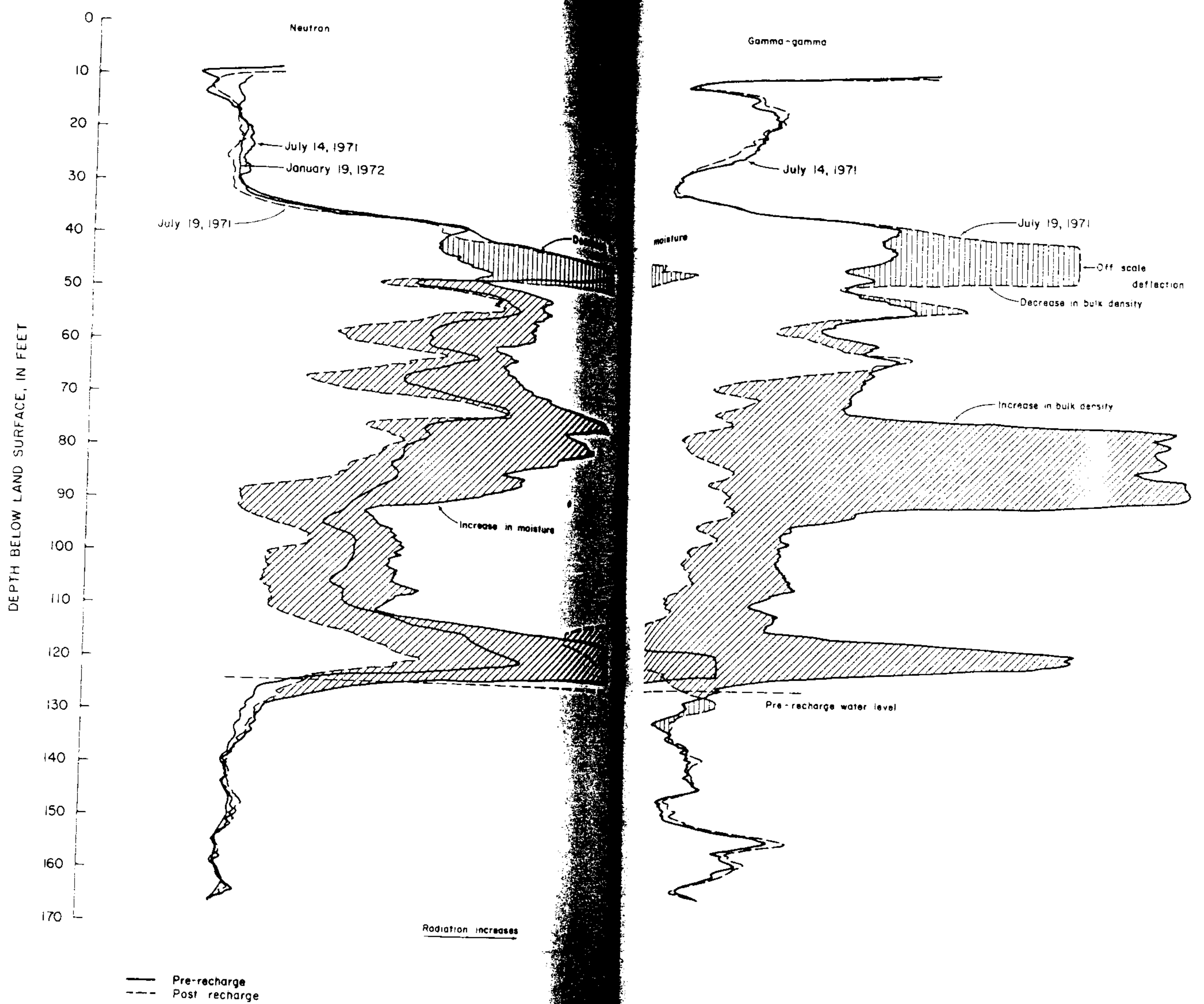

FIG. 11--Neutron and gamma-gamma logs made in observation well Du-4 near [illegible] forth, Texas; 20 ft from injection well. Logs made before and after rech[illegible] 2 days at 1,800 gpm.

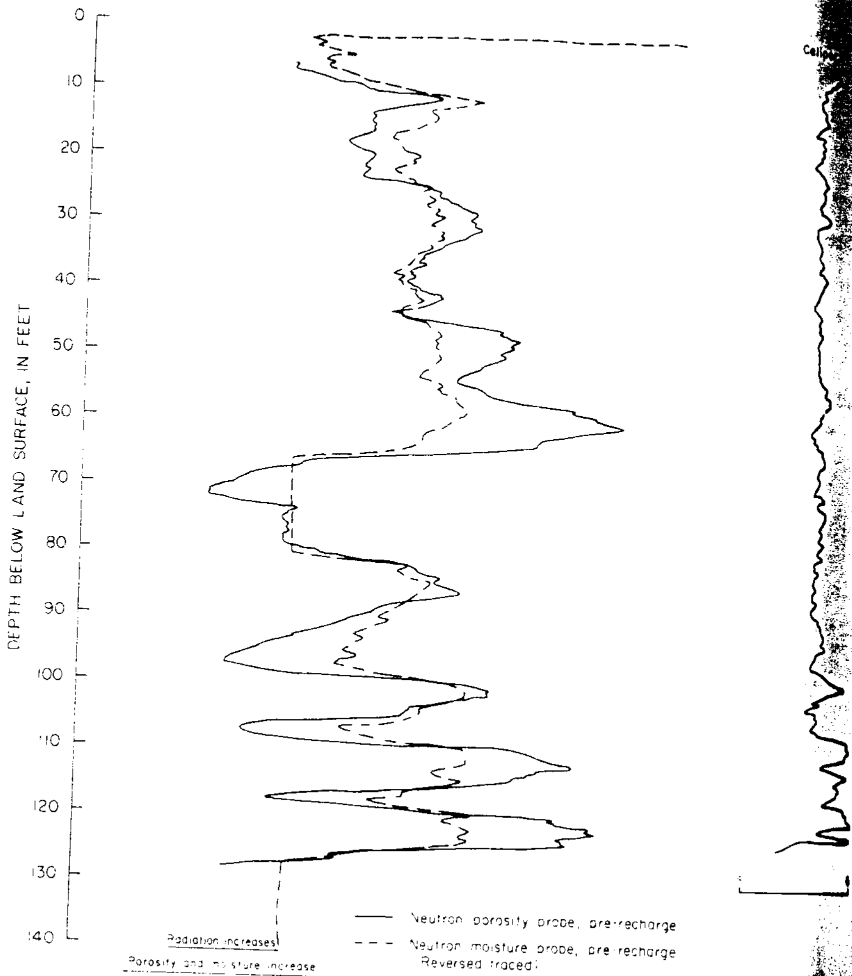

FIG. 12--Neutron porosity log and neutron moisture log made in same hole.

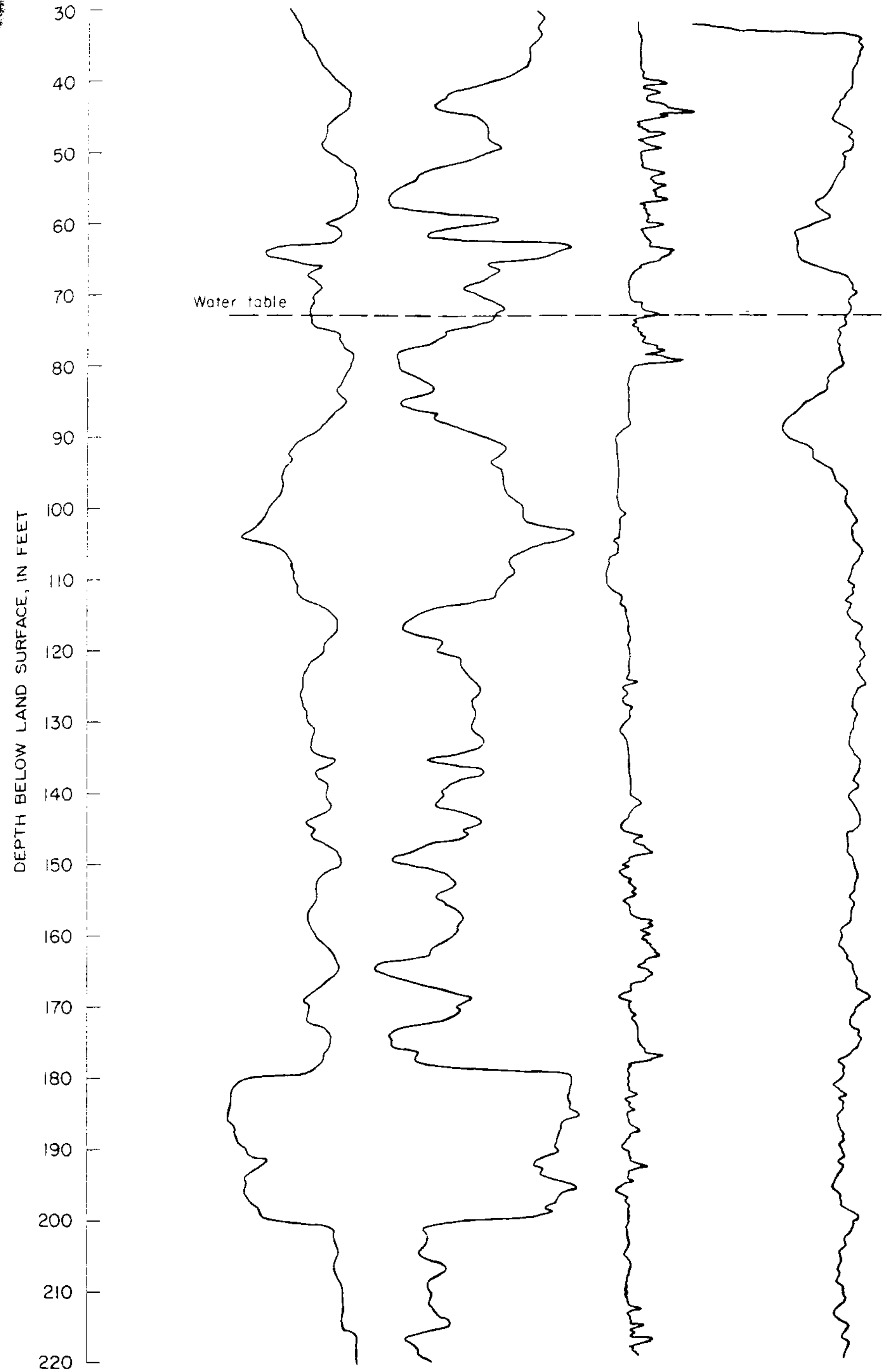

FIG. 13--Neutron moisture log in mud-filled rotary hole shows a minimal response to lithologic changes shown on the SP, single point and caliper logs.

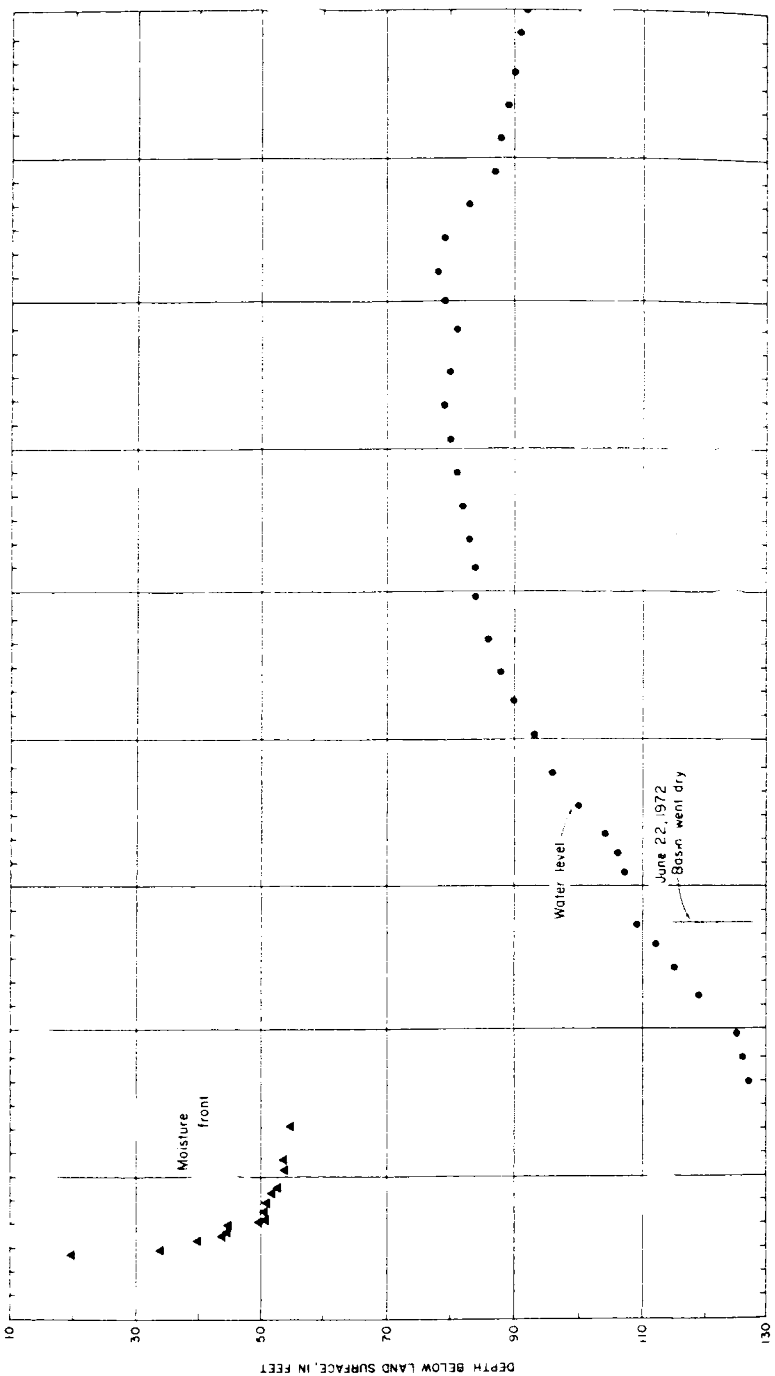

FIG. 14--Observation well S-7 near Lubbock, Texas; downward movement of moisture front and rise in water table under spreading basin recharge facility measured by neutron moisture probe. Lower limit of moisture front decline is local zone of perching.

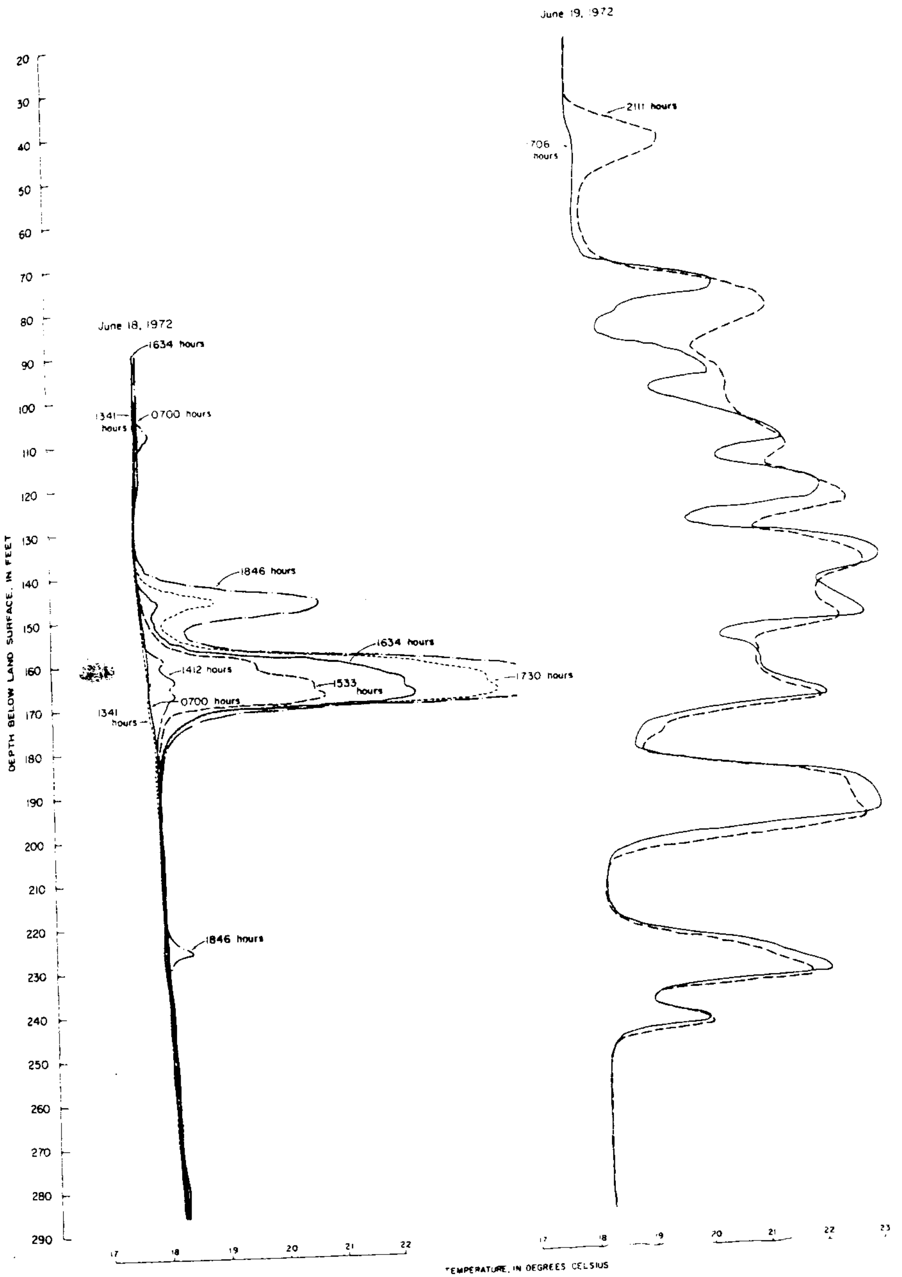

FIG. 15--Temperature changes in observation well 20 ft from injection recharge well, resulting from injection of warm recharge water. Thermal changes on June 18 show position of most permeable zones. Zones of minimum change June 19 are least permeable.

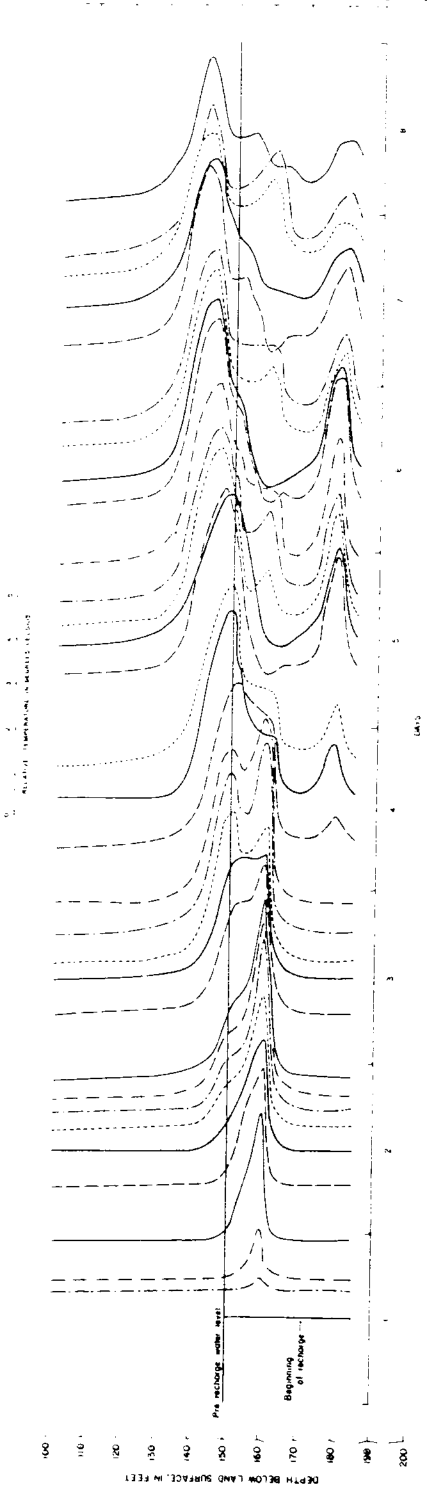

FIG. 16--Temperature logs in observation well 35 ft from injection well. Effect of diurnal thermal fluctuations of recharge water can be seen in temperature changes at a depth of 160 ft. Logs are plotted at approximate time they were made. Actual range in temperature is 15 to 20°C.

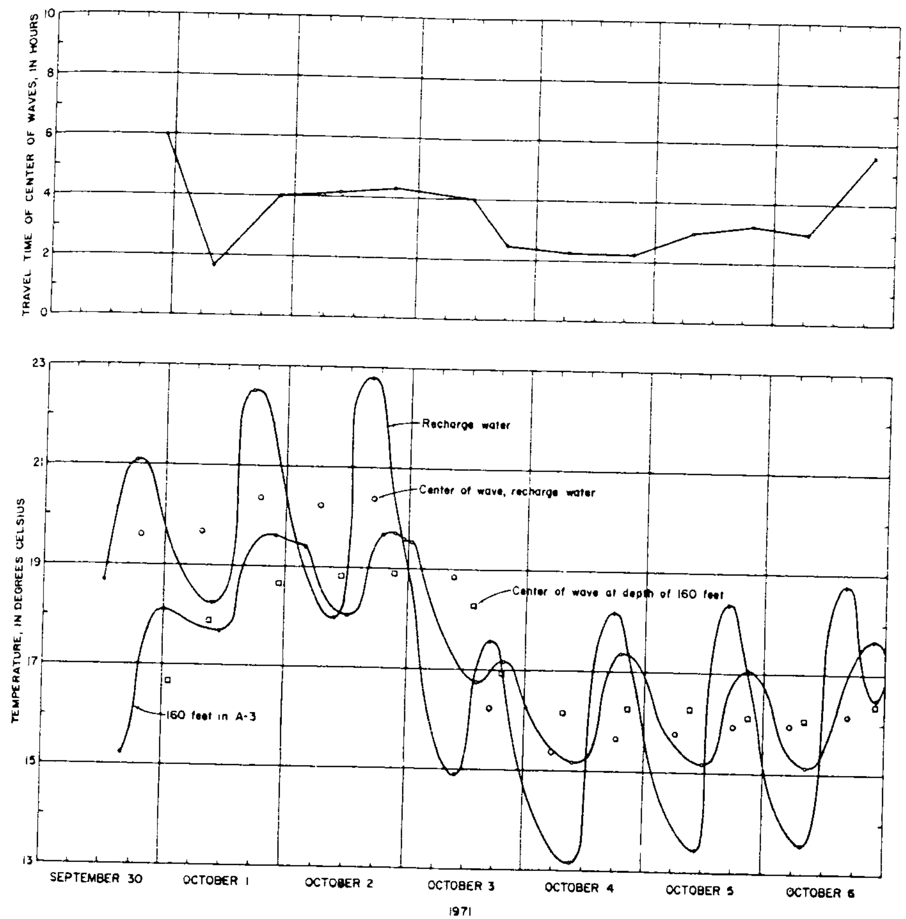

FIG. 17--Travel time for recharge water to move from injection well to observation well 35 ft away at a depth of 160 ft. Travel time computed from center of thermal wave.

SALINE AQUIFERS--FUTURE STORAGE RESERVOIRS FOR FRESH WATER?[1]

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ABSTRACT In the advanced industrial countries the most favorable, least expensive sites for surface reservoirs already are in use or the land already is preempted for other uses and is unavailable for the storage of water. Many flat areas in coastal zones, underlain by saline aquifers, are unsuitable for water storage although a surplus of fresh water is available in such areas at certain times of year. The lack of a reliable, year-round supply of water has been a major factor in preventing commercial and residential development in these areas.

It is suggested that saline aquifers may be used for freshwater storage. To study the physical process in the laboratory we have constructed and operated several mini-aquifers and, simultaneously, have devised some approximate mathematical models. The annual cycle of injection, storage, and withdrawal of the fresh water has been found to be feasible under the idealized assumptions normally found in groundwater hydrology--a horizontal, isotropic, homogeneous aquifer of uniform porosity, transmissivity, and storativity. Laboratory experiments on a single-well system built into a mini-aquifer constructed of epoxy-consolidated, uniform blasting sand show that the efficiency of the process, per cycle, increases as the number of cycles increases. Our computational procedure verifies this and has enabled us to change inexpensively and quickly such parameters as density difference, dispersion coefficient, input rate and period, withdrawal rate, storage period, etc. The studies show that storage of fresh water in an aquifer that contains brine is feasible, if a sufficient number of cycles is considered. The cost, in terms of irretrievable fresh water, is calculable under these conditions.

Additional studies were and are being made on a 9-unit well field. Preliminary results show that, although the recovery percentage at the end of the first cycle is smaller than that of a single well operating by itself, the recovery rate of a multi-well configuration increases as the number of cycles increases.

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INTRODUCTION

The storage of water in times of plenty for use in times of drought is not a novelty. In many areas of the world the topography is suitable for the construction of dams and the creation of reservoirs. In the coastal zones of the world, however, the land is too flat to accommodate dams and impoundments, although fresh water may be available in surplus during certain times of year. In many industrialized areas, although the topography may be suitable, the land has been preempted for use by industry, commerce, or residences.

The problems of reservoir storage for surplus water must be solved, as a water supply that is not available on a daily basis will not support a population. Thus, the engineering profession is being increasingly confronted by the physical problem of topography in the estuarine, coastal areas of the world and the sociological problem of population distribution in the industrialized areas of the world when it is called upon to provide a perennial water supply of determinable quantity.

The conventional answer to these two types of circumstance is to provide surface storage by brute force and awkwardness. In the coastal zone, the answer might be to construct reservoirs within ring levees on the existing soil of muck or clay. Under such circumstances, an average water depth of about 15 ft (5 m) approaches the economic maximum, when the hazard posed by hurricane winds is taken into account. Moreover, such reservoirs are readily contaminated, not only by air-borne or people-borne pollutants, but also by fecal matter from water fowl and aquatic life. In addition, such shallow reservoirs tend to provide favorable conditions for the growth of algae and water-loving weeds.

The evaporation from such reservoirs, as much as 0.25 in./day (5 or 6 mm/day) during warm, dry periods, constitutes a significant water loss and reduces the reliable yield.

In industrialized areas, although local topographic conditions might have been suitable, reservoirs have had to be constructed at long distances from the point of use and large pipelines built to convey the water from the dams to the populated areas.

Many of the areas where such engineering problems exist are underlain by water-bearing formations that contain saline water. The purpose of this paper is to outline the possibilities of using such formations for the storage of fresh water. The problems inherent in bringing such a technologic development from theory to practice under a wide range of hydrogeologic conditions are also outlined.

AQUIFER CHARACTERISTICS

An aquifer that contains salt water (a salaquifer) possesses all of the characteristics of a water-supply aquifer except that the water is not potable. Salaquifers are usually artesian, which means that the water is stored in the aquifer's pore spaces and is compressed slightly. The quantity of water that must be removed from the aquifer to produce a pressure decline of 1 ft over 1 sq ft of aquifer area (measured in the horizontal plane) is called the "storativity" of the aquifer and is designated by the symbol, S. Conversely, the quantity of water that must be injected into an aquifer to produce a water-level rise of 1 ft in 1 sq ft of aquifer is also S. The hydrologic constant, S, is expressed as a decimal fraction, and field tests of aquifers have yielded values that range from as little as 5×10^{-5} to as much as 2×10^{-3} or more. The other hydrologic constant normally encountered in groundwater engineering is the transmissivity of the aquifer, T, and is the product of the hydraulic conductivity, K, of the system and its thickness in feet. In water-supply work it is usually expressed as gallons per day per foot of aquifer, measured at right angles to the direction of groundwater flow, under a gradient of one foot per foot (unity gradient). Transmissivity is usually expressed in Meinzer-feet. The petroleum engineer is familiar with the concept of transmissibility, which is $k \cdot h/\mu$, where k is the intrinsic permeability, in darcys, of the formation and h is the formation thickness in feet. Hence the product $k \cdot h$ is in darcy-feet. Transmissivity as used in water-supply work and the transmissibility as used in petroleum engineering are related by the following equation:

$$K \cdot h = 0.3287 \frac{\gamma}{\mu} (k \cdot h),$$

where:

- K = hydraulic conductivity of system in Meinzers;
- h = formation thickness in feet;
- γ = unit weight of fluid at temperature of interest in lbs/ft^3;
- μ = absolute viscosity of fluid at temperature of interest in centipoises;
- k = intrinsic permeability of formation in darcys.

The relationship between the withdrawal (or injection) of water from (into) an artesian aquifer and the pressure change produced throughout the aquifer is well known (Theis, 1935) and will not be repeated here. Thus, for a system that contains only one type of water, engineering calculation techniques are available and proved so that well-field design can be undertaken with predictable results. The potentiometric surface produced by the injection of fresh water into the aquifer can be computed. For the injection phase of cyclic water storage, a limiting criterion is that the confining formation shall not be fractured by an excess of pressure at the point of injection into the aquifer. To satisfy this criterion safely, the pressure at the well face should not exceed 75 percent of the computed geopressure at the site. This means that if the top of the salaquifer is at a depth of 600 ft (180 m) below land surface, the bottom-hole pressure during injection should not be permitted to exceed 450 psi (32 kg/cm^2).

The unique computational problems associated with the storage of fresh water in salaquifers, assuming that the viscosities of the native and injected waters are approximately the same, stem in part from the requirement that 100 percent of the native water be replaced by the injected fresh water within the zone of storage. In computing the recovery efficiency (quantity of water recovered divided by quantity injected) the following parameters were found to be important--(1) the dispersion coefficient of the aquifer material; (2) the difference in density between the native and the injected water; (3) the rate and duration of injection, the duration of storage, and the rate of offtake (withdrawal); (4) the dip of the salaquifer; (5) the direction and rate of groundwater flow under undisturbed conditions; and (6) the homogeneity of the salaquifer. In computing recovery efficiency, we have assumed that offtake will stop when 3 percent of the native water appears in the discharge stream ("breakthrough" of the native groundwater). One of our findings is that the effect of molecular diffusion (between salt and fresh water) is of negligible importance.

HORIZONTAL, HOMOGENEOUS AQUIFER

Attempts to write equations that subsume all of the parameters listed in the previous paragraph were, and still are, unsuccessful. So a step-by-step approach was adopted, starting with a minimum combination of parameters. After physical verification of the results of the computational process, the additional parameters were systematically included, one at a time. The simplest set of conditions consists of a horizontal, isotropic, homogeneous aquifer where one well is installed for injection and offtake purposes, and there is no pre-existing groundwater flow.

The physical verification of proposed equations has required the construction of miniature aquifers ("mini-aquifers") whose boundaries and hydrologic characteristics could be determined using the field-test techniques of groundwater engineering. As used by the writers, the mini-aquifers have been constructed of an epoxy-consolidated, uniform blasting sand, by a technique similar to that used by Caudle (1963) and by Esmail and Kimbler (1967). However, our mini-aquifers have generally been larger than those described in the literature, the most recent having dimensions of 305 cm by 146 cm by 3.8 cm, a porosity of 25 percent, a permeability of 5.67 darcys (approximately 96 gal/day per sq ft at 72°F), and a transmissivity of 0.71 darcy-ft (11.95 gal/day per ft at 72°F). Figure 1 is a photograph of the largest mini-aquifer that we have constructed. Half of a nine-well array is installed in it and the mini-aquifer's boundary is an isopotential. For reasons of symmetry our construction must be considered as half of a mini-aquifer 305 cm by 292 cm, bordered completely by an isopotential.

Fluid is injected into or withdrawn from each well by means of constant-speed positive-displacement pumps (Fig. 2), each of which has been precisely machined and calibrated. Each pump consists of a cylinder and a piston powered by a synchronous motor. By means of properly selected gear ratios the rate of injection or offtake can be set at a predetermined value. The available injection and production rates range from 8.05 cm^3/minute to 1.34×10^{-2} cm^3/minute. The output of any well can be individually monitored by a chemical oscillometer to detect changes in fluid composition, as the native and injected fluids not only

are different in color but also possess different electrical characteristics.

The frontal position of the fluid can be seen at the upper and lower surfaces of the mini-aquifer, and as part of our study we photograph both surfaces of the model simultaneously. In this way we can determine the leading and trailing positions of the interface and calculate the angle of lay-down that results from the density difference between the injected and native fluids. Figure 3 shows the position of the front at two points in the injection-withdrawal cycle on a test of a single-well operation. Note the circularity of the front, which demonstrates the homogeneity of the model.

To minimize the effect of the boundary conditions, the front of the injected fluid is never allowed to move more than 60 cm from the center of the well field. Our observations of the frontal position and shape of the front, plus the results of the computational model (using the isopotential boundary as compared to the assumption of an infinite aquifer), show that, as long as the front is never allowed to move more than 40 percent of the distance to the boundary, there should be no significant difference in the recovery ratios for the horizontal-aquifer, single-well case.

The characteristics of the mini-aquifer are listed in Table 1. The recovery rates of two representative tests of the mini-aquifer are compared with the computed rates and are shown in Table 2.

THE POTENTIAL OF SALAQUIFER STORAGE

On the basis of the results shown in Table 2, the practical significance of the process can be approximately evaluated. Let us assume that a municipal water-treatment plant, with unused capacity during the flood season, requires additional storage. Table 3 extends the results of our computational procedure to field conditions.

Evaluation of the cost of aquifer storage shows that the density difference between the native water and the injected water is the most important single variable that affects cost. For a single well the calculable capital cost might be estimated at $200,000, which would include construction of the well and cost of pump, piping, appurtenances, and controls. Even if we estimate a 20-year life for this system, the annual interest and amortization cost would be $17,500. As water would be stored during times of surplus, only the incremental cost of chemicals, power, and personnel used to process the extra water would be properly chargeable--there would be no overhead and no capital recovery on the basic treatment plant (we recognize that this approach is subject to argument and that divergent opinions exist as to how to apportion such charges).

If as much as 200 million gal were irrecoverably lost in reaching a steady-state, 95-percent-recovery, cyclic operation, at an incremental cost of water treatment of 5 cents per kilogallon, a cost increase of only $40,000 for water would be chargeable to the capital account--about 20 percent of the original cost of construction. This would be due to the density of the native groundwater at 35,000 ppm total dissolved solids (TDS). The 3,500-ppm native water would require less irrecoverable water to be lost in reaching the 95 percent cyclic efficiency. Thus the annual cost of storing 120 million gal might be placed as high as $22,000 or $23,000 a year. Annual cost of power for groundwater production and replacement of irrecoverable water lost during each cycle after steady state is reached might be as much as an additional $1,000/year.

In the New Orleans area the alternate possibility to the storage of water in a saline aquifer, which would protect the water from contamination during hurricanes and evaporation during periods of drought, is the construction of steel tanks on the land surface. According to Peter Russo, Superintendent of the Jefferson Parish Water Department, who recently completed construction of steel storage tanks, a good cost estimate is ten cents per gallon of water stored. On the basis that cyclic operation ultimately would result in 95 percent recovery or an effective storage of 120 million gal, it would be necessary to construct steel tanks with a capacity of approximately 120 million gal for surface storage of the water. The cost would be approximately $12 million. On the basis of a 6 percent interest rate and a 50-year life for the tanks, the annual cost of water storage would be approximately $760,000/year and the cost of the land must be added to these figures.

In summary, there is potentially an overwhelming economic advantage to the use of subsurface storage as compared to surface tanks or even as compared to well-protected surface reservoirs. The loss of irrecoverable water would probably not be in excess of water loss due to evaporation from a surface reservoir of reasonable depth. In an industrialized, populated area, however, the proper comparison must be with steel tanks.

THE MULTI-WELL CASE

In a field application of the process, however, it is unlikely that a single well will suffice to establish storage in a salaquifer. Although

the water might be recharged at a slow rate over a long period of time, the water demand is likely to be high over a shorter time interval. So it is likely that a number of wells will be installed, for the most part to improve water deliverability during time of use. In fact, should a number of such well fields be established as part of a municipal water-supply system, they might be used to supply the peak demands as well as long-term water supply when the surface source is temporarily shut down.

The configuration or size of a well field cannot be determined in advance. However, we have studied the operation of a single well and of multiple wells in the mini-aquifer and have attempted to devise computational models of such well fields. As might be anticipated, the exact method of operation affects the recovery rate. For instance, injection might be started simultaneously in all of the wells (Fig. 4) and, after a storage period, the demand might be met by starting the wells in some convenient sequence. Or, the center well of the array might be used for injection until the freshwater front passed through the closest injection wells and then, as the front passed through additional wells, each would, in turn, be used for injection. A reverse procedure might be used in satisfying demand--first all of the wells would be on, then, consecutively as breakthrough occurred in each well, the wells would be turned off.

Owing to the number of operational possibilities and the difficulties with the computational model (it is still being worked on and improved), it is simplest to present the results by comparing the multi-well recovery rate with the single-well recovery rate (the injected volume of water, in each instance and in each cycle, being approximately the same). Table 4 can be used as a guide to potential recovery rates of multi-well systems as compared to those of single wells in the same system. Our series of experiments is not yet complete, but we can make some predictions. The first-cycle recovery rate will inevitably be less than that of a single well that injects the same total quantity. However, study of the data from the multi-well runs indicates that, as the number of cycles increases, the per-cycle recovery of a multi-well configuration will equal, or surpass, the performance of a single well.

It should be noted that, except for Run No. 7, when all wells were used for injection and production, the multi-well tests used the center well for injection and all wells for production to obtain the results shown. Since the mini-aquifer is really half of a symmetrical mini-aquifer, the well fields are really half the size of the complete well fields. Thus injection and production of the wells situated on the boundary--the line of symmetry--is one-half of the corresponding production or injection of the other wells. The recovery rates, however, are applicable to the entire well field. In producing the well field in a multi-well run, all wells were operated until breakthrough occurred at a particular well. The particular well was then shut in and production continued until breakthrough occurred at another well, which was, in turn, shut in. The production rates given in the table are those at the start of the production period in each instance. The total production recorded is the result of following the shut-down procedure described previously.

In the foregoing comparison, based on Table 4, the dispersion, which depends solely on travel distance, a length which in turn is determined by the volume injected, is almost constant. So the difference in recovery rates is probably due to the effect of density difference, thus lay-down. We surmise, however, that the larger the mix zone, the more favorable the recovery rates during the cyclic operation of a well field for subsurface storage. Cyclic operation, which increases the distance traveled by the front, hence the mix zone, therefore can be expected to improve recovery rates.

The printout of the computer program used for computing recovery rates for one cycle is available on request. The card deck can be supplied at nominal cost.

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Table 1. Aquifer Characteristics

Thickness	3.8 cm
Porosity	0.25
Permeability	5.67 darcys
Transmissivity	11.95 Meinzer-ft at 72°F

Table 2. Recovery Rates--Horizontal Aquifer, Single Well, One Cycle

	Run No. 1		Run No. 6	
	Observed	Computed	Observed	Computed
Total quantity injected, cm^3	5170	5170	4300	4300
Rate of injection, cm^3/min	24.138	24.138	17.880	17.880
Density difference $\Delta\rho$	0.0019	0.0019	0.0780	0.0780
Storage period, sec.	0	0	0	0
Offtake rate, cm^3/min	24.138	24.138	17.880	17.880
Quantity recovered, cm^3	4797	4653	3194	2795
Recovery rate	0.93	0.90	0.74	0.65

Table 3. Aquifer Characteristics and Computed Recovery Rate

Aquifer thickness:	75 ft	
Aquifer transmissivity:	5.0×10^4 gal/day/ft	
Aquifer storativity:	5×10^{-4}	
Aquifer porosity:	.30	
Native water density:	1.025 (35,000 ppm TDS)	1.0025 (3,500 ppm TDS)
Static water level:	10 ft below land surface	
Depth to roof of aquifer:	300 ft	
Mobility ratio $\left(\frac{\mu_n}{\mu_i}\right)$:	unity	
Wellbore radius:	1 ft	
Injection rate:	1,000 gpm	
Max. surface injection pressure computed:	41 ft or 18 psi	
Max. allowable pressure:	190 ft or 82 psi	
Storage volume:	130 million gal (90 days at 1.44 million gal/day)	
Static storage:	150 days	
Withdrawal rate:	800 gpm	
Recovery rate, first cycle:	.11	.59

	Run Number 1	2	3	4	5	6	7
Density Difference $\Delta\rho$	0.0019	0.0015	0.0022	0.0023	0.0112	0.0780	0.0780
1st Cycle:							
Quantity injection (cm^3)	5170	5170	5174	4754	5181	4300	5932
Injection rate (cm^3/min)	24.138	24.138	6.705	18.285	18.285	17.880	18.084
Static storage (sec)	0	0	0	585	515	0	0
Production rate (cm^3/min)	24.137	24.138	6.705	18.285	18.285	17.880	18.084
Quantity production (cm^3)	4797	4867	4707	4082	4675	3194	1517
Recovery efficiency	0.93	0.94	0.91	0.86	0.90	0.74	0.26
2nd Cycle:							
Quantity injection (cm^3)		5170	5174		5181	4300	
Injection rate (cm^3/min)		24.138	6.705		18.285	17.880	
Static storage (sec)		0	0		325	0	
Production rate (cm^3/min)		24.138	6.705		18.285	17.880	
Quantity production (cm^3)		4932	4866		4672	3760	
Recovery efficiency		0.95	0.94		0.90	0.87	
3rd Cycle:							
Quantity injection (cm^3)		5170			5181	4300	
Injection rate (cm^3/min)		24.138			18.285	17.880	
Static storage (sec)		0			420	0	
Production rate (cm^3/min)		24.138			18.285	17.880	
Quantity production (cm^3)		5036			5083	3937	
Recovery efficiency		0.97			0.98	0.92	

Run No. 1--Single-well injection, single-well production.
Run No. 2--Single-well injection, single-well production.
Run No. 3--Single-well injection, single-well production.
Run No. 4--Single-well injection, three-well production.
Run No. 5--Single-well injection, three-well production.
Run No. 6--Single-well injection, single-well production.
Run No. 7--Multiple-well injection, multiple-well production, run terminated prematurely.

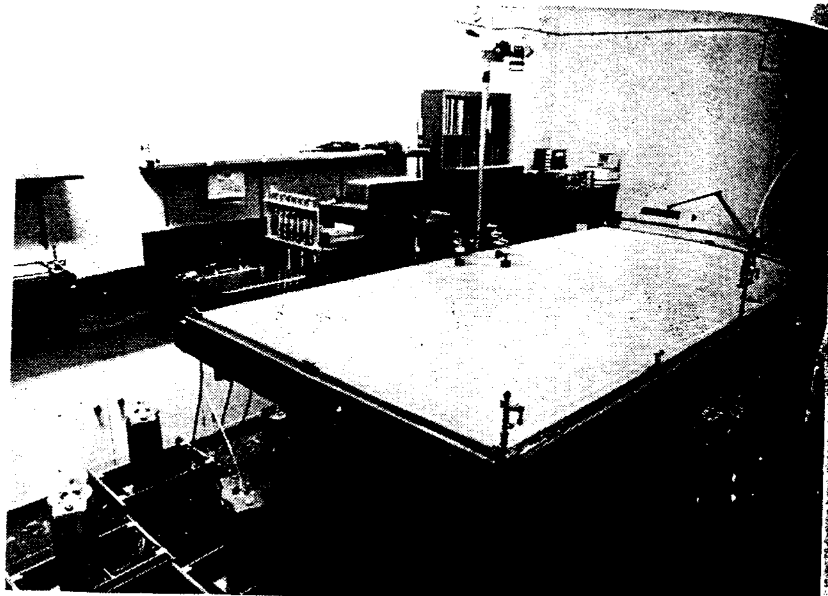

FIG. 1--View of mini-aquifer, pumps, and instrumentation. Note camera on support for photographing frontal position of injected fluid. In left fore-ground are three large-barrel pumps and one small-barrel pump. Chemical oscillometer and recorder are on stand just above well field. To left of oscillometer are individual controls for each well.

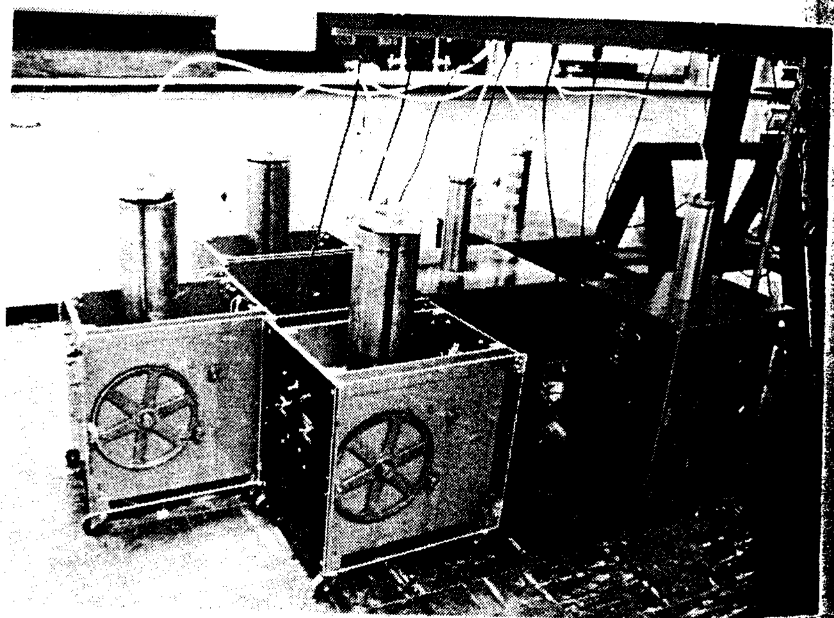

FIG. 2--View of three large-barrel and three small-barrel pumps used in multi-well test in mini-aquifer.

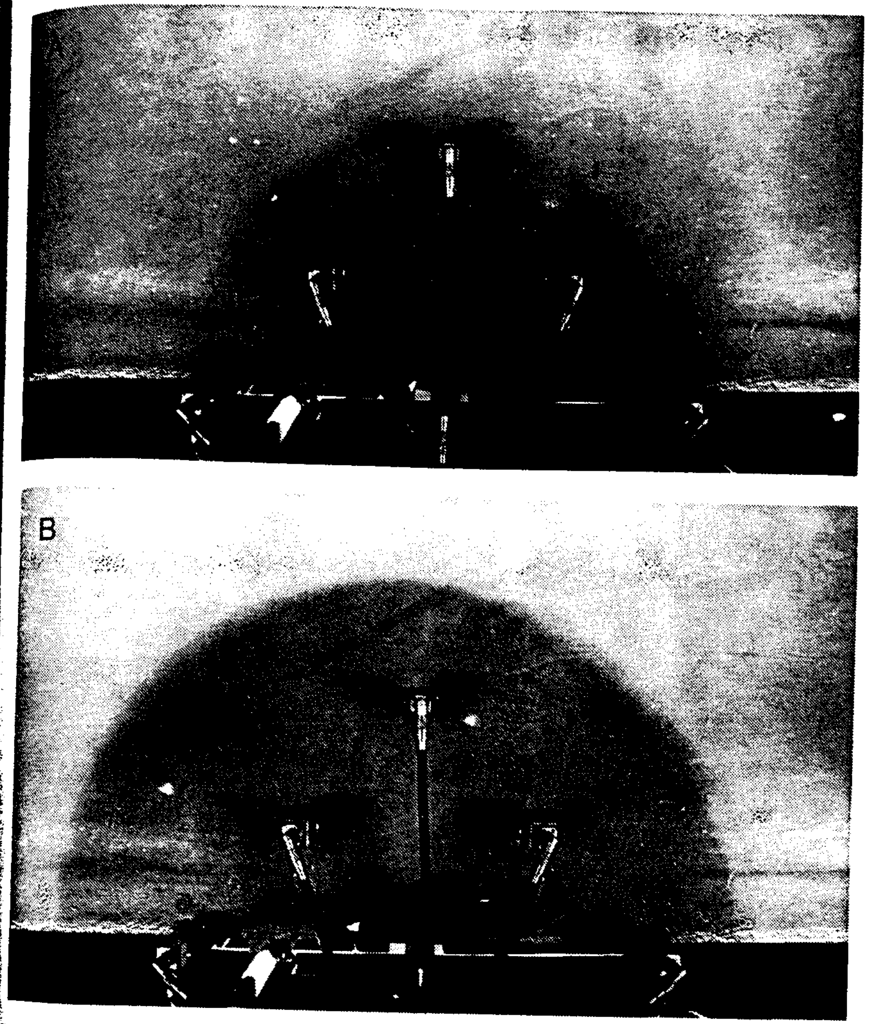

FIG. 3--Movement of front at two times during injection of fluid from center well of array. Note circular appearance of front, which indicates homogeneity of mini-aquifer in vicinity of well field.

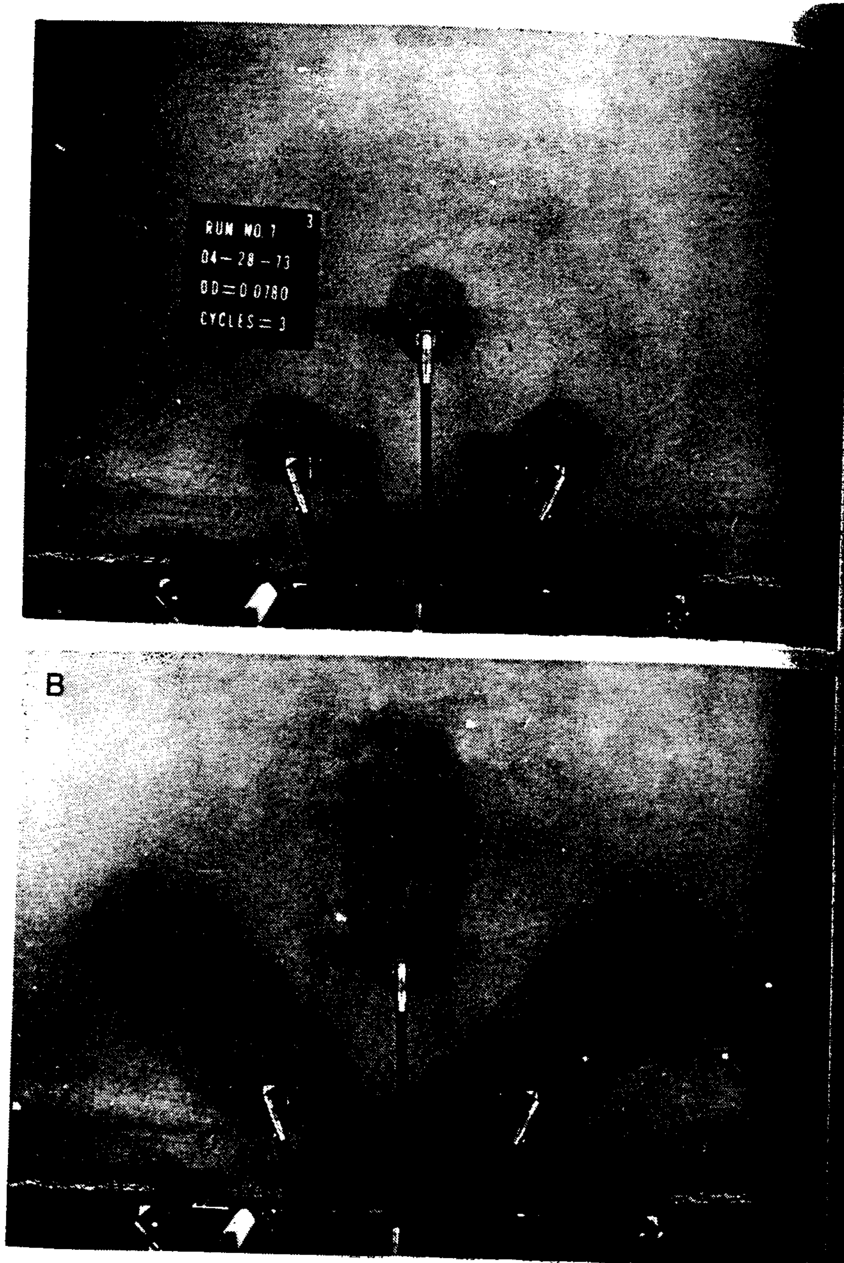

FIG. 4--A. Frontal positions during multi-well injection. Center [illegible] a circular front; frontal shapes at all other wells show distortion [illegible] interference. B. Frontal positions after a long period of injection [illegible] tortion due to well interference has increased. Almost square front [illegible] tion associated with center well of array is faintly visible.

FEASIBILITY STUDY OF A SEISMIC REFLECTION MONITORING SYSTEM FOR UNDERGROUND WASTE-MATERIAL INJECTION SITES[1]

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ABSTRACT Injection of waste materials into deep subsurface formations is becoming an increasingly popular method of waste disposal. Attendant to this growing practice is an increasing possibility of accidental damage to the subsurface and surface environments. An effective method of monitoring the movement and distribution of these injected waste materials is needed.

In most cases, the acoustical properties of the receiving formation (i.e., its density and the velocity of propagation of compressional sound waves) will be changed upon contact with the injected waste materials. These changes will subsequently change the reflection coefficient encountered by a vertically traveling sound wave at the receiving formation. It is this change in acoustical properties that suggests the application of modern seismic reflection and data-processing techniques to this monitoring problem.

The seismic reflection technique involves the introduction of acoustical energy into the earth from the surface and the recording of signals at or near the surface which are indicative of the travel time and amount of energy reflected from each of many closely spaced points along each subsurface formation. The use of the seismic method on a periodic basis is therefore proposed to detect changes of acoustical properties in the receiving formation and surrounding formations to monitor effectively the movement and position of the waste materials.

The assumed seismic field system for this monitoring system includes the following components: (1) permanent arrays of velocity geophones buried a small distance below the surface of the earth, (2) a multichannel digital recording system of the instantaneous-floating-point type, (3) truck-mounted seismic surface energy sources, and (4) the use of a digital

[1]Manuscript received, June 8, 1973.

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seismic data-processing center with special software.

The feasibility of the monitoring system is studied by means of an acoustical model derived from well-log information for a typical Gulf Coast injection well. It is determined that, under the most severe expected field conditions, a change of at least 7.34 percent in the magnitude of the reflection coefficient at the injection zone is required to monitor reliably the waste material in this particular injection well. As this amount of change in the reflection coefficient might well be expected, the feasibility of the seismic reflection monitoring system appears to be confirmed.

INTRODUCTION

The use of deep-well waste-emplacement systems is increasing in the United States, and with it the need for an effective method of monitoring the subsurface position of injected waste materials. The present practice of drilling monitor wells to the injection zone is unsatisfactory for technical, economic, and safety reasons.

In an effort to develop an effective monitoring system, this study attempts to determine the feasibility of using the seismic reflection method (a type of acoustical radar) and modern data-processing techniques to determine the subsurface position and movement of injected waste materials.

SEISMIC REFLECTION METHOD

As illustrated in Figure 1, the seismic reflection technique involves the introduction of acoustical energy into the earth at or near its surface. This acoustical energy propagates away from the seismic energy source and is partially reflected back to the surface from each interface between materials which have dissimilar acoustical properties. For simplicity, only the ray paths of acoustical waves reflected from the top of the injection zone are shown. Figure 2 illustrates the relations between the amplitudes of the incident, reflected, and transmitted sound waves at each such interface. Transducers, called "geophones," at or near the earth's surface convert this reflected acoustical energy to an electrical signal which is subsequently amplified and recorded on magnetic tape by digital recording instruments. As illustrated in Figure 1, by recording as separate signals the voltages of several geophones placed increasing distances from the seismic energy source, a measure of the reflected

energy from as many separate points along each reflecting interface can be obtained. A display of such signals resulting from the use of an impulsive type of seismic energy source is shown on the right side of Figure 1.

The use of an impulsive type of seismic energy source, such as dynamite buried at a small depth, in the vicinity of a waste-injection well might physically damage the well at or near its surface. Therefore, the use of a low-amplitude, long-duration, frequency-modulated sinusoidal energy source is more desirable. This type of source is called a Vibroseis[3] vibrator and has an output signal modeled after that of the "chirp" radar (Klauder et al., 1960). The long-duration sweep signal illustrated in Figure 3A is applied as a force to the surface of the earth. The resulting sound waves then propagate downward and are partially reflected back to the surface, transformed to time-varying voltage signals by the geophones, and amplified and recorded on magnetic tape by the digital recording instruments. Inasmuch as each reflection is represented by a long-duration oscillatory signal rather than an impulse, the resulting recorded signals bear no resemblance to the type recorded from an impulsive energy source. However, when, as a data-processing step, a representation of the sweep signal is cross-correlated with each of the recorded signals, each long-duration sweep signal is collapsed into a short-duration wavelet and the resulting signal appears almost identical to one recorded with the use of an impulsive energy source (Geyer, 1970). This data-processing step is illustrated in Figures 3B and C.

INJECTED WASTE MATERIALS AND THEIR EFFECT ON REFLECTION COEFFICIENT

As illustrated in Figure 2, the amplitude and polarity of a reflected sound wave depend upon the densities and velocities of sound propagation of the two materials involved. Assume, for example, that material no. 1 in Figure 2 is a shale ($\rho_1 = 2.3$ g/cm^3, $c_1 = 9{,}100$ ft/sec) and material no. 2 consists of unconsolidated sand saturated with salt water ($\rho_2 = 2.15$ g/cm^3, $c_2 = 7.400$ ft/sec). A sound wave reflected from the interface between these two materials would be opposite in polarity and 0.13627 times the amplitude of the incident sound wave.

If the unconsolidated sand's connate fluid were displaced by a liquid waste which had a different density and sonic velocity than those of salt

[3]Registered trademark and service mark of Continental Oil Company.

water, the overall density and sonic velocity of the unconsolidated would be changed, causing the negative reflection coefficient to change in magnitude. This, in turn, would cause the amplitude of the reflected sound wave to be changed. The possible success of the seismic reflection system as a waste-material monitoring system is based on the possibility of this phenomenon.

SEISMIC REFLECTION MONITORING SYSTEM

Assuming that the injected waste materials change the reflection coefficient at the injection zone and therefore indicate their location, periodic use of the seismic reflection monitoring system is suggested in order to detect changes as a function of time in reflection coefficients at closely spaced points along the injection zone. In other words, the first survey would be performed and the returned acoustical energy from each of the closely spaced points along the injection zone would be cataloged. Then at a later date the survey would be repeated in order to determine the points at which the reflection coefficient had changed. In this manner, the location of the waste materials might be traced as a function of time.

The required components for this monitoring system would include (1) permanent arrays of velocity geophones buried approximately 100 ft beneath the earth's surface, (2) a multichannel amplifier and digital recording system of the instantaneous-floating-point type, (3) three truck-mounted vibrator seismic energy sources, and (4) the use of a digital seismic data-processing center with special software.

In order to obtain information on the movement and location of the waste materials in all directions from the well, arrays of geophones such as the one illustrated in Figure 1 could be placed in a configuration similar to the spokes of a wheel with the well head at the hub, as illustrated in the plan view of Figure 4.

ACOUSTICAL MODEL

The feasibility of the seismic monitoring system depends on the ability of the recording system to register a very small change in signal amplitude due to a small change in reflection coefficient in the presence of random noise. Because of the extreme complexity of the received signal from the Vibroseis energy source and the presence of random recording-instrument and environmental noise, the only means of estimating the recording system's ability to register this small change in signal amplitude

is to simulate such a received signal from the earth in the vicinity of a typical waste-injection well. In order to do this, a valid acoustical model is required.

The acoustical model used in this study is derived from sonic and density well logs from a typical waste-injection well in the Texas Gulf Coast area. These well logs are illustrated in Figure 5. It can be seen that the injection zone is a 90-ft-thick sandstone at a depth of 6,275 ft. It can also be seen that the well-log information begins at a depth of 1,400 ft and ends at 6,500 ft. It is unfortunate that this information is incomplete because it decreases the accuracy of the model. However, quantitative estimates of densities and velocities from the surface to 1,400 ft are available and are used (Tullos and Reid, 1969).

In order to generate a representation of the voltage signal recorded by the digital recording instruments, the following steps are performed.

1. Reflection coefficients are computed by substituting velocity and density values from the well logs in the equation shown in Figure 2. The reflection coefficients are illustrated in Figure 6A.

2. Two-way travel times for the seismic energy are computed using the distance traveled by the sound waves and the average velocity to each reflector, which is determined from the sonic-velocity log.

3. Spherical-divergence losses are computed for each reflector according to the distance traveled by the sound waves (Dobrin, 1952).

4. Transmission-coefficient losses are computed for each reflector according to the transmission coefficients (Fig. 2) encountered by the sound waves in traveling to and from each reflector.

5. Inelastic-attenuation losses (which increase with the frequency of the sound waves; Dobrin, 1952) are computed for each reflector according to the distance traveled and frequency content of the sound waves, using the following estimates for the Gulf Coast area (Tullos and Reid, 1969): (a) 0-10 ft: -13.1 decibel/wavelength traveled, (b) 10-100 ft: -0.1508 decibel/wavelength traveled, (c) 100-500 ft: -0.3641 decibel/wavelength traveled, and (d) 500-6500 ft: -0.2004 decibel/wavelength traveled.

6. The force applied to the earth's surface by the seismic energy source (Fig. 3A) is converted to units of particle velocity by the expression (While, 1965):

$$v(t) = \frac{1}{2\pi\rho c}\, 2\, \frac{d}{dt}\,\{f(t)\}\ \text{ft/sec},$$

where ρ = density of the surface layer, slugs per cu ft;

c = sonic velocity of surface layer, ft/sec;

$f(t)$ = force as a function of time (Fig. 3A), lbs.

7. For each reflection coefficient, $v(t)$, multiplied by that reflection coefficient and attenuated by its corresponding spherical-divergence loss, transmission-coefficient loss, and inelastic-attenuation losses, is algebraically added into the received signal beginning at the two-way travel time corresponding to that reflection coefficient. For this study, $f(t)$ was chosen to be a 30,000-lb, 10-40-Hz sweep signal of one second duration.

8. Direct-arrival and refracted sound waves, illustrated in Figure 7, are added to the received signal.

9. The received signal is converted to units of voltage by using the transfer function of a typical velocity geophone, 0.5 volts/in./second.

The resulting geophone voltage signal is illustrated in Figure 6B.

RECORDING SYSTEM

In the actual field system, the geophone output voltage is input to the digital recording system whose block diagram is shown in Figure 8. For the purpose of this study, the specifications of a state-of-the-art instantaneous-floating-point system called the "Futura," manufactured by Electro-Technical Labs of Houston, Texas, were used. The geophone voltage signal is initially amplified by a fixed-gain preamplifier with a selectable gain from 18 to 42 decibels in 6-decibel increments. The resulting signal is then bandpass-filtered and sampled at a uniform rate (typically 500 samples per second). Each of these voltage samples is amplified in the instantaneous-floating-point amplifier by the amount necessary to increase its magnitude from 2.048 to 4.096 volts. The available amplification for this purpose is 0-90 decibels in 6-decibel increments. The value of the gain used is recorded on magnetic tape for each voltage sample. The amplified voltage is then converted to a 15-bit binary number (14 bits plus sign) which is equal to that voltage divided by 0.25×10^{-3} volts. This number is recorded on magnetic tape along with its associated gain value. This process is carried out for each sample of each geophone signal being recorded.

It can be seen from the above explanation that the amplified geophone voltage has been quantized into discrete 0.25×10^{-3} volt levels. It is at this point that the danger of failing to record a small change in signal amplitude occurs. However, a more troublesome complicating factor introduced by the recording instruments is the addition of random thermal

noise to the recorded data. With the preamplifier set to 42 decibels of gain, the root-mean-square value (Brenner and Javid, 1967) of this random noise is approximately 0.1×10^{-6} volts.

DATA PROCESSING AND SMALLEST DETECTABLE CHANGE IN REFLECTION COEFFICIENT

If there were no instrumental and environmental random noise recorded along with the desired geophone voltage signal, the necessary data-processing steps would be uncomplicated. They would simply consist of transforming the information on magnetic tape back to the actual geophone voltage signal, cross-correlating a representation of the vibrator sweep signal with this signal (Fig. 6C), and computing the energy contained in the resulting signal within a gate centered at the two-way travel time for the injection-zone reflection. This energy would then be compared to the energy obtained from the previous survey. However, the presence of the random noise causes the amount of energy possibly contained in that gate to be a random variable. Therefore, a means of determining whether a change in energy was caused by a change in reflection coefficient is required. In order to overcome this problem, the field technique and data-processing steps must be expanded. At this point, it is convenient to explain simultaneously the necessary added steps and the method used in this study to estimate the smallest detectable change in reflection coefficient.

As the energy of the signal within the gate has an infinite number of possible values for any one noise-to-signal ratio, it is necessary to sample this "infinite population" in order to attempt to determine its mean value. This is accomplished by taking multiple rather than single field recordings at each vibrator location. If the number of field records is chosen to be ten, this means that during the survey ten traces will be obtained for each reflecting point along the injection zone instead of one.

Now, let us concentrate on ten traces corresponding to one particular reflecting point on the injection zone. For the purpose of this study, these ten traces were produced by adding ten different sets of 0.1×10^{-6}-volt root-mean-square random instrument noise plus random environmental noise at a specified level to the trace shown in Figure 6B. For the example to be explained, this level was chosen such that the root-mean-square value of the environmental noise was equal to that of the received reflection energy from the injection zone. This is referred to as a noise-to-signal ratio equal to 1.0. These traces were then quantized and cross-correlated, and the energy within the gate was computed. The ten resulting energies are shown in Figure 9 to be randomly distributed between 0.032633471 joules and 0.03879338 joules. The average of these energy values is computed to be 0.035297337 joules. However, this is only an estimate of the true mean value of all the possible values of energy obtainable and is therefore fairly meaningless without some measure of its accuracy. It is possible to compute from the ten energy values a range of energy values which we are 95 percent confident contains the true mean (Ostle, 1963). For this example, as shown in Figure 9, this 95-percent confidence interval is the range of energies between 0.0338571627 joules and 0.0367373047 joules.

The crucial question now presents itself. If we return a month later and obtain ten traces for the same reflection point, and compute their ten energies and their 95-percent confidence interval, how large a change in reflection coefficient would be necessary in order that this new 95-percent confidence interval would not overlap the one obtained a month earlier and displayed in Figure 9? It was determined by use of the acoustical model that a change of approximately 7.34 percent in the reflection coefficient at the injection zone would be necessary with ten samples and a noise-to-signal ratio of 1.0. Figure 10 illustrates the relation between this minimum change required in reflection coefficient and the environmental noise-to-signal ratio for ten samples as well as twenty samples. It can be seen that smaller changes in reflection coefficient can be detected with twenty samples than with ten at all values of noise-to-signal ratios. This difference is due to the fact that the larger number of energy values yields a better estimate of the true mean value and a correspondingly narrower 95-percent confidence interval.

So, the expanded field and data-processing technique involves obtaining multiple recordings for each reflecting point along the injection zone, reconstructing the original geophone voltage signals, cross-correlating these signals with the vibrator sweep signal, computing the desired energies, computing their 95-percent confidence interval, and comparing this confidence interval with the one computed from the previous survey. If they do not overlap, there is an extremely good probability that a change in reflection coefficient has been detected.

EXPECTED QUANTITATIVE EFFECTS OF WASTE MATERIALS ON REFLECTION COEFFICIENT

For the typical Texas Gulf Coast injection well used in this study, the injection zone is a thick sandstone of density 2.15 g/cm^3 and sonic

velocity 7,400 ft/sec. The overlying material is a shale of density 2.3 g/cm^3 and sonic velocity 9,100 ft/sec. Sandstones at about 6,000 ft of depth which are used for waste injection in this area generally have a porosity of approximately 30 percent and a salt-water connate fluid having a density of about 1.07 g/cm^3 and a sonic velocity of approximately 5,250 ft/sec (Press, 1966).

According to the Texas Water Quality Board, which regulates injection well activities in Texas, most of the waste materials being injected in the Gulf Coast area are mixtures of substances such as glycol and phenol and have densities typically in the range of 0.993-1.02 g/cm^3. If the waste material were to have an identical sonic velocity to that of salt water, this range of densities would cause the reflection coefficient to increase in magnitude by 2.5-3.87 percent. For an expected environmental noise-to-signal ratio between 0.5 and 1.0, it can be seen from Figure 10 that the effect of the density of the waste material alone would not be sufficient. However, it is unreasonable to expect the sonic velocity of the waste material to be identical to that of the connate salt water. It happens that to obtain a 6.73 percent increase in reflection-coefficient magnitude, the sonic velocity of the waste material would have to be 195.5-288.34 ft/sec slower than that of salt water for the corresponding range of densities 0.993-1.02 g/cm^3. Alternatively, to obtain a 6.73 percent decrease in reflection-coefficient magnitude, the sonic velocity of the waste material would have to be 736.5-640.1 ft/sec faster than that of salt water for the same range of densities. Since most of the typical waste-material constituents have sonic velocities less than or equal to that of salt water (Gray, 1957), the increase in reflection-coefficient magnitude is probably the change that is more likely to occur. The preceding calculations ignore the possibility that the waste materials may be miscible with the connate fluid. Of course, if they are, greater contrasts in sonic velocities would be necessary.

CONCLUSIONS

Inasmuch as the required sonic-velocity contrasts computed in the previous section are not unreasonable and as many injection wells are shallower than 6,275 ft, it can be concluded from this study that the seismic reflection monitoring system appears to be feasible. This conclusion is strengthened by the fact that a substance might be added to the injected waste material to insure the necessary contrasts in acoustical properties. However, due to the inherent limitations of a "paper-andpencil" feasibility study concerning acoustics in a material as complex as the earth, the actual feasibility of such a seismic reflection monitoring system must be determined by an actual field test.

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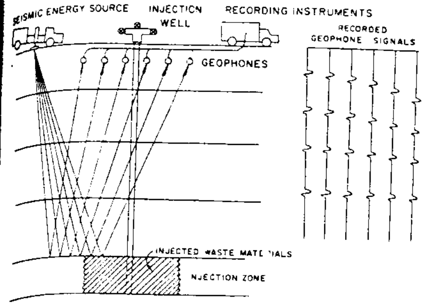

FIG. 1.

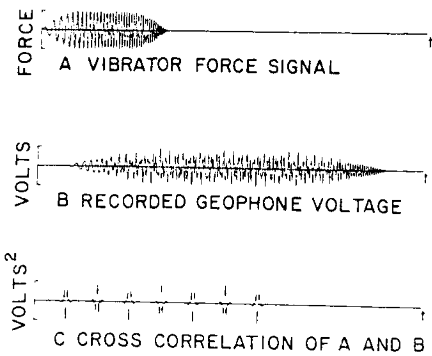

FIG. 3.

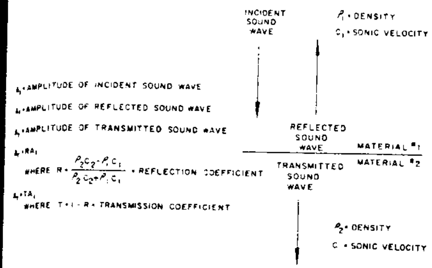

FIG. 2.

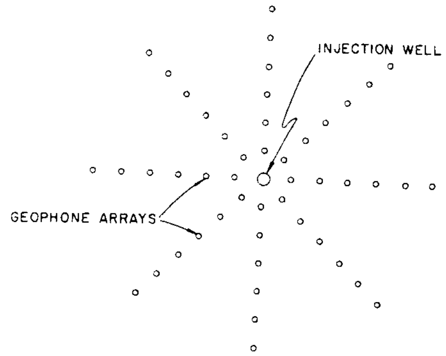

FIG. 4.

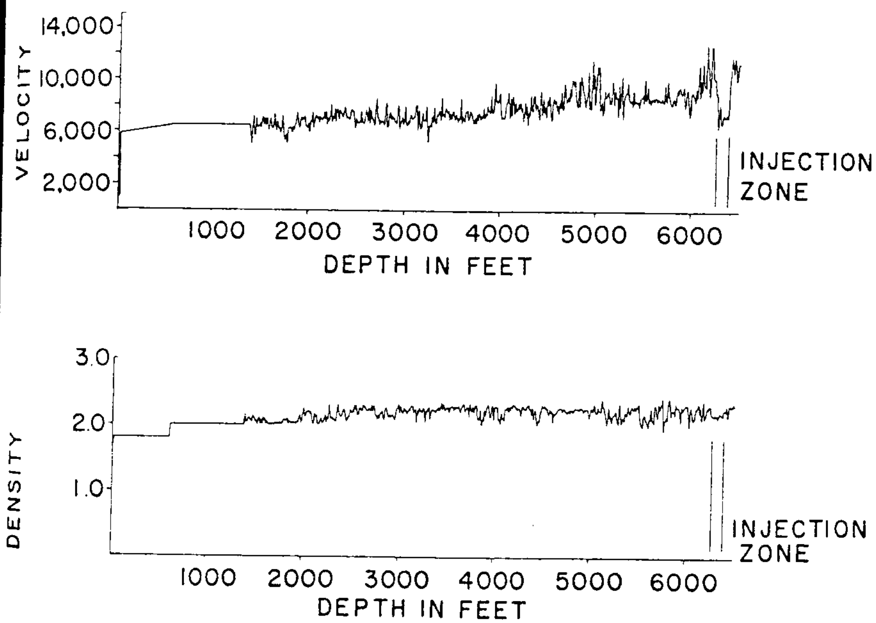

FIG. 5.

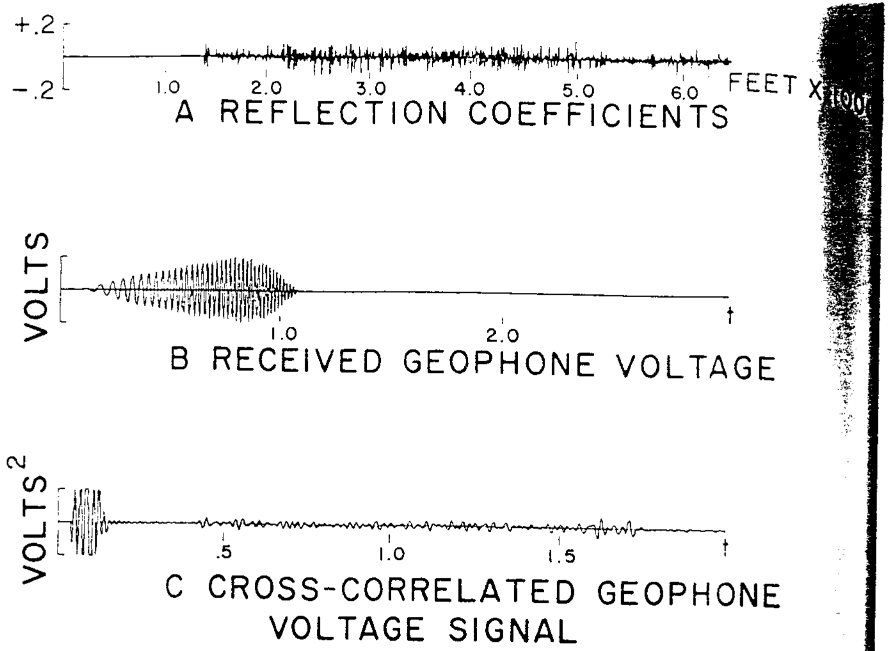

FIG. 6.

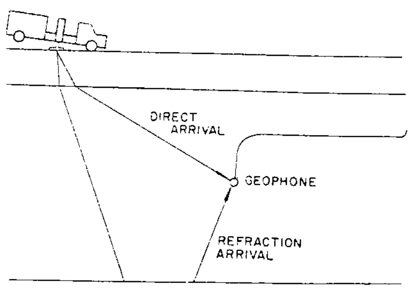

FIG. 7.

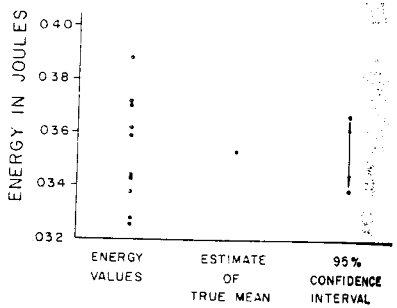

FIG. 9.

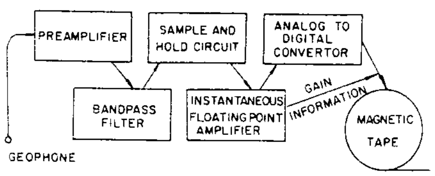

FIG. 8.

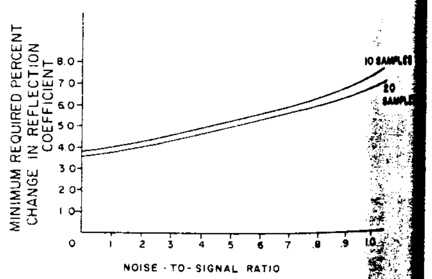

FIG. 10.

HYDRAULIC FRACTURING AS A TOOL FOR DISPOSAL OF WASTES IN SHALE[1]

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The writer is especially indebted to Dr. Wallace de Laguna, ORNL, for providing the injection data and gamma-ray logs, as well as his thought on the pressure-decay method. The writer wishes to thank his colleague, George H. Chase, for furnishing geologic information, which he interpreted from geophysical logs or obtained in the field, as well as for his beneficial work with the writer in the field. The writer is grateful to his supervisor, George D. Debuchananne, USGS, and his former supervisor, Robert Schneider (presently at Office of Water Resources Research, Department of the Interior), for their patience and encouragement on this study. K. A. Slagle (Halliburton Co.) provided the information on the characteristics of the grout which was injected on July 23, 1971, and his help also is gratefully acknowledged.

ABSTRACT The injection of wastes mixed with cement grout into thick shale formations is a promising method for effective immobilization of toxic wastes in a nearly impermeable medium. Hydraulic fracturing provides openings in the shale during the grout injection. To retain certain types of wastes better, ion-exchange and adsorption agents can be added to the grout when it is mixed. After solidification of the grout, the injected wastes will become an integral part of the shale and remain there as long as the shale is not subject to erosion.

Problems concerning the safety of the method are: (1) the possibility that liquid wastes might separate from the grout during and after the injections and migrate through natural fractures; and (2) the reliability of methods of determining that orientation of the hydraulically induced fractures is horizontal or nearly so. During solidification of the grout, liquid wastes might separate from the grout. In such an instance, the mobility of wastes in the separated liquid would be limited by the very low permeability of shale. If the separated liquid wastes were to reach a groundwater reservoir, the concentration of contaminants would be greatly reduced further by hydraulic dispersion in the native groundwater.

In bedded shale there is a great difference in tensile strength between the direction normal to and the direction parallel with bedding planes. This difference in tensile strength may favor the formation of hydraulically induced fractures along bedding planes within a zone of limited vertical extent. However, hydraulic fracturing may not produce bedding-plane fractures in all shale formations; therefore, before construction of a waste-disposal facility it is necessary to test the site by injecting water or non-toxic grout tagged with radioactive tracers to judge whether a "zone" of fractures can be induced parallel with the shale bedding planes. Injection pressure, pressure decay, movement of the ground surface, and gamma-ray logs of observation wells may be used to interpret the orientation of the hydraulically induced fractures during the site-selection tests. A case history of hydraulic fracturing at West Valley, New York, illustrates the method of evaluating a site.

Waste disposal through an injection well should be conducted in multiple-layer injection stages. The first injection starts at the greatest depth, then the injection zone is plugged off by cement, and the second injection is started at a suitable distance above the first one. The repeated use of the injection well distributes the high cost of construction of injection and monitoring wells over many injections, thereby making hydraulic fracturing economically attractive as a tool for the disposal of certain types of wastes.

INTRODUCTION

The growth of modern society and technology has given rise to a growing problem of waste disposal, particularly of radioactive and toxic industrial wastes. As surface disposal is becoming more restricted to protect the environment, underground disposal has become more attractive.

One of the underground-disposal methods is to inject wastes through a deep well into a permeable formation which is sufficiently removed from points of water discharge or potential use so that, should the wastes reach the biosphere, it is hoped that their concentration will have been reduced to a harmless level by dispersion or physical and chemical changes. However, the concept of disposing of waste into shale by hydraulic fracturing is entirely different. Here wastes, mixed with cement as a grout, are injected into a thick, nearly impervious, shale formation. The grout is allowed to solidify under pressure to form grout sheets which will become an integral part of the shale, thereby immobilizing the wastes in the impervious medium. Hydraulic fracturing during the injection serves as a tool to provide openings in the shale.

The technique of hydraulic fracturing is widely used in the petroleum industry for recovery of oil. However, disposal of wastes in shale by grout injection using hydraulic fracturing as a tool has never been used except at the Oak Ridge National Laboratory (ORNL). In 1958 D.A. Shock of the Continental Oil Company suggested to the U.S. Atomic Energy Commission (AEC) that hydraulic fracturing with grout injection into well-bedded, nearly impermeable rock, such as shale, might be used in the disposal of highly toxic radioactive wastes (de Laguna et al., 1968). The AEC accepted the suggestion and the ORNL was chosen as the experimental site.

From 1959 through 1960, three experimental injections without actual wastes were made into the Conasauga Shale at two different locations at the ORNL. The grout was tagged with the radioactive isotope Cs^{137}. The first injection was at shallow depth, 290 ft below land surface. After the injection, 22 core holes were drilled in the vicinity of the injection area. The core data show that the solidified grout sheet was formed nearly parallel with the bedding plane. The second and third experiments were made at a new well which was drilled about 6,000 ft east of the first injection well. Similar grout was used in these two injections, which were made at depths of 934 ft and 694 ft, respectively. After the injections, 24 core holes were drilled in the vicinity of the second experimental area. The core-hole data confirmed that the grout sheets were again formed nearly parallel with the bedding planes.

From 1964 through 1965, eight experimental injections, with actual radioactive wastes produced at the ORNL, were made through a third well drilled into the Conasauga Shale about 0.5 mi west of the second experimental well. The injections were made at progressively shallower depths from 945 ft to 872 ft. From 1966 through 1970, regular waste-disposal operations were carried out through this well from depths of 872-842 ft. From 1964 through 1970, a total volume of 1,712,966 gal of grout containing 1,061,440 gal of intermediate-level radioactive wastes produced at ORNL was injected. These wastes contained 355,820 curies of Cs^{137}, 27,932 curies of Sr^{90}, 1,491 curies of Ru^{106}, 1,041 curies of Co^{60}, and 68.83 g of Pu^{239}. Core drilling and gamma-ray logs in observation wells also suggested that the grout sheets were formed parallel with bedding planes (de Laguna et al., 1963; 1971).

The demonstration at the ORNL shows that disposal of wastes in bedded shale using hydraulic fracturing is feasible. However, through the National Academy of Sciences, earth scientists advised the AEC not to extrapolate the results obtained at the ORNL site to other locations in the nation (Belter, 1972).

In view of the earth scientists' advice, the AEC sponsored an experimental program carried out jointly by the ORNL and the U.S. Geological Survey (USGS), with the permission of the State of New York, to test the concept further at the Western New York Nuclear Fuel Service Center near West Valley in Cattaraugus County, New York. The objectives of this program were (1) to demonstrate the applicability of disposal of wastes in bedded shale through hydraulic fracturing at another location, (2) to develop a method for judging the orientation of the induced fractures, and (3) to develop economical site-evaluation procedures (Belter, 1972).

The primary purpose of this paper is to evaluate the feasibility of disposal of wastes in shale through hydraulic fracturing on the basis of the experience at ORNL and at West Valley, New York.

SAFETY CONSIDERATIONS

Ideally, after solidification of the grout the injected wastes should become an integral part of the shale and remain there as long as the shale is not subject to erosion, or the wastes are not subject to leaching by percolating groundwater. However, two problems concerning the safety of the method are frequently raised. They are (1) the possibility that liquid wastes might separate from the grout during and after the injections

and migrate through natural fractures; and (2) the reliability of methods of determining with certainty that orientations of the hydraulically induced fractures are horizontal, or nearly so.

The injected wastes may separate from the grout either during the solidification of the grout when part of the fluid containing wastes may separate from the grout, or after solidification of the grout when waste materials held in the grout may be leached out by groundwater. Waste separation can be minimized by using a suitable grout mixture. The grout mixture has two essential characteristics: (1) it must hold large percentages of wastes; and (2) it must provide sufficient mobility during injection. The best grout mixture can be designed from laboratory tests. For example, the most important constituents of wastes produced at the ORNL are Cs^{137} and Sr^{90}. Laboratory tests at ORNL showed that the retention of radiostrontium in cement mixtures was high and that higher cement content and higher chemical concentration of the wastes favored retention; in low-cement mixtures the addition of fly ash improved retention. Radiocesium retention in cement was poor but could be improved by the addition of illite. Attapulgite used instead of bentonite as a cement substitute also was found to play a part in the retention of radiocesium (Tamura, 1971).

At the ORNL, after each hydraulic fracturing injection the wellhead of the injection well was closed. The grout was then allowed to solidify under pressure. After the scheduled time for solidification had passed, the well-head valve was opened and fluids remaining in the injection well flowed back to the surface under pressure. About 5 to 10 percent of the total injected volume of fluids flowed back. However, the waste concentration found in the flowback liquid suggested that only a few tenths of a percent of the total injected wastes failed to be contained in the grout (de Laguna et al., 1971). The large amount of flowback suggests that the permeability of the shale at the site of ORNL is very low; otherwise the fluid pressure in the injection well would have dropped as the fluid moved through the shale, and it would have been impossible to have such a large amount of flowback fluid. De Laguna (1966) suggests that the permeability of the Conasauga Shale at ORNL is about 10^{-4} md.

To evaluate the phenomena of leaching of wastes from the solidified grout, the ORNL laboratory used grout which had been cored from wells in the vicinity of the injection well. The cores were ground and sieved in a leachability test. Particles passing a 60-mesh screen were put into a bottle containing 1 g of solids per 100 ml of distilled water. The results of tests of a grout containing illite show that less than a few hundredths of a percent of the original radioactivity was leached during a 504-hour test (Tamura, 1971). However, conditions of this test deviate greatly from those in the ground. Cores of the grout obtained at ORNL suggest that the solidified grout sheets are strongly integrated with the nearly impervious shale, and therefore groundwater probably has only a small chance to move through the solidified grout sheets. The laboratory test at ORNL demonstrates only that, if the fine particles obtained from the ground-solidified grout are in contact with distilled water for 21 days, the leachability of the wastes is small. The actual problem of leaching from grout sheets in shale involves time scales and other conditions which cannot be adequately modeled in the laboratory. Therefore, present knowledge must be considered to be limited, and the possibility of leaching should be considered carefully in any waste-disposal project using grout injection.

The low permeability, the high ion-exchange and adsorption capacity of shale, the size of the shale body (several hundreds of feet in thickness), and the low concentration of radioactive wastes in the liquid separated from the grout--all suggest that the possibility of contamination of the biosphere by the injected radioactive wastes at ORNL is likely to be remote. If the separated liquid which contained a very low concentration of the injected radioactive wastes did reach a water source, then the concentration of the contaminants would be further reduced by hydraulic dispersion in that water body and by radioactive decay.

The second problem concerning the safety of this disposal method is the orientation of the induced fractures. If hydraulically induced fractures are propagated vertically during the injection, they possibly may extend into an adjacent permeable formation. Should this occur, liquid wastes could be released to the groundwater in the permeable formation before the grout has had time to solidify. For safe disposal it is mandatory that the induced fractures be formed along bedding planes in shale, thereby restricting the injected grout to a known layer of low permeability.

The overwhelming experience of hydraulic fracturing indicates that the fractures are generally vertical (Hubbert, 1971). Because of this experience some earth scientists take a negative attitude toward disposal of waste by hydraulic fracturing. However, in thick shale formations, because of the difference in tensile strength in the direction normal to the bedding plane compared with that in the direction parallel with the bedding plane of the shale, under certain conditions bedding-plane fractures are more easily induced hydraulically in shale than in other formations.

STATE OF UNDERGROUND STRESSES

The amount of stress at a given point in a rock mass generally results from superposition of stress components of three types. These stress components are (1) gravitational stress, (2) tectonic stress, and (3) fluid pressure within the rocks.

Gravitational stress results fundamentally from the weight of the overburden. Two aspects of this stress need to be differentiated: (a) stress effects that result from the present condition of the overburden and (b) stress effects that result from the preexisting condition of the overburden. Tectonic stress is induced by mobility of the earth, resulting from various influences, and fluid pressure is caused by fluid, usually water, that is found in pore spaces of the rock mass.

Vertical Stress

Consider a small element underground at a depth z in a Cartesian coordinate system with its z-axis in the vertical direction (Fig. 1). The equation of equilibrium in terms of vertical stresses (Jaeger and Cook, 1969) is given by

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \tau_{xz}}{\partial x} - \rho g = 0, \qquad (1)$$

where σ_z is the vertical stress, τ_{yz} and τ_{xz} are shear stresses, and ρ and g are density of the rock and acceleration of gravity, respectively.

Integrating Equation 1 with respect to z,

$$\sigma_z = \rho g z - \int_0^z \frac{\partial \tau_{yz}}{\partial y}\, dz - \int_0^z \frac{\partial \tau_{xz}}{\partial x}\, dz;$$

$$\sigma_z = \gamma z - \int_0^z \frac{\partial \tau_{yz}}{\partial y}\, dz - \int_0^z \frac{\partial \tau_{xz}}{\partial x}\, dz, \qquad (2)$$

where γ is the weight density of the rock. Howard (1966) states that there are only three special cases of Equation 2 in which the vertical stress is equal to the weight of overburden per unit area, but only two of them are condisered to be geologically acceptable.

Case 1. In regions of gentle topography and simple geologic structure there are no shear components along the air-earth interface, and then

$$\tau_{xz} = \tau_{yz} = 0. \qquad (3)$$

In this case the vertical stress is simply equal to the weight of overburden per unit area.

Case 2. The condition for $\sigma_z = \gamma z$ is

$$\int_o^z \frac{\partial \tau_{xz}}{\partial x} dz = \int_o^z \frac{\partial \tau_{yz}}{\partial y} dz = 0. \qquad (4)$$

This condition can be achieved through relaxation of rocks by creep over long periods of time.

Case 3. The third case for $\sigma_z = \gamma z$ is

$$\int_o^z \frac{\partial \tau_{xz}}{\partial x} dz = - \int_o^z \frac{\partial \tau_{yz}}{\partial y} dz . \qquad (5)$$

This case is very restrictive, especially when compounded by choice of geographic direction for the x and y axes. Howard regards this case as geologically improbable.

It is concluded that, in a relatively flat area with simple geologic structure, the vertical stress may be calculated as the weight of overburden per unit area. However, in a topographically irregular area or in a region having complex geologic structures, the vertical stress may or may not be the overburden pressure alone.

Horizontal Stress

No adequate analytical models are available today to estimate horizontal stress. However, the theoretical equation frequently used in rock mechanics for the prediction of horizontal stress, which is induced by the vertical stress, is based on two assumptions: (1) behavior of the rock under stress is purely elastic, and (2) lateral extension of the stressed rock is constrained by the surrounding material (Harrison et al., 1954; Price, 1966; Jaeger and Cook, 1969). Most horizontal stresses measured in situ are much higher than the stresses calculated by the theoretical equation (Obert and Duvall, 1967).

The reason for the discrepancy between the theoretical horizontal stress and the stresses measured in situ is that the first assumption used in the development of the theoretical equation is valid only when the process of compaction of the rocks is completely finished. Horizontal stress is produced during the accumulation of sediments while they are undergoing compaction; therefore, the above-mentioned theoretical equation is not adequate to predict horizontal stress induced by weight of overburden in sedimentary rocks.

An empirical formula used in soil mechanics to describe the relation between the horizontal stress and the vertical stress during the process of compaction under the condition of zero lateral strain (Brooker and Ireland, 1965) is as follows:

$$\sigma_h = (0.95 - \sin \varphi)\sigma_z, \qquad (6)$$

where σ_h is horizontal stress which is to be the same in both x and y directions; φ angle of internal friction of the sediments.

When sediments are buried at a great depth they are compacted. Horizontal stress may be influenced by shear failure. The critical value to avoid shear failure can be estimated from the geometry shown in Mohr's circle (Fig. 2). The expression is as follows:

$$\sin \varphi = (\sigma_z - \sigma_h)/(2x + \sigma_z + \sigma_h). \qquad (7)$$

Through mathematical rearrangement Equation 7 can be written as

$$\sigma_h = \sigma_z(1 - \sin \varphi)/(1 + \sin \varphi) - (2C \cos \varphi)/(1 + \sin \varphi), \qquad (8)$$

in which C is cohesive strength.

If C is small and can be neglected, then

$$\sigma_h/\sigma_z = (1 - \sin \varphi)/(1 + \sin \varphi). \qquad (9)$$

It is believed that the ratio between the horizontal stress and the vertical stress, σ_h/σ_z, should lie in between the two extreme conditions described by Equations 6 and 9 (Voight, 1966) and shown as follows:

$$(1 - \sin \varphi)/(1 + \sin \varphi) \leq \sigma_h/\sigma_z \leq (0.95 - \sin \varphi). \qquad (10)$$

In general, the angle of internal friction, φ, is about 27-30° for hard sedimentary rocks such as sandstone and limestone, and about 0-20° for soft sedimentary rocks such as shale and clay (Fenner, 1938; Harrison et al., 1954; Perkins, 1967; Jaeger and Cook, 1969). Therefore, the ratio of horizontal to vertical stress, σ_h/σ_z, may be between 0.33 and 0.55 for hard sedimentary rocks, and between 0.49 and 0.95 for soft rocks.

Laboratory experiments show that a hysteresis effect exists during loading and unloading (Brooker and Ireland, 1965; Voight, 1966), thus horizontal stress is higher during unloading than during loading (Fig. 3). If this hysteresis effect also exists in denuded areas, then horizontal stress probably would be higher than the stress as calculated on the basis of the present weight of overburden.

In the above discussion, horizontal stresses are assumed to be equal in both the x and y directions; however, in reality they may not be. In tectonically active areas, horizontal stresses may be affected by tectonic stresses, but unfortunately no analytical theory has been developed to predict these tectonic stresses.

EARTH STRESS AFFECTED BY CONSTRUCTION OF INJECTION WELL

The presence of a borehole in the rock distorts the stress distribution in the vicinity of the borehole. The effects of a borehole on the preexisting horizontal stresses in the rock can be calculated by analogy with an infinitely large plate, subjected to uniaxial stress, containing a circular hole with its axis perpendicular to the plate, which was solved by Kirsch in 1898 (Timoshenko and Goodier, 1951).

The solution of a borehole subjected to two horizontal stresses can be obtained by the Kirsch solution and the law of superposition. The results are:

$$\sigma_r = [(\sigma_x + \sigma_y)/2](1-a^2/r^2) + [(\sigma_x - \sigma_y)/2](1+3a^4/r^4-4a^2/r^2)\cos 2\theta, \quad (11)$$

$$\sigma_t = [(\sigma_x + \sigma_y)/2](1+a^2/r^2) - [(\sigma_x - \sigma_y)/2](1+3a^4/r^4)\cos 2\theta, \text{ and} \quad (12)$$

$$\tau_{r\theta} = [(\sigma_x - \sigma_y)/2](1-3a^4/r^4+2a^2/r^2)\sin 2\theta, \quad (13)$$

where a is the radius of the borehole (Fig. 4), r the radial distance from the center of the borehole, θ the polar angle from the x-axis moving in counterclockwise direction; σ_x and σ_y are horizontal stresses along the x-axis and y-axis, respectively; σ_r is the radial stress, σ_t the tangential stress, and $\tau_{r\theta}$ the shear stress.

From Equations 11 and 13 it can be seen that the radial and shear stresses are zero at the edge of the borehole and increase rapidly toward the undisturbed stress field within a few borehole diameters. Equation 12 shows that, at the circumference of a borehole (r=a), the stress reaches a minimum value at θ=0° and θ=180°, with a magnitude of $(3\sigma_y - \sigma_x)$, if $\sigma_x > \sigma_y$. This minimum tangential stress can be either compressional or tensional depending on the ratio of σ_x/σ_y. When σ_x is greater than $3\sigma_y$ the tangential stress is in tension. The maximum tangential stress on the circumference of the borehole is at θ=90° and θ=270°, with a magnitude of $(3\sigma_x - \sigma_y)$ and is always in compression. In the case where both horizontal stresses are equal, $\sigma_x=\sigma_y$, then the tangential stress around the borehole circumference is the same everywhere, with a magnitude of $2\sigma_h$.

In conclusion, the effect of a borehole on horizontal stresses is localized within a few borehole diameters of the borehole. Beyond that distance stresses will be undisturbed.

Some investigators believe that, if there is a thick clay or shale formation above the injection zone, some material could flow into the borehole during the process of drilling, essentially decreasing the overburden prssure near the borehole (Zheltov and Khristianovich, 1955; Barenblatt, 1956, personal commun., 1970). There are no data from the field tests discussed below to verify this reduction in overburden pressure, and it was therefore not taken into consideration in the following calculations. If it does occur it would be favorable to the formation of horizontal fractures and would increase the confidence in the prediction of horizontal fractures based on calculations that ignore this effect.

GENERAL THEORY OF FRACTURING MECHANICS

The following discussion of mechanics of hydraulic fracturing is based on the assumptions that shale is a brittle material and that it behaves elastically. Price (1966) states that, "the arenaceous rocks with a clay matrix can not be considered as brittle materials while they are undergoing compaction and 'volume flow.' But after their compaction, i.e., following a phase of uplift or during any phase subsequent to subsidence, they may possibly behave as brittle materials." Therefore, shale probably can be considered to be a brittle material or at least a semi-brittle material. Although shale definitely behaves more plastically than elastically, nevertheless the existing elastic theories should yield approximate solutions on hydraulic fracturing.

Two stages are involved in the formation of a fracture--fracture initiation and fracture propagation. Fracture initiation is defined as the failure process by which a preexisting plane of weakness in the rock starts to open. Fracture propagation is a stage subsequent to fracture initiation in which the fracture is extending. Two kinds of fracture propagation may be distinguished--stable and unstable propagation (Bieniawski, 1967).

Stable propagation is the process of rupture in which the extension of a fracture is a function of applied stress and can be controlled accordingly. If the fracture extension is governed by parameters other than loading, e.g., the fracture growth velocity, then the fracture becomes uncontrollable. The fracture extends rapidly to complete rupture of the material under a constant applied load. This kind of propagation is called "unstable propagation" and has dynamic characteristics.

The extension of a hydraulically induced fracture is essentially dependent on the applied hydraulic load; that is, the fracture will cease to propagate when the injection is stopped. Therefore, it can be concluded that the extension of a hydraulically induced fracture is a case of stable propagation without dynamic characteristics. The pressure required for stable propagation is that needed to overcome the cohesive forces at the fracture tip.

When sediments are deposited, individual grains, although in contact with neighboring grains, are discrete, and sediments are without cohesion. As sediments are buried beneath later deposits they become compacted. Compaction and consolidation result in interlocking of grains so that the resulting rock takes on cohesive strength. The greater the compaction, the greater is the cohesion.

In the formation of a hydraulically induced fracture, work must be done against the cohesive forces on the grains at the fracture-tip area. As the hydraulic load increases, the preexisting plane of weakness in the rock does not open until the applied hydraulic load reaches a critical level, at which time the maximum cohesive forces at the tip of the plane of weakness are overcome and the preexisting plane of weakness starts to open.

The cohesive forces under tensile strength are a function of the intermolecular distance b (Fig. 5). The molecular cohesive forces are zero before the tensile stresses are applied, then they rise in proportion to the amount of separation between the pulled molecules. They reach the maximum value when the separation has reached approximately one intermolecular distance from the equilibrium condition. A further increase of separation of the pulled molecules will diminish rapidly the cohesive forces. When the separation between the pulled molecules is greater than three intermolecular distances from the equilibrium condition, the cohesive forces are approximately equal to zero (Cottrell, 1964; Kunz, 1971).

The maximum cohesive force is defined as the ideal tensile strength of a rock which has a perfect molecular structure. The actual tensile strength of a rock is usually several orders of magnitude lower than the ideal tensile strength because of defects of the molecular structure.

Figure 6 shows a schematic, idealized structure of molecules around a fracture tip. Consider the fracture extending from the left toward the right; the first pair of molecules at the fracture tip begins to separate

as the applied hydraulic load increases. A rise of the applied hydraulic load increases the separation between the pair. The intermolecular distance increases from the equilibrium condition, b, to 2b then to 4b; at this stage the rupture process is completed and the fracture extends one intermolecular distance b, at the same time that the applied hydraulic load falls. The second pair of molecules now becomes the new fracture tip. The applied hydraulic load again rises and falls. This fracturing process goes on from one pair of molecules to another pair.

Neither the distribution of cohesive forces over the fracture surface nor the dependence of these forces on the separation between the opposite fracture faces is clearly known. Even if these factors were known, a very complex non-linear integral equation would be involved in solving the fracture problem. To avoid these difficulties Barenblatt (1962) divided the fracture area into two regions (Fig. 7). In the inner region, the opposite fracture faces are relatively far apart, hence there are no molecular cohesive forces and the fracture can be considered as free of stresses. The linear theory of elasticity can be applied fully to this region. In the outer (edge) region, the opposite fracture faces are sufficiently close to each other so that there are strong molecular cohesive forces between them. Plastic yielding occurs in this region. To avoid the complex non-linear theory of elasticity and to work within the framework of the linear theory, Barenblatt made two assumptions to solve the fracture problem: (1) the width of the edge region is small compared to the size of the whole fracture; and (2) when the fracture extends, the shape of the section normal to the fracture surface in the edge region (and consequently the local distribution of the cohesive forces over the fracture surface) does not depend on the pressure in the fracture and is always the same for a given material under given conditions of temperature and composition.

Based on these two assumptions, the average cohesive forces at the fracture tip can be defined as (fT), where T is the average tensile strength of the rock; f depends on the characteristics of the rock and the radius of the plastic zone at the fracture tip. The value of f is between zero and one—$0 \leq f \leq 1$ (Barenblatt, 1962; Kenny and Campbell, 1967; Perkins and Krech, 1968; Rice, 1965).

If a is the radius of the area of the inner region of the fracture, a' is the radius of the area of both inner and edge regions, and α represents a/a', then the value of f is equal to one if α reaches zero (Fig. 8), i.e., a' approaches infinity; or f approaches zero as α approaches to one, i.e., a approaches a'.

Perkins and Krech (1968) found the average value of f to be 0.3.

MECHANICS OF HYDRAULIC FRACTURING

According to the lithologic characteristics of the injection formation, fracture walls can be classified as (1) both walls impermeable, as in fractures formed in shale; and (2) one or both walls permeable, as in fractures formed along the interface of shale and sandstone, or within sandstone.

Waste injection is conducted in thick shale with water-based grout, therefore, only the case of hydraulic fractures with impervious walls will be discussed.

Vertical Fracture

In case of the initiation of a vertical fracture, the initiation pressure, P_i, which is commonly called the "breakdown pressure," should be equal to or greater than the sum of the minimum effective tangential earth stress at the well face and the tensile strength of the rock in the direction normal to the fracture plane. The mathematical expression for initiation of a vertical fracture can be obtained from Equation 12, when r=a, θ=0°, and θ=180°, and the result is

$$P_i - p_o \geq 3(\sigma_y - p_o) - (\sigma_x - p_o) - T, \quad \text{if } \sigma_x > \sigma_y,$$

or

$$P_i \geq 3\sigma_y - \sigma_x - p_o - T, \tag{14}$$

in which T is the tensile strength of the rock in the direction parallel to the stress σ_y (normal to the fracture plane), considered to be negative; p_o is the pore pressure.

Because the presence of a borehole distorts the natural stresses only within a few borehole diameters of the injection well, the pressure P required to propagate the initiated fracture is the sum of the earth stress normal to the fracture plane, the average cohesive force at the fracture tip, the fluid resistances developed in the fracture, and the fluid pressure within the rock. The mathematical expression is as follows:

$$P - p_o \geq \sigma_y - p_o - fT + F(Q,L,W),$$

or

$$P \geq \sigma_y - fT + F(Q,L,W), \tag{15}$$

where F(Q,L,W) is flow resistance, a function of injection rate and the length and width of the vertical fracture.

Horizontal Fracture

The pressure required to initiate a horizontal fracture is equal to the sum of the vertical stress and the tensile strength of the rock in the vertical direction. The mathematical expression can be written as follows:

$$P_i \geq \sigma_z - T_z, \tag{16}$$

where T_z is the tensile strength of the rock in the vertical direciton.

The propagating pressure of a horizontal fracture is written as

$$P \geq \sigma_z - fT_z + F(Q,r,W), \tag{17}$$

where F(Q,r,W) is flow resistance, a function of injection rate, radial distance and width of the horizontal fracture.

The disposal well should be constructed with strong casing, which is cemented in place. Before the injection, a horizontal cut (360°) should be made through the casing and cement at the desired injection depth using a hydraulic-jet tool. This cut, which extends a short distance into the rock, serves as a preexisting plane of weakness. Injection fluid enters the cut and creates the vertical stress without the use of a packer in the well.

Owing to the additional tensile strength provided by the strong casing in the tangential direction and a weak plane existing in the horizontal direction, the fracture initiated at the well face is probably horizontal in spite of the direction of the least principal earth stress. However, at a great depth, this additional tensile strength of the casing might be overcome by the great overburden pressure; thus a vertical fracture would be initiated at the well face.

In areas where the least stress (sum of earth stress and the tensile strength of the rock) is in the horizontal direction, even if the fracture initiated at the well is horizontal, the fracture plane will become vertical when the fracture propagates away from the injection well. The fracture plane will be normal to the least stress because this requires the least amount of work to rupture the rock.

Induced Fracture in Laminated Rock

Experimental results show that laminated rocks have directional tensile strength. Because of low cohesion between laminations and the presence of micro-fractures parallel with laminations, laminated rocks commonly have the least tensile strength in the direction normal to the laminations and have the greatest tensile strength in the direction parallel with the laminations. Laboratory data show that for laminated rocks such as siltstone and shale, the value of tensile strength in the direction normal to the bedding plane is about 20-80 percent of the value in the direction parallel with the bedding plane, depending on the condition of the lamination. In most cases, the value is in the range of 30 percent (Hobbs, 1964; Chenevert and Gatlin, 1965; Youash, 1965; Obert and Duvall, 1967).

If it is assumed that (1) the dominant principal earth stress σ_1 is in the vertical direction parallel with the well axis; (2) the least principal earth stress σ_3 is in one of the horizontal directions; and (3) the plane of lamination makes an angle ω with the axis of the injection well, then the general stresses acting on an element in the earth below land surface can be expressed in polar coordinates (Fig. 9). They are:

$$\sigma_r = (\sigma_1 + \sigma_3)/2 + [\sigma_1 - \sigma_3)/2]\cos 2\omega, \tag{18}$$

$$\sigma_t = (\sigma_1 + \sigma_3)/2 - [\sigma_1 - \sigma_3)/2]\cos 2\omega, \text{ and} \tag{19}$$

$$\tau_{r\theta} = [(\sigma_1 - \sigma_3)/2]\sin 2\omega. \tag{20}$$

Pressure needed to propagate fractures along or normal to the lamination can be computed from Equation 19. The results are:

$$P \geq (\sigma_1 + \sigma_3)/2 - [(\sigma_1 - \sigma_3)/2]\cos 2\omega - fT_n + F(Q,L,W),$$

$$\text{fracture formed along lamination,} \tag{21}$$

$$P \geq (\sigma_1 + \sigma_3)/2 + [(\sigma_1 - \sigma_3)/2]\cos 2\omega - fT_a + F(Q,L,W),$$

$$\text{fracture formed normal to lamination,} \tag{22}$$

where T_n is the tensile strength of the rock in the direction normal to the lamination and T_a the tensile strength in the direction parallel with the bedding plane.

Equations 21 and 22 clearly show that when the angle of the bedding plane, ω, is equal to or less than 45°, the induced fractures are always formed along bedding planes; however, when $\omega > 45°$, the condition for forming a bedding-plane fracture is as follows:

$$(\sigma_1+\sigma_3)/2 - [(\sigma_1-\sigma_3)/2]\cos 2\omega - fT_n < (\sigma_1+\sigma_3)/2 + [\sigma_1-\sigma_3)/2]\cos 2\omega - fT_a;$$

$$(\sigma_3 - \sigma_1) < f(T_n - T_a)/\cos 2\omega. \tag{23}$$

For example, in the laboratory Youash (1965) found T_n to be 400 psi and T_a to be 1,600 psi for shale of the Green River Formation. Perkins and Krech (1968) state that the average value of f is 0.3. If the bedding-plane angle ω of the Green River shale is assumed to be 10°; if σ_h/σ_z is assumed to be 0.5; and if σ_z is simply equal to the weight of overburden and is assumed to be 1.0 psi per foot of depth, then the maximum depth at which a bedding-plane fracture can be hydraulically induced, even in a normal-fault area, can be estimated from Equation 23. The result is

$$z \geq [0.3(1{,}600 - 400)]/(-0.94 \cdot 0.5);$$
$$z \geq 766 \text{ ft.}$$

Effect of Natural Fractures

Most if not all rocks contain fractures or joints. Bugbee (1953) has assumed that "the internal pressure to break down rock formations in a well depends primarily upon the presence of a fracture, fault or joint system in the structure and to a lesser degree upon the extent of intrusion of fluid, the position of bedding plane, etc. ...." Is this statement true, and if so under what conditions?

If it is assumed that (1) the dominant principal stress σ_1 is in the vertical direction and the least principal stress σ_3 lies in one of the horizontal directions (these are the conditions most favorable for producing vertical fractures), and (2) the natural fracture plane makes an angle β, and the bedding plane makes an angle ω with the borehole axis, which is in the vertical direction (Fig. 10), then the pressure required to induce a fracture along a natural fracture plane is given by

$$P \geq (\sigma_1 + \sigma_3)/2 - [(\sigma_1 - \sigma_3)/2]\cos 2\beta - T_n\cos(\omega-\beta) - T_a\sin(\omega-\beta). \tag{24}$$

The pressure required to induce a bedding-plane fracture is as follows:

$$P \geq (\sigma_1 + \sigma_3)/2 - [(\sigma_1 - \sigma_3)/2]\cos 2\omega - T_n. \tag{25}$$

The condition for natural fractures not to be propagated is that Equation 24 be greater than Equation 25. The result is given by

$$[(\sigma_3 - \sigma_1)/2]\cos 2\beta - T_n\cos(\omega-\beta) - T_a\sin(\omega-\beta) > [(\sigma_3 - \sigma_1)/2]\cos 2\omega - T_n. \tag{26}$$

In case of vertical joints, Equation 26 can be written as

$$(\sigma_3 - \sigma_1)/2 - T_n\cos\omega - T_a\sin\omega > [(\sigma_3 - \sigma_1)/2]\cos 2\omega - T_n. \tag{27}$$

If the bedding plane of the rock is vertical, i.e., $\omega = 0°$, then joints will be extended unconditionally. If the bedding plane is horizontal, i.e., $\omega = 90°$, then the condition for joints not to be extended can be obtained from Equation 27. The result is

$$(\sigma_3 - T_a) > (\sigma_1 - T_n). \tag{28}$$

For example, if σ_h/σ_z is assumed to be 0.5, σ_z to be 1.0 psi per foot of depth, T_n and T_a to be 400 psi and 1,600 psi, respectively, then the maximum depth range within which vertical joints will not be extended is calculated as follows:

$$(0.5)(1.0)z + 1{,}600 > (1.0)z + 400;$$
$$z < 2{,}400 \text{ ft.}$$

INTERPRETATION OF INJECTION DATA

Before a site is selected for waste injection, it is desirable to run test injections to evaluate whether or not bedding-plane fractures can be produced in the shale. The best method for determining the orientation and extension of hydraulically induced fractures during a site-selection study is to drill a series of coreholes after some grout injections. However, this procedure is costly and therefore other techniques are desirable.

Injection pressure and pressure decay can be measured without difficulties during and after injections. Injection pressure is composed of three elements: (1) the breakdown pressure, (2) the propagation pressure, and (3) the instantaneous shut-in pressure. The breakdown is the pressure needed to overcome the earth stresses around the wall of the injection well and the tensile strength of the rock normal to the fracture plane. In areas where rock has very low tensile strength, the breakdown pressure may be difficult to differentiate from the propagation pressure.

Propagation pressure is the pressure needed to overcome the earth stress normal to the fracture plane, the cohesive force at the fracture tip, and the flow resistance in the fracture. Instantaneous shut-in pressure is the pressure at the time the injection pump has just been stopped. At this time no fluid is entering the injection well, and

therefore the fluid resistance in the fracture can be considered to be zero. Thus the instantaneous shut-in pressure is simply the sum of the earth stress normal to the fracture plane and the cohesive forces at the tip.

De Laguna (1972) proposed that pressure decay after water injection may yield some information useful in judging whether or not the fracture formed was horizontal. He plotted rate of change of pressure decay against time on log-log paper. A point of discontinuity on the data curve would suggest that an induced fracture may be closed by the overburden pressure at that time.

Pressure decay in an injection well is not only the result of flow of water from the fracture into the formation, but is also affected by the reduction of fracture volume due to the rebound of elastic energy stored in the rock during the injection, as well as the effects of earth stress normal to the fracture plane. At the present time, no simple solution has been found. It is hoped that the empirical method proposed by de Laguna may yield valuable information.

If t is time and P is the difference between the observed bottom-hole pressure and the formation-fluid pressure at time t during the pressure-decay period, then from data obtained at the New York test site (described in detail below) the author found that $t_i/t_{i-1} = r$ and $P_i/P_{i-1} = r^{-k}$. Therefore, the relation between P and t can be expressed as follows:

$$P = Ct^{-k}, \tag{29}$$

where r, C, and k are constants. Plotting P and t on log-log paper should result in a straight line, if k remains unchanged.

The New York data show that the P-t curve may yield some information on the overburden pressure as well as the horizontal stress. However, owing to lack of a theoretical formulation, this relation is not considered reliable without further tests.

During the experiments at Oak Ridge, Tennessee, it was observed that the ground surface had been uplifted in the vicinity of the injection site. Sun (1969) developed an analytical model to predict the amount of uplift produced by a horizontally induced fracture which is shaped as a thin disc. From the analysis of field data from nine hydraulic-fracturing injection tests made at the ORNL, he obtained reasonable agreement between predicted and observed data (Fig. 11). It is believed that the uplift produced by a discrete vertical fracture should have a different pattern from that produced by a horizontal disc-shaped fracture.

From 1969 through 1971, six hydraulic-fracturing injections were made at the Western New York Nuclear Service Center near West Valley in Cattaraugus County, New York, about 35 mi southeast of Buffalo, New York. The injection depths ranged from 500 to 1,450 ft. The first two injections were conducted at 1,450 ft in one slot and the last two injections were made at 500 ft from one slot. All injections consisted of water except the last one, which was a grout injection.

Geology at Site

The geologic structure in the vicinity of the test site is that of a simple monocline with no noticeable folds and faults. The bedding planes of the shale are nearly horizontal, probably dipping southwards by only one or two degrees (written commun., G. H. Chase, USGS, 1969).

Three sets of principal joints have been identified by Chase at the outcrop area near the test area. Their trends are N68°E, N45°W, and N13°W, in descending order according to the frequency of occurrence. All joints are vertical or nearly so. The average spacing of joints is about 2 ft and the separation is about 0.01 ft. The vertical extent of the joints ranges from less than 1 ft to about 8 ft; most extend about 1 ft or less. From cores and geophysical logs, it was estimated that 20 percent of the joints probably are open (written commun., G. H. Chase, USGS, 1969).

The area is blanketed by as much as 200 ft of glacial drift. Underlying the glacial material are deposits of Late Devonian age (Fisher et al., 1961). The rocks at the injection depth are either silty shale or petroliferous shale with some interbedded siltstones. A layer of 100 ft of siltstone was logged in the corehole at a depth of 950-1,050 ft. The siltstone contains thin layers of silty shale (written commun., G. H. Chase, USGS, 1969).

Well Construction

A corehole about 3 in. in diameter was drilled to a depth of nearly 1,500 ft in order to obtain lithologic information before the drilling of the injection well. This corehole was later constructed as one of the four observation wells, namely, the East observation well. The injection well, which was constructed with good-quality 4.5-in. steel casing cemented in full depth in an 8-in. hole, was located 150 ft west from the corehole.

Three more observations wells were constructed, to the south, the west, and the north, each 150 ft from the injection well. These observation wells were also cased with strong 2-in. tubing and cemented in full depth (about 1,520 ft) in a 6-in. hole.

The East observation well (the corehole) was cased with rather weak 1 1/4-in. tubing but was also cemented in full depth (about 1,520 ft).

Before converting the corehole to the East observation well, gamma-ray, electric, 3-D sonic, density, and caliper logs were made in the hole to obtain geological and geophysical information. Density, caliper and hole-alignment logs were made at the injection well.

Interpretation of Injection Data

A total of six injections was made at the site. Only four injections will be discussed in this paper; the interpretation of the other two tests, one of them made in a siltstone formation, was presented in another paper (Sun and Mongan, 1973).

The first injection was made on October 9, 1969. About 114,300 gal of water were injected through a pre-cut slot, at a depth of 1,450 ft. The rock at this depth is well-bedded petroliferous shale; however, a zone of vertical joints from 1,440 to 1,450 ft had been observed from geophysical logs (written commun., G. H. Chase, USGS, 1969).

The injection was started at a very low rate which could not be detected on the Halliburton flow meter. At this low rate the injection pressure increased rapidly. Twenty-two minutes after the injection started, the pressure at the well head reached 1,400 psi and a trace of flow was detected on the flow meter. The injection rate was progressively increased in three steps from 16.5 gpm (gallons per minute) to 30 gpm and then to 50 gpm. Each injection step lasted about 10 minutes. After the 50-gpm step, an injection pattern consisting of flow rates of 100, 200, and 400 gpm each at intervals of 1/2 hour was established. This pattern was repeated over and over until the end of the injection (Fig. 12).

Six hours after the start, when a total of 59,000 gal of water had been pumped into the formation, the injection was stopped to allow for the adjustment of one of three liquid-level tiltmeters. After the required adjustments were made, 45 minutes later, the injection was resumed with the regular injection pattern, but this time starting at the rate of 400 gpm.

Injection pressure, rate of injection, volume of fluid, and tilt of ground surface in the vicinity of the injection well were all recorded during the injection. After the injection operation ceased, the pressure decay was observed for a period of 8 days.

Precise leveling surveys (close to 0.001 ft) were made by the USGS before and after the injection to observe the ground movement which had been caused by the injection.

Since all injection pressures were observed at the head of the injection well, the bottom-hole pressures must be computed from the observed surface pressures by adding the static pressure and by subtracting the pressure loss due to friction in the casing. For water injections the pressure loss in the casing can be calculated by the well-known Darcy-Weisbach equation (King and Brater, 1963), which is given as follows:

$$\Delta p = 0.00673\ fLv^2/D, \tag{30}$$

where Δp = pressure loss due to friction, psi,
f = Fanning frictional factor of casing, dimensionless,
L = length of casing, ft,
D = inside diameter of casing, ft,
v = flow velocity, ft/sec.

The computed bottom-hole pressure and the observation times are given in Figure 12. All pressures used in the interpretation of the test data are computed as bottom-hole pressure.

If flow through an induced fracture is assumed to be laminar and to obey Darcy's law, then the injection pressure at the well should be linearly proportional to the injection rate. Excluding the pressure involved in the initiation of the fracture, a linear regression equation is found (Fig. 13) and

$$P = 1{,}735 + 0.22Q, \tag{31}$$

where P is bottom-hole injection pressure in psi and Q injection rate in gpm. The statistical significance of the regression coefficients has been tested based on a 95-percent confidence level (Krumbein and Graybill, 1965; Kenny and Keeping, 1966).

Because of the simple geologic structure and the relatively flat topography at the test site, it is reasonable to consider the vertical earth stress to be equal to the weight of overburden. The average specific gravities of the shale and the glacial drift were found to be 2.6 and 2.0, respectively (de Laguna, 1972). Therefore, the overburden pressure at the injection depth can be calculated as follows:

$$\sigma_z = 0.4331[2.0\cdot 200 + 2.6(1{,}450 - 200)] = 1{,}581 \text{ psi}.$$

At Q = 0, the observed injection pressure was 2,028 psi (Figs. 12, 13), which was the breakdown pressure. As discussed before, owing to the high tensile strength of the casing, horizontal fractures were probably initiated at the well face. Therefore, the tensile strength of shale normal to the fracture plane, T_z, can be calculated by Equation 16, and the result is

$$2{,}028 = 1{,}581 - T_z;$$

$$T_z = -447 \text{ psi}.$$

After about 700 gal of water had been injected, the injection pressure increased rapidly beyond the propagation pressure predicted by the regression equation (Equation 31; Fig. 13). This high injection pressure suggests the formation of additional fractures.

From Equation 31, the normal propagation pressure at Q = 100 gpm is 1,755 psi; however, the highest observed pressure at Q = 100 gpm was 2,203 psi (Figs. 12, 13). This pressure could be the breakdown pressure at the stage of formation of additional fractures. The tensile strength of shale calculated on the basis of this breakdown pressure is 602 psi, which is about 155 psi higher than the value calculated on the basis of the first breakdown pressure.

When the injection pump is stopped, Q will be zero. The instantaneous shut-in pressure, which is 1,735 psi, can be obtained from Equation 31. The observed instantaneous pressure was 1,748 psi (Fig. 12).

The average cohesive force at the fracture tip can be calculated from Equation 17, and the result is

$$fT_z = 1{,}581 - 1{,}735 - 154 \text{ psi, and}$$

$$f = 0.34,$$

which is close to the average value found by Perkins and Krech (1968).

After the 45-minute pause to adjust the tiltmeters, the injection was resumed at a rate of 400 gpm. The propagation pressure for this rate should be 1,823 psi (Equation 31); however, the observed pressure at this rate was 2,229 psi, which was the breakdown pressure at the re-injection stage. The estimated tensile strength based on this breakdown pressure is 568 psi. This is 121 psi higher than the first value that was calculated, 447 psi, but it is close to the value calculated on the basis of the breakdown pressure observed at Q = 100 gpm.

After the fracture was initiated, the injection pressure diminished to the normal propagation pressure. The regression equation (Fig. 14) for the injection period after the pause is

$$P = 1{,}735 + 0.55Q. \qquad (32)$$

After the completion of the injection, the well was shut in at the well head. Pressure decay was observed for about 8 days (Fig. 15).

Groundwater level was measured in the corehole by G. H. Chase, USGS, on May 22, 1969, and was found to be 58.5 ft below land surface (written commun., 1970). This would indicate a depth to water of about 80 ft at the injection well because of differences in land surface altitudes. On the basis of this water level the fluid pressure, p_o, in the formation at the injection depth of 1,450 ft was calculated as 593 psi.

The log-log plot of $(P-p_o)$ against observation time t (Fig. 15), where $(P-p_o)$ represents the difference between the bottom-hole pressure in the well and the formation-fluid pressure at the injection depth, appears to fall on two straight lines which intersect at t = 450 minutes and $P-p_o$ = 980 psi.

As the well is shut in, the fluid pressure in the induced fracture is much higher than the fluid pressure in the surrounding formation; thus water in the fracture starts to flow into the formation, despite the low permeability of the fractured shale. Fluid pressure in the injection well will decline and the plot of the observed data will fall on a single line. As soon as the fluid pressure in the fracture is reduced to a value which is less than the effective earth stress normal to the fracture walls, the fracture is reduced in size. Thus the decay of the fluid pressure in the injection well is affected not only by water leaking out of the fracture but also by the reduction of fracture volume, and therefore the slope of the line will change. The pressure at the discontinuity point of the $(P-p_o)$-t curve can be interpreted as the effective earth stress normal to the fracture plane. If there is more than one discontinuity on the plotted curve, it can be interpreted as indicating that there is probably more than one set of directional fractures, such as horizontal fractures with expanded vertical joints (Sun and Mongan, 1973).

For this injection (Oct. 9, 1969), $(P-p_o)$ at the intersection of the two lines is 980 psi, and the formation-fluid pressure is 593 psi; therefore, the earth stress normal to the fracture plane is 1,573 psi (593 + 980 = 1,573), which is close to the calculated overburden pressure, 1,581 psi. It can be concluded, therefore, that the vertical stress at the test site is simply equal to the weight of overburden, and the induced fracture is nearly horizontal. However, this does not assure that vertical joints were not expanded by the injection fluid. As discussed before, if the shale bedding is horizontal, vertical joints are not likely to be extended if an injection is made at shallow depth. However, if a vertical joint is open and intercepted by part of the induced fracture, injection fluid will enter the joint, which will be expanded without being extended.

According to Equation 10, the lower limit of σ_h/σ_z is 0.5 for soft rocks such as shale. Therefore, the time of observation of pressure decay of the injection was not long enough to cover the interval of time in which $P-p_o < 490$ psi. The longest decay observation was made after the third and fourth injections, where the second discontinuity has been noted (Sun and Mongan, 1973).

Three water-level tiltmeters were in operation during the first injection test. Tiltmeter 1 was installed east of the injection well near the South observation well, tiltmeter 2 was placed south of the injection well near the West observation well, and tiltmeter 3 was installed west of the injection well near the North observation well. Tilting was observed at tiltmeter 3 during the entire injection, the ground surface tilting toward the injection well. The other two tiltmeters registered some possible tilting during the latter part of the injection, but the amount of tilting was not large enough to be clearly differentiated from thermal effects.

Precise-leveling surveys were made before and after the injection. The uplift was concentrated in an area west of the injection well (Fig. 16), which is consistent with the results shown by tiltmeter 3. The amount of uplift and the distribution pattern could not be predicted with the analytical model developed by Sun (1969). The amount of uplift is three times greater than the value calculated by the analytical model. An explanation for this discrepancy is that the analytical model is based on a circular, disc-type, horizontal fracture, whereas the uplift pattern produced by this injection shows that most of the injection fluid probably moved westward and concentrated in a relatively small area, probably in some joints. Nevertheless, it is believed that the injection fluid was confined to a thin horizontal zone. Unfortunately, the injection fluid was not tagged with a radioactive tracer and no field evidence is available to support these conclusions.

A second injection was made on June 26, 1970, when a total of 112,200 gal of water tagged with radioactive isotope Zr^{95}-Nb^{95} (65-day half life) was injected into the same slot used for the first injection at a depth of 1,450 ft. The radioisotope was used as a tracer in order to locate the position of the induced fracture at the observation wells by gamma-ray logs.

About 0.1 lb of Grundite (illite) clay per gallon of water was added to the injection water, so that the clay might absorb the isotope while the injection water was moving down the well. Illite has a strong absorption capacity for zirconium and it was hoped that this clay would help carry the isotope to the farthest extent of the injection. Red dye was also added to the injection fluid for safety precautions. The radioactive isotope was added at the head of the injection well at a constant rate by an electrically driven pump (de Laguna, 1972).

Owing to lack of funds, land-surface-movement observations were not made during this injection. However, injection pressure, rate of injection, volume of fluid, and pressure decay were observed as in the first injection test.

Ten days after the end of the injection, on July 6, 1970, gamma-ray logs were made by Piper Well Survey Co. in all four observation wells. Unfortunately, the East observation well (the original corehole) was plugged at 1,391 ft by cement and the South observation well was also blocked at 1,452 ft. No significant radioactivty was recorded in either of these wells. However, strong gamma activity was recorded in the North observation well with a maximum intensity of 6,000 API G.R. units (gamma unit). The activity was observed over a vertical distance of about 20 ft, from 1,430 to 1,450 ft. Gamma activity was also recorded at the West observation well with a maximum intensity of 4,000 API G.R. units, and a spread of about 20 ft, from 1,440 to 1,460 ft.

On August 24, 1970, about 2 months after the injection and after the South observation well had been cleaned out, gamma-ray logs were again run in the three open observation wells. This time, gamma activity was observed in the South observation well with a maximum intensity of 1,600 API G.R. units and a vertical spread of less than 20 ft from 1,450 to 1,470 ft. The gamma activity observed at the other two wells, North and West, was approximately at same depth as surveyed before; however, the intensity was reduced to less than half of the values observed on July 6, 1970. The gamma-ray activities observed in the three observation wells on August 24, 1970, are shown in Figure 17.

During the second test (June 26, 1970) the injection was started at a very low rate; however, injection pressure built up consistently.

The rock apparently was ruptured at 2,328 psi (1,700-psi surface pressure). After the breakdown, the injection rate was increased to 34 gpm. This rate was maintained for about 70 minutes. When the injection rate was again increased, the pressure rose quickly until it reached a peak of 3,516 psi (2,900-psi surface pressure); at this time Q was 160 gpm. This second peak may indicate the formation of additional fractures. Thereafter the pressure dropped to normal propagation pressure (Fig. 18). The regression equation of the relation between P and Q is found (Fig. 19) to be

$$P = 2,024 + 2.05Q. \tag{33}$$

Instantaneous shut-in pressure obtained from Equation 33 is 2,024 psi; however, the observed value was only 1,928 psi (Fig. 18). There is no ready explanation for this discrepancy.

The injection well was shut in under pressure on October 9, 1969, after the first injection stopped. Two weeks before the second injection, the well was bled and was shut in again on June 25, 1970, 1 day before the injection. A residual pressure of 150 psi was noted at the wellhead. It was believed that at least 778 psi pressure (150 + 1,450 · 0.4331 = 778) remained in the fracture (written commun., G. H. Chase, USGS, 1970).

If the shale is assumed to have ruptured at 2,328 psi (Fig. 18), then the tensile strength of shale is calculated to be 562 psi, which is close to the value calculated from the first injection (Oct. 9, 1969).

Average cohesive force (fT_z) at the fracture tip is calculated to be 258 psi and f to be 0.46. The value of f is about 35 percent higher than the value obtained from the first injection data (Oct. 9, 1969). However, if the observed shut-in pressure is used, f will be equal to 0.29, which is close to the value found in the first injection.

No attempt was made to use the peak pressure at Q=160 gpm (3,516 psi) to estimate the tensile strength, because the injection rate at this time was increased from 34 gpm to 160 gpm in 5 minutes (Fig. 18). Actual flow resistance in the induced fracture would be much higher than the expected flow resistance calculated from Equation 33 during this time interval.

Pressure-decay data ($P-p_o$) are plotted against time t on a log-log paper (Fig. 20). All data apparently fall on two straight lines. ($P-p_o$) at the point of intersection of the two lines is 920 psi; therefore, the estimated overburden pressure from the pressure-decay data would be 1,513 psi (920 + 593 = 1,513), which is only 68 psi less than the calculated weight of overburden, 1,581 psi.

Gamma-ray logs surveyed in the three observation wells (Fig. 17) show that several horizontal fractures had been formed. Some of the injection fluids probably had also entered joints between the horizontal fractures. However, there is no evidence that the injected fluid had moved more than 20 ft in the vertical direction.

Two injections, the fifth and sixth, were made at the 500-ft depth. The rock at this depth is thin-bedded silty shale, interbedded with siltstone (written commun., G. H. Chase, USGS, 1969). The first injection at this depth was made on May 29, 1971. A total of 51,500 gal of water without radioactive isotope was injected. The breakdown pressure was 617 psi (Fig. 21). The regression equation of this injection (Fig. 22) is

$$P = 563 + 0.10Q. \tag{34}$$

Shut-in pressure obtained by Equation 34 is 563 psi, against the observed value 551 psi (Fig. 21).

The weight of overburden at the injection depth is calculated by the specific gravity of the rocks to be 511 psi.

Tensile strength of the rock at this injection depth (500 ft) is calculated to be 106 psi, and the average cohesive force at the fracture tip to be 52 psi. The calculated value of f is 0.49, about 44 percent higher than the value determined with the data obtained at the first (Oct. 9, 1969) and second (June 26, 1970) injections. This suggests that the rock at this depth produces more strain than the rock at 1,450 ft. A 3-D sonic log shows that the Young's modulus of the rock is 3.9×10^6 psi at 500 ft against 4.9×10^6 psi at 1,450 ft (written commun., G. H. Chase, 1970).

The calculated formation-fluid pressure at the 500-ft depth is 182 psi (420 x 0.4331 = 182). When $(P-p_o)$ is plotted against t on log-log paper (Fig. 23) all data appear to fall on two straight lines.

The overburden pressure obtained from the intersection of the lines is 499 psi (317 + 182 = 499), which is close to the weight of overburden calculated from the specific gravity of the rocks.

A second injection at the 500-ft depth was made on July 23, 1971, in the same slot as the previous injection (May 29, 1971). A total of 41,000 gal of grout, made up of water, cement, and bentonite, was injected. The grout was tagged with the radioactive isotope of Zr^{95}-Nb^{95}.

The injection was started with water first, and after 2,350 gal of water had been injected, cement and bentonite were added to the water (Fig. 24). Because the grout has a high viscosity its flow is non-

Newtonian and the frictional loss in the casing should be calculated by the following expression (Slagle, 1962):

$$R_e = (1.86\, v^{2-n'}\rho)/[K'(96/D)^{n'}], \tag{35}$$

$$\Delta p = (0.039\, L\rho v^2 f)/D, \tag{36}$$

$$f = 16/R_e, \text{ for laminar flow when } R_e < 2{,}100,$$

$$f = 0.0045 + 0.645(R_e)^{-0.7}, \text{ for turbulent flow,}$$

where

R_e = Reynold's number, dimensionless,

Δp = friction loss, psi,

v = flow velocity, ft/second,

ρ = fluid density, lb/gal,

D = inside diameter of casing, in.,

f = Fanning frictional factor, dimensionless,

K' = fluid-consistency index, lb-second$^{n'}$/sq ft,

n' = fluid-flow behavior index, dimensionless.

The value of K' and n' of this injection are 0.6792 and 0.09, respectively (written commun., Slagle, Halliburton Co., 1971). The computed bottom-hole pressures and time are plotted in Figure 24. The regression equation of the relationship between P and Q is shown by the following expression (Fig. 25):

$$P = 580 + 0.12Q. \tag{37}$$

No clearly defined breakdown pressure was found during the early part of the injection (Figs. 24, 25). However, several high pressures indicate the formation of new fractures during the latter part of the injection. The tensile strength of the rock can be estimated from these high pressures. For example the pressure at Q = 45 was 617 psi (Fig. 25), and the tensile strength of the rock is calculated to be 101 psi. By using the other high pressures, the calculated values of tensile strength are between 120 and 110 psi. All the calculated values are very close to the tensile strength estimated from the previous injection data (May 29, 1971), i.e., 106 psi.

The injection pressure during the latter part of the injection test was lower than the pressure indicated by the 95-percent confidence level of the regression equation (Fig. 25). Some of the injection pressures were even below the instantaneous shut-in pressure, 580 psi, estimated from Equation 37. This discrepancy suggests that the induced fracture probably entered some open joints during the latter part of the injection.

Precise leveling (close to 0.001 ft) was carried out by the USGS before and after the injection and the results are shown in Figure 26. The correlation between the observed uplift and the uplift predicted by the analytical model (Sun, 1969) is shown in Figure 27. Whereas the correlation is not as good as hoped, the uplift data strongly suggest that the induced fractures are in a horizontal zone.

Gamma-ray logs were made on July 28, 1971, in three observation wells. The East observation well was found to have been plugged at 495 ft by cement. The plugging at 495 ft suggests that the well casing in the East observation well was ruptured at that depth by the induced fracture.

The results obtained from the other three wells are shown in Figure 28. These results indicate that a zone (15 ft) of bedding-plane fractures had been formed within a radius of 150 ft from the injection well.

All calculated results of the four injection tests at West Valley, New York, are summarized in Table 1.

SITE EVALUATION

Well-bedded shale with nearly horizontal bedding planes, several hundred feet thick, without known faults or massive joints, appears to be the best injection formation. At least one corehole should be drilled at a proposed injection site to obtain geophysical logs and in-situ subsurface geologic information including frequency and condition of natural joints and fractures, as well as the natural characteristics of rock such as elastic constant and directional tensile strengths. Detailed surface studies of joints and faults should also be made in the vicinity of the proposed site.

If possible, the corehole should be cleared of all drilling fluid to determine whether the shale yields a significant amount of fluid. An interconnected joint or natural fracture system which was transmitting water would be sufficient cause to condemn the site for disposal purposes.

In addition to the injection well, at least four observation wells should be constructed, using strong casings and good cement, at a radial distance of about 200 ft from the injection well. These wells will be used to locate the position of induced fractures after each injection test.

At least one water injection or one grout injection, or both,

should be made at the site to determine whether a zone of bedding-plane fractures can be induced in the shale. Each of these injections should be made with radioactive isotope as a tracer. The altitude and orientation of induced fractures should be determined by gamma-ray logs made in the observation wells, as well as by injection pressure, pressure decay, and uplift. Pressure decay should be observed long enough to judge whether there are effects of horizontal stresses. The longer the pressure is maintained in the injection well, the less the leakage of the system. The absence of fast leakage suggests that no interconnected joints or natural fractures exist in the vicinity of the injection well.

SAFETY MONITORING

In addition to the four observation wells constructed during site evaluation, four or more additional observation wells should be constructed at a radial distance far enough away from the injection well so that no grout would be expected to reach them. The distance beyond which the injected grout is not expected to extend during horizontal fracturing can be estimted by the analytical models (Sun, 1969). Each waste injection should be tagged with radioactive isotope as a tracer. After each injection, all observation wells should be monitored with geophysical probes.

Bedding-plane fractures should be indicated in the wells which are near the injection well by a zone of radioactivity near the injection depth; however, the observation wells at the greater distance from the injection well should not show evidence of the radioactivity above the original background level of the rock.

During the injection operations, wellhead pressure should also be observed carefully, and, if the injection pressure is lower than the value expected for horizontal or bedding-plane fractures, the operation should be suspended immediately to ascertain whether induced fractures have entered interconnected joints.

Precise leveling may be carried out after several injections to judge whether the observed uplift has followed a pattern predicted by the analytical models.

CONCLUSION

On the basis of the experience at ORNL and West Valley, New York, bedding-plane fractures can be hydraulically induced in a shale formation, at least within moderate depth, 2,000-3,000 ft, because of the great difference between tensile strength of shale in the direction parallel with, and that in the direction normal to, bedding planes. Injection pressure and pressure decay, as well as uplift produced by injections, can be used as indirect evidence to judge the orientation of induced fractures. Gamma-ray logs made in observation wells, however, should give direct evidence of the fractures' orientation, if injections include an appropriate radioactive tracer.

Wastes intimately mixed with grout, when injected into shale, can be immobilized by grout sheets and become an integral part of the shale. Therefore, it can be concluded that grout injection using hydraulic fracturing would be a safe and feasible disposal method--if the shale formation is carefully selected, the grout is properly mixed, the fracturing characteristics of the rocks are carefully tested and evaluated, and the injection is cautiously conducted.

Pressure-decay data obtained from water injection can be used not only to judge the orientation of induced fractures but also to determine the effective earth stress normal to the fracture plane. If the induced fracture is proved to be horizontal, then the vertical stress can be determined from pressure-decay data. This can be used to check whether the vertical stress at the site is simply equal to the weight of overburden. The pressure-decay method, however, needs further tests because it is developed empirically without a theoretical analytical basis.

Waste injection should be made repeatedly at different depths in an injection well, starting near the bottom of the well and moving upward, thus distributing the high cost of construction of the injection and monitoring wells over many injections, and making hydraulic fracturing economically attractive as a means for the disposal of wastes. For example, the projected cost at the ORNL is about $0.30 to $0.35 per gallon of waste for a facility disposing of 400,000 gal/year, with a total designed disposal capacity of 8 x 10^6 gal. This cost is well within the range of costs of competitive treatment and disposal methods for the type of wastes produced at the ORNL (de Laguna et al., 1968).

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Table 1. Calculated Overburden Pressure, Tensile Strength of Shale, Average Cohesive Forces at Fracture Tip, and Value of f at West Valley, New York

Injection Date	Depth ft	Injection fluid	Total injection volume, gal	Overburden pressure, psi by specific gravity	Overburden pressure, psi by pressure-decay data	Tensile strength, psi	Average cohesive forces at fracture tip (fT), psi[1]	f	Remarks
Oct. 9 1969	1,450	Water	114,300	1,581	1,573	447 602[2]	154	0.34	before 45 minute pause
						568	154	0.34	before 45 minute pause
June 26 1970	1,450	Water	112,200	1,581	1,513	562	258 162[3]	0.46 0.29[3]	
May 27 1971	500	Water	51,500	511	499	106	52	0.49	
July 23 1971	500	Water and Grout	2,350 water 39,000 grout	511	--	101	69	0.68	

[1]Average cohesive forces at fracture tip (fT) are calculated on the basis of the regression equations of the relation between P and Q. The values of (fT) calculated on the basis of the observed shut-in pressures are essentially the same as those based on the regression equations.
[2]**Calculated on the basis of the breakdown pressure at Q=100 gpm.**
[3]**Calculated on the basis of the actual observed shut-in pressure.**

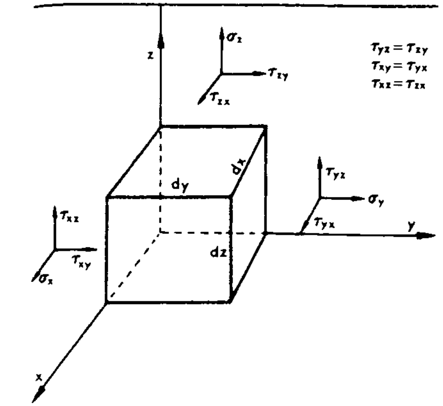

FIG. 1--Stresses on a small rectangular parallelepiped element located underground.

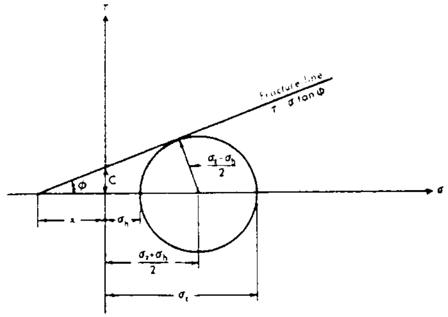

FIG. 2--Mohr's circle showing shear-failure envelope.

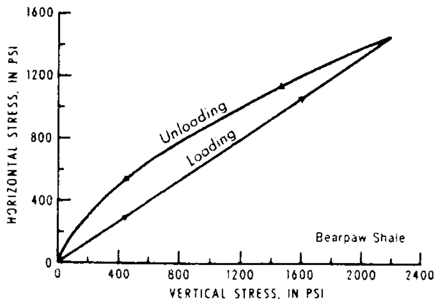

FIG. 3--Hysteresis during loading and unloading in uniaxial compression test.

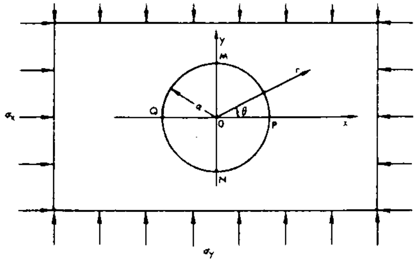

FIG. 4--Stresses on infinitely large plate with circular hole.

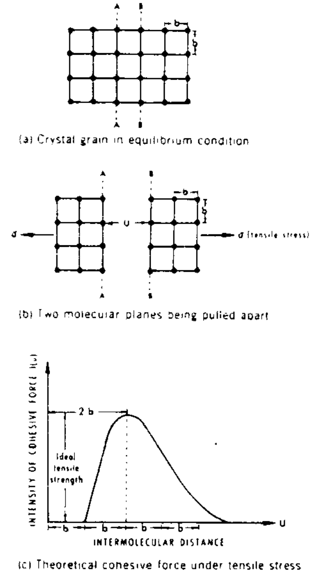

FIG. 5--Fracturing a material under tensile stresses.

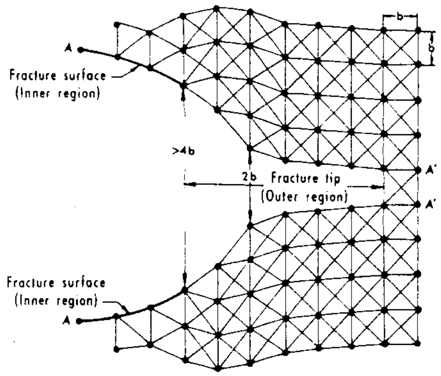

FIG. 6--Molecular structures around a fracture tip.

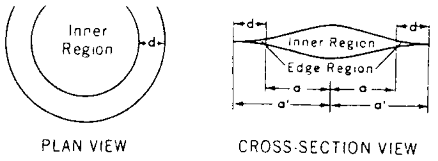

FIG. 7--Two regions of a fracture.

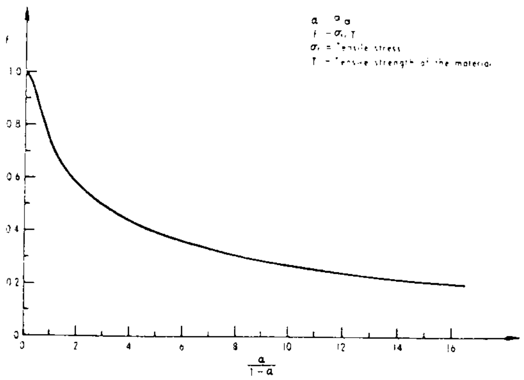

FIG. 8--Schematic diagram showing relation between f and α.

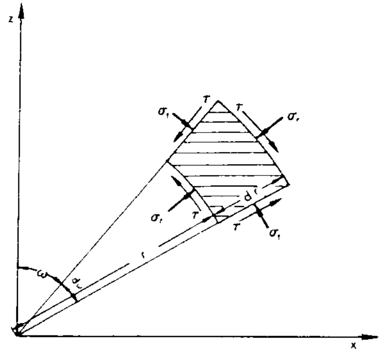

FIG. 9--Stresses on underground elements, expressed in polar coordinates.

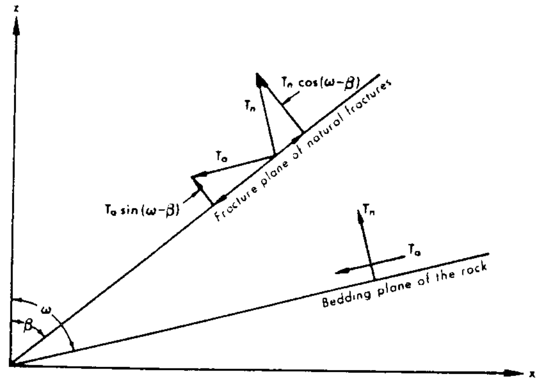

FIG. 10--Schematic diagram showing fracture plane of natural fractures and bedding plane of rock with respect to axis of injection well.

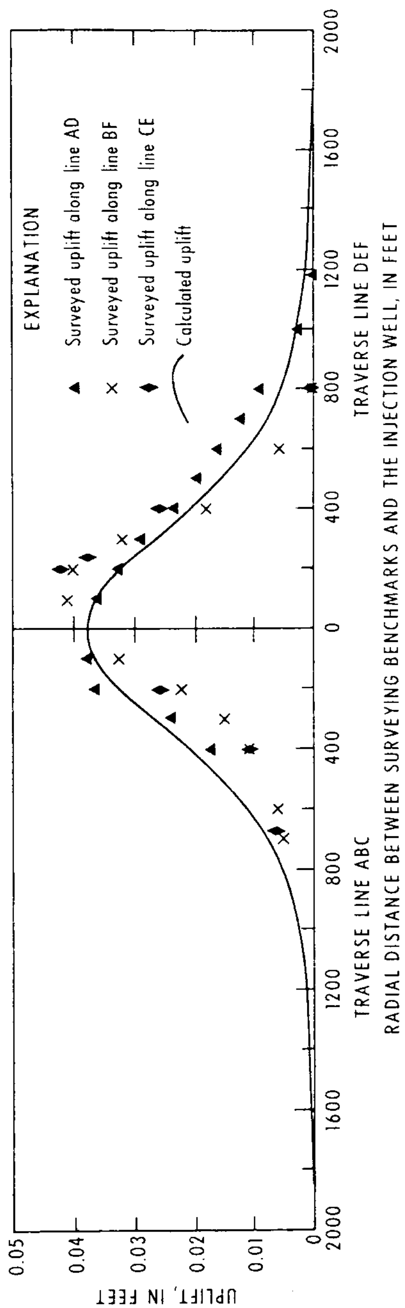

FIG. 11--Calculated and surveyed uplift produced by injection 2, second experiment at Oak Ridge National Laboratory, Oak Ridge, Tennessee, September 10, 1960.

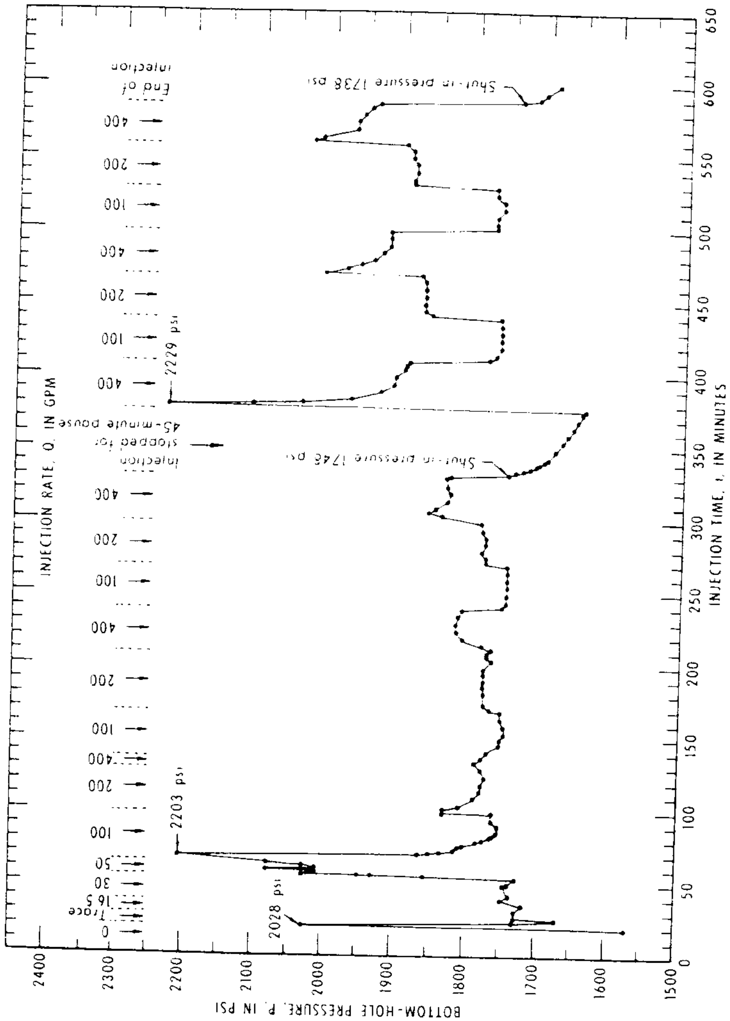

FIG. 12--Injection pressure versus time, first water injection at 1,450 ft, West Valley, New York.

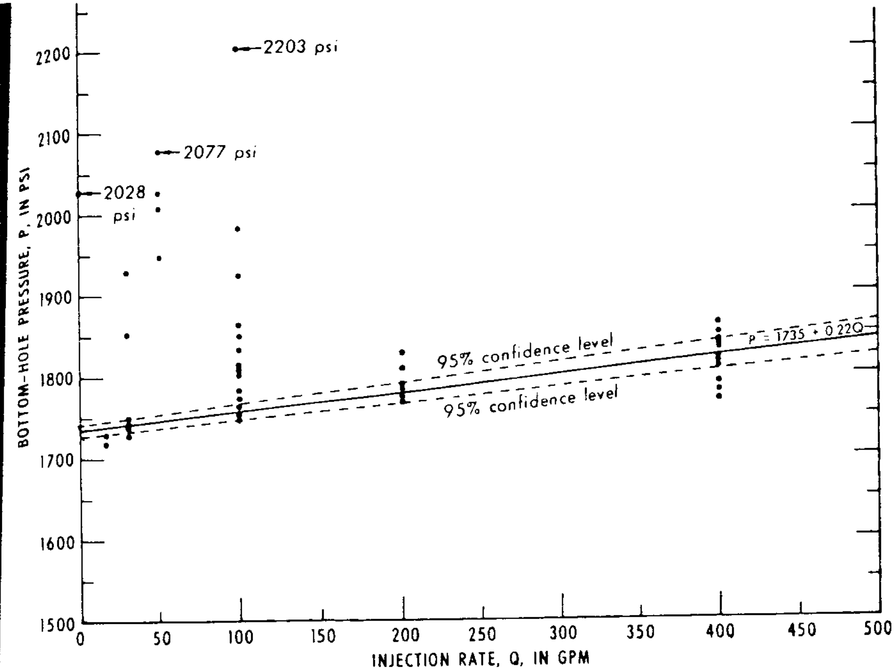

FIG. 13--P versus Q, before 45-minute pause, first water injection at 1,450 ft, West Valley, New York.

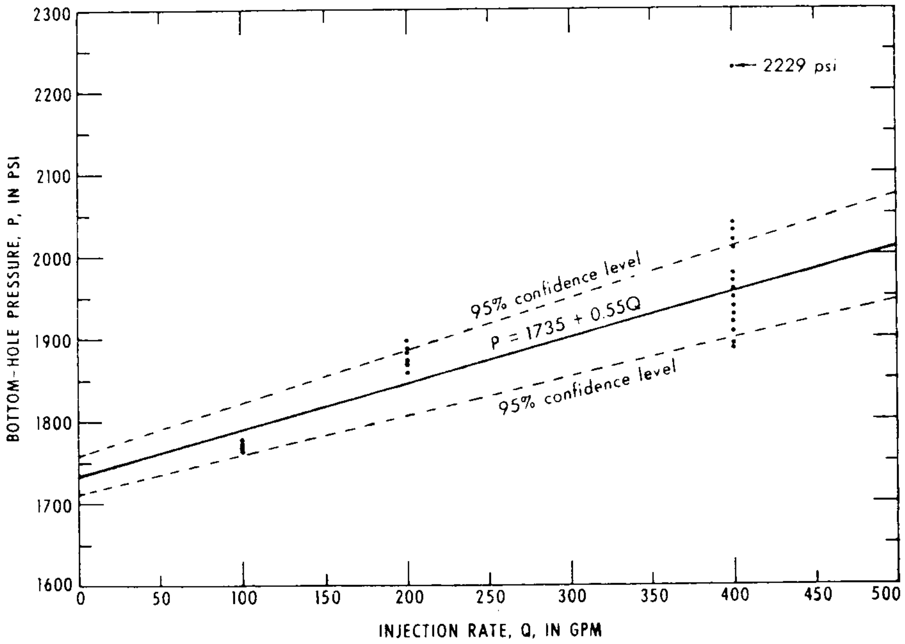

FIG. 14--P versus Q, after 45-minute pause, first water injection at 1,450 ft, West Valley, New York.

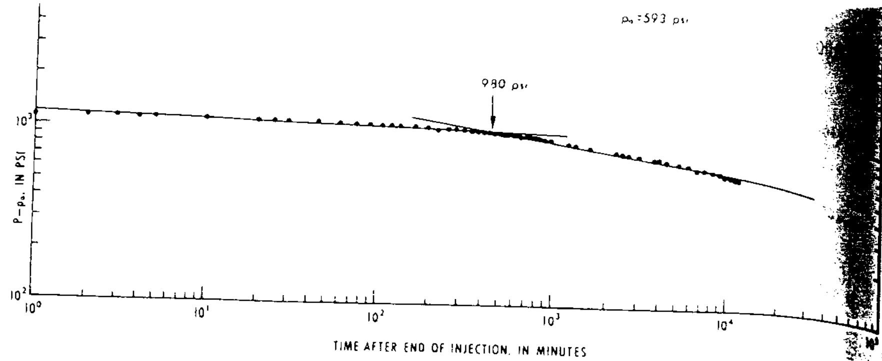

FIG. 15--Pressure decay versus time, first water injection at 1,450 ft, West Valley, New York.

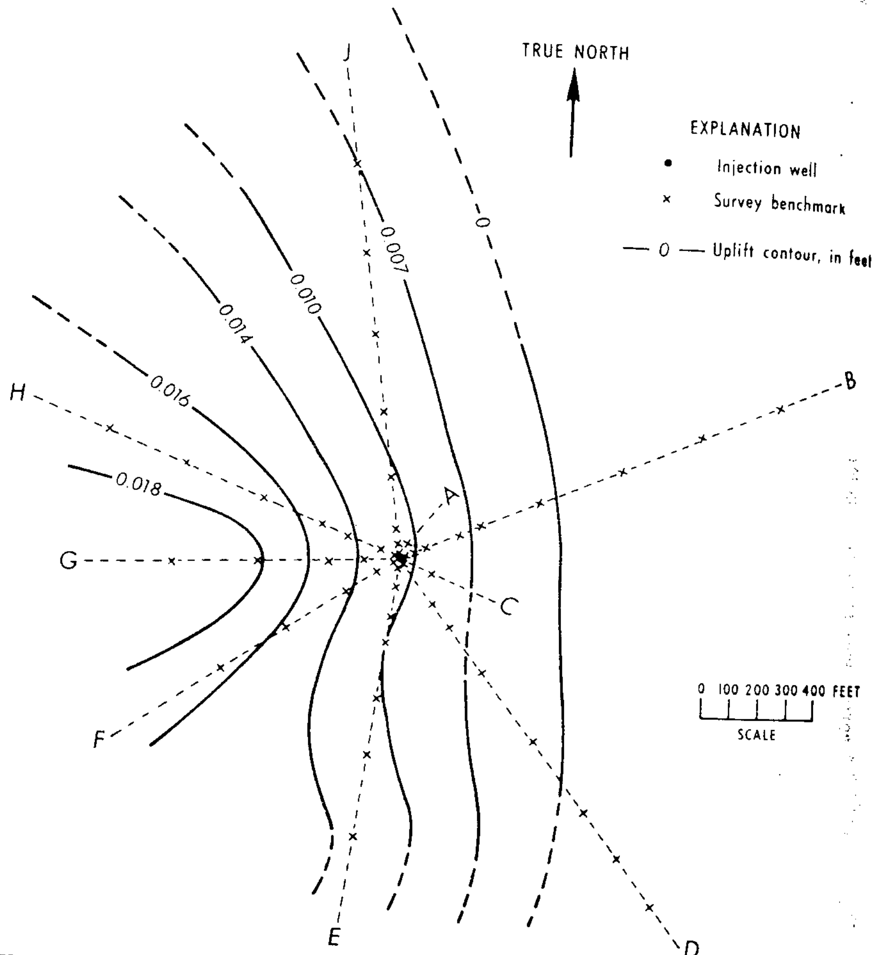

FIG. 16--Uplift produced by first water injection at 1,450 ft, West Valley, New York.

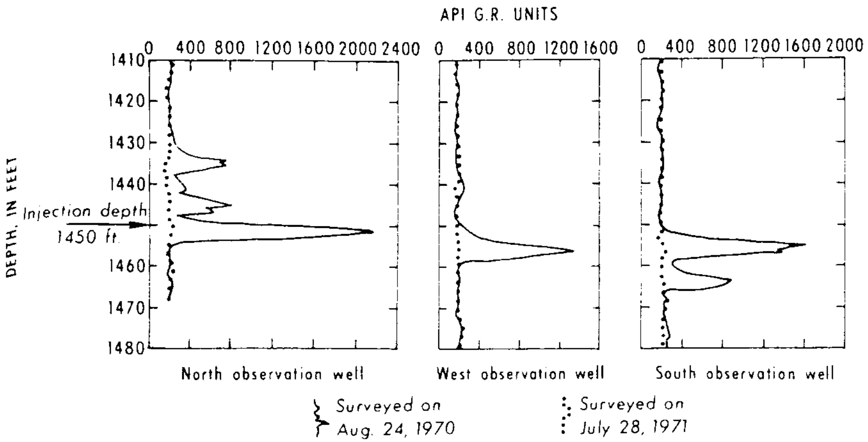

FIG. 17--Gamma-ray activity observation wells, second water injection at 1,450 ft, West Valley, New York.

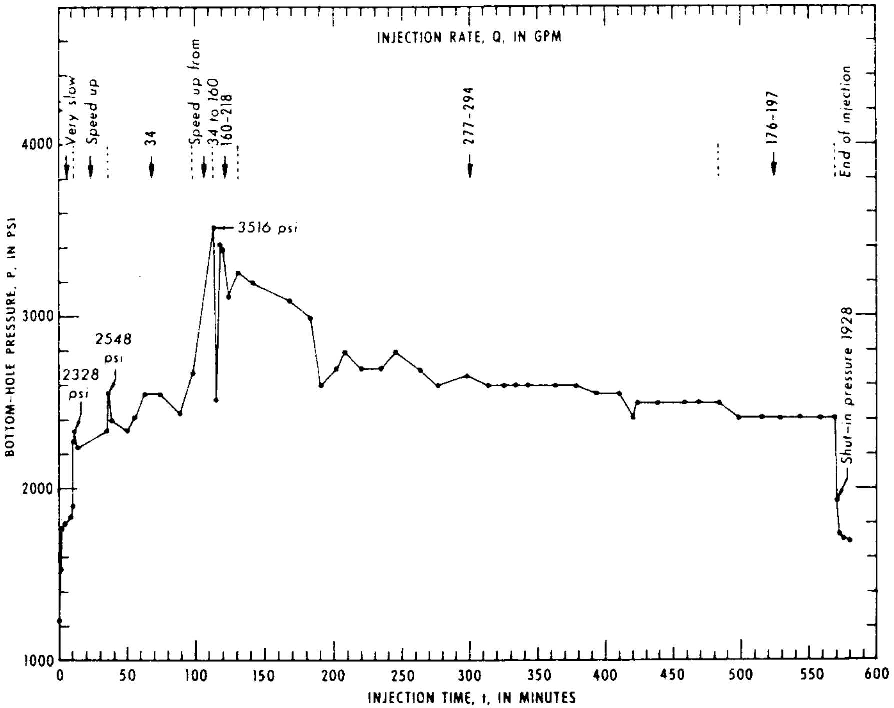

FIG. 18--Injection pressure versus time, second water injection at 1,450 ft, West Valley, New York.

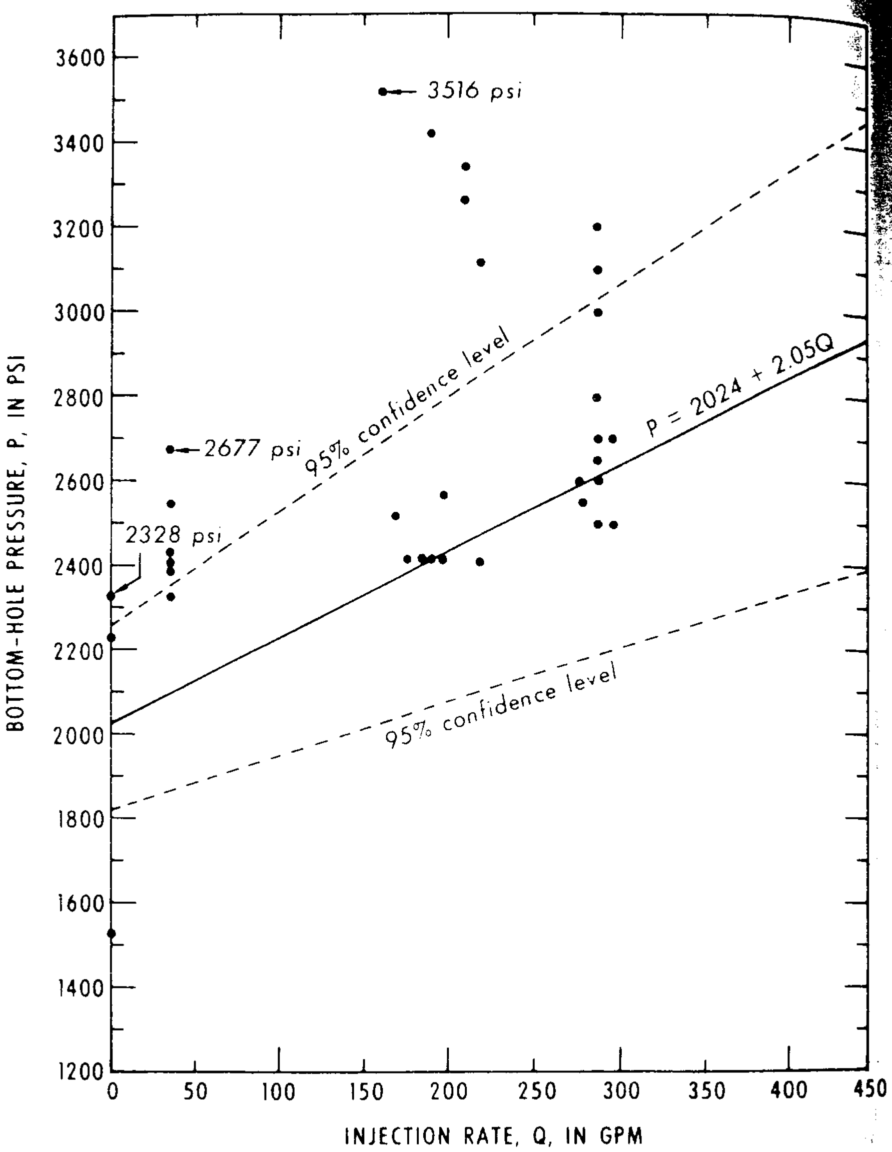

FIG. 19--P versus Q, second water injection at 1,450 ft, West Valley, New York.

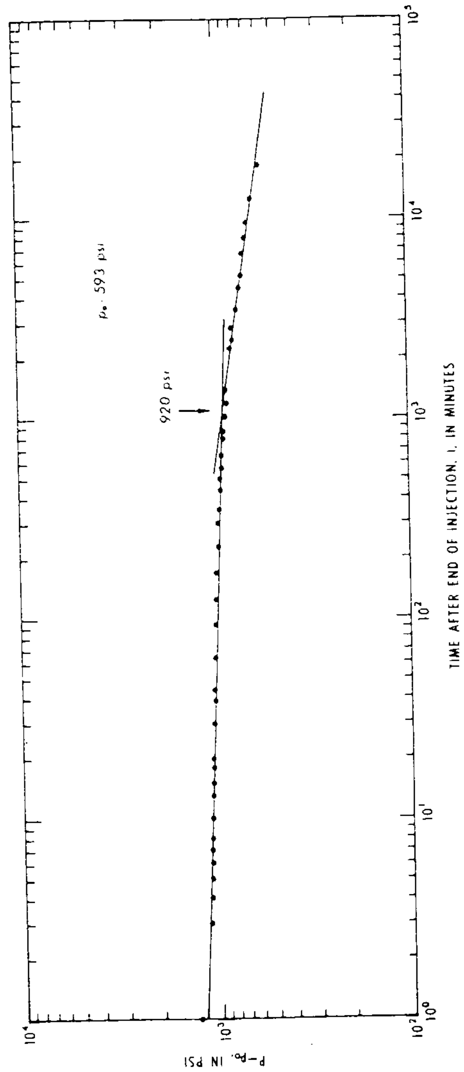

FIG. 20--Pressure decay versus time, second water injection at 1,450 ft, West Valley, New York.

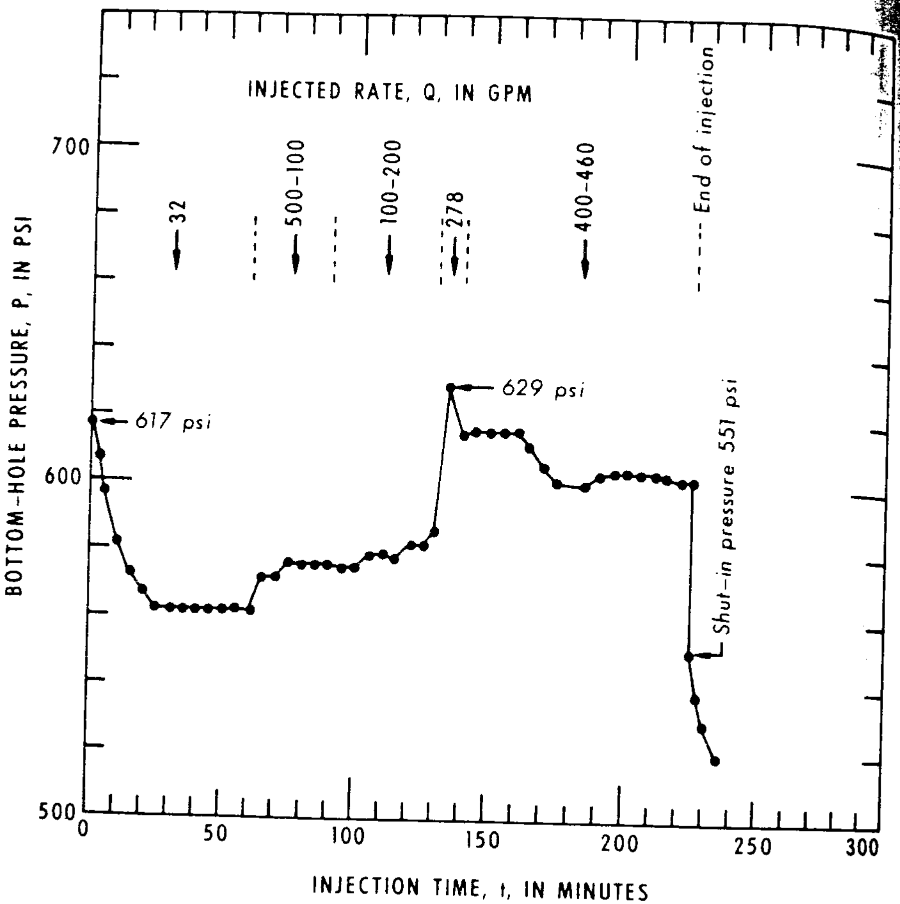

FIG. 21--Injection pressure versus time, water injection at 500 ft, West Valley, New York.

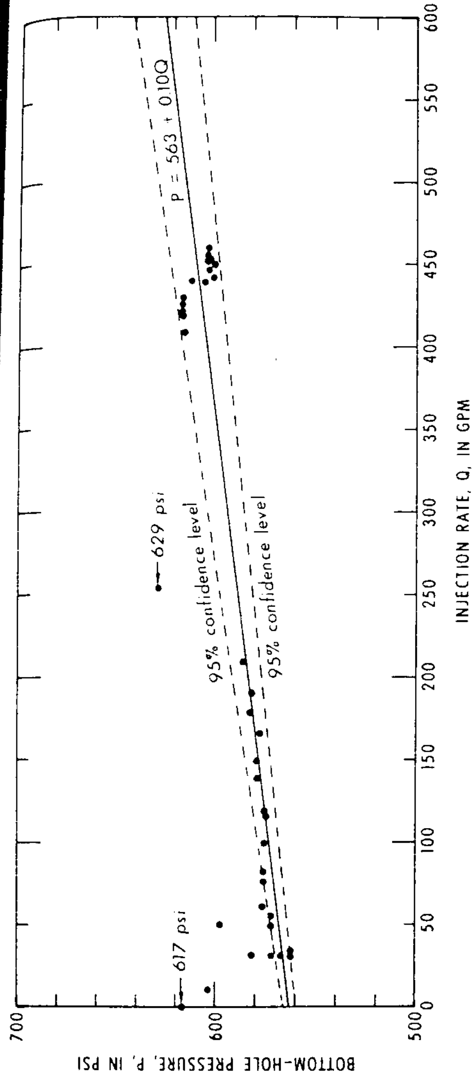

FIG. 22--P versus Q, water injection at 500 ft, West Valley, New York.

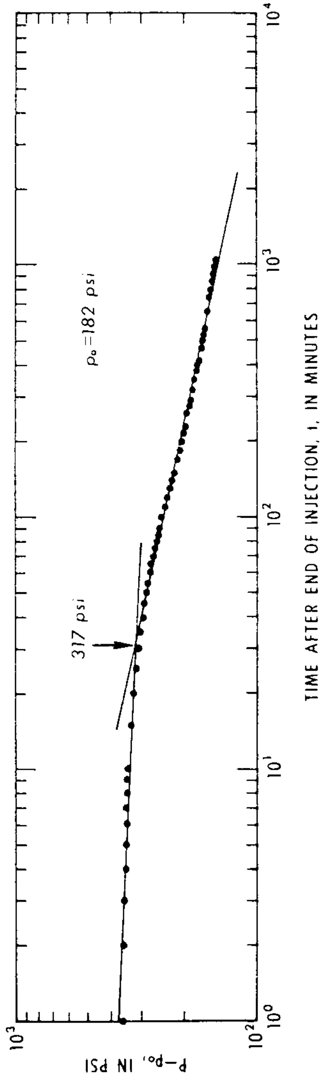

FIG. 23--Pressure-decay versus time, water injection at 500 ft, West Valley, New York.

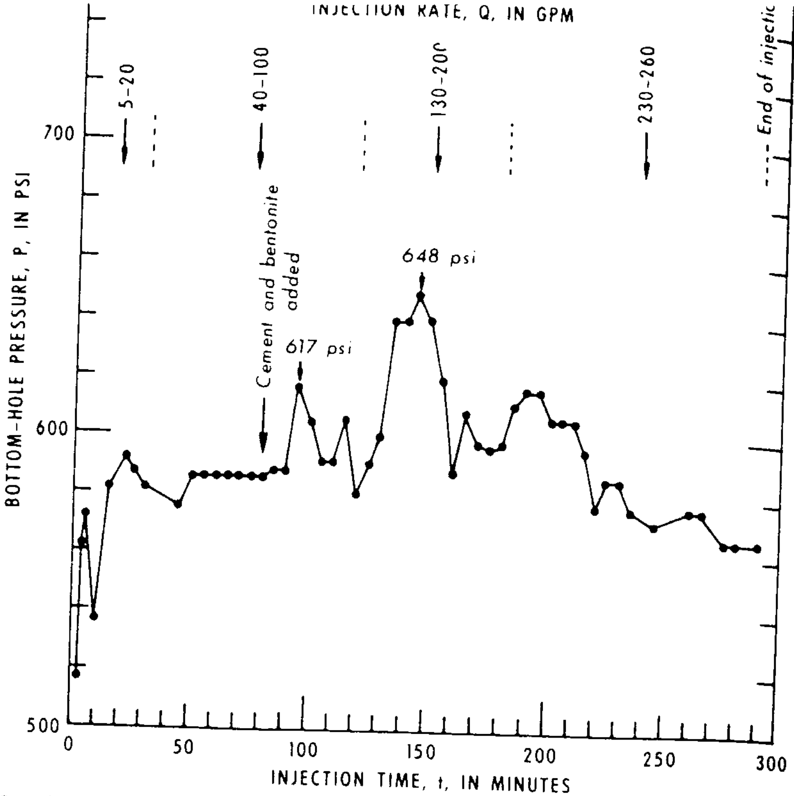

FIG. 24--Injection pressure versus time, grout injection at 500 ft, West Valley, New York.

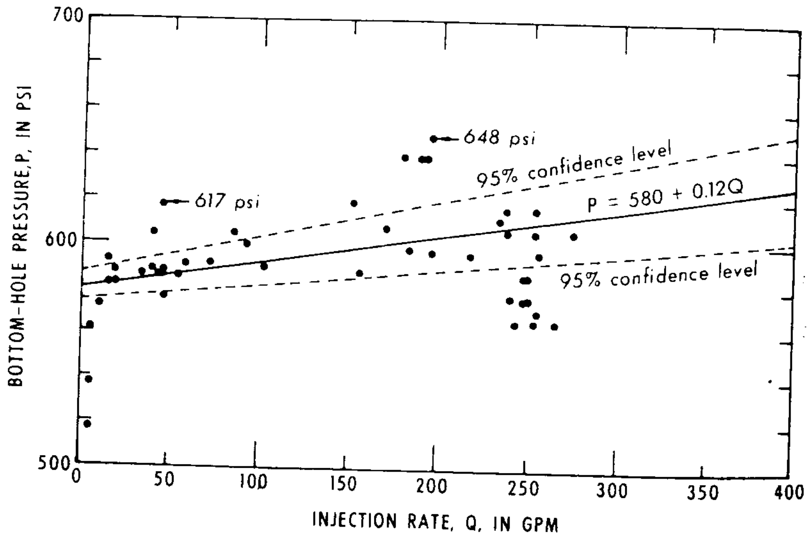

FIG. 25--P versus Q, grout injection at 500 ft, West Valley, New York.

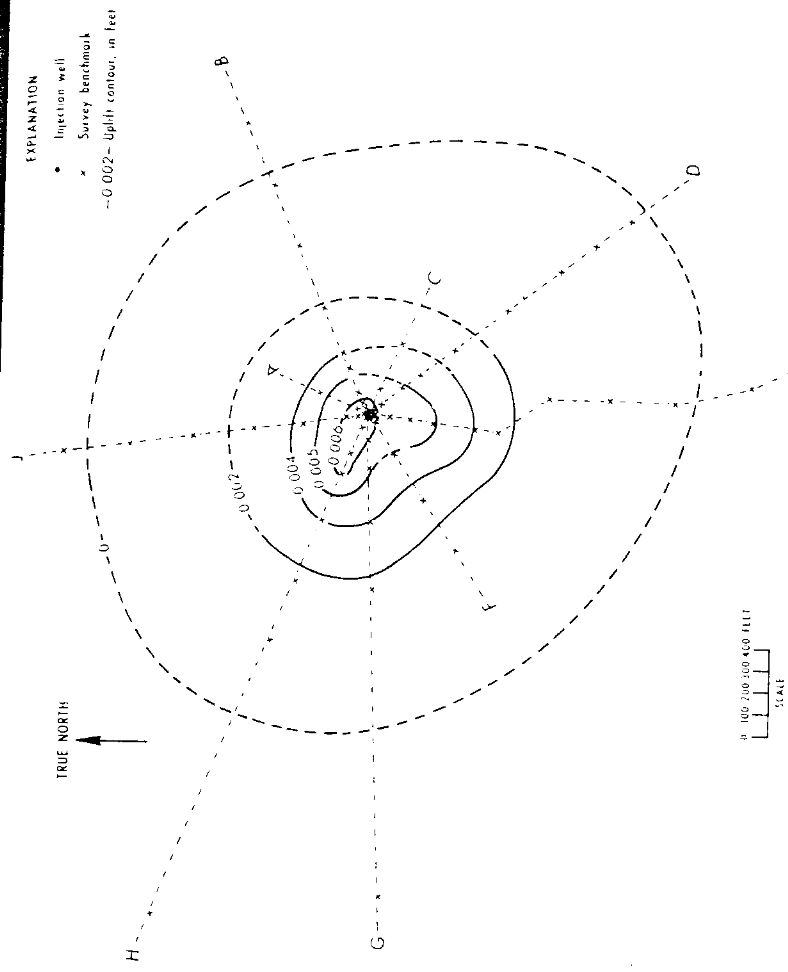

FIG. 26--Uplift produced by grout injection at 500 ft, West Valley, New York.

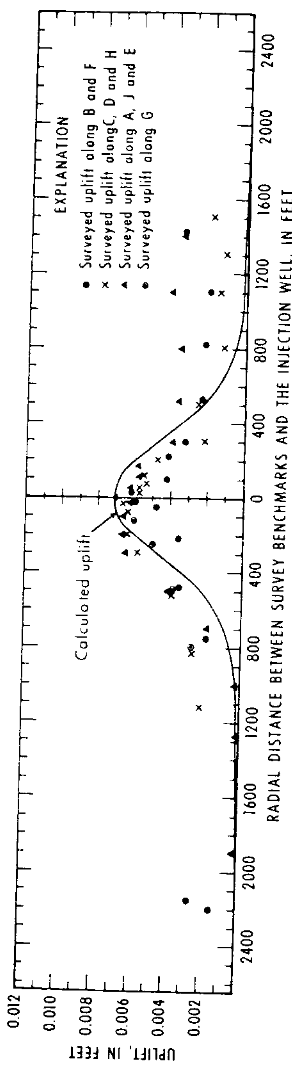

FIG. 27--Calculated and surveyed uplift produced by grout injection at 500 ft, West Valley, New York.

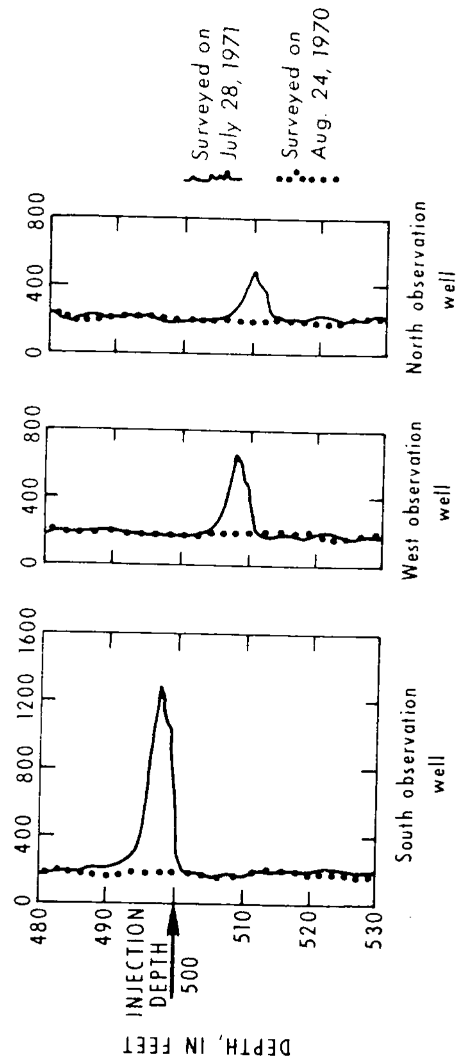

FIG. 28--Gamma-ray activity surveyed at the observation wells, grout injection at 500 ft, West Valley, New York.

APPENDIX I. Nomenclature

a	radius of fracture (inner region only); or borehole radius
a'	radius of fracture (both inner and outer region)
C, k	constants
D	inside diameter of casing
$F(Q,L,W)$	flow resistance in a vertical fracture
$F(Q,r,W)$	flow resistance in a horizontal fracture
fT	average cohesive force at fracture tip
f	$0 \leq f \leq 1$; or Fanning frictional factor, dimensionless
g	acceleration due to gravity
K'	fluid-consistency index, lb-second $^{n'}$/sq ft
L	length of casing; or length of vertical fracture
n'	fluid-flow behavior index, dimensionless
P	propagation pressure
P_i	breakdown pressure
$(P-p_o)$	pressure difference between the bottom-hole pressure in the injection well and the formation-fluid pressure (pore pressure) at the injection depth during the pressure-decay period
P_o	pore pressure
Δp	fluid-frictional loss in the casing
Q	rate of injection
R_e	Reynold's number, dimensionless
r	radial distance
T	tensile strength of rock
T_z	tensile strength of rock in the vertical direction
T_n	tensile strength of rock in the direction normal to the lamination of the rock
T_a	tensile strength of rock in the direction parallel to the lamination of the rock
t	time
v	flow velocity
W	width of fracture
x, y	distance along x-, y-axis, respectively; or x-, y-axis
z	depth
α	a/a', dimensionless
β	angle of natural fracture plane with the borehole axis
γ	weight density of rock
θ	polar angle
ρ	density

σ	normal density
σ_1, σ_2, σ_3	principal stress
σ_h	horizontal stress
σ_r	radial stress
σ_t	tangential stress
σ_z	vertical stress
σ_x, σ_y	stress along x-, y-axis, respectively
τ	shear stress
φ	angle of internal friction of rock
ω	angle of lamination of rock with the borehole axis

OPERATIONAL CASE HISTORIES

SHORT-TERM EFFECT OF INJECTION OF TERTIARY-TREATED SEWAGE ON IRON CONCENTRATION OF WATER IN MAGOTHY AQUIFER, BAY PARK, NEW YORK[1]

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Mineola, New York 11502

ABSTRACT Tertiary-treated sewage (reclaimed water) has been recharged by a deep well into the Magothy aquifer, the primary water-supply source for Nassau County, Long Island, New York. As of September 1972, 12 recharge tests have been run since the inception of the recharge program in September 1968. Although the iron concentrations of reclaimed and native water averaged 0.44 mg/l and 0.24 mg/l, respectively, the iron concentration of the mixed (native and reclaimed) water at times exceeded 3 mg/l. Several sources can account for the increase in iron concentration, but the most probable source is the pyrite that is native to the Magothy aquifer. During recharge, the natural reducing condition in the aquifer is replaced by a progressively more oxidizing environment. The initial response to this change is the oxidation of pyrite, which released Fe^{+2}, SO_4^{-2}, and H^+ into solution. Eventually, ferric hydroxide precipitates, and the Fe^{+2} concentration decreases. The exact oxidation mechanism apparently involves inorganic and (or) organic constituents in the reclaimed water, because water from the public potable water-supply system that is injected into the aquifer does not cause an increase in iron concentration.

[1]Manuscript received, June 6, 1973. Publication authorized by the Director, U.S. Geological Survey.

[2]Chemist, U.S. Geological Survey.

[3]Hydrologist, U.S. Geological Survey.

The authors gratefully acknowledge the work of John H. Peters, Commissioner, Nassau County Department of Public Works, in supporting the project. Thanks are given also to Ms. Lillian B. Maclin, U.S. Geological Survey, for her diligent efforts in making the numerous on-site chemical analyses on which this report is largely based.

INTRODUCTION

The U.S. Geological Survey, in cooperation with the Nassau County Department of Public Works, is studying the feasibility of recharging tertiary-treated sewage (reclaimed water) into the Magothy aquifer, Nassau County's primary source of potable water (Cohen and Durfor, 1967; Peters and Rose, 1968). Water is reclaimed at the Nassau County Bay Park sewage-treatment plant in Bay Park, New York, and is recharged through a well half a mile away. As of September 1972, 12 recharge tests ranging in duration from 2 to 33 days have been made since the inception of the recharge program in September 1968. Although the iron concentration of both reclaimed water and native water in the Magothy aquifer is generally less than 0.5 mg/l, the iron concentration in the mixed (reclaimed and native) water at times exceeded 3 mg/l.

In one of the more recent tests (from November 1, 1971, to November 11, 1971), detailed sampling at an observation well (N7886) 20 ft from the recharge well showed that the iron concentration began to increase with the first appearance of the reclaimed water. The iron concentration increased to more than 3 mg/l three days after the start of injection and then decreased. After 10 days of injection, the iron concentration of the mixed water remained higher than that of either the reclaimed water or the native water.

This paper describes the short-term changes in the concentration of iron in relation to the movement of the reclaimed-water front through the aquifer, and interprets these changes in terms of physico-chemical processes in the aquifer system.

PROCEDURES

Water Reclamation and Recharge

A small part of the effluent from the Nassau County Bay Park sewage-treatment plant is diverted to a pilot 400-gpm (gallons per minute) tertiary-treatment plant, where the effluent is further treated to produce virtually potable water (Peters and Rose, 1968). The reclaimed water is pumped half a mile to a storage tank at the recharge plant. In addition to the storage tank, the recharge plant has (1) an 18-in.-diameter fiberglass water well, with a 16-in.-diameter by 62-ft-long stainless steel screen at a depth of 418-480 ft; (2) various equipment for further treatment of the reclaimed water (dechlorination, degasification, pH, and [or] Eh adjustments); and (3) 14 observation wells at different

distances, ranging from a few inches to 200 ft from the recharge well and screened at various depths in the Magothy aquifer. More detailed descriptions of this plant have been published previously (Perlmutter et al., 1968; Cohen and Durfor, 1966, 1967).

Sampling and Analytical Methods

At appropriate time intervals during recharge, samples were collected from the injectant and from observation well N7886, which is 20 ft from the recharge well and is screened at the same depth interval as the recharge well. Procedures outlined by Brown et al. (1970) were used to determine total iron, ferrous iron, bicarbonate, sulfide, dissolved oxygen, chlorine residual, temperature, specific conductance, pH, Eh, and turbidity at the recharge site. Additional analyses (Tables 1, 3) were performed by the Geological Survey Laboratory in Albany, N.Y.

RESULTS AND DISCUSSION

Comparison of Chemical Quality of Native and Reclaimed Water

Analyses of native-water samples collected at observation well N7886 before and during the early hours of recharge test RW 10,[4] part 1, indicate the water quality to be similar to that previously reported for the Bay Park site (Table 1; Vecchioli et al., in press; S. D. Faust and John Vecchioli, written commun., 1971).

The chemistry of the native water in the Magothy aquifer is controlled by precipitation recharging of the aquifer and by subsequent reactions between the mineral constituents of the aquifer and the water moving through it (Pearson and Fisher, 1971). The Magothy aquifer at Bay Park consists of very fine to medium quartzose sand, containing some clay, lignite, muscovite, and pyrite-marcasite (Vecchioli et al., in press).

The pH of groundwater is buffered by the H_2CO_3-HCO_3^- equilibria. Carbon-dioxide partial pressure is one to two orders of magnitude higher than that in equilibrium with air (S. D. Faust and John Vecchioli, written commun., 1971; Pearson and Friedman, 1970). This increased partial pressure results from the oxidation of lignite by the dissolved oxygen in the native recharge water some distance north of Bay Park (Pearson and Friedman, 1970). Clay minerals, principally kaolinite, con-

[4]RW 10 means reclaimed water test 10 in a series of tests in which reclaimed water was injected.

trol the concentrations of alkali and alkaline-earth elements by cation exchange (Pearson and Fisher, 1971).

The dissolved-oxygen concentration of native water of the Magothy aquifer at Bay Park is zero. Eh measurements indicate that the system is near equilibrium for pyrite oxidation:

$$FeS_2 + 8H_2O = Fe^{+2} + 2SO_4^{-2} + 16H^+ + 14e \qquad (1)$$

(Vecchioli et al., in press). This equation suggests that under suitable conditions of Eh and pH the concentration of dissolved iron could be substantially changed by the oxidation of pyrite.

The concentration of most constituents was greater in the reclaimed water than in the native water. Except for COD (chemical oxygen demand), the chemical quality of water injected in this test was similar to that injected in the other tests (S. D. Faust and John Vecchioli, written commun., 1971; Ehrlich et al., 1972; Vecchioli, 1972). COD concentration was about twice the usually observed concentration because filtration through activated carbon columns was omitted from the tertiary-treatment process. Of particular importance in this study are the differences between the reclaimed water and the native water in pH: 6.28 as compared with 5.50; dissolved oxygen: 6.6 as compared with 0; bicarbonate concentration: 88 as compared with 8; and COD: 22 as compared with 0. The reclaimed water contained iron in both filterable (greater than 0.45 micrometer) and unfilterable (less than 0.45 micrometer) fractions.

Changes in Iron Concentration at Observation Well N7886 During Recharge

The effect of recharge on pH, specific conductance, and the concentrations of bicarbonate and total iron in water from observation well N7886, 20 ft from the recharge well, during part 1 of test RW 10, is indicated in Figure 1. The curves represent typical kinds of changes during the first several days of recharge. Similar changes have been observed in other tests, but they were documented most thoroughly in this one.

During the first 17 hours of recharge, total iron concentration was 0.21 $\pm$ 0.03 mg/l (Fig. 1A). The iron was in solution in the Fe^{+2} state; this condition, together with the observed concentration, is typical for native water at Bay Park. After 18.5 hours of recharge, an increase in iron concentration, which coincided with the arrival of the reclaimed-water front, was observed. Arrival of the reclaimed-water front is indi-

cated by the sharp rise in specific conductance (Fig. 1B), which reflects the tenfold-higher dissolved-solids concentration of the reclaimed water as compared with the native water. Fe^{+2} concentration and specific conductance continued to rise until t = 31.5 hours (small "t" is time after recharge began). At this time, the Fe^{+2} concentration reached 3.05 mg/l and then began to decrease. Fe^{+2} concentration decreased sharply at first (to 1.55 mg/l after 48 hours) and then gradually to 0.72 mg/l after 239 hours, when recharge was stopped.

Specific conductance, on the other hand, continued to increase after 31.5 hours or recharge, but leveled off at about 700 micromhos after about 80 hours. Consequently, most of the large increase and the decrease in the iron concentration occurred at the reclaimed-water front, where the reclaimed water mixed with the native water. On the arrival of the reclaimed-water front at 18.5 hours, pH and HCO_3^- concentrations decreased slightly but they began to increase at about 31.5 hours (Figs. 1C, 1D). These changes reflect the type of chemical reactions occurring in the front and are discussed in a following section.

Redevelopment of Recharge Well

After 10 days of recharge of the Magothy aquifer with reclaimed water, redevelopment of the recharge well by pumping was begun. Samples were collected during five pumping surges (Fig. 2). The well was first surged immediately after recharge ended, at 300 gpm; it was surged a second time about a month later, at the same rate. The well was surged three additional times at 950 gpm, beginning about 3 weeks after the second surge. The iron concentration of the initial slug of water repumped in the first three surges was high (Table 2). This effect of surging has been reported previously (Vecchioli and Ku, 1972; S. D. Faust and John Vecchioli, written commun., 1971). Little increase in the iron concentration was observed in the last two short surges, presumably because the iron that had collected around the well-screen - aquifer interface had been removed during the previous surges.

INTERPRETATION OF DATA

Knowledge of the source of the iron is necessary for an understanding of the increase in concentration of iron in the mixed-water system. Three possible sources of iron were considered: (1) colloidal iron in the reclaimed water, (2) dissolved iron in the reclaimed water, and (3) native pyrite in the Magothy aquifer. Neither iron from the recharge well nor that from the observation wells was considered a possible source because the wells were constructed of either nonferrous or noncorrodible materials (Cohen and Durfor, 1966).

Although the concentration of iron in the reclaimed water is relatively low with respect to the concentration in the mixed water, the authors considered the possibility that iron might accumulate at some site in the vicinity of the well and subsequently be released to produce the observed iron peak. Distinction was made between colloidal iron and dissolved iron in the reclaimed water because of the different mobility of the phases with respect to the aquifer. The colloidal-iron phase is the iron that is removed by a 0.45-μ filter and, because of its size, is trapped at the gravel-pack-aquifer interface. The dissolved-iron phase is Fe^{+2}. This phase will move into the aquifer.

The pyrite in the Magothy aquifer seems to be the most probable source of iron for the observed iron peak. A more detailed discussion of the three possible iron sources follows.

Colloidal Iron in Reclaimed Water

The fine- to medium-grained Magothy aquifer is an effective filter for the particulate matter in the reclaimed water. Earlier tests showed that as injection proceeds a mat of organic material containing high concentrations of iron, phosphate, and sometimes aluminum accumulates at the gravel-pack - aquifer interface (Vecchioli and Ku, 1972). Thus, the colloidal iron in the reclaimed water is probably trapped at this point in the aquifer. The buildup of the mat results in a reduction of the specific capacity of the well. Even after prolonged redevelopment, the specific capacity is not restored to pre-injection levels (Vecchioli and Ku, 1972; Vecchioli, 1972), suggesting that a small amount of the mat remains. After the organic mat has been exposed to anaerobic bacteria in the aquifer (Ehrlich et al., 1972), the mat may be degraded, and the iron may be released into solution. Analyses of water samples collected from the recharge well about 30 days after the first two redevelopment surges after RW 10 support the occurrence of this degradation process, as the concentrations of dissolved iron, TOC (total organic carbon), and COD were 40 mg/l, 21 mg/l and 50 mg/l, respectively, at this time. Thus, if the colloidal iron that is trapped at the gravel-pack - aquifer interface were released into solution by biological activity, a subsequent injection could carry the iron into the aquifer.

There are several quantitative difficulties with the colloidal-iron

model, however. First, all the iron that was injected into the well in colloidal form during part 1 of RW 10 can be accounted for by the iron recovered during the redevelopment surges. Whereas 5.5×10^6 mg of colloidal iron was injected, 5.8×10^6 mg of iron was recovered (Table 3). The table also shows that when the colloidal iron content of the injectant is low, as for part 2 of RW 10 and for RW 11 and RW 12, the iron recovered during redevelopment surging is low. Thus, all the colloidal iron in the reclaimed water seems to have been filtered and trapped at or very near the aquifer-well interface, and to have been recovered during redevelopment.

Several other points should be noted. The iron concentrations observed between parts 1 and 2 of RW 10, although helpful in establishing the kinds of chemical and biological activities occurring during shutdown of the recharge well, represent an extremely high rate of biological activity because the well was not fully redeveloped after part 1 of the test. The anaerobes were exposed to high concentrations of organic and inorganic nutrients, so their activity was not restricted. Usually the well is redeveloped for a prolonged period after injection ends, and the well and its environs are restored to virtually native conditions (except for the slight decrease in specific capacity, as noted earlier). Under these conditions, anaerobic activity is restricted. Samples taken before RW 12, from a completely redeveloped well, show the effect of this restricted activity; the iron concentrations of these samples are only 1-2 mg/l.

No iron peak occurred when city (drinking) water was injected into the aquifer under the same conditions used to inject the reclaimed water. The peak probably does not result, therefore, from the physical action of injection alone, as it would if the colloidal-iron model were correct.

Dissolved Iron in Reclaimed Water

The other iron phase in the reclaimed water, dissolved ferrous iron, was also considered to be a source of iron. Eh-pH conditions of the reclaimed water indicate that the Fe^{+2} is unstable with respect to the precipitation of $Fe(OH)_3$. Consequently, ferrous iron is probably being converted to ferric iron and is being precipitated as ferric hydroxide. However, the point of accumulation is not restricted to the well-screen-aquifer interface as in the colloidal-iron model. The aquifer acts as a medium on which precipitation can occur. This has been at least partly substantiated in preliminary laboratory-column studies. As injection proceeds, the iron precipitate could accumulate as far as 20 ft from the recharge well. After redevelopment of the well, the aquifer is returned to its native, reducing state. One might expect that the precipitated ferric iron would be reduced under these conditions and that the Fe^{+2} concentration would be increased. As pyrite does not readily precipitate from solution (R. A. Berner, written commun., 1972; Barnes and Clarke, 1969), this supply of iron-rich water could be carried into the formation with any subsequent recharge.

The difficulties with this model are similar to those cited for the colloidal-iron model. The dissolved-iron content of the reclaimed water is inadequate to account for the iron content of the peak (Table 3), and, also, the city-water test should have carried this iron into the formation.

Native Pyrite as Iron Source

The iron pyrite in the Magothy aquifer is the most likely source of iron for the iron peak. Recharging the aquifer with reclaimed water changes the environment of the aquifer from a reducing to an oxidizing one. The change is abrupt at the well-screen - aquifer interface, but becomes more gradual as the reclaimed water moves into the formation because of the mixing action between the reclaimed water and the native water. Eventually, an oxidizing zone, with a radius of about 12 ft, is established for a 10-day test. This radial distance is determined from the dissolved-oxygen concentration of repumped water, as follows: water pumped from the recharge well immediately after completion of injection RW 12 contained detectable dissolved oxygen. The dissolved oxygen persisted until about 67,500 gal had been recovered. Assuming that this volume was contained uniformly in a 60-ft-high cylinder with an estimated porosity of 30 percent, the radius of this oxidizing zone extended about 12 ft away from the recharge well.

In terms of iron chemistry, during recharge a reduced condition, in which pyrite is the stable mineral assemblage, changes to a more oxidized one, in which ferric hydroxide becomes stable. The iron peak results from this redox change. In response to the higher redox potential, pyrite becomes thermodynamically unstable and reacts to release Fe^{+2}, SO_4^{-2}, and H^+ to solution (Equation 1). This reaction explains the Fe^{+2} and H^+ increase during the interval t = 18.5-31.5 hours (Figs. 1A, 1C).

Eventually, ferric hydroxide begins to precipitate, and the concentration of iron in solution decreases. This precipitation explains the iron decrease after t = 31.5 hours in Figure 1A. Although the reaction also produces H^+ (Equation 2), the pH increases after this time because

of the increasing percentage of reclaimed water in the mixed-water system. The reclaimed water has a higher pH than the native water (Fig. 1D):

$$Fe^{+2} + 1/4\ O_2 + 5/2\ H_2O + Fe\ (OH)_{3(s)} + 2H^+. \qquad (2)$$

A qualitative conception of this reaction sequence may be obtained from the diagram based on the data for the reclaimed water, which shows iron solubility in relation to Eh and pH (Fig. 3). The diagram represents an equilibrium condition in which a constant total alkalinity, sulfide concentration, and ionic strength are specified. However, the real (mixed) system is not at equilibrium, and total alkalinity, sulfide concentration, and ionic strength vary with the degree of mixing of reclaimed water and native water. No iron phosphate species is shown in the diagram because virtually all phosphate is lost from solution before it reaches the "20-ft" well (Vecchioli, 1972); consequently, the phosphate is not correlated with the iron peak. In addition, the diagram does not show the role of organic compounds in the iron chemistry.

Nevertheless, the diagram (Fig. 3) is useful in establishing thermodynamic tendencies, with respect to the oxidation of pyrite and precipitation of ferric hydroxide under different Eh and pH conditions. The symbol "X" indicates the equilibrium (reducing) conditions in the native water system. The symbol "Y" indicates the Eh-pH condition of the reclaimed water. The pH of this water is known. The Eh of the water is estimated to be at least +0.35 mv (millivolt). This value is based on the minimum Eh reported for natural water in contact with the atmosphere at a pH of 6 to 8 (Hem, 1970).

Even a small increase in Eh will increase the thermodynamic tendency for pyrite to oxidize and to release Fe^{+2}, SO_4^{-2}, and H^+ into solution (Fig. 3). An iron concentration between 200 and 300 mg/l could be retained in solution at equilibrium if the S_T (total sulfur) and C_T (total carbon) values cited in Figure 3 prevailed. This level is not likely to be attained, however, because the available concentration of oxidizing material is relatively small. A DO concentration of 6.6 mg/l is equivalent to a Fe^{+2} concentration of 3 mg/l in the pyrite-oxidation reaction.

Above an Eh of +0.2 mv, the thermodynamic tendency would be to precipitate ferric hydroxide and to decrease the concentration of Fe^{+2} in solution.

The thermodynamic model, however, does not completely explain the formation of the iron peak. The authors found that injection of city (drinking) water, with a dissolved-oxygen concentration of 4-6 mg/l and a pH of 6.65, does not cause an iron peak to form, even though Eh-pH considerations suggest that it should. The fact that city water did not produce an iron peak, whereas reclaimed water did, may result from the different rates of pyrite oxidation and ferric hydroxide precipitation. An iron peak, such as that shown in Figure 1A, will develop when the rate of Fe^{+2} production (pyrite oxidation) exceeds that of Fe^{+2} consumption (Fe^{+2} oxidation and subsequent precipitation as ferric hydroxide). Several inorganic and microbiological agents have been observed to accelerate pyrite oxidation (Walsh and Mitchell, 1972; Stumm and Morgan, 1970; Baker and Wilshire, 1970; Singer and Stumm, 1968; Smith et al., 1968; Garrels and Thompson, 1960). In terms of the present study, the presence of Fe^{+3} in the reclaimed water may play a role in accelerating the pyrite oxidation by the following reaction:

$$FeS_{2(s)} + 14Fe^{+3} + 8H_2O = 15Fe^{+2} + 2SO_4^{-2} + 16H^+. \qquad (3)$$

The rate-determining step in the precipitation of ferric hydroxide in acidic solutions is the oxidation of the Fe^{+2} produced by pyrite oxidation (Singer and Stumm, 1968). The many factors affecting the rate of Fe^{+2} oxidation include PO_2 (partial pressure of oxygen) and Fe^{+2} concentration of the solution (Singer and Stumm, 1968; Stumm and Lee, 1961), microbiological agents (Walsh and Mitchell, 1972; Baker and Wilshire, 1970; Oborn and Hem, 1961), solution pH (Singer and Stumm, 1968; Stumm and Lee, 1961), solution buffer capacity (Jobin and Ghosh, 1972), the nature of the anionic species in solution (Singer and Stumm, 1968), and organic matter (Jobin and Ghosh, 1972; Oborn and Hem, 1961; Hem, 1960). Organic matter decreases Fe^{+2} oxidation rate, whereas anionic species may either increase or decrease the oxidation rate. Increased concentrations of the remaining components tend to accelerate Fe^{+2} oxidation.

In terms of the present study, organic matter in the reclaimed water and the water's slightly acidic pH could sufficiently retard the Fe^{+2} oxidation reaction to produce the iron peak.

The effect of organic matter on oxidation of Fe^{+2}, although not thoroughly studied, has been at least partly substantiated in the recharge tests. As shown in Table 4, the iron content in the peak observed during part 1 of RW 10 is greater than that for all other recharge tests. The organic load was greatest in this test because the reclaimed water had not been passed through the carbon filters (Table 4).

The effect of pH also agrees, at least qualitatively, with the model. When the pH was increased to 8 in RW 12, the iron peak was reduced substantially, to a maximum concentration of only 1.37 mg/l and a total iron content of only 5 x 10^6 mg (Table 4). Earlier tests using city water, in which the pH was decreased to 5.5, showed a substantial increase in iron, although not necessarily an iron peak, as well (F. J. Pearson, Jr., and G. D. Bennett, written commun., 1971).

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Table 1. Comparison of the Chemical-Quality Characteristics of Reclaimed Water to Native Water at Observation Well N7886[1]

	Reclaimed water[2]			Native water at observation well N7886[3]
	AVERAGE		STANDARD DEVIATION	AVERAGE
Silica (SiO_2)	13	±	1	7.2
Iron (Fe), total	0.44	±	0.36	0.24
Iron (Fe), dissolved	0.16	±	0.08	0.25
Manganese (Mn), total	0.07	±	0.02	0.01
Manganese (Mn), dissolved	0.10	±	0.04	0.01
Calcium (Ca)	15	±	2	1.2
Magnesium (Mg)	5.8	±	1.0	0.7
Sodium (Na)	81	±	6.0	4.4
Potassium (K)	12	±	0.5	0.8
Bicarbonate (HCO_3)	88	±	15	8
Sulfate (SO_4)	136	±	13	5.2
Chlorine, total residual	2.4	±	0.5	0.0
Chloride (Cl)	85	±	9	3.6
Fluoride (F)	0.2	±	0.0	0.0
Ammonium (NH_4)	34	±	2	0.85
Nitrate (NO_3)	0.4	±	0.0	0.0
Nitrite (NO_2)	0.01	±	0.01	0.00
Phosphate (PO_4), total	0.69	±	0.46	0.03
Dissolved solids, residue @ 180°C	369	±	17	31
Dissolved solids, sum	428	±	16	30
Specific conductance (micromhos per cm at 25°C)	760	±	31	48
pH	6.28[4]	±	0.14	5.50[4]
Temperature (°C)	21.4	±	0.7	17
Turbidity (mg/l as SiO_2)	0.49	±	0.44	0
Dissolved oxygen	6.6	±	0.5	0
Chemical oxygen	22	±	1	0

[1]All data except pH, specific conductance, and temperature reported in mg/l.

[2]Based on the analysis of 10 daily composite samples collected during RW 10, part 1, Nov. 1 to Nov. 11, 1971.

(Footnotes continued on following page.)

Table 2. Concentration of Iron Recovered During the Initial Redevelopment Surges

Elapsed time between time pumping was started and sample was collected, in minutes	Dissolved iron (mg/l)	Total iron (mg/l)	Remarks
Surge 1 (300 gpm)			Immediately after recharge (November 11, 1971)
[1]14	0.82	21	
90	0.10	0.19	
Surge 2 (300 gpm)			One month after recharge ended (December 15, 1971)
[1]14	36	40	
60	15	15	
Surge 3 (950 gpm)			Seven weeks after recharge ended (January 5, 1972)
[1]5	21	44	
60	6.6	6.7	
Surge 4 (950 gpm)			Seven weeks after recharge ended (January 5, 1972)
[1]5	8.0	8.5	
10	- -	7.0	
Surge 5 (950 gpm)			Seven weeks after recharge ended (January 5, 1972)
[1]5	5.75	5.75	
10	- -	5.25	

[1]Time required to evacuate water standing in well casing at the prevailing pumping rate. Prevailing pumping rates are given in parentheses.

Footnotes for Table 1 (continued):

[3]Based on the analysis of samples collected either before or after the beginning of RW 10, part 1.

[4]Calculated after converting pH value from logarithmic form to exponential form.

Table 3. Iron Mass-Balance Data for Tests RW 9 to RW 12, April 1971 to June 1972[1]

Test	Date of Test	Injected Water[2]			Water From Well N7886, Peak Phase[3]	Water From Redevelopment Surges[4]
		Total Iron	Dissolved Iron	Colloidal Iron		
RW 9	4/21-5/7/71	5.8	3.5	2.3	Insufficient data	Insufficient data
RW 10, part 1	11/1-11/11/71	8.7	3.2	5.5	21	5.8
RW 10, part 2	1/10-1/14/72	0.6	0.7	0.0	2.1	0.7
RW 11	3/6-3/16/72	1.8	1.4	0.4	14	0.1
RW 12	6/5-6/15/72	2.6	2.0	0.6	5.0	0.2

[1]Values in 10^6 mg.

[2]Total iron is the average total iron concentration for the injection period multiplied by volume of injected water. Dissolved iron is the average dissolved-iron concentration for the injection period multiplied by volume of injected water. Colloidal iron is the total iron minus dissolved iron.

[3]Peak phase is the area under the iron curve from the time of arrival of the injected water at the 20-ft observation well N7886 to the end of the injection test. In RW 10, part 1, for instance, the iron content was calculated by multiplying the iron concentration from Figure 1A, at hourly intervals, by the volume of water injected per hour from $t = 17.5$ to $t = 239$ hours.

[4]Redevelopment surges represent the area under the iron-redevelopment curve or curves. The iron content was calculated by multiplying the iron concentration at 2-minute intervals on the curve by the redevelopment rate.

Table 4. Effect of Different Treatments of the Reclaimed Water on the Maximum Iron Concentration and Iron Content of the Observed Iron Peak

Recharge Test	Treatment	Maximum Iron Concentration (mg/l)	Iron Content of Peak (mg)
RW 10, part 1	Carbon filters bypassed	3.05	21×10^6
RW 10, part 2	Unchlorinated	1.20	2.1×10^6
RW 11	Dechlorinated using sodium thiosulfate	2.60	14×10^6
RW 12	pH raised to 8 using sodium hydroxide	1.37	5×10^6
City water test 6	No treatment	No peak	0

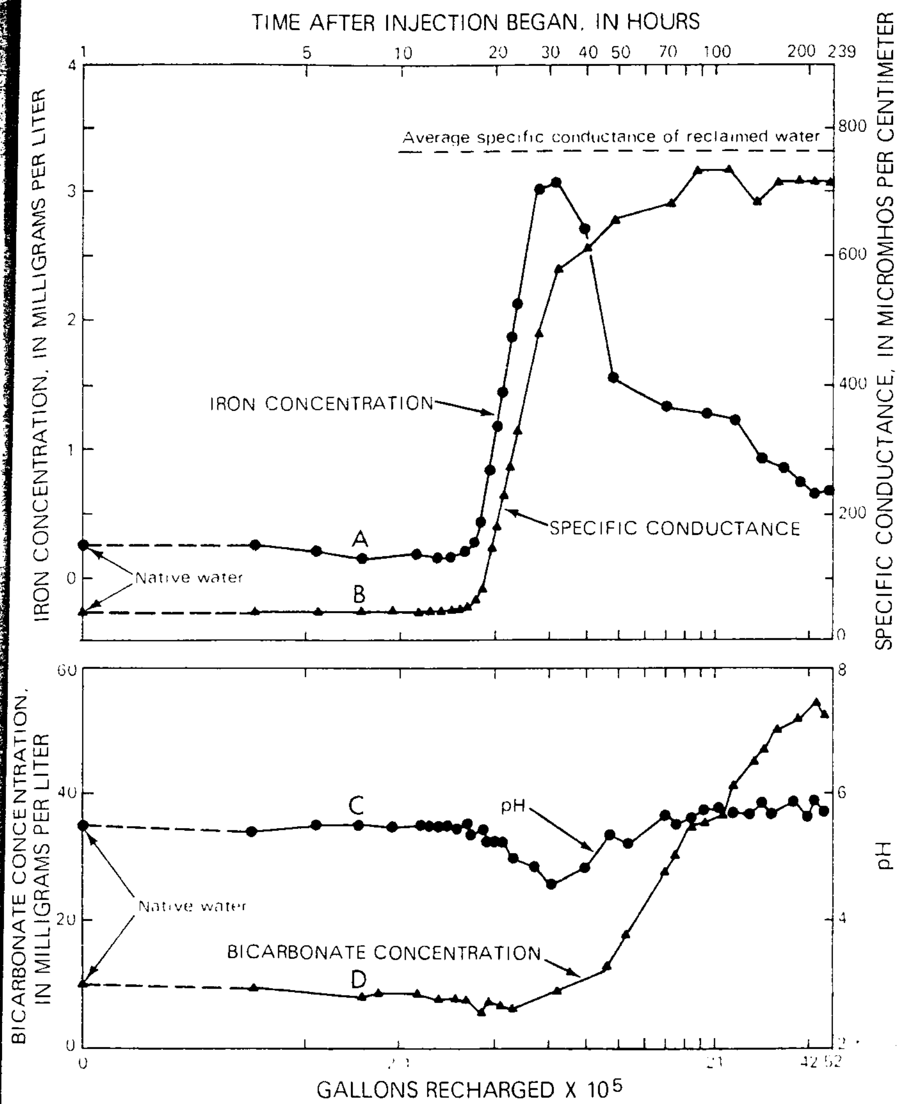

FIG. 1--Change in total iron concentration, specific conductance, bicarbonate concentraion, and pH of water from observation well N7886, 20 ft from recharge well during test RW 10, part 1.

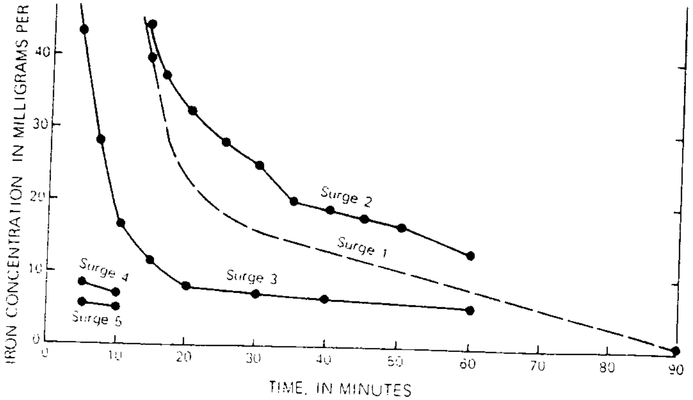

FIG. 2--Change in iron concentration of repumped water during initial redevelopment surges of recharge well.

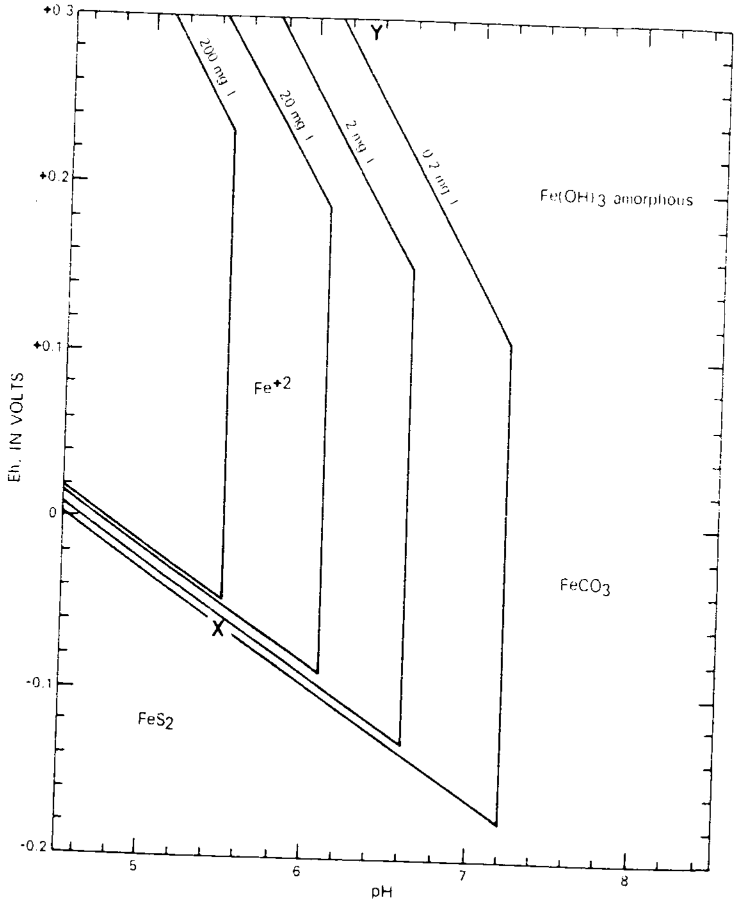

FIG. 3--Graph showing equilibrium solubility of iron in relation to pH and redox potential in reclaimed water, I = 0.009, $C_T = 10^{-2.64}$, and $S_T = 10^{-2.84}$

RADIOACTIVE- AND CHEMICAL-WASTE TRANSPORT IN GROUNDWATER AT NATIONAL REACTOR TESTING STATION, IDAHO: 20-YEAR CASE HISTORY AND DIGITAL MODEL[1]

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[2]U.S. Geological Survey, National Reactor Testing Station.

The hydrologic data upon which the 20-year case history is based derived principally from the efforts of many U.S. Geological Survey investigators including A. E. Peckham, P. H. Jones, and D. A. Morris. R. G. Jensen of the U.S. Geological Survey staff has collected and compiled much of the vital hydrologic and hydrochemical data. C. V. Theis (USGS, Albuquerque, New Mexico) has provided much encouragement and helpful advice over many phases of these studies. Most of the development of the hydrologic and waste-transport digital model was done by J. H. Bredehoeft (USGS, Denver, Colorado). He provided invaluable assistance in adapting this modeling technique to the NRTS problem.

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ABSTRACT Industrial and low-level radioactive liquid wastes at the National Reactor Testing Station (NRTS) in Idaho have been discharged to the Snake River Plain aquifer since 1952. The movement and distribution of these wastes have been monitored. The aquifer is large and has a high transmissivity. The total waste discharge to the aquifer at NRTS, which has averaged about 1×10^9 gal (4×10^9 l) per year, contained small quantities of tritium, Sr^{90}, Cs^{137}, Co^{60}, chloride, hexavalent chromium, various acids and bases, and heat. Tritium and chloride have dispersed over a 15-sq mi (39 km^2) area of the aquifer in low but detectable concentrations and have migrated as much as 5 mi (8 km) downgradient from discharge points. A remarkable degree of lateral dispersion has diluted and spread the waste products rapidly. The movement of cationic waste solutes, particularly Sr^{90} and Cs^{137}, has been significantly retarded owing to sorption phenomena, principally ion exchange. Cs^{137} has shown detectable migration in the aquifer and the Sr^{90} has migrated only about 1.5 mi (2 km) from a discharge well. The Sr^{90} plume covers an area of only 1.5 sq mi (4 km^2) of the aquifer.

Digital modeling techniques have been applied successfully to the analysis of this complex waste-transport system by numerical solution of the coupled equations of groundwater motion and mass transport. The model includes the effects of convective transport, flow divergence, two-dimensional hydraulic dispersion, radioactive decay, and reversible sorption. The 20-year transport and distribution history of waste chloride and tritium has been successfully simulated by the model. The less conservative cationic solutes have also been successfully modeled. The modeling results indicate that hydraulic dispersion (especially transverse) is a much more significant influence than has been suggested by earlier studies. The model may be used to project future waste-migration patterns for varied hydrologic and waste conditions.

INTRODUCTION

The National Reactor Testing Station (NRTS) occupies 894 sq mi (2,320 km^2) of gently rolling, semiarid land on the eastern Snake River Plain in southeast Idaho (Fig. 1). The NRTS is operated by the U.S. Atomic Energy Commission (AEC) and its contractors for the construction, operation, and testing of various types of nuclear reactors. It is one of the principal centers for the development of peaceful uses of atomic energy. The first NRTS reactor was finished in 1951 and, since then, 50 other reactors have been built.

The eastern Snake River Plain is a large graben or downwarped structural basin 12,000 sq mi (31,000 km^2) in area (Fig. 1). It has been filled to its present level with perhaps 5,000 ft (1,500 m) of thin basaltic lava flows and interbedded sediments. Nearly all of the eastern Snake River Plain is underlain by a vast groundwater reservoir known as the "Snake River Plain aquifer," which may contain more than 1 billion acre-ft (1,230 km^3) of water. The aquifer yields, and is recharged by, an average of about 6.5 million acre-ft (8 km^3) of water per year. The flow of groundwater in the aquifer is principally to the southwest (Fig. 1) at high velocities (generally 5-25 ft/day, or 1.5-8 m/day). Transmissivity of the aquifer is high, generally ranging from 1 million to 100 million gal/day per foot (1×10^4 to 1×10^6 m^2/day).

Several facilities at the NRTS generate and discharge low-level radioactive and dilute chemical liquid wastes to the subsurface by means of seepage ponds or wells. The two most significant waste-discharge facilities are the Test Reactor Area (TRA) and the Idaho Chemical Processing Plant (ICPP), as shown on Figure 2. Liquid-waste discharge has been continuous at these facilities since 1952. The U.S. Geological Survey has been monitoring and analyzing the fate of these wastes in the aquifer since that time. Material in this report, except for digital modeling, is covered in greater detail in a report by Robertson, Schoen, and Barraclough (1973).

WASTE DISPOSAL

Although several NRTS facilities discharge liquid wastes to the subsurface, only the two facilities with the most significant discharges are considered in this study--the Test Reactor Area (TRA) and the Idaho Chemical Processing Plant (ICPP). Discharges from these two facilities comprise 75 percent of the total volume of liquid waste discharged to the ground at NRTS and include 80 percent of the total chemical wastes and over 90 percent of the total radioactive waste.

Test Reactor Area

The TRA generates several different types of liquid waste and uses four different disposal systems. Low-level radioactive wastes are discharged to three interconnected seepage ponds and allowed to percolate to the water table 450 ft (137 m) below the land surface. Corrosive chemical wastes (nonradioactive) are discharged to a separate seepage pond. Nonradioactive cooling-tower blowdown wastes are discharged directly into the aquifer through a deep disposal well. The fourth disposal system is a seepage pond for sanitary wastes, which are not included in this study.

Waste water discharged to seepage ponds must percolate through a complex sequence of basalt and interbedded sediment layers before reaching the Snake River Plain aquifer. Fine-grained sediment layers generally have a much lower permeability than do the basalt layers and, consequently, tend to retard and perch downward-percolating waste water. One major perched water body about 1/2 sq mi (1.3 km^2) in area has developed beneath the TRA ponds on a sediment layer about 150 ft (46 m) deep. Other perched zones occur at other depths. It is estimated that the average travel time of water from the ponds to the aquifer is 1-3 years. Although the fate of the wastes between the ponds and the aquifer is an important and interesting problem, this study is limited only to the fate of wastes after they enter the Snake River Plain aquifer.

Radioactive-waste ponds--An average of about 200 million gal (7.6×10^8 l) of water per year have been discharged to the TRA radioactive-waste ponds since 1952. Since 1962, the water has contained an average of about 3,300 curies (Ci) per year of various activation and fission products, of which about 70 percent are short-lived products of little significance.

The significant longer-lived nuclides in the TRA waste are listed in Table 1. Of the above-listed isotopes, only tritium (as tritiated water) has entered the Snake River Plain aquifer in detectable quantities, because it is not affected by ion exchange or other sorption reactions. All the other nuclides are cationic and are removed from solution by sorption before reaching the water table.

Chemical waste pond--About 44 million gal (1.7×10^8 l) per year have been discharged to the TRA chemical waste-seepage pond since its construction in 1962. The wastes originate principally from ion-exchange system regenerations and, on the average, consist of 1,000 tons (9.1×10^5 kilograms) per year of sulfuric acid, 500 tons (4.5×10^5 kilograms) per year of sodium hydroxide, and 50 tons (4.5×10^4 kilograms) of sodium chloride.

Deep disposal well--A 1,300-ft-deep (395 m) disposal well has been used at TRA since 1964 for disposal of about 150 million gal (5.7×10^8 l) per year of nonradioactive waste water. The principal source of the waste is cooling-tower blowdown water. It generally contains about 1,200 mg/l of naturally occurring dissolved solids (about five times as much as

the natural groundwater). Prior to 1972 it also contained 1-2 mg/l of hexavalent chromium, used as a corrosion inhibitor. The chromium treatment has since been replaced by a phosphate treatment.

The well discharges directly into the aquifer, whose water level is about 450 ft (137 m) below the ground surface. Most of the discharge occurs between the depths of 500 and 700 ft (150 and 210 m). The well can accept 1,000 gallons per minute (gpm; 60 l/second) with no detectable head build-up.

Idaho Chemical Processing Plant

The Idaho Chemical Processing Plant (ICPP) currently discharges all its low-level or dilute effluents directly to the Snake River Plain aquifer through a 600-ft-deep (180 m) disposal well (Fig. 2). Nearly all radioactivity is removed from the effluents by distillation and ion exchange before discharge. Tritium (as tritiated water) is the principal waste isotope that is not removed by these treatments. In previous years, small amounts of other significant isotopes, such as Sr^{90} and Cs^{137}, were discharged also. Chemical wastes carried in the effluent consist primarily of sodium chloride, with some sulfates and other chemicals, plus small amounts of heat.

The water table is about 450 ft (137 m) below land surface at the ICPP. The disposal well extends about 150 ft (46 m) below the water table. The average annual discharge to the well has been about 300 million gal (1.1×10^9 l) per year. Table 2 indicates the principal characteristics of the effluent going down the well.

Until 1967, small but significant quantities of liquid wastes were also discharged to a shallow seepage pit at ICPP; these are not included in the above data. Current and future discharge rates of Sr^{90}, Cs^{137}, and other cationic radionuclides will probably be lower than the previous average, owing to improvements in waste management and treatment processes.

WASTE DISTRIBUTION IN AQUIFER

During the 20 years of waste discharge at TRA and ICPP, periodic studies of the migration and distribution of various waste products in the Snake River Plain aquifer have been made. Some complex and interesting phenomena have been observed. These studies are based on samples obtained from about 45 observation wells near to, and downgradient from, points of discharge (Fig. 2).

Chloride

Perhaps the best tracer of waste behavior in the aquifer is ordinary chloride. Chloride (from sodium chloride) has been a continuous waste product at both TRA and ICPP. On a week-to-week basis, it has probably been discharged in more consistent and uniform quantities than any other product. Once in the aquifer, it is also relatively conservative--that is, free from chemical reactions, such as ion exchange or precipitation, which could complicate its behavior.

Natural (background) chloride concentration in the aquifer water ranges from 10 to 20 mg/l but generally averages near 12 mg/l in the TRA-ICPP vicinity. A value of 15 mg/l was used as a lower limit to indicate waste contamination.

The first year for which good areal chloride data were available was 1958. The interpreted distribution pattern for that year is shown on Figure 3A. Chloride distributions for two other representative years, 1969 and 1972, are also shown on Figure 3. These maps indicate a rather rapidly moving and dispersing chloride "plume." The degree of lateral dispersion is particularly remarkable.

Figure 3A suggests that ICPP waste chloride first arrived at the Central Facilities Area (CFA) production wells (CFA-1 and CFA-2) in 1958. This arrival time agrees with that of waste tritium also. That arrival time corresponds to an average velocity of 7 ft (2.1 m) per day between ICPP and CFA. However, this is not necessarily the average groundwater velocity for that region, because first arrivals are generally faster than the average velocity and, furthermore, CFA may not be directly downgradient from ICPP.

Mass balances have been calculated for chloride, comparing the quantity discharged to the quantity apparently present in the aquifer at different times, and agreements within 8 percent were obtained (Robertson, Schoen, and Barraclough, 1973).

The western half of the TRA part of the plume is poorly defined because of lack of observation wells. It probably has a wider extent than that shown on these maps.

Sodium

Sodium is discharged in quantities correlating with chloride, because their common source is sodium chloride. However, sodium has displayed a somewhat different behavior in the aquifer than has chloride.

Although the shape of the sodium plume and the degree of dispersion are similar to that of chloride, the sodium migration appears to have been retarded by sorption (principally ion exchange). Therefore, the ratio of sodium concentration to chloride concentration becomes smaller and smaller downgradient. For instance, in 1968, water from production well CFA-1, which is 2.5 mi (4 km) downgradient from the ICPP disposal well, contained 14 mg/l sodium, whereas the chloride concentration was 60 mg/l. A pure sodium chloride solution with 60 mg/l chloride should contain about 40 mg/l sodium.

Chromium

Hexavalent chromium (nonradioactive) was a waste product at TRA until 1972. From 1952 to 1963, the chromium wastes were discharged to the radioactive-waste seepage ponds. Since 1963, the chromate wastes have been discharged to the TRA deep-disposal well. Chromium wastes serve as a good tracer of the TRA waste plume in the aquifer because the ICPP does not discharge chromium waste. Typical TRA chromium plumes are shown on Figure 4. As with chloride, the west half of the chromium plume is poorly defined and probably covers a bigger area than is indicated on the map.

There appears to be no significant reduction of hexavalent chromium to lower oxidation states in the aquifer. Sorption does not appear to have a major effect on the chromate migration.

Heat

The temperature of the effluent discharged down the ICPP disposal well averages about 70°F (21°C), whereas the natural water temperature in the Snake River Plain aquifer is about 53°F (12°C). Although the 17°F (9°C) differential is relatively small, it has been enough to create a detectable plume of warmer water in the aquifer about 2 sq mi (5 km^2) in area. The amount of excess heat discharged has amounted to about 1.5×10^{14} calories.

A heat balance calculated for the system indicates that perhaps 20 percent of the waste heat has been conducted out of the aquifer (Robertson, Schoen, and Barraclough, 1973).

Tritium

Tritium (H^3), as tritiated water, is the most abundant waste radioisotope at the TRA and ICPP. Although it was not identified as a waste product until 1961, it has undoubtedly been discharged since 1952. It makes a very good waste tracer in groundwater because it is not subject to ion exchange or other interfering reactions. For this reason, plus its abundance and long period of discharges, waste tritium is as widespread in the aquifer as chloride and has a similar distribution pattern.

Tritium discharge to the aquifer has been much more variable than other waste products, such as sodium chloride. For example, in 1970, 75 curies of tritium was discharged into the ICPP well, whereas, in 1967, 860 curies was discharged. These variations, plus radioactive decay (half-life = 12.3 years), complicate the behavior of tritium in the aquifer compared to that of chloride.

The first year that waste-tritium distribution was mapped in the aquifer was 1961. This distribution is shown in Figure 5A; subsequent plumes are shown in Figures 5B and 5C, to represent the long-term changes in the plume.

Mass balances calculated for tritium indicate that a semi-equilibrium quantity of 13,000-14,000 curies has been maintained since 1961 in the aquifer waste plume, which occupies an area of about 15 sq mi (39 km^2). The decay rate for that quantity is approximately equal to the long-term average discharge rate so that the total amount in the aquifer is not increasing. In fact, the quantity in the aquifer has generally decreased since 1967 because of lower tritium discharge rates.

The background concentration of tritium in the aquifer water (from fallout and natural sources) averages less than 0.2 picocuries per milliliter (pCi/ml). The normal detection limit in NRTS tritium analyses is 2 pCi/ml, and is therefore used as the lower mapping limit. All the tritium concentrations in the aquifer are well below the recommended drinking-water guide line of 3,000 pCi/ml (U.S. Atomic Energy Commission, 1968).

Strontium-90 and Cesium-137

Since 1958, Sr^{90} and Cs^{137} have been discharged at the TRA and ICPP. Because of an additional waste ion-exchange system installed at ICPP in 1970, these two cationic products will no longer be discharged in significant quantities to the ICPP well.

The first extensive analysis of Sr^{90} in the aquifer was completed in 1964 and several analyses have been carried out in subsequent years, such as 1972 (Fig. 6). Variations in the interior concentrations of the

Sr^{90} plume are due to long-term variations in discharge rates.

The detection limit for Sr^{90} has ranged from 0.002 pCi/ml to 0.005 pCi/ml, so that a lower limit of 0.005 was used for mapping. It is obvious that the Sr^{90} plume is much smaller in area than the tritium or chloride plumes. The Sr^{90} plume occupies an area of about 1.5 sq mi (4 km^2). This is due mainly to the effects of sorption (principally ion exchange) that attenuate the concentrations and retard the movement of Sr^{90}. Although the Sr^{90} plume will continue to migrate in the aquifer, it will be at a very slow rate (compared to tritium or chloride), so that radioactive decay will be much more effective in reducing its concentration and limiting the ultimate extent of the plume migration.

Significant quantities of Sr^{90} have also been discharged to the TRA ponds, but it is essentially all sorbed out of solution on the sediment and basalt layers between the pond and the aquifer. Therefore, no detectable Sr^{90} from TRA has been observed in the aquifer. Mass balances calculated for Sr^{90} indicate that only 3 percent of the Sr^{90} discharged down the ICPP well is in the aquifer water; the other 97 percent is apparently sorbed.

Cs^{137} is even more affected by sorption than Sr^{90}. Although Cs^{137} and Sr^{90} are produced and discharged in nearly equal quantities, Cs^{137} has never been detected in any aquifer samples near TRA and ICPP. Under present waste-management policies, Cs^{137} probably will never show detectable migration in the aquifer.

Studies of waste behavior and aquifer characteristics indicate that the waste plumes shown on the following maps generally remain as relatively thin lenses in about the upper 250 ft (75 m) of the aquifer. Although there is considerable dispersion laterally and longitudinally, there appears to be little vertical migration because of low vertical permeability.

DIGITAL MODEL OF WASTE TRANSPORT

Although good documentation of changes in waste distribution in the aquifer over the 20-year period of disposal has been obtained, little quantitative understanding of the processes involved was acquired until recently. Little could be said of why the wastes behaved as they had or how they could be expected to behave in the future. The hydraulic and chemical processes were too complex for simple analysis.

In order to gain better understanding of the processes involved and to gain predictive capability, a digital model has been developed to simulate the Snake River Plain aquifer in the NRTS vicinity, and the significant influences on solute transport in the aquifer.

The modeling involves two principal phases--hydrology and solute transport. The hydrology phase of the model solves the following transient partial differential equation of groundwater flow (Equation 1) for a bounded, two-dimensional, one-layer aquifer by finite-difference techniques.

$$\nabla\cdot\bar{\bar{T}}\cdot\nabla h = S\frac{\delta h}{\delta t} + W(x,y,t), \qquad (1)$$

where $\nabla = \frac{\delta}{\delta x}\bar{i} + \frac{\delta}{\delta y}\bar{j}$,

$\bar{\bar{T}}$ = transmissivity tensor (Lt^{-1}),
h = hydraulic head (L),
S = storage coefficient (L°),
x,y = cartesian coordinates (L),
t = time, and
W = flux of a source or sink (Lt^{-1}).

The method uses the iterative-alternating-direction implicit scheme, described by Pinder and Bredehoeft (1968) and Bredehoeft and Pinder (1970). The hydrology model generates velocity vectors for groundwater flow at each finite grid point and each time step. These flow rates and directions are then used in the solute-transport segment of the model to simulate the migration of waste in time and two-dimensional space.

The solute-transport portion of the model solves the following transient partial differential equation (Equation 2) of solute movement in groundwater by the method of characteristics, as developed and described by Pinder and Cooper (1970) and by Bredehoeft and Pinder (1973). The model includes the effects of convective transport, velocity divergence, two-dimensional hydraulic dispersion, radioactive decay, and reversible sorption (instantaneous-equilibrium, linear-isotherm type).

$$\frac{\delta\varepsilon C_n}{\delta t} = \nabla\cdot\bar{\bar{D}}\cdot\nabla C_n - \nabla\cdot C_n\bar{q} - \varepsilon C_n\alpha_c\frac{\delta\rho}{\delta t} - QC_s - \lambda_n\varepsilon C_n - \frac{\delta(1-\varepsilon)N_n}{\delta t}, \qquad (2)$$

which converts to Equation 3, assuming instantaneous-equilibrium, reversible, linear adsorption isotherm:

$$\frac{\delta\varepsilon C_n}{\delta t} = (\nabla\cdot\bar{\bar{D}}\cdot\nabla C_n - \nabla\cdot C_n\bar{q} - \varepsilon C_n\alpha_c\frac{\delta\rho}{\delta t} - QC_s - \lambda\varepsilon C_n)\left(\frac{\varepsilon}{\varepsilon+K_d-K_d\varepsilon}\right), \qquad (3)$$

where $\bar{\bar{D}}$ = dispersion coefficient tensor (L^2t^{-1}),
C_n = concentration of a particular solute, n, (ML^{-3}),
C_s = concentration of solute n in a source or sink (ML^{-3}),
q = Darcy velocity vector of groundwater flow (Lt^{-1}),
α_c = compressibility of medium ($M^{-1}Lt^2$),
ρ = density of solution (ML^{-3}),
Q = production rate of source or sink (t^{-3}),
E = effective porosity (L),
λ_n = radioactive-decay constant for solute n (t^{-1}),
N_n = concentration of sorbed solute n on solid phase (ML^{-3}), and
K_d = sorption distribution coefficient for particular system (L).

A model of this type requires two forms of verification before it can be used with confidence. First, the hydraulic behavior of the model must correspond accurately to historical observations of the aquifer and, second, the historically observed behavior of waste in the aquifer must be adequately simulated by the model. The verifications must be accomplished using only realistic values for the various physical characteristics and parameters of the system, such as transmissivities and dispersivities.

Hydrology

Although several variations of a digital hydrologic model have been used for this study, one of the typical finite-difference grid systems is shown in Figure 7. This particular model is 36 by 39 nodes and covers an area of 2,600 sq mi (6,600 km^2). The model therefore encompasses less than 25 percent of the total aquifer area. It is assumed that the model area is large enough so that aquifer effects outside the area may be ignored. Each node is assigned transmissivity, initial head, storage coefficient, and recharge values. The outer-border nodes are assigned zero transmissivity. Most of the nodes next to the outer border are designated "constant head boundaries." The only significant surface-water influence on this portion of the aquifer is the Big Lost River, whose intermittent recharge is simulated by "recharge wells" at each node along the reach of the river.

Transmissivities for the model were obtained principally by flow-net and pumping-test analyses, and all other physical characteristics were based on the best field information available concerning the aquifer. Although the real aquifer system is probably more than 1,000 ft (300 m) thick, a thickness of 250 ft (76 m) was used in this study, on the basis of apparent layering effects in the aquifer. This upper layer appears to be somewhat separated from, and more permeable than, the lower zones. The model assumes that flow in the aquifer obeys Darcy's law, which, for this aquifer, may not be completely valid.

Verification--Verification of the hydrologic model was based on steady-state and transient performance of the model. In 1965, a good map of a near steady-state condition of the potentiometric surface of the aquifer was obtained (Fig. 8). The model was then given the corresponding head elevation around the constant head boundary nodes and allowed to bring itself to a steady head distribution. After some minor adjustments in transmissivity, the head pattern was nearly identical to that shown in Figure 8. The simulated potentiometric surface showed no significant deviations from the well data.

In 1965, the Big Lost River recharged the largest quantity of water on record to the aquifer. This large perturbation caused major transient changes in head distribution over the aquifer, which were fairly well documented (Barraclough et al., 1967). The amount of measured head increase from December 1964 to December 1965 is shown on Figure 9. When the same amount of recharge was artificially induced on the model, it generated corresponding head rises for the same period (Fig. 9). These results give an excellent transient verification of the model. Although it is not accurate in every detail, the general areal response of the model is excellent. With these results, the model was accepted as valid hydrologically, and verification procedures were applied to the waste-transport segment of the model.

Waste Transport

Although the waste-transport segment of the model depends directly on the hydrology segment, the area of concern is much smaller. Solute transport was modeled only for the area shown by the shaded grids in Figure 7. The input to the solute-transport model included initial concentrations of the solute of interest at each node, transverse and longitudinal dispersivities (or characteristic mixing lengths), location of waste sources and sinks, radioactive-decay factors, and distribution coefficients for sorption. Porosity of the aquifer is a critical factor

in the waste-transport model and was assumed to be 10 percent, based on the best available evidence. The most speculative of these inputs are the dispersivities and distribution coefficients. No effective way of measuring these coefficients in the field is presently practical because of the large-scale aquifer inhomogeneities. It is therefore invalid to extend ordinary laboratory measurements to field conditions. These parameters must therefore be estimated. The longitudinal and transverse dispersion coefficients are related to the dispersivities by $D_L = \alpha_L V$ and $D_T = \alpha_T V$, where V is the groundwater flow velocity, relative to the grains, D_L and D_T are the dispersion coefficients (longitudinal and transverse, respectively), α_L and α_T are dispersivities.

Chloride--Because of its consistent discharge and conservative nature, waste chloride was first used to verify the transport model. It is neither radioactive nor subject to significant sorption, so those factors were ignored. Therefore, the principal unknowns in modeling chloride transport in the aquifer are the dispersivities. Based on the results of Bredehoeft and Pinder (1973) and Webster et al. (1970), a longitudinal dispersivity (α_L) of 300 ft (91 m) was initially estimated. Little is known of large-scale transverse dispersivities (α_T) but, in the case of the Snake River Plain aquifer at NRTS, they appear to be larger than longitudinal dispersivities. Both numbers are much larger than indicated by small-scale laboratory studies. The reasons for this are not clear. The effects of varying the dispersivities in the model are shown in Figure 10, which reproduces ICPP chloride plumes generated by the model for 1968.

In comparing simulated chloride plumes to those actually mapped from field data for 1958 and 1969, the best fits were obtained using $\alpha_L = 300$ ft (91 m) and $\alpha_T = 450$ ft (137 m; Figs. 11, 12). Although the value for α_L appears reasonable, α_T would appear high compared to Bredehoeft and Pinder's (1973) value of 60 ft (18 m) in a limestone aquifer. However, the fact is that no one seems to know what is reasonable for α_T. It is therefore assumed for this study that the α_L and α_T values that give the best fits are the most reasonable, as they are at least in the acceptable order of magnitude.

Although not accurate in every detail, the model-simulated chloride distributions match well the historical observations (Figs. 11, 12). Part of the inaccuracy is due simply to the coarseness of the model grid. The largest area of discrepancy is the west-to-southwest side of the TRA part of the plume. This discrepancy is probably due to conservative interpretation of field data, because that area is not covered by observation wells. In other words, the simulated plume is probably more realistic than the one mapped from well data.

Tritium--Using the same dispersivities, good simulations were also made for tritium. Figure 13 demonstrates a typically good fit. Again, the match is not perfect in all details, but for the purposes of this analysis it is considered excellent--better than expected. Other good tritium matches were also obtained for different years. The radioactive-decay part of the model was shown to function properly.

Strontium-90--Verification of a sorption routine in the model was obtained with the simulation of Sr^{90} transport. Because of the small area of contamination by Sr^{90}, it is more difficult to obtain accurate simulation, owing to the coarseness in the model grid. The 1964 plume covers only an area of about 10 nodes. Nevertheless, good matches were obtained for 1964 and 1972 (Figs. 14, 15). The results were obtained using a sorption distribution coefficient (K_d) of 3.0. K_d is the ratio of the adsorbate equilibrium concentration on the solid-mineral phase (mg/cm^3) to its concentration in the solution (mg/cm^3) and is therefore dimensionless. The value of 3.0 for K_d was the estimate based on extrapolation of measured K_d's, porosity of the aquifer, and apparent behavior of Sr^{90} in the aquifer. K_d did not have to be adjusted arbitrarily to force a good fit.

The 1972 Sr^{90} pattern is more complex to match because of changes in discharge and local heterogeneities in the aquifer. If there were no sorption involved, the Sr^{90} plume would be much larger in area (similar to tritium) than is indicated.

The maps indicate no Sr^{90} plume from TRA. This is because no detectable quantities have yet penetrated the lithologic column between the disposal ponds and the aquifer.

Future Projections

After extensive verification of the model for chloride, tritium, and Sr^{90}, it appears to be a valid tool for estimating future projections of waste distribution in the aquifer. However, the future behavior of waste in the aquifer is highly dependent on future hydrologic and waste-disposal conditions, neither of which can be accurately predicted. Therefore, in making future projections, the hydrology and waste-disposal conditions must be assumed. This presents no particular technical problems, because projections can be simulated for any number of assumed future conditions.

However, the qualifying assumptions for each projection must be kept carefully in mind. The biggest hydrologic variable is recharge from the Big Lost River. In some years, it is highly significant and in other years, not. Several future projections up to the year 2000 have been made for chloride, tritium, and Sr^{90} under various conditions with good results. However, only one example will be used here to demonstrate the results.

There has been a tendency in recent years for lower waste-discharge rates to the aquifer and attempts are being made to discontinue all waste disposal to the aquifer within the next few years. For the assumption that waste-chloride discharges were discontinued in 1973, the simulated 1979 distribution of the previously discharged chloride is shown in Figure 16. Tritium displays a similar pattern. The chloride plume gradually grows bigger and more dilute as it moves and disperses down-gradient; however, because of decay, the tritium plume eventually declines in size and declines more rapidly in concentration than does chloride. Recharge from the Big Lost River tends to accelerate the southerly movement of the plume and divert it more to the southeast.

CONCLUSIONS

These studies have led to a well-documented case history of large-scale, long-term waste transport in a large groundwater system. There are relatively few histories of this type available, particularly involving several different waste products with different migration properties. The lateral or transverse dispersion process in this aquifer is considerably larger than expected from classic theory or laboratory studies. Longitudinal dispersivities appear to be similar to those found in some other large-scale dispersion studies.

Results of these studies indicate that recently developed digital modeling techniques have a very useful application to the study of complex heterogeneous systems of this type. The modeling has proved helpful in understanding the system hydrologically and chemically. This provides a good example of digital-model applications to actual large-scale problems of waste and water-resources management.

Although a basaltic aquifer with only a few interbedded sediment layers has relatively low ion-exchange capacity, it is apparent that sorption is a significant large-scale influence on the migration of cationic solutes such as Sr^{90}.

It should be reemphasized that models of this type are only as good as the input information that they are based upon. Successful modeling generally requires personnel who are intimately familiar with the hydrologic system.

The computer work was done on the AEC IBM model 360/75 in Idaho Falls. A typical 30-year waste-transport simulation requires 500K-600K bytes of core and about 5 minutes in the central processing unit.

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Table 1. Principal Waste Nuclides in TRA Radioactive Liquid Waste[1]

Waste Nuclide	Half-life (years)	Approximate Average Discharge, 1962 through 1972 (curies per year)	Average Concentration, 1962 through 1972 (picocuries per ml)	Approximate Discharge since 1952 (curies)
Tritium (H^3)	12.3	500	615	8,300
Strontium-90 (Sr^{90})	28.9	5	6	70
Cesium-137 (Cs^{137})	30	5	6	110
Cobalt-60 (Co^{60})	5.2	20	25	400

[1]Based on Data from Robertson, Schoen, and Barraclough (1973) and AEC files.

Table 2. Approximate Average Composition of Liquid Waste Effluent Discharged at ICPP Disposal Well[1]

Waste Product	Half-life	Approximate Average Annual Discharge, 1962 through 1972	Average Concentration in Effluent, 1962 through 1972	Approximate Total Discharge since 1952
Tritium (H^3)	12.3 yrs	505 Ci	430 pCi/ml	22,000 Ci
Strontium-90 (Sr^{90})	28.9 yrs	4 Ci	3 pCi/ml	18 Ci
Cesium-137 (Cs^{137})	30 yrs	4 Ci	3 pCi/ml	18 Ci
Zirconium-niobium-95 (Zr^{95}-Nb^{95})	65 days	2 Ci	1.5 pCi/ml	62 Ci
Cerium-144 (Ce^{144})	284 days	1 Ci	1.5 pCi/ml	96 Ci
Ruthenium-rubidium-106 (Rh^{106}-Ru^{106})	368 days	1 Ci	1 pCi/ml	23 Ci
Total radioactivity	----	520 Ci	440 pCi/ml	23,000 Ci
Sodium chloride ($NaCl$)		390 tons	160 mg/l Na 245 mg/l Cl	7,800 tons
Sulfuric acid (H_2SO_4)		40 tons	45 mg/l sulfate	800 tons
Sodium hydroxide ($NaOH$)		14 tons		280 tons
Dissolved solids			600 mg/l	
Temperature			70°F (21°C)	

[1]Based on Data from Robertson, Schoen, and Barraclough (1973) and AEC files.

Note: Recent improvements in ICPP waste treatments essentially have eliminated subsurface discharge of all radioisotopes except tritium (for instance, the 1971 and 1972 Sr^{90} discharges were less than 1 curie). There are future plans to reduce greatly or to eliminate discharge of tritium and chemical wastes also.

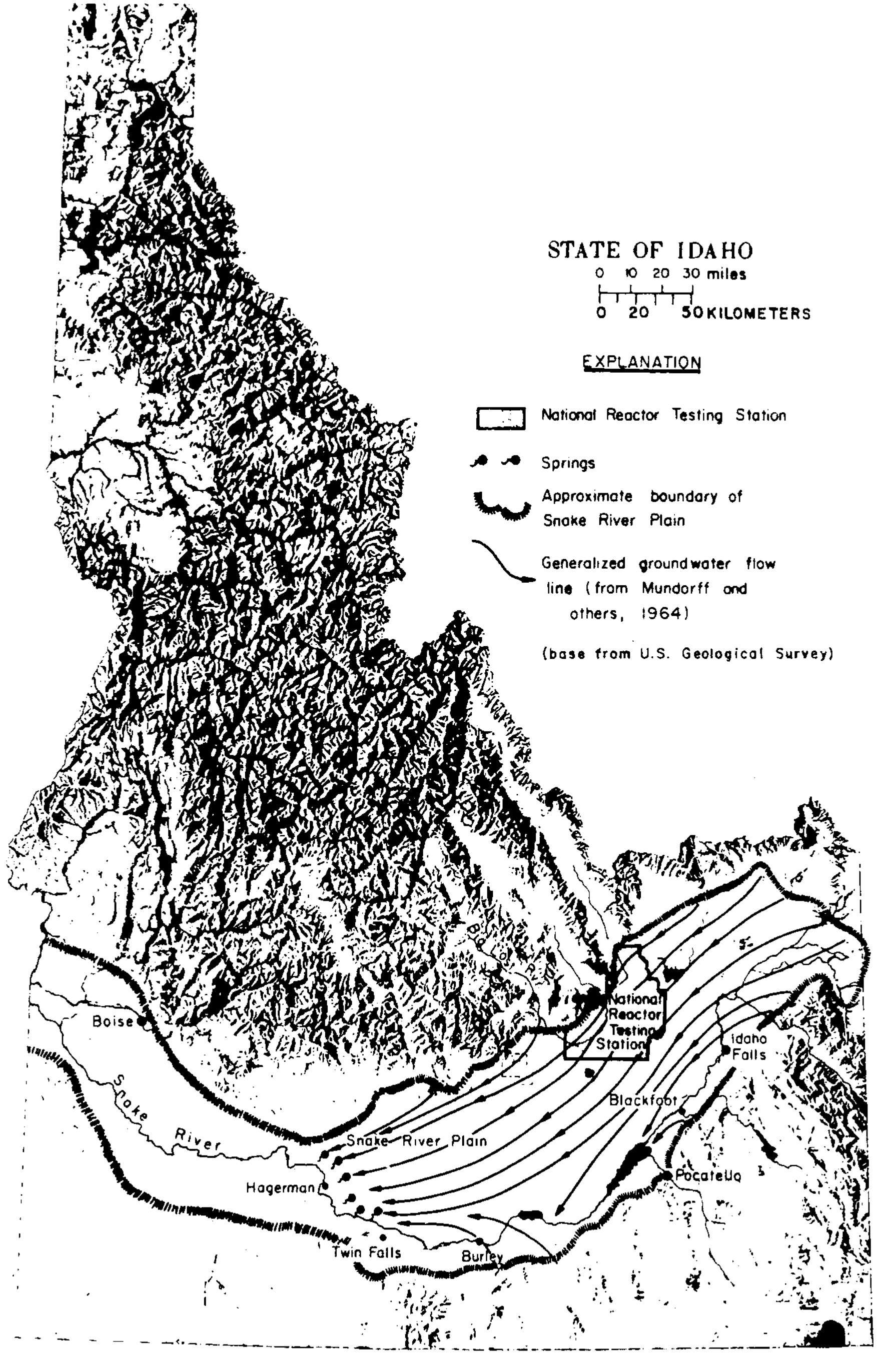


FIG. 1--Relief map of Idaho showing location of the NRTS, Snake River Plain, and inferred groundwater flow lines of Snake River Plain aquifer.

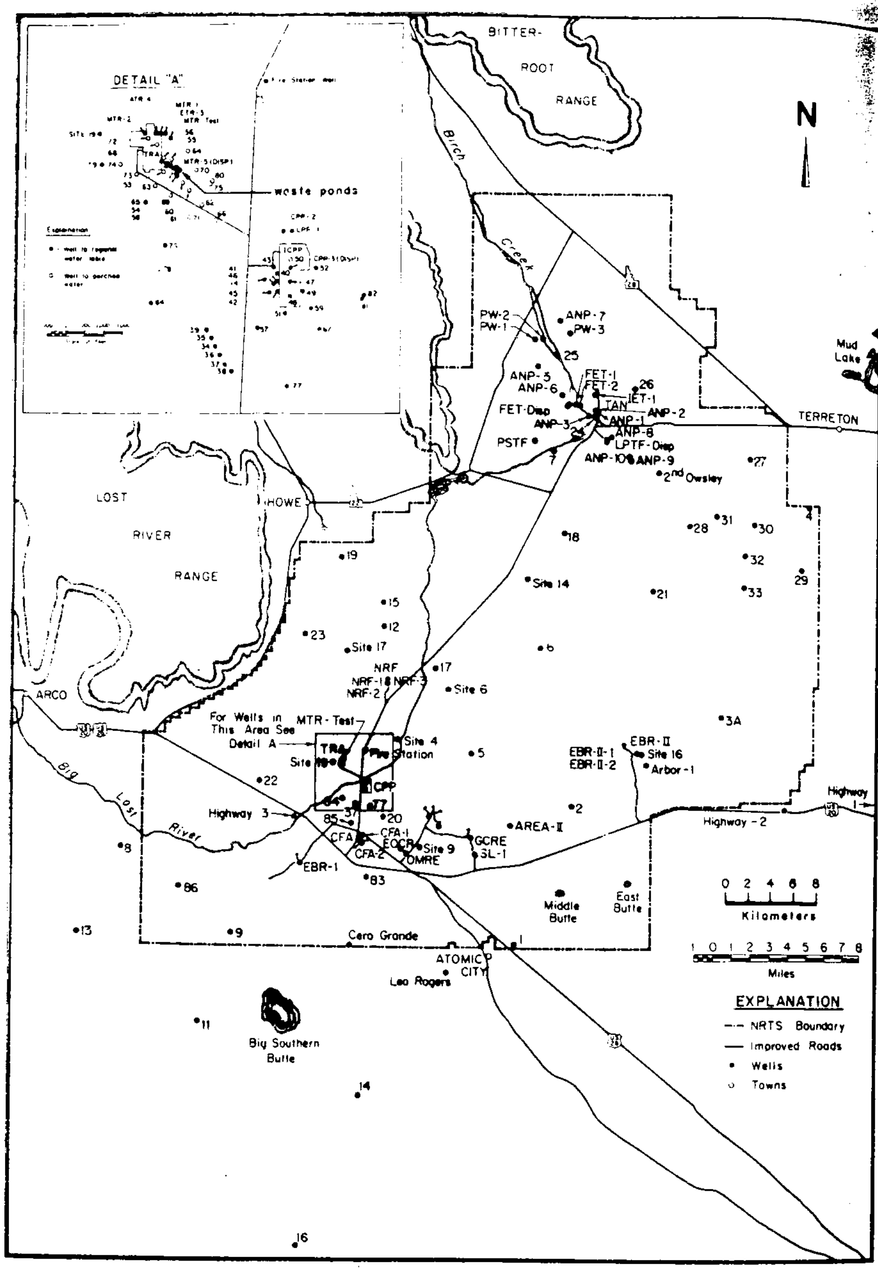
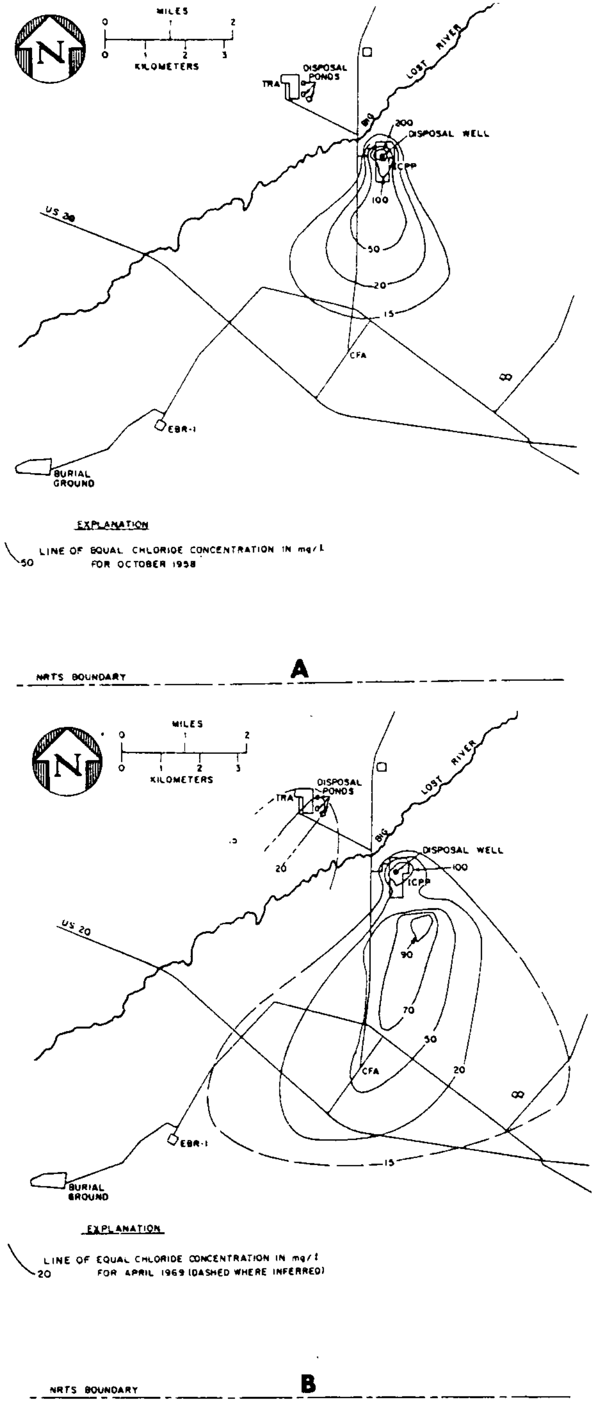


FIG. 2--Map of NRTS vicinity showing numbers and locations of NRTS production and observation wells.



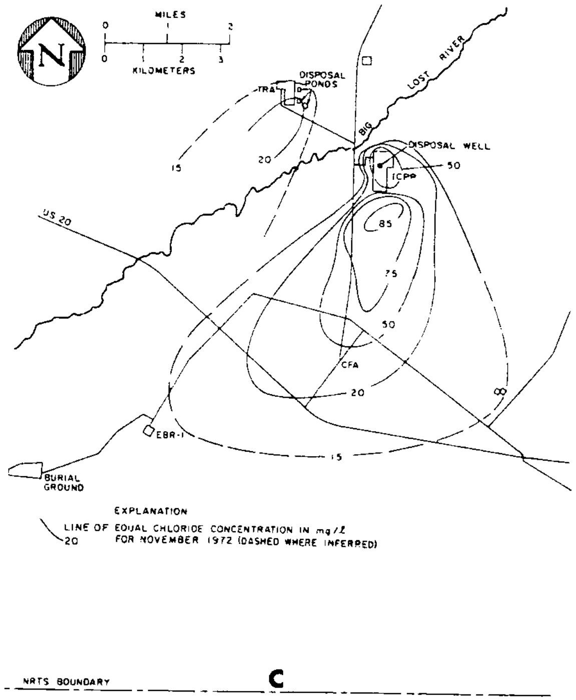


FIG. 3--Maps of ICPP-TRA vicinity showing distribution of waste chloride in Snake River Plain aquifer water in 1958 (A), 1969 (B), and 1972 (C; parts A and B on preceding page).

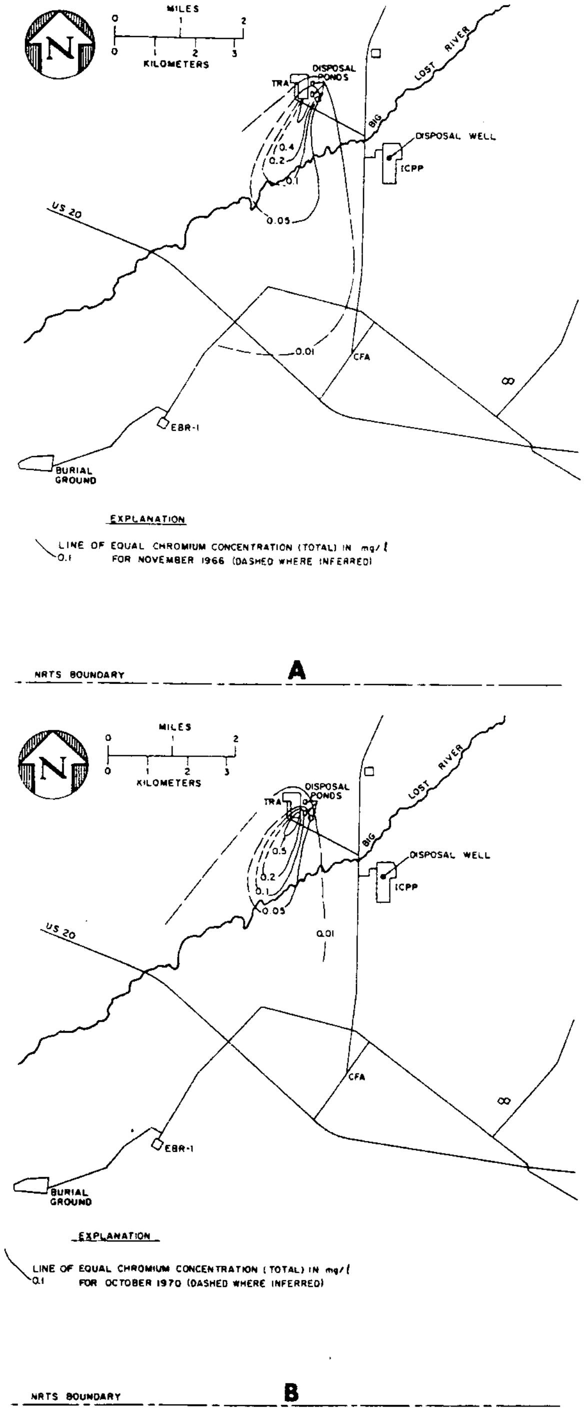
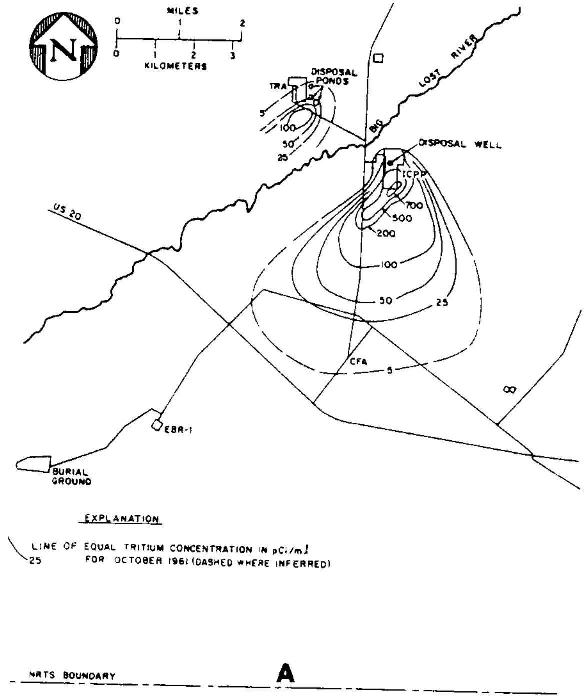
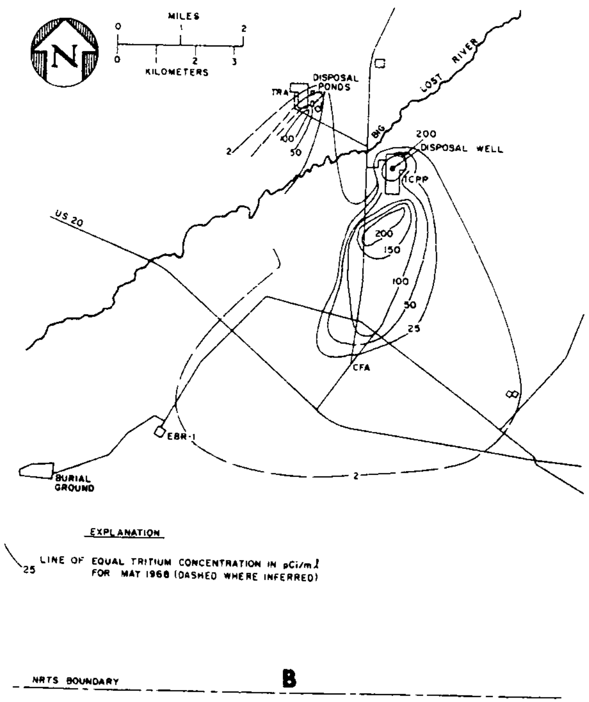


FIG. 4--Maps of ICPP-TRA vicinity showing distribution of waste chromium in Snake River Plain aquifer water in 1966 (A), and 1970 (B).





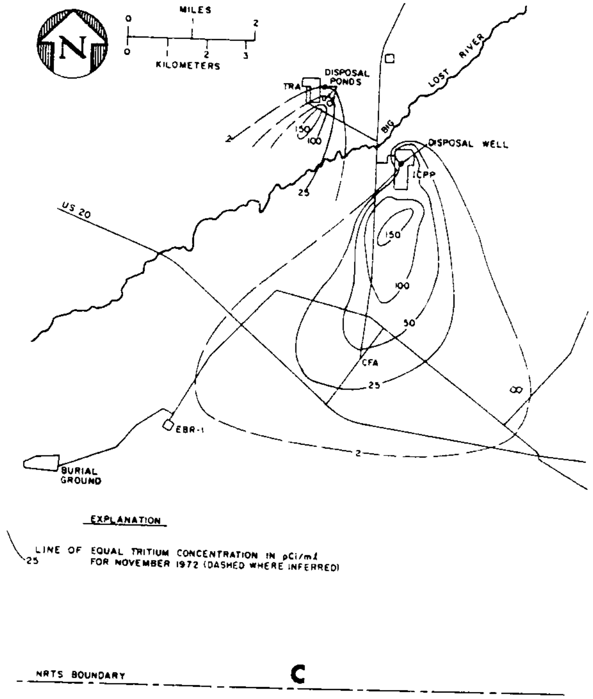


FIG. 5--Maps of ICPP-TRA vicinity showing distribution of tritium in Snake River Plain aquifer water in 1961 (A), 1968 (B), and 1972(C; parts A and B on preceding page).

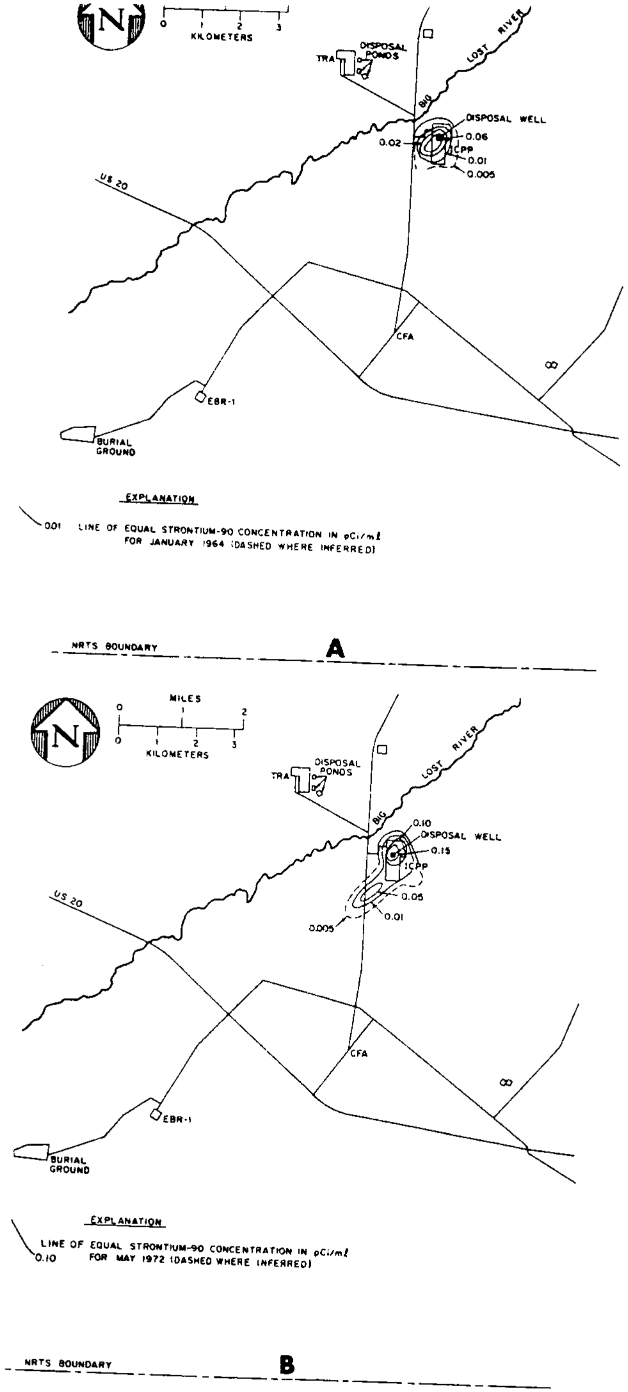


FIG. 6--Maps of ICPP-TRA vicinity showing distribution of Sr^{90} in Snake River Plain aquifer water in 1964 (A), and 1972 (B).

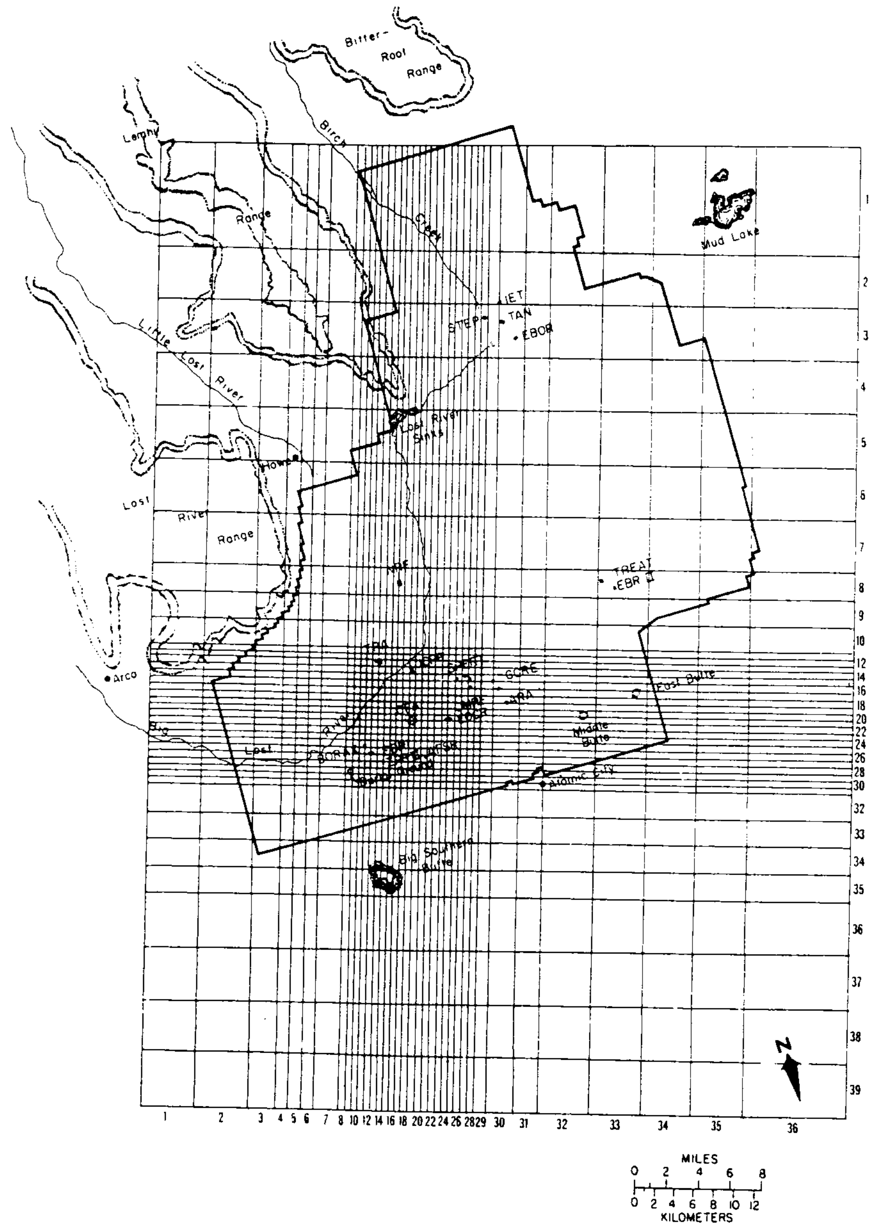


FIG. 7--Finite-difference grid map for one version of hydrology and solute transport model. Solute transport module includes only shaded part (smallest nodes).

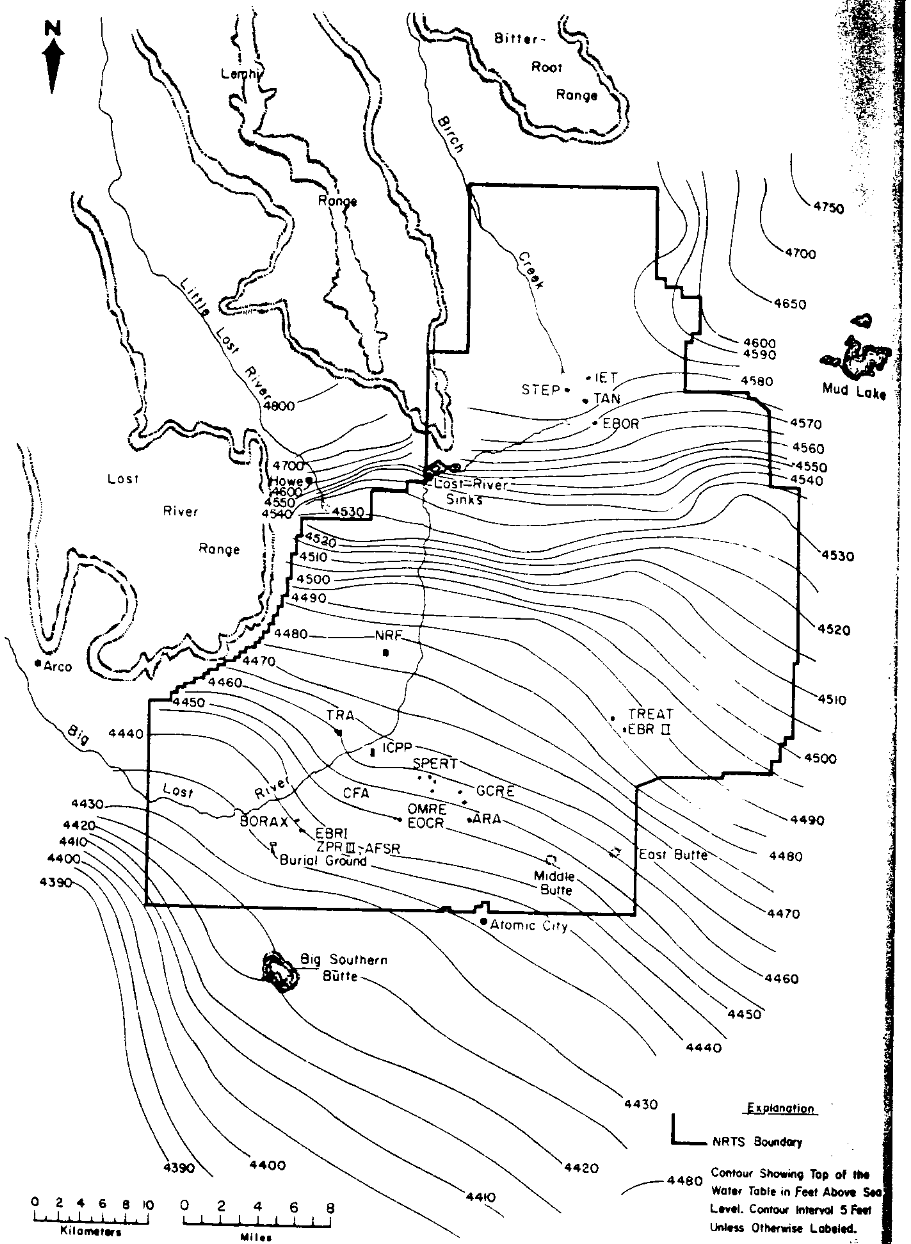


FIG. 8--Map of NRTS and vicinity showing contours of regional water table, May through June 1965, upon which steady-state verification of hydrologic model was made.

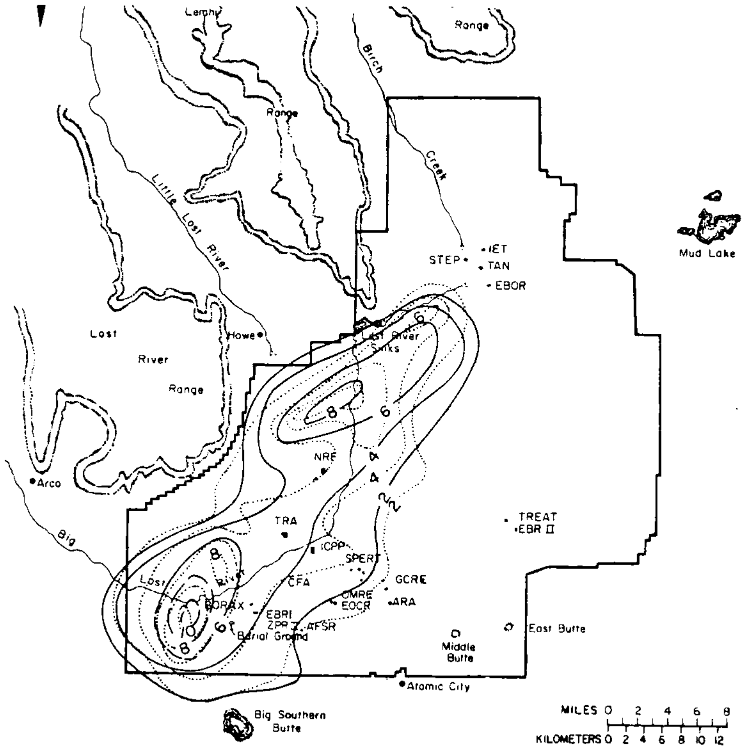


FIG. 9--Water-level rise in Snake River Plain aquifer from December 1964 to December 1965, resulting from Big Lost River recharge. Solid-line contours based on well measurements (dashed where inferred); dotted-line contours based on digital model simulation. Contour interval 2 ft.

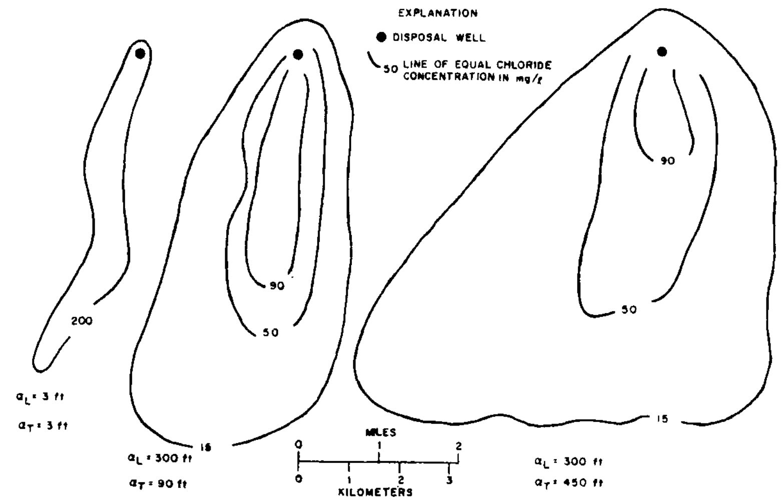


FIG. 10--Model-simulated waste-chloride plumes from ICPP for year 1968, showing effects of varying dispersivities γ_L and γ_T. Chloride concentrations in mg/l.

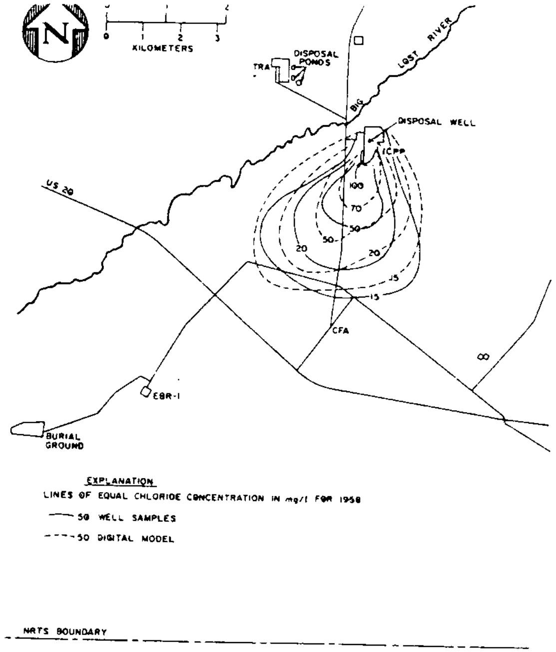


FIG. 11--Comparison of ICPP-TRA waste-chloride plumes in Snake River Plain aquifer for 1958, based on well-sample data and computer model.

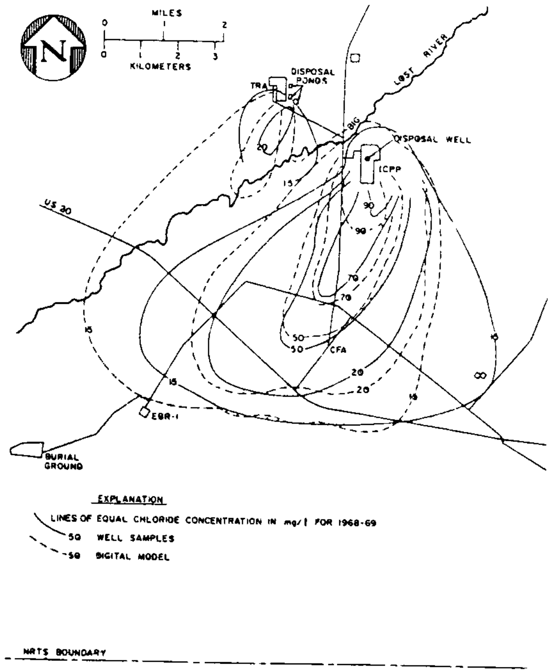


FIG. 12--Comparison of ICPP-TRA waste-chloride plumes in Snake River Plain aquifer for 1968-1969, based on well-sample data and computer model.

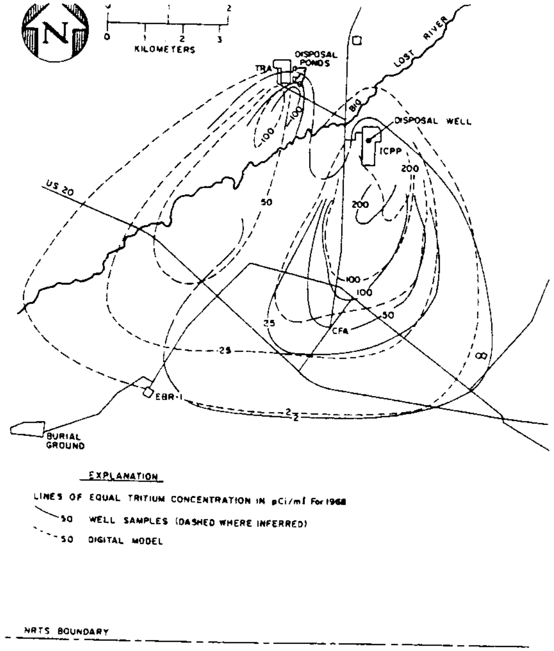


FIG. 13--Comparison of TRA-ICPP waste-tritium plumes in Snake River Plain aquifer for 1968, based on well-sample data and computer model.

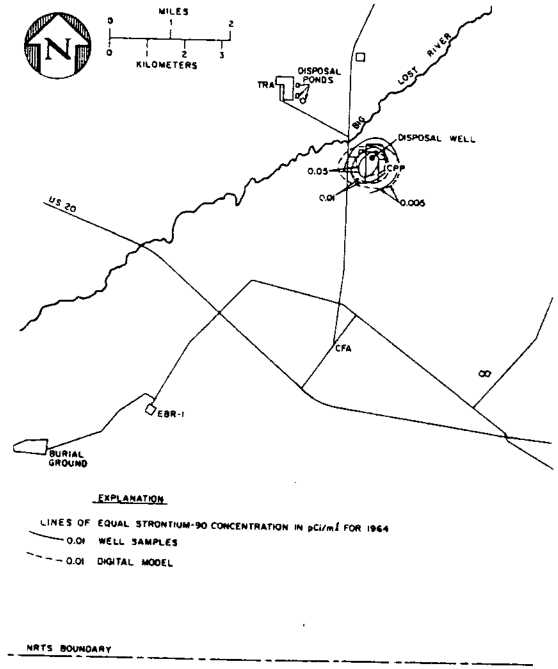


FIG. 14--Comparison of ICPP waste Sr^{90} plumes in Snake River Plain aquifer for 1964, based on well-sample data and computer model.

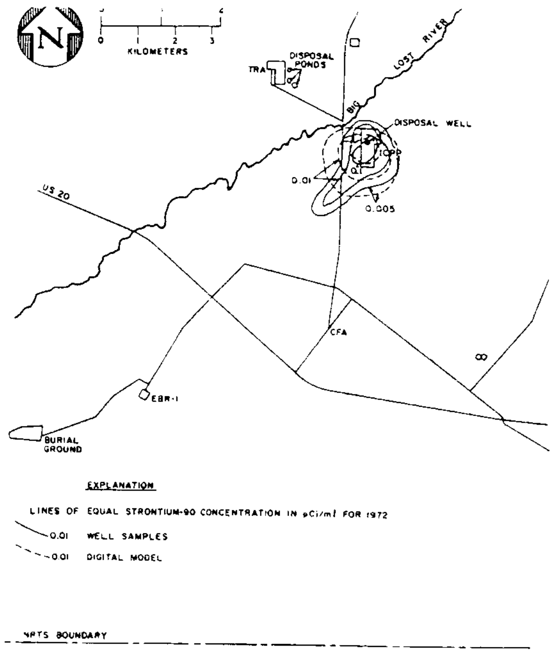


FIG. 15--Comparison of ICPP waste Sr^{90} plumes in Snake River Plain aquifer for 1972, based on well-sample data and computer model.

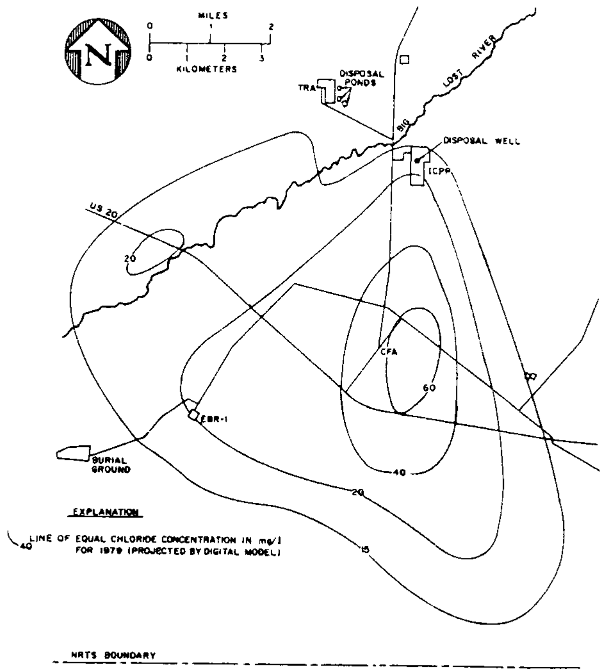


FIG. 16--Model-projected chloride plume in Snake River Plain aquifer for year 1979 assuming disposal ceases in 1973 and Big Lost River recharges aquifer during odd-numbered years.

MODIFICATION OF ARTIFICIALLY RECHARGED WATER IN SWITZERLAND[1]

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ABSTRACT Performance data from certain artificial-recharge plants, where Pleistocene gravels are artificially replenished by polluted river water and where recharge capacities are from 1 to 25 million gal/day, demonstrate the importance of retention time and oxygen balance in highly permeable aquifers. The retention time of the individual water particles is considerably varied and depends on hydraulic dispersion, which can be proved by tracer investigations. The oxygen content of the artificially recharged groundwaters depends on the oxygen concentration and on the biochemical oxygen demand of the raw water. From these data the oxygen balance of a groundwater can be evaluated. Insofar as the oxygen balance is concerned, there are no significant differences between a percolative and a direct infiltration, probably because significant quantities of oxygen gas are no longer present after the first percolative seepage of a polluted raw water in an unsaturated zone above the water table. Poor oxygen-balance systems must be improved by different processes or raw-water pretreatment.

After the infiltration, the aerobic decomposition of organic matter and the elimination of bacteria generally require short flow times. The equalized temperature of artificially recharged groundwater is another important advantage over mechanical and chemical treatment of river water.

High oxygen content and low biochemical oxygen demand of the raw water, together with long retention times, are the most important factors for the modification of river water to a wholesome drinking water by artificial recharge.

[1]Manuscript received, June 14, 1973.

INTRODUCTION

Artificial replenishment of groundwater by the infiltration of river water is being practiced at various localities in Switzerland. A few examples should serve to demonstrate the quality changes that river water undergoes after infiltration into its underground course. The aquifers described here consist of Pleistocene gravels with high permeabilities and standard coefficients ranging between K = 2,000 and 20,000 gal/day/ft^2 (K = 10^{-3} to 10^{-2} m/sec). The quality of the artificially recharged water is governed mainly by retention time and oxygen balance.

RETENTION TIME

Between a recharge plant and a water-intake well, the retention times of the individual water particles are highly variable. The resulting hydraulic dispersion of the infiltrated water can be demonstrated by tracers.

In Example A, presented in Figures 1-3, the groundwater flows in a Pleistocene river channel. The main natural recharge of this groundwater takes place as a result of infiltration from the river. Additional groundwater occurs independently at a higher level on the right side of the old river channel (Fig. 1). At the groundwater divide, the main flow is additionally recharged by artificial infiltration of river water into seepage basins. The weekly average infiltration of artificially recharged water is 1.0 mgd (43 l/sec). To this infiltrated water was added, in a few hours time, a sodium chloride brine containing about 10 tons of chloride. For a period of 28 weeks, the course of the salt tracer was observed in 19 intake and observation wells situated downstream from the infiltration basins in a stretch of 1.75 mi (2,820 m). Salt brine is one of the substances most suitable for tracing for such long distances and time periods.

The chloride was detected along the entire investigated stretch of 9,250 ft (2,820 m) though it was restricted to the right side of the groundwater flow (we had introduced the tracer on this border of the flow). As the travel-time diagram of the chloride propagation (Fig. 2) shows, near the site of the tracer introduction the velocities of the individual water particles have a strong scattering. Up to a distance of 180 ft (55 m), the fastest water particles traveled at a rate of about 550 times that of the slowest detectable ones. The maximum velocity was 12.5 times higher than the median value.

The analysis of the changes in the tracer concentrations with time shows that, at greater distances from the recharge plant, the flow times

of the different particles generally follow a log-normal distribution (Fig. 3). At a distance of 1,350 ft (410 m), the highest measured velocity of water particles was still 6 times that of the median value. Against the median flow time of 22 days, the minimum flow time here was only 3.4 days.

It is basically the minimum, and not the average, flow velocity which should be considered in the hygienic assessment of water. For Example A, the mean values of velocity in the vicinity of the recharge plant were somewhat less than those at greater distances. There was, however, a decrease in the maximum velocities as the distance from the recharge plant increased. The following values of maximum velocities were determined:

0-180 ft	900 ft/day
180-1,350 ft	365 ft/day
1,350-4,725 ft	180 ft/day

It follows, therefore, that increased distances lead not only to longer average retention times, but also to the prevention of short circuits of water particles into locally extremely permeable layers.

BIOCHEMICAL DECOMPOSITION AND OXYGEN BALANCE

In percolative seepage through a filter column with a mixture of air and water, the decomposition of the organic and bacterial pollutants takes place predominantly in the upper layers. In the case of induced direct infiltration of river water, however, the decomposition also takes place under aerobic conditions during a short lateral flowage.

It was observed in Example A that, in the well 180 ft (55 m) distant, the average coliform count fell to 1/500 of the river-water value after the percolative infiltration and after a minimum flow time of only 5 hours and a mean flow time of 2.5 days (Fig. 4). In the wells 1,350 ft (410 m) apart, the coliform count determined after a minimum flow time of 3.4 days remained at less than one in 100 ml, which is the drinking-water standard. At the relatively short distance of 180 ft (55 m), the amount of the biochemical oxygen demand (BOD), representing the decomposable organic pollutants, was also distinctly reduced. The oxygen consumed from permanganate was relatively high, because resistant organic matter from chemical pulp-factory wastes was present in the river water; however, at 180 ft distance the oxygen content also had decreased considerably, because of the decomposition processes in the underground. Contrary to commonly expressed opinions, it is not possible that only the percolative seepage through a filter medium with a mixture of air and water prevents oxygen decline if the crude water is heavily contaminated by organic pollutants. During biochemical decomposition, the loss of oxygen in the pores of the soil is too great to be made up by the small amounts of oxygen released by diffusion.

If instead of percolative seepage a direct infiltration of river water to the groundwater is carried out, under aerobic conditions, a quick decomposition could be achieved even at small lateral flow distances. However, after the beginning of an artificial-recharge operation, when the groundwater rises and inundates dry gravels the biological activity in this natural filter is accelerated only at successive intervals of time with increasing length of operation. In Example B (Fig. 5), long recharge tests were undertaken. In two tests, one involving a direct infiltration in recharge wells and the second a percolative infiltration through basins, river water was infiltrated at a rate of 2.3 million gal/day (100 l/sec). Up to the next intake well, the water had a minimum retention time of 1 day and an average of 7 days. Here, within 7 days after the commencement of both the tests, there was a significant increase in the BOD, in bacterial count, and in the oxygen concentration (Fig. 5). The artificially recharged river water, rich in oxygen, had replaced the groundwater previously formed by natural infiltration. In those parts of the aquifer which were flooded 6 to 12 ft above the natural groundwater level during the recharging process, an active biological filter could be formed only with time. In such parts, the organic matter and the bacteria initially were insufficiently eliminated by the aerobic biochemical decomposition processes. Therefore, at the beginning the oxygen content remained high; it was reduced only successively, after longer periods of operation (Fig. 5). A corresponding gradual reduction in the bacterial count, as well as BOD, was observed. Optimal coliform results were obtained only after an operation time of 70 days. In the artificially recharged groundwater, a significant correlation between the oxygen content and the BOD can be demonstrated (Fig. 6). With the increasing decomposition of the organic matter, indicated by decrease of BOD, there was a marked decline in the oxygen content. Statistically, the relation between the observed values, in parts per million, can be expressed as BOD (5 days) = 0.164 O_2 - 0.40. The average oxygen balance (Table 1) confirms that the amount of oxygen which remains in the groundwater depends on the reduction of the BOD along the total stretch of flow.

A groundwater of high quality standards must have a high oxygen content. Oxygen content below about 5 ppm leads to insufficient formation of protective scales on the interior walls of iron pipes. Low oxygen concentrations in the groundwater also retard the elimination of bacteria, and of taste and odor. Absence of oxygen in a groundwater would even permit solution of iron and possibly of manganese also. Many examples of this kind are known from Switzerland. Therefore, before artificially recharging river water, it is necessary to improve poor oxygen-balance systems by different processes of raw-water pretreatment.

Example C (Fig. 7) is instructive in showing the effect of water pretreatment on the oxygen balance. In this case the groundwater was recharged till the middle of 1958 by induced direct infiltration, i.e., by bank-filtered river water. The percolative infiltration of the same polluted, untreated river water in ditches and with a vertical passage through about 50 ft (15 m) of gravel layer caused no substantial improvement of the oxygen balance. Higher oxygen contents have been present only since 1960. From that year to the present, the polluted river water has been brought to a higher degree of purity by rapid filtering before being infiltrated in an amount of 25 million gal/day.

Though groundwater obtained after subjection to all three infiltration types used in this example had a normative bacteriologic standard and was free of taste and odor, there were marked differences in oxygen balance. Even the thick, porous, and seemingly aerated layers above the groundwater table could not prevent oxygen depletion in the water charged with organic pollutants. This fact confirms the hypothesis that the purity of the water used for infiltration is more important in determining the end oxygen balance than is the difference between percolative and direct infiltration types.

TEMPERATURE

Equalization of temperature is one of the important advantages of the artificially recharged groundwater over mechanically and chemically treated river water. The seasonal temperature variations of the infiltration water are transmitted to the groundwater in a delayed manner not corresponding to flow times, because heat is exchanged from the water to the soil and vice versa. In Example A (Fig. 8), the extreme temperature values already had diminished considerably after a flow time of 2 1/2 days. After average flow times of 50 to 100 days, the amplitude of temperature variation in the groundwater produced by river-bank infiltration and by artificial recharge was about 10 times less than that in the river. In Example B, with an average flow time of 7 days, the maximum temperature in the groundwater was 5°C lower and occurred 2 months later than in the river. In Example C, the flow times toward different wells are in the order of only a few weeks. In contrast to the summer maximum of river temperatures, however, the maxima in the wells are observed with considerable delay, mostly in the months of October to January. This delay is advantageous, because water as cool as possible is desirable in midsummer. In winter, however, higher groundwater temperatures are preferred because they lower the risk of freezing of supply pipes.

PUBLIC HEALTH AND ECONOMY

In Switzerland the public water supply is obtained exclusively from groundwaters, springs, and lakes. Mechanically and chemically treated river water is utilized only in industry, and not as a source of supply of drinking water. Because many of the existing natural groundwater reserves of the country have already been depleted or are going to be used up in the near future, new artificial-recharge plants are under consideration in different regions. In such plants, preference will be given to the direct treatment of river water, because this method offers a greater degree of operational safety and yields a water which is hygienically better and possesses equalized temperatures.

In general, production of drinking water by artificial replenishment is also economical. The largest existing plant in Switzerland, with a capacity of 25 million gal/day (Example C) involved an investment of about 20 million Swiss francs (S. Fr.) or about $6 million. This facility includes a rapid filter plant for water pretreatment, infiltration ditches and basins, and intake wells. The water so obtained is sold to the water-distribution works at a rate of S. Fr. 0.14 per m^3, i.e., about 4 cents per ton. The water which is produced at this cost has properties similar to those of natural groundwater and, directly from the intake wells, meets the standards for drinking water. Moreover, it is free of odor and taste and has fairly constant temperatures throughout the year.

Table 1. Example B--Average Oxygen Balance

O_2 in artificially infiltrated river water		12.6 ppm
Total BOD (empirically 2.15 x determined BOD for 5 days):		
In the artificially infiltrated river water	9.7 ppm	
In the groundwater after 7 days of average flow time	0.5 ppm	
Reduction = oxygen consumed along the total flow stretch		9.2 ppm
O_2 in the groundwater after an average 7 days flow time; theoretical value = determined value		3.4 ppm

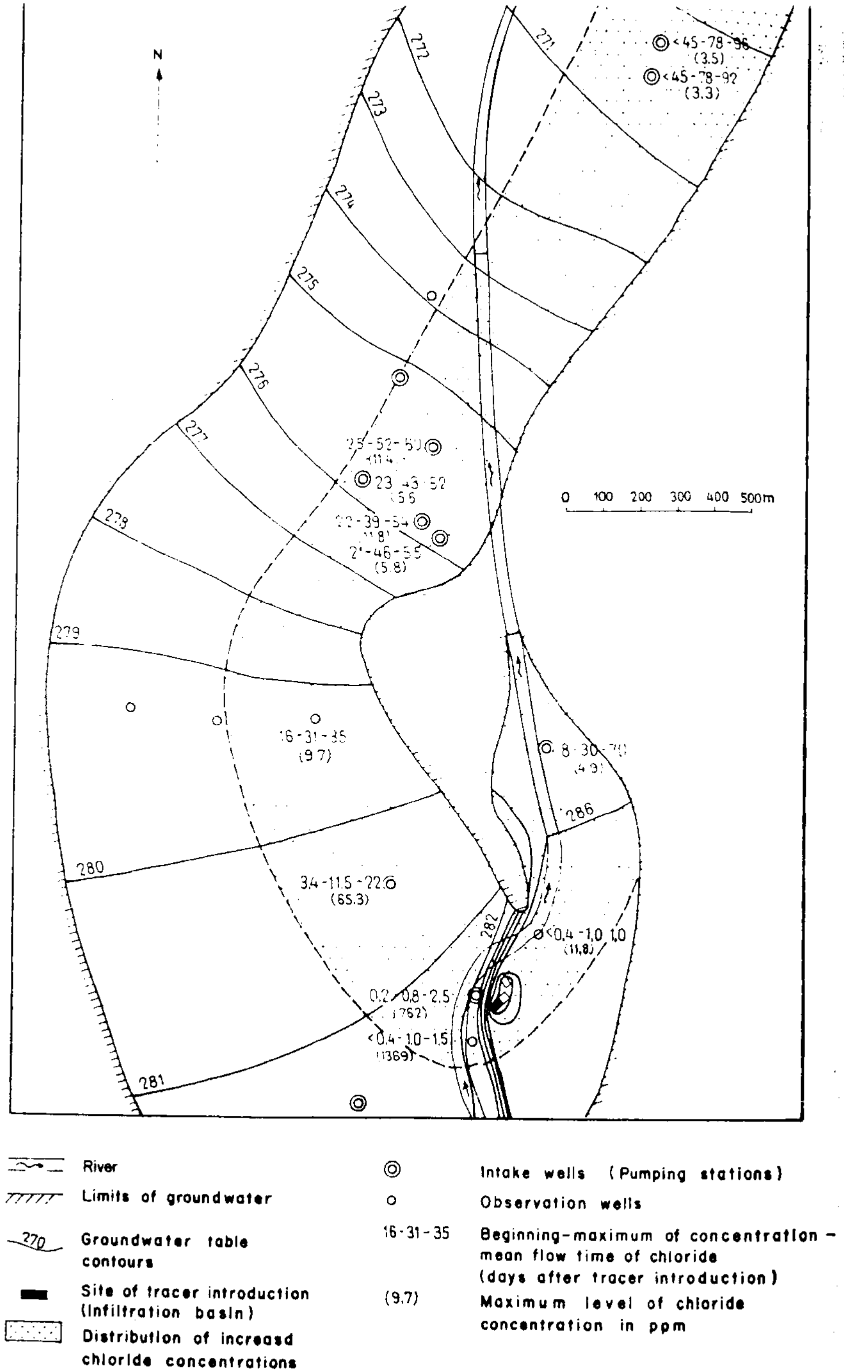

FIG. 1--Example A: hydrologic situation with recharge and well sites.

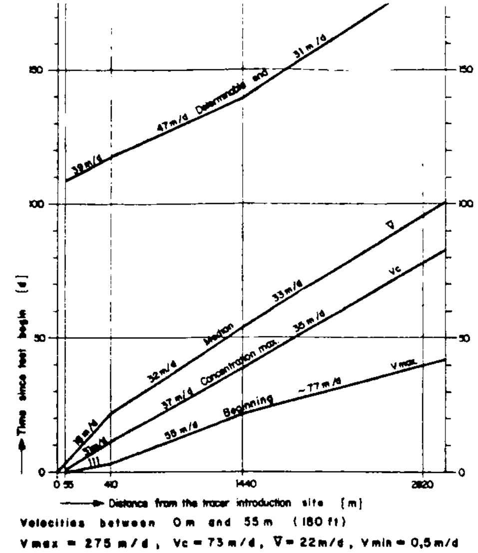

FIG. 2--Example A: travel-time graph of chloride tracer.

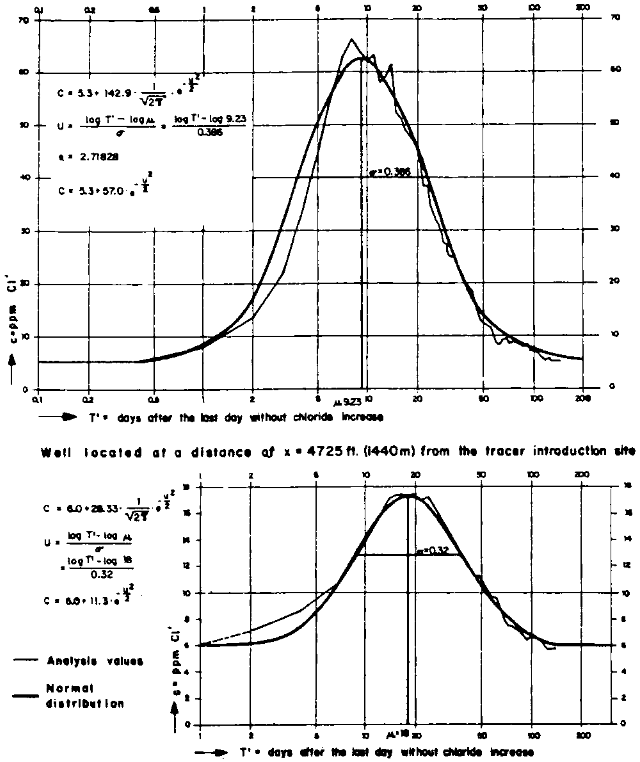

FIG. 3--Example A: log-normal distribution of concentration increase during chloride propagation.

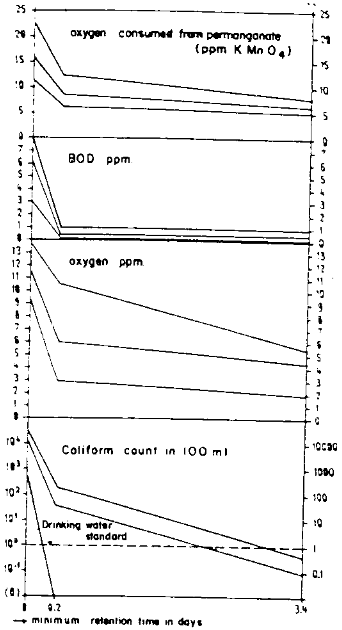

FIG. 4--Example A: chemical and bacteriologic modification of river water after infiltration. Maximum, average, and minimum values in 1 year.

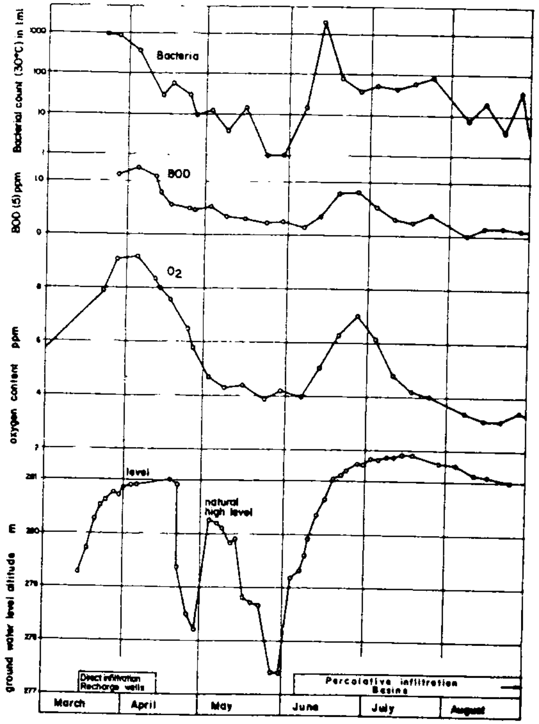

FIG. 5--Example B: quality change of groundwater during direct and percolative recharge operations.

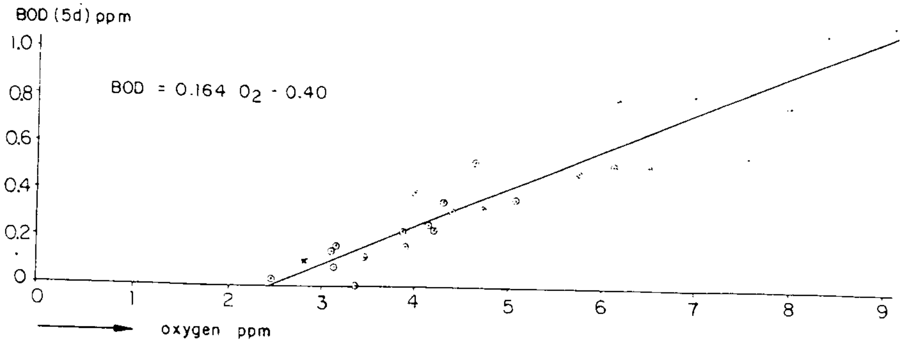

FIG. 6--Example B: correlation between oxygen content and BOD in artificially recharged groundwater after a mean retention time of 7 days.

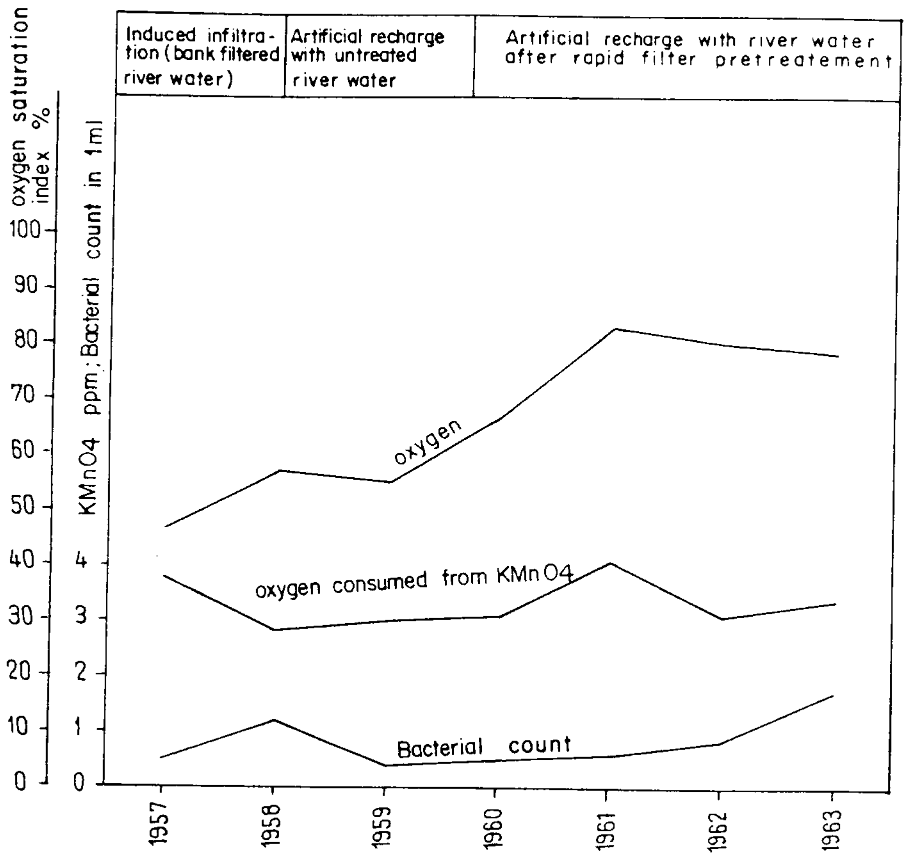

FIG. 7--Example C: quality of groundwater during different recharge processes.

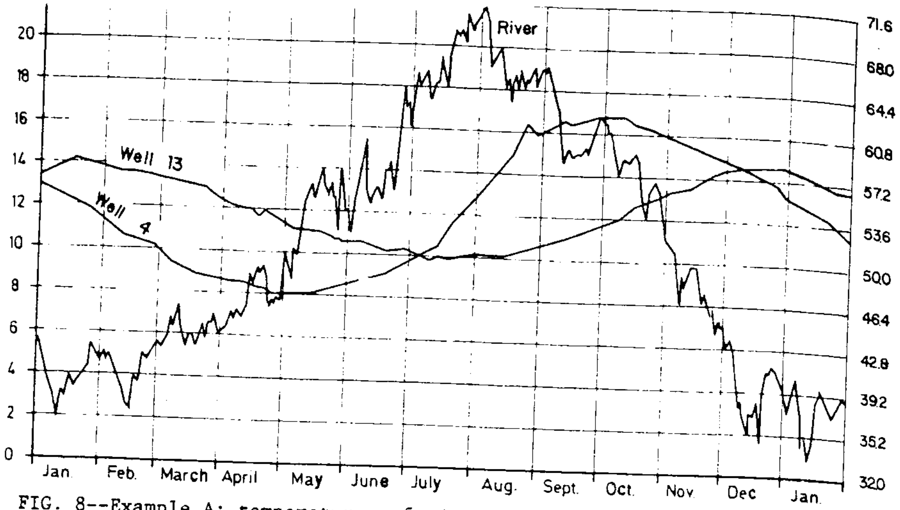

FIG. 8--Example A: temperatures of river water and recharged groundwater.

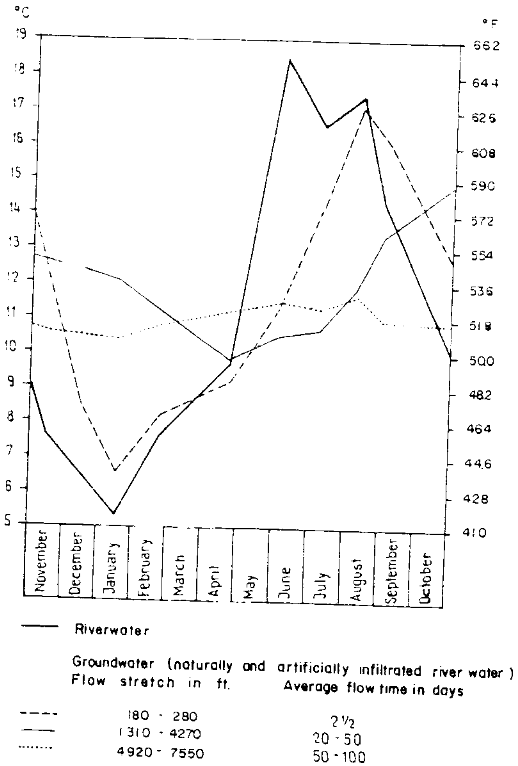

FIG. 9--Example C: annual temperature variations of river water and artificially recharged groundwater.

DESIGN, DRILLING AND COMPLETION, OPERATION, AND COST OF UNDERGROUND WASTE-DISPOSAL WELLS IN GULF COAST REGION OF TEXAS AND LOUISIANA[1]

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ABSTRACT The first factor to be considered in determining the feasibility of underground waste disposal is the quality of the waste stream. A practicable method or methods of removing suspended solids must be planned. Equally important is the necessity that the effluent be chemically stable, after filtration, under elevated temperature conditions of the injection zone. Compatibility of the waste with the indigenous brine is necessary to avoid plugging. The disposal well is the final filter in the waste-disposal system; it is the nature of filters to become plugged, and a filter several thousand feet underground is difficult and expensive to clean.

Once the suitability of the waste stream for underground waste disposal has been determined, the reservoir must be selected. Existing knowledge of the subsurface gained from oil and gas exploration provides enough data to determine a well depth sufficient to penetrate several probable reservoirs. Sand parameters measured in the disposal well permit selection of the most suitable reservoir. Geologic study of the subsurface provides information as to the areal extent and thickness of probable reservoirs.

Well design must meet state requirements for protecting surface freshwater sands and confining the waste to the selected reservoir. Drilling and well-completion techniques, including casing and cement selection to meet corrosion-protection needs, should be planned to offer maximum protection against failure of any part of the waste system.

[1]Manuscript received, May 4, 1973.

[2]Pollution Control and Waste Disposal, Inc.

The quantity and quality of the waste stream, the type and size of drilling equipment, and the type of contract used are the principal factors affecting the cost of a disposal well. Past experience in the area should provide data on drilling conditions and potential problems. Such data, together with proper equipment and material selection, should significantly reduce costs.

Operating an underground disposal well properly is just as important to success as good well design and good reservoir selection. Operating personnel should receive careful training in how to handle new waste sources and maintain good instrumentation and records. A dependable underground disposal system should include a standby or alternate well.

INTRODUCTION

In considering the feasibility of underground waste disposal, it is assumed that a reasonably thorough effort has been made to find a practicable and economic alternative disposal technique which will not create additional waste-disposal problems.

In the Gulf Coast region of Texas and Louisiana, the disposal reservoir may be expected to respond very much like a filter. Inasmuch as filters become plugged, it is important to remember that a surface filter is much easier to clean than one several thousand feet underground--and much less expensive.

The question arises as to what limits may be placed on filtration requirements. What maximum particle size will the receiving reservoir tolerate? What total-solids content of the filtered stream is acceptable? The answers to these questions depend on the volume of the waste stream, the size and density of solids, and the reservoir permeability and porosity. Two general observations can be made:

1. Any volume of solids of any size injected into a sandstone reservoir ultimately will have a plugging effect.

2. Solids removal from wells is usually more effective if the particle size is large enough so that penetration into the sandstone is limited to the well bore.

There are many successful injection operations which require periodic cleaning to restore original injectability. Depending on the type of solids involved, cleaning may range from a simple acid flush or backwash to removal and replacement of the screen and gravel pack. High-pressure jet cleaning of the screened section is commonly effective and less expensive than removing the screen. Generally, the necessity for some type

of cleanout procedure should be anticipated and budgeted.

Compatibility of the waste with the indigenous brine is also an essential factor. Usually a sample of brine from the chosen reservoir is not available at the time of well planning. Small samples taken at the well bore may be expected to be contaminated by drilling fluids and, therefore, may not be representative. If the nature of the waste suggests a possible incompatibility, it is desirable to flow the well (gas lift) until a true brine sample is obtained before injecting waste. If necessary, a suitable buffer material should be selected. However, if an incompatible condition is known to exist, it is much better to correct the condition by pretreatment if possible. The use of a buffer poses the problems of volume and distribution, and at a certain distance from the well bore the buffer is no longer effective. In a recent buffer application, it was observed that only the top 20 ft of a 100-ft vertical section had received the injected buffer material. The heterogeneous nature of porosity and permeability in most sandstone reservoirs prevents uniform distribution of the buffer material. Since liquid flows preferentially along a course of least resistance, the success of the buffer may be attributed to the probability that the waste stream follows the buffer, so that the plugging effect is minimized as the buffer is dissipated with distance from the well bore.

Bacteriologic control is usually not difficult or expensive. It is important that recognition be given to potential plugging by bacterial action and that necessary treatment be provided.

In planning underground waste disposal, knowledge of available storage reservoirs is essential. In the Gulf Coast region of Texas and Louisiana, there is a minimum of 1,000 ft of porous sandstone in the zone between 2,000 and 6,000 ft depth. Extensive exploratory drilling in this region has yielded enough subsurface information to permit adequate mapping of subsurface structure and reservoirs, so that the drilling of exploratory wells usually need not be considered prerequisite to waste disposal. However, selectivity should be practiced in choosing optimum reservoir parameters. The storage capacity of the reservoir depends on porosity, thickness, compressibility of water, and compressibility of the rock matrix. This combined effect, peculiar to each aquifer, results in appreciable storage space only where the reservoir has wide areal extent.

It is generally predictable from the subsurface geologic study that several sandstone strata are present which offer adequate storage parameters and capacity. Electric-log information from wells in the vicinity may be used to plan the depth so that the well will penetrate at least two such reservoirs below 2,000 ft. Although the freshwater level is usually above 1,000 ft depth, it is good practice to inject below 2,000 ft, thereby leaving several impermeable strata separating the reservoir from the freshwater zones.

Although the data from nearby wells are useful for disposal-well planning, specific reservoir data should be obtained from all penetrated prospective sandstone strata before casing is set. In addition to the electric log, enough core samples should be taken to establish knowledge of the variations in reservoir characteristics throughout the entire vertical section to be used for injection. The samples should be analyzed for porosity, permeability, sand-grain size, and silt content. Very silty sandstones should be rejected as disposal reservoirs, because they usually are less permeable and react adversely to most waste streams. Sand-sieve analysis is required for proper screen and gravel sizing. Liquid samples taken at the well bore may be contaminated with drilling fluid and thus not reliable for compatibility studies. A liquid sample in sufficient volume may be obtained by backwashing the well after gravel packing and before any waste injection. Following the backwash operation, a static bottom-hole pressure and temperature measurement should be made. Also, bottom-hole pressure should be measured during the final test injection with a pressure bomb in the hole. From these data, the initial capacity index of the well is determined for use as a reference point in evaluating future well performance.

Obtaining reservoir data from all penetrated sandstones which are known to have wide areal extent will permit maximum selectivity in choosing the primary storage reservoir. Also, more flexibility in planning additional wells is afforded. If all critical parameters are equal, the lowest sandstone penetrated should be selected as the initial disposal zone. This choice permits plugging back to the next higher zone if performance of the lower zone ultimately fails, owing to plugging.

Every well design must meet state regulations providing for the protection of surface freshwater reservoirs and confinement of the waste in the designated reservoir. These requirements involve setting the first casing string--the surface pipe--through all freshwater zones and circulating cement to the surface. A light-weight cement slurry is usually required, and a second-stage cementing from the surface is commonly required to refill the annular space when subsidence of the primary cement occurs.

For hydraulic reasons, the volume of the waste stream dictates the size of the injection tubing. Clearance between concentric strings of pipe dictates the size of the primary casing string and the surface pipe.

Typically, a well having 7-in. injection tubing has a 9 5/8-in. primary casing and 13 3/8-in. surface pipe. Corresponding bit sizes are 17 1/2 in. and 12 1/4 in. Because the surface pipe is encased in cement, inside and outside, it is not affected by corrosion. The annular space between the injection tubing and the primary casing string is filled with fluid. Therefore, this fluid should be selected and treated for corrosion control. Clean brine with a corrosion-inhibitor additive is a commonly used annular fluid. The selection of the material for the injection tubing depends on the corrosion characteristics of the waste stream. The tubing may be made of carbon steel or of many other materials, such as fiberglass or one of the grades of stainless steel. A variety of materials is also available for equipment such as the wettable parts of the wellhead equipment, packer, screen, and liner.

Use of a screen, liner, and packer implies that an open-hole type of well design is contemplated. This type of completion is preferred in the Gulf Coast region, as it offers a minimum restriction to injection and permits backwashing as a cleanout technique. Unconsolidated sands tend to flow into the well bore if not restricted by a screen and gravel pack. The use of a packer at the top of the liner, with a polished receptacle to receive the tubing-seal assembly, is recommended as a preferable design (Fig. 1). The use of a packer permits positive pressure monitoring of the system for leaks. The injection tube is subject to expansion or contraction owing to changes in temperature and pressure; therefore, the annular pressure is not constant. To assure that no injection-tubing leak will be undetected, it is desirable to maintain annulus pressure at a fixed differential of 100 psi above injection pressure. Thus, if a tubing or packer leak develops, it is known immediately because of the inability to maintain annulus pressure, and remedial action can be taken. The integrity of the injection tube is of paramount importance in successful waste injection; thus, continuous monitoring is necessary.

The cost of an underground waste-injection well varies with the volume and quality of the waste stream. A recent well with an injection rate of 600 gal/minute (gpm), drilled to 3,850 ft and equipped with 7-in. carbon-steel injection tubing, was completed and tested at a total cost of $36.50 per foot. Total time to move in equipment, drill, and complete the well was 15 days. This cost probably represents a minimum for this size and depth of well. Costs increase if special material to handle corrosive waste is required. Costs also vary with the type of drilling equipment used and the type of contract. Usually, turnkey contracts result in much higher costs than daywork or footage contracts. The depth and size of most disposal wells fall within the capability of the drive-in type rig used for drilling to depths of 7,500-10,000 ft. Moving and rigging-up costs and well-site preparation are less expensive than with larger rigs which are not truck mounted. Another factor affecting costs is the prevention of trouble while drilling and completing. Personnel experienced in drilling in the area, and the use of good equipment, minimize possible "fishing" jobs, etc.

The necessity for cementing both the surface casing and the primary casing from bottom to ground surface requires that the open hole be kept in good condition. Special attention should be given to the drilling mud to minimize problems of hole washout and lost circulation. Mud treatment should begin before the surface-pipe cement plug is drilled. A drilling mud with a low filtration rate should be used. Use of such a mud results in good hole conditions which enhance the success of core sampling and cementing. It is also important to avoid circulating entrained sand in the mud, because the increased mud density is conducive to lost circulation. A desander in the mud-circulating system is essential. As soon as the hole is drilled to the desired depth, an electric log and a caliper log should be run. A sidewall-coring program should follow, designed to get representative samples from selected sandstone strata as determined from the electric log. It is difficult to get core samples from unconsolidated saltwater-saturated sands. By controlling the penetration of the core bullets and pulling the cores out of the hole slowly, it is not unusual to achieve 80-100 percent recovery.

Cementing of the primary casing should receive special attention. Adequate centralizers should be applied and a cement float collar should be positioned 30 or 40 ft above the cement shoe to assure good placement of uncontaminated cement around the casing shoe, which should be positioned at least 5 ft below the top of the sandstone reservoir. A light-weight cement should be selected, and the final 100 sacks should be neat cement. If a corrosive waste is to be injected, the bottom joint of casing should be of corrosion-proof material. As further protection, the final cement slurry may be an epoxy-resin cement which is acid resistant. Because the casing must be free from leaks, a good teflon-filled thread lubricant should be used and the casing couplings should be hydraulically tested as the casing is run. After the cement has set, the casing should again be pressure-tested before the cement is drilled.

Before the cement is drilled, all the drilling mud in the system should be displaced with a 50-60 viscosity, 9-lb/gal brine. This fluid

will be used to clean out the open hole below the casing and to underream the hole. The largest available underreamer that can be run through the casing is recommended. After underreaming, a hole caliper should be run to determine the volume of gravel required and to prove the effectiveness of the underreamer.

Running of the screen and liner follows the underreaming operation. A prompt core analysis is required in order to provide the necessary information for sizing of the screen. Prior arrangements with the screen manufacturer should assure procurement of the required material so that only the slotting or wire wrapping are needed to complete the order. Allowance of one day following coring to condition the hole and run casing, one day for cement to set, and one day for underreaming and caliper survey leaves very little time to secure the screen. Underreaming should not be commenced until the arrival time of the screen is known, because sloughing of the sand may occur during any waiting time, thus necessitating additional cleanout procedures.

After the screen and liner are run in the hole, the drilling fluid should be displaced with a clean 35-38-viscosity brine during the gravel-packing process, thus further cleaning the hole as the gravel packing progresses. After the liner packer is set and the injection tubing is run, the well may be backwashed, with the use of nitrogen as a gas-lift medium, through 3/4-in. tubing. At this time, samples of the indigenous brine may be collected for compatibility studies. Following the backwash operation, an injection test should be performed and bottom-hole static and injection pressures should be measured with a bottom-hole pressure bomb. As a final cleanup before placing the well in operation, an acid treatment with a combination of HCl and HF acid should be performed; a diverter or a selective injection packer should be used to improve the distribution of the acid.

At the time the well is placed in service, all filtering and waste treatment should have been perfected. The tubing-casing annulus should have been pressurized and all automatic valves checked. Pressure-recording instruments and flowmeters necessary to record well performance and satisfy record-keeping requirements should have been properly calibrated. All surface piping should have been tested and flushed to clean out all construction debris. Operating personnel should have been trained in operating procedures and advised that the new facility is just as important to the plant operation as is any other segment of the plant. Almost invariably the response will be one of disbelief. Psychologically, the disposal well is viewed as a part of the plant drainage system--a necessary nuisance. When a plant process failure occurs (particularly on the 12-to-8 shift) and a quantity of new waste is created, somebody says: "Send it to the well." Or, if the filters become plugged and waste storage is critically high, somebody says: "Open the by-pass valve." Filter precoat material escaping from improperly installed filters becomes an effective plugging agent.

My point is that disposal-well plugging is likely to occur, since the well becomes just another filter in the system. Time required to clean a plugged well may vary from a few hours for a simple backwash or acid treatment--if successful--to several days if the plugging is so severe that removal of the screen and mechanical cleaning are necessary. Therefore, a second well as a standby or alternate facility is recommended.

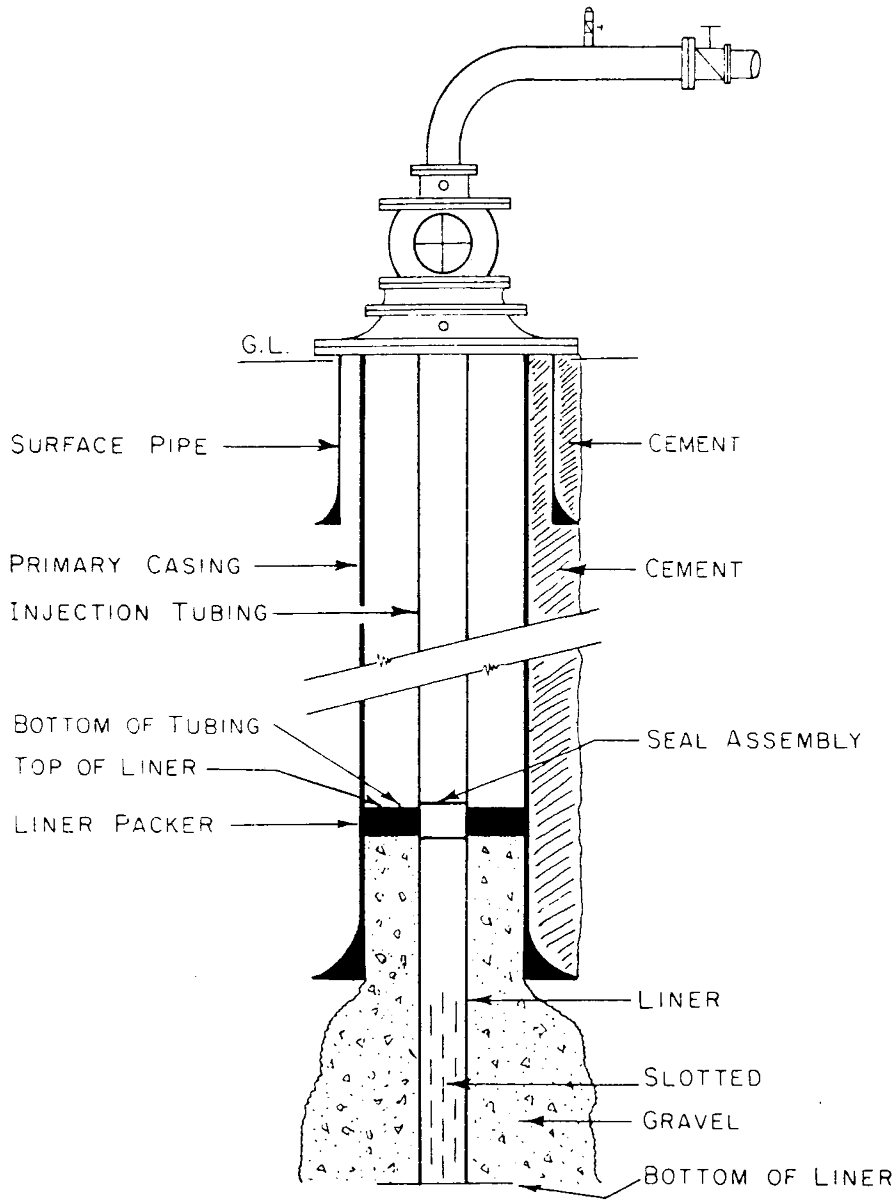

FIG. 1--Schematic drawing of typical waste-disposal well.

LABORATORY AND FIELD INVESTIGATIONS

HYDRODYNAMICS OF MOUNT SIMON SANDSTONE, OHIO AND ADJOINING AREAS[1]

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ABSTRACT The Mount Simon Sandstone (Cambrian), the most favorable stratum for waste injection in Ohio, presently accepts about 250 x 10^6 gal of industrial waste per year. Concern has been expressed about the transport of these fluids by natural hydrodynamic flow.

The potentiometric-surface map of the Mount Simon reservoir of Ohio has a form which mirrors the structural configuration--highest values are in the deeper part of the Appalachian basin and lowest values are on the Indiana-Ohio platform. Flow direction in central Ohio is indicated to be west or northwest. Head difference is 2-7 ft. Porosity and permeability data combined with this information (Darcy's law) yield velocities of less than 6 in./year.

Because the assumptions involved in determining velocity in this manner are questionable, the resulting values should be considered rough approximations. Nevertheless, the calculations generally show that transport of injected fluids by hydrodynamic flow is not presently a serious hazard in Ohio.

[1]Manuscript received, May 7, 1973.

[2]Ohio Division of Geological Survey.

The Cambrian Mount Simon Sandstone is the most favorable zone for deep injection in Ohio; presently, about 250 million gal of industrial waste per year are injected through six wells. Concern has been expressed about the fate of waste fluids in the subsurface as a result of transport by hydrodynamic movement. This report discusses the theoretical average direction and rate of flow of fluids in the Mount Simon Sandstone. Modification of the natural flow by injection is not discussed; at present rates of injection, such modification is only a local phenomenon.

The direction of flow and potential head between points within a formation are determined from potentiometric-surface maps. Pressures recorded from wells penetrating the formation are related to a common reference--the height, relative to sea level, to which a column of fresh water would rise in a well bore open to that formation as a result of the pressure encountered in the formation. The different freshwater-head values are contoured to form a surface. Flow is considered to be at right angles to the contour lines and to be directed from higher to lower values. By use of Darcy's Law, head difference between points can be combined with porosity and permeability data to calculate theoretical velocity of flow.

Pressure data were available from seven drill-stem tests and from two static water levels from wells in and near Ohio (Table 1). The pressures recorded by the drill-stem tests were extrapolated to infinite time by the testers or by the writer, or were recorded for sufficient duration that they were believed to be accurate. The two static water levels were not verified. A map of the potentiometric surface was constructed from these data (Fig. 1).

In general form, the potentiometric surface closely mimics the structural configuration of the underlying Precambrian surface (Fig. 2). The close match was partly forced by the data, but an intentional effort was made to emphasize the fit where data were sparse.

Part of the apparent head difference between wells in Figure 1 is not real, at least insofar as head difference is supposed to cause flow. The reason is that salinity, hence density, variations are quite large; in general, the deeper wells encounter fluid of higher salinity. Specific gravity ranges between about 1.05 and 1.23 in the area under consideration. Bond and Cartwright (1970) and Bond (1972) discussed this problem and reviewed previous work dealing with density variations in potentiometric data from deep wells.

Figure 3, a cross section of a formation in hydrostatic equilibrium, was constructed to illustrate the problem and to show a means of correcting

for it. If density increases with depth, as indicated on the left of the figure, then a pressure gage placed opposite the formation in the deeper well will record a pressure which will convert to a higher freshwater head (P_1) than is indicated for the shallower well (P_2). This behavior occurs because the average density of the formation water acting on the deep well is higher. The potentiometric surface will slope from the deep to the shallow well, though no flow exists. To correct for this effect, the average change in density (G_1-G_2/2) times the interval over which the change occurs (D_1-D_2) is converted to feet of freshwater head and subtracted from the potential head of the deeper well.

This correction is made with the assumption that salinity varies as a function of depth, which is generally true of the Mount Simon of the Appalachian basin (Clifford, 1972) and also of the Illinois basin (Bond, 1972, p. 9). The salinity variation seems to be most nearly linear for wells located along lines at right angles to dip.

Density corrections were made between pairs of wells using the method shown in Figure 3. The pairs are indicated in Figure 4, along with scalar arrows showing remaining head difference, head in feet per mile, and amount of correction.

The corrected values for head difference were then form-line contoured in the style of the uncorrected map. The result, Figure 5, shows a similar picture except that the contours are spaced somewhat more widely. The flow direction is indicated to be toward potential lows in northwest Ohio and northwest Indiana. The gradient in the corrected potential averages about 4 ft/mi and does not exceed about 7 ft/mi.

According to Bond (1972) and Bond and Cartwright (1970), it is possible that flow may not occur, even though differences in potential exist. These authors pointed out that density differences coupled with structural irregularities can effectively block flow. If the potential energy available to lift denser water out of a structural trough is not high enough, flow will not take place. Permeability variations can have the same effect as structural irregularities.

If it is assumed that flow does occur, it is possible to combine the head difference between wells with the porosity and permeability data shown in Table 2 and, by use of Darcy's Law, to calculate the theoretical velocity of flow between these wells. By generalizing the data, it is possible to determine the velocity between any two points within the mapped area. A graph was prepared (Fig. 6) to show velocity as a function of head for various values of porosity and permeability. Within the Mount Simon Sandstone of Ohio, theoretical flow rate probably does not exceed about 6 in./year. At that rate, flow between two points a mile apart would take over 10,000 years.

Some assumptions necessary to determine velocity in this manner are that the reservoir is isotropic and homogeneous, that the viscosity of the reservoir fluid is that of fresh water, and that a sloping potentiometric surface actually indicates flow. Because these assumptions are questionable to some degree, the resulting value of average velocity is only a first approximation. Nevertheless, the calculations do tend to show that possible transport of injected fluids by natural hydrodynamic flow is not presently a serious hazard of deep-well waste injection in Ohio.

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Table 1. Potentiometric-Surface Data

State	Well name	Surface elevation[1]	Reservoir pressure (psi)	Gage depth[3]	Gage elevation[4]	Top Mt. Simon[3]	Elevation Mt. Simon[4]	Specific gravity[5]	Potentiometric surface[6]	Source of data
Ohio	Empire-Reeves #1	1177KB[2]	2050	4961	-3784	4982	-3805	1.20(.520)	952	DST (extrapolated)
	Vistron	864GR[2]	1100	----	-----	2783	-1919	1.10(.476)	622	Est. from static water level
	U.S.S. Chem. #1	557KB	2633	5545	-4988	5514	-4957	1.225(.530)	1094	DST (extrapolated)
	Calhio #1	701KB	2760	5886	-5185	5930	-5229	1.218(.527)	1191	DST (extrapolated)
	Hoelscher #1 (East Ohio)	896KB	1093	2772	-1876	2799	-1903	1.047(.453)	644	DST (extrapolated)
	Ohio Liquid Disposal #1	623KB	1132	2745	-2122	2810	-2187	1.089(.472)	493	DST (not extrapolated but curve is flat)
	Armco #1	---	----	----	-----	----	-----	----------	---	No data recorded
Indiana	U.S. Steel	600KB	1420	3300	-2700	3289?	-2689	1.072(.461)	580	DST (extrapolated)
	F.M.C.	650KB	2437	----	-----	5264	-4614	1.148(.497)	1015	Static water level (Hundley & Matulis, 1963)
Michigan	Consumers Gas	616KB	2145	4498	-3882	4576	-3960	1.195(.517)	1073	DST (extrapolated)
Kentucky	DuPont	462KB	2571	5397	-4935	5408	-4946	1.14 (.494)	1004	DST (not extrapolated, curve reported flat)

[1]Feet above sea level
[2]Depths measured from Kelly bushing (KB) or ground surface (GR)
[3]Depth in feet below surface elevation
[4]Elevation in feet below sea level
[5]Numbers in parentheses are pressure gradients in psi/ft
[6]Potentiometric surface (in feet above sea level) = (reservoir pressure X 2.31) + gage datum

Table 2. Porosity and Permeability Data, Mount Simon Sandstone, Ohio

Well name	County	Permit number	Interval analyzed (ft)	Average porosity (%)	Average permeability (millidarcys)	Source of data
Vistron #1	Allen	P-67 IWDW #4	334	14.4	80 (to gas)	Core analysis
Hoelscher #1 (East Ohio)	Auglaize	P-71	148 127	12.3	— 44	Density log analysis Drill-stem test
Armco #1	Butler	P-4 IWDW #2	? 217	13.1	25.1 (to water) —	Consultant's report Core analysis
Herman #1 (Sun Oil)	Erie	P-19	44	13.2	32.6	Unpublished consultant's report
Ullman #1 (Amerada)	Noble	P-1278	175 159	2.6	— very low[1]	Cross-plot log analysis Drill-stem test
Calhio #1	Lake	P-142 IWDW #7	165 102	8.4	3.0 11.6	Drill-stem test Core analysis
Empire-Reeves #1	Richland	P-448 IWDW #1	108 90	10.4	24 9	Drill-stem test Core analysis
Ohio Liquid Disposal	Sandusky	P-210	79 122	15.5	30 41	Core analysis Drill-stem test
U.S.S. Chem. #1	Scioto	P-212 IWDW #5	29	12	27 (to gas)	Core analysis

[1]Drill-stem test of Mt. Simon recovered only 90 feet of drilling mud over interval 11,283-11,442 ft

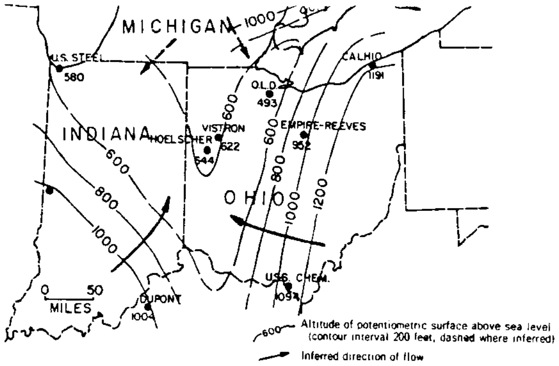

FIG. 1--Potentiometric surface of Mount Simon Sandstone; not corrected for density variation.

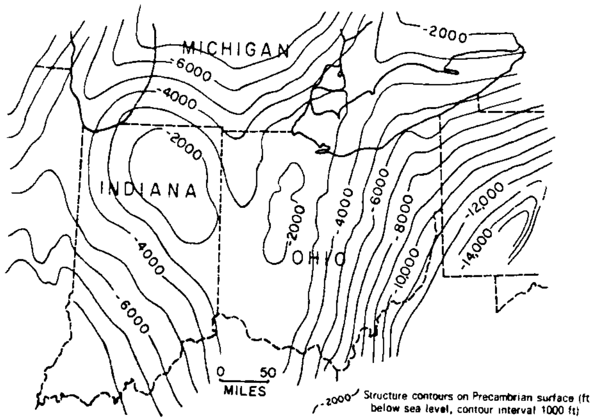

FIG. 2--Structure of Precambrian surface (from Hennington, 1973).

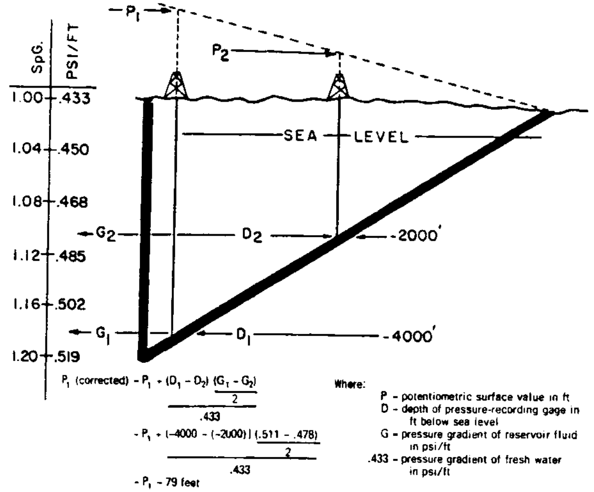

FIG. 3--Effect of density variation on potentiometric surface and method of correction.

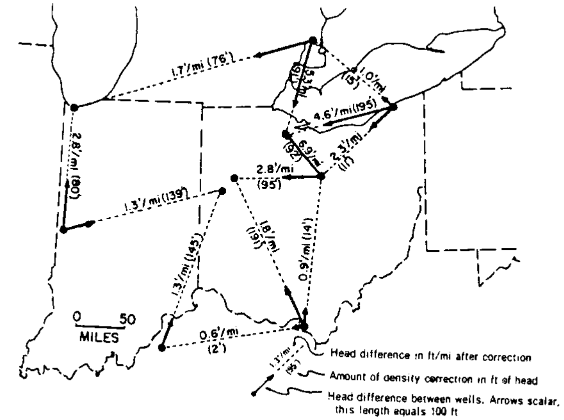

FIG. 4--Density corrections between pairs of wells and resulting head differences.

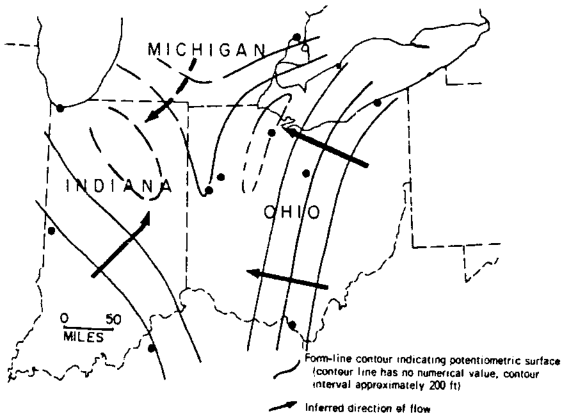

FIG. 5--Form-line contour map showing potentiometric surface of Mount Simon Sandstone corrected for density variation.

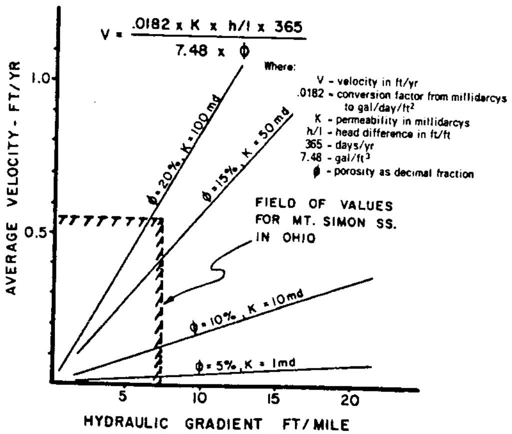

FIG. 6--Average velocity of flow as a function of hydraulic gradient for several values of porosity and permeability.

DEDUCTION OF FLOW PATTERNS IN VARIABLE-DENSITY AQUIFERS FROM PRESSURE AND WATER-LEVEL OBSERVATIONS[1]

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ABSTRACT In previous potentiometric studies of variable-density aquifers, particularly studies related to oil exploration, certain gravitational effects apparently have been ignored. These include the effects of troughs formed by permeability barriers within the aquifers and the effects of structural troughs, saddles, anticlines, and synclines. In intermontane regions these gravitational effects probably are negligible in comparison with observed head differences; in most other regions they can appreciably change the heads, or the potentials, that are available to cause flow.

A gradient in potential is not necessarily associated with flow, even though corrections are made for the average rate of change in density of water. Gravitational effects can cause the interface between water and an oil or gas deposit to be tilted, even if the water under the deposit is static. These effects can reduce the rate of flushing of brine by fresh water, or they can prevent flushing.

Previous potentiometric studies should be reevaluated to ensure that all gravitational effects have been taken into account.

INTRODUCTION

Information about the flow of groundwater is important in the oil and gas industry and in related industries. Such information can be helpful in predicting possibilities for oil accumulation. It can be used to determine the tilt of the interface between a natural deposit of oil or gas and the underlying water, or the tilt of the bottom of a gas-storage bubble. It may help to explain certain anomalies in the growth of gas-storage bubbles. Furthermore, knowledge of the direction and rate of flow

[1]Manuscript received, May 24, 1973.

[2]Illinois State Geological Survey.

of underground water is essential in the handling of problems related to underground disposal of industrial wastes. For these reasons, a study has been made of some of the problems encountered in the flow of groundwaters, in particular, those waters of which the densities vary because of variations in salinity.

Several investigators have made studies of flow in variable-density systems (see Bond, 1972, p. 4-5). Thus far, no one appears to have studied critically the criteria for flow in such systems, nor has anyone detailed the consequences of variations in water density.

The purposes of the present report are:

1. To inspect the basis for our ideas about the forces that control the flow of groundwater in aquifers in which the density of the water is variable;
2. To estimate the magnitude of gravitational effects, previously ignored, that can occur in various kinds of aquifers;
3. To present methods of handling head and density data to give valid conclusions about flow, and to outline the limitations of these methods;
4. To raise questions about the validity of some studies that have been made of variable-density aquifers; and
5. To explain certain anomalous situations that have been revealed by previous hydrodynamic studies.

Definitions of certain terms to be used herein are as follows.

Aquifer--"An aquifer is a formation, group of formations, or part of a formation that contains sufficient saturated permeable material to yield significant quantities of water to wells and springs" (Lohman et al., 1972, p. 2).

In this paper the word "aquifer" is used to designate a porous, permeable, water-saturated rock unit, without implication as to its potential for yielding a water supply, particularly a potable water supply or a supply that could be made potable economically. Under the general conditions considered here, an aquifer might contain no potable water or, more commonly, considered regionally, it might contain waters of markedly different mineral qualities, ranging from potable water in and near the area of outcrop to concentrated brine downdip in a sedimentary basin. We are concerned primarily with portions of aquifers wherein the water varies in mineral quality and density.

Potential, Φ--The potential, Φ, of water at a given point in an aquifer is given by:

$$\Phi = gZ + \frac{\text{(Pressure at observation point)}}{\text{(Density of water at observation point)}}, \text{ or}$$

$$\Phi = gh \text{ (h = static-water level above standard datum).}$$

The definition is from Hubbert (1953, p. 1960).

Elevation, Z--The letter "Z" is used to denote elevation with respect to sea level, given in feet.

$H^{1.00}$--$H^{1.00}$ is the static head at a point, relative to sea level, in terms of water having a relative density of 1.00.

Relative density, ρ--The relative density, ρ, of a given sample of groundwater is the ratio of the density of the water, under the temperature and pressure existing at the point of sampling, to the density of fresh water at 75°F and 1 atm pressure.

Distance, s--The letter "s" denotes the distance, in miles.

Head available to cause flow, Ψ_ρ--The following equation applies:

$$\Psi_\rho = \left[H_2^{1.00} - H_1^{1.00}\right] - \int_{P_2}^{P_1} (\rho - 1)\, dZ,$$

where $H_2^{1.00}$ and $H_1^{1.00}$ are the values of $H^{1.00}$ at points P_2 and P_1, respectively, and $\int_{P_2}^{P_1} (\rho - 1)\, dZ$ is the integral along a flow path from P_2 to P_1 (Bond, 1972).

Trough--A trough is a part of an aquifer where permeable rock is partially encased in impermeable, or relatively impermeable, strata. When liquid flows from one end of the trough to the other, the impermeable strata force the liquid to flow down and then up. The shapes of the impermeable strata prevent horizontal flow through the sides of the trough. Any configuration of impermeable strata that causes such up-and-down flow is a trough. In its simplest form, a trough is a U-tube, bounded on its top, bottom, and sides by impermeable strata, with the ends of the U-tube open to flow. The cross section of the U-tube can have any form and can vary in any way from one end of the trough to the other. A trough can be of any size. It can be bounded by the top and bottom of an aquifer, or it can be contained within the body of the aquifer.

DIFFERENCES BETWEEN CONSTANT-DENSITY AND VARIABLE-DENSITY AQUIFERS

Hubbert (1953) studied the behavior of ambient groundwater, which he considered to be a homogeneous fluid of constant density. He assumed that no impermeable barriers isolated the various parts of the space that was under consideration. Further, he assumed that a number of observations were made in scattered wells that penetrated a stratum. The observations could be the static-water levels in the wells, or they could be equilibrium reservoir pressures at known depths--pressures which could be converted into equivalent water levels.

Hubbert (1953, p. 1973) considered the values of the potential or of the head at three points forming the apices of a triangle along the upper surface of the stratum. He showed that the water would be static if all three values were the same; if any two values were different, the water would be flowing. Furthermore (p. 1958), if the water was in motion in one region, it had to be in motion throughout all space that was not isolated by impermeable barriers.

Besides constant-density systems, Hubbert briefly analyzed two variable-density systems. In one case (1953, p. 1993-1995), he discussed the flow of one liquid above another, denser, liquid and showed that the flow of the lighter liquid caused a tilt in the interface between the two liquids.

In the other case (p. 1995), Hubbert described a method of detecting flow in a basin in which the salinity of the water increases with depth. Measurements of pressure and of water density are taken in a row of wells extending downdip from the flank of the basin. A well in the middle of the row is used as a reference well; the density of the water in this well is used as a reference density. For each of these wells, the potential, Φ, is calculated by use of this reference density and the pressure measured in the well. The plot of potential versus distance is said to show whether the water is static (curve has a minimum point at the reference well) or flowing (flow is in direction of downward slope of tangent to curve).

Hubbert did not give the justification for this method, nor did he give its limitations. For certain relatively simple systems, especially aquifers that are quite homogeneous in lithology and have simple structure, the method appears reasonable. However, until we know the assumptions upon which it is based, as well as its limitations, we cannot use Hubbert's method universally without risking erroneous conclusions.

Several researchers have made studies of aquifers in which the salinity, and therefore the density, of the water varies (e.g., McNeal, 1965, 1969; Hitchon, 1969a, b; Hanshaw and Hill, 1969). Although some of them have presented methods for making potentiometric maps for such variable-salinity aquifers, none has shown how his method was derived; nor has

anyone outlined the assumptions that he made or the limitations of his method. These methods, like Hubbert's method, cannot be used indiscriminately without risking erroneous conclusions.

In aquifers filled with water that has the same density throughout, if Hubbert's assumptions about constant density and absence of barriers are valid, one needs to know nothing about the rock in the space between the observation wells and in the rest of the aquifer. Hubbert showed that the difference in potential from one point in the aquifer to another point was equal to the work done in moving a unit mass of the liquid from one of the points to the other point. It made no difference by what path the liquid might flow between the two points: if work was done against gravity in lifting the liquid over a given vertical distance in one part of the aquifer, the same amount of work was done by gravity and was recovered when the liquid flowed down the same vertical distance in another part of the aquifer. Thus, in a constant-density aquifer, a difference in potential was necessarily associated with flow; the difference in potential represented the work that was done in overcoming viscous forces as unit mass of the liquid was transported from one point to another.

However, in a variable-density aquifer, Hubbert's reasoning does not hold. (Hubbert [1953] emphasized that his conclusions [p. 1965] were derived for a <u>constant</u>-density system.) Consider two observation points, P_1 and P_2, on a possible flow path in a variable-density aquifer. When water flows from P_1 to P_2, work is done in overcoming viscous forces. Work is also done either by or against gravity; the sign and the magnitude of this work depend upon the manner in which the density of the water varies with the elevation along the flow path taken by the water. The location of this flow path is influenced by, among other factors, (1) the shape of the relatively impervious beds along the boundaries of the aquifer --that is, structure on the top and bottom of the aquifer, and (2) the shape of the impermeable or slightly permeable beds within the aquifer. Of course, no gravitational work is done if the flow path is horizontal. A given difference between the heads at P_1 and P_2, $(H_2^{1.00} - H_1^{1.00})$, can be accompanied by (1) flow from P_1 to P_2, (2) flow from P_2 to P_1, or (3) a static condition with no flow, depending on the relative values of $(H_2^{1.00} - H_1^{1.00})$ and $\int_{P_2}^{P_1} (\rho - 1)\, dZ$ along the flow path (Bond and Cartwright, 1970, p. 1493).

DEDUCTION OF FLOW PATTERNS IN A VARIABLE-DENSITY AQUIFER

Our problem is to deduce useful information about the flow patterns in an aquifer from the results of measurements of head or of pressure made at a limited number of observation points in the aquifer. We can approach the problem by considering the potential, the force, or the head at various points in the aquifer. To some degree, each of these is used in the discussion that follows. However, because the effects of head are easiest to visualize without ambiguity, $H^{1.00}$, the head in terms of fresh water, is used here in most cases.

In a constant-density aquifer, a potential difference has significance with respect to flow--a potential difference is a measure of the work done against viscous forces as water flows. In a variable-density aquifer, a potential difference can give a measure of the work done against viscous forces, as water flows only if the work done against gravity is negligible or if this work can be evaluated and subtracted from the observed potential difference.

We can look at the problem in another way: in a variable-density aquifer the water exists in a field of force in which the force at any point can be considered to be the vector sum of two forces. One of these forces causes work to be done against viscous forces as water flows through the aquifer. The other force results from the head differences that are caused by variations in water density. For a given flow path between two points, if we can evaluate the head difference that is caused by variations in density, we can calculate the head that is available to cause flow along that path. If we can make such a calculation for representative possible flow paths, we may be able to estimate the approximate directions of flow and the magnitudes of the rates of flow in parts of the aquifer. If we cannot make such an evaluation, we have no valid way of determining what flow, if any, is caused by the observed head differences.

Following are descriptions of four types of aquifers and suggested methods of handling flow problems for each type. For each type of aquifer the assumption is made that no permeability barriers exist to isolate one part of the aquifer from another part.

1. Aquifer rock is homogeneous; interstitial water has same density throughout the aquifer. Hubbert's procedure is valid. That is, values of Φ are plotted on a map and equipotential lines are drawn; flow direction is perpendicular to equipotential lines. Hubbert's conclusions (1953, p. 1962, 1992) about forces exerted on the water are valid.

2. Aquifer rock is not homogeneous; interstitial water has same density throughout aquifer. The procedure outlined in (1) can be used here also.

3. Aquifer rock contains no barriers that prevent horizontal flow; density of interstitial water is variable. Two possibilities exist. In the first (a), the gravitational effects caused by structure on top and bottom of the aquifer are negligible.

Method I for Case 3(a)--The aquifer is divided into n horizontal layers, of thickness ΔZ, in which the average relative density is ρ_1, ρ_2, $\rho_3, \ldots \rho_n$. Let the mass of unit volume of fresh water (75°F and 1 atm pressure) be m_s. We can show that the quantity $(\rho_2 - \rho_1) \times \Delta Z \times m_s \times g$ must be substracted from observed values of Φ in Layer 2 in order to make values of Φ in Layers 1 and 2 comparable for purposes of predicting flow. For the $n^{\underline{th}}$ layer the quantity to be subtracted is $(\rho_n - \rho_1) \times \Delta Z \times m_s \times g$.

The observed values of Φ are corrected as outlined above and are plotted on a map; equipotential lines are then drawn. Flow direction is perpendicular to these equipotential lines. Rate of flow is proportional to the gradient in corrected values of Φ.

Several investigators have used methods that appear to be similar to the one outlined above (Foulks and Brown, 1962; McNeal, 1969; Hitchon, 1969a).

I emphasize that this method, as well as the one that follows, is valid only if the aquifer rock contains no barriers that interfere with horizontal flow and if the structural effects on flow are negligible.

Method II for Case 3(a)--A second method (Bond, 1972) is based on consideration of the quantity Ψ_ρ, the head available to cause flow from P_2 to P_1; Ψ_ρ is defined by the equation:

$$\Psi_\rho = \left[H_2^{1.00} - H_1^{1.00}\right] - \int_{P_2}^{P_1} (\rho - 1)\, dZ,$$

where the integral is taken along a flow path between two points, P_1 and P_2, in the aquifer. In practice, we seldom know ρ as a function of Z, so we cannot evaluate the quantity $\int_{P_2}^{P_1} (\rho - 1)\, dZ$ exactly. Let us assume that ρ is an approximately linear function of Z in the region between P_1 and P_2 and that the flow line is a straight line from P_1 to P_2. We can show that if lines proportional to $\left(\frac{\Psi_{\rho\,ave.}}{s}\right)$ in length are plotted between various pairs of observation points on a map, the resulting family of lines gives vectors from which one can deduce the general direction and the approximate rate of flow in various parts of the aquifer. If sufficient data are available, Methods I and II should give similar results.

In most of the discussion that follows, "gravitational effects" refers to the head differences that are caused by water-density variations, excluding the long-range, gradual variations that can be handled by the incremental averaging techniques described. The concern is primarily with aquifers in which the quantity $|\Psi_\rho - \Psi_{\rho_{ave.}}|$ for pairs of points along possible flow paths is appreciable in comparison with observed head differences.

The second possibility for Case 3 is that (b) structure of top and bottom of the aquifer involves troughs, saddles, anticlines, and synclines that can cause gravitational effects. Even where the water is tatic, a trough that contains dense water (that is, water that is more dense than an invading water) can cause a difference in head from one end of the trough to the other end; in a series of troughs the head differences are additive (Bond and Cartwright, 1970). Where waters of different density exist on either side of a saddle, a head difference can be maintained even if the waters are not flowing; the saddle may be the spillpoint between two anticlines (Bond, 1972). This concept is especially important because, in practice, most observations of pressure and head are made at or near the tops of anticlines.

If the water is not static but is flowing, the head differences that are caused by troughs, saddles, anticlines, and synclines usually reduce the head that is available to cause flow.

Thus, in a variable-density aquifer, if appreciable differences in head are introduced because of the structure of the top and bottom of the aquifer (that is, because of the presence of troughs, saddles, anticlines, and synclines), these differences must be evaluated and taken into account in any attempt to prepare a "corrected" potential map or head map for use in deducing flow patterns in the aquifer.

In considering the possible effects of structure on flow, we should weigh carefully the meaning of "structure." The surface of an aquifer marks the boundary between two lithologic units, one of which is permeable and hydraulically interconnected throughout and the other of which is relatively impermeable. The structure of this surface is the structure with which we are concerned. Use of structural data based on other concepts--e.g., the idea of a formational contact or a marker bed--could lead to erroneous conclusions about flow. For example, in the Illinois basin the Mount Simon aquifer (as defined on the basis of permeability) can exhibit structural changes as great as 300 to 400 ft over a distance where little or no structure is evident from the contours of the top of the Mount Simon formation.

4. Aquifer rock contains permeability barriers, which prevent horizontal flow within the aquifer or divert flow from horizontal paths; density of interstitial water is variable. Note that this is the case in which barriers occur within the main body of the aquifer; it is not concerned with the effects of structure that are caused by barriers on the top and the bottom of the aquifer. Valid flow patterns cannot be deduced unless the geometry of the permeability barriers and the variations in water density are known in detail, or unless the head differences that are caused by gravitational effects are known to be negligible in comparison with the observed head differences.

Any impermeable barrier rock or slightly permeable rock within the body of the aquifer that causes the flow of water to deviate from the horizontal can cause a difference in head, because it causes work to be done either by or against gravity as water flows through the aquifer. If the water is static, the barrier rock must have practically zero permeability if a head difference is to be maintained indefinitely; if the water is flowing, the barrier rock needs to be only sufficiently impermeable to cause flow to deviate from the horizontal. If appreciable paths exist for horizontal flow from one part of the aquifer to another part, no gravitational effects are observed along those paths, but such effects can still be observed in regions where barriers to horizontal flow exist.

In an aquifer in which such permeability barriers are distributed more or less randomly, we can expect flow to be diverted down and then up, over and over again, to give an undulatory, "corrugated" flow. That is, the water flows through a series of irregular U-shaped troughs. In a region where dense water is being displaced by a lighter water, the dense water in the troughs is not necessarily displaced completely (see Fig. 3). The net result is a series of troughs, with light water on the inlet side and dense water on the outlet side of each. Each trough produces a head difference, $\Delta H^{1.00}$, equal to $(\rho_D - \rho_L) \times \Delta Z$, where ρ_D and ρ_L are the densities of the dense and light waters, respectively, and ΔZ is the depth of the trough. In a series of troughs, the head differences are additive (Bond and Cartwright, 1970). Thus, anything within the aquifer that causes flow to deviate from the horizontal can introduce gravitational effects that result in differences in head.

In summary, in a variable-density aquifer, gravitational effects can be simple (no appreciable structure at top and bottom of aquifer, no barriers to horizontal flow); they can be moderately complex (aquifer contains saddles, domes, etc., but has no barriers to flow within the aquifer); or they can be quite complex (barriers to flow exist within the aquifer, structural effects are significant). For the first two cases if enough information about structure and density variations is available we can prepare useful corrected potentiometric maps, which may yield valid conclusions about flow. For the third case, the available information usually does not permit the preparation of potentiometric maps that can yield valid flow data, unless the gravitational effects can be shown to be negligible in comparison to the observed differences in potential. These ideas are illustrated in Figures 1 and 2.

In Figure 1, the arrows show the directions of increasing values of $|\Psi_\rho - \Psi_{\rho_{ave.}}|$. In general, information about the aquifer is insufficient to enable us to evaluate the quantity $|\Psi_\rho - \Psi_{\rho_{ave.}}|$ for possible flow paths within the aquifer. Therefore, we are led to the conclusions in Figure 2.

For aquifers in which the differences in head that are caused by gravitational effects are negligible in comparison to the observed differences, areas A and B (Fig. 2) disappear; that is, ordinary potentiometric maps, corrected for average density changes, give valid conclusions about flow, regardless of the other properties of the aquifers.

DISPLACEMENT OF DENSE WATER BY LIGHT WATER IN AN AQUIFER

When dense water is displaced by a lighter water as a result of natural forces, most of the flow that takes place occurs near the roof of the aquifer. The lighter water rides along the roof of the aquifer because the vertical force due to difference in water density is usually much greater than the force available to cause lateral movement. For example, a difference in relative density of 0.02 units yields 1 ft of vertical head difference per a 50-ft difference in elevation; a horizontal gradient of 2.5 ft/mi gives about 1 ft head difference over a horizontal distance of 2,000 ft. Thus, the ratio of vertical to horizontal forces is about 40 to 1 for this case.

When flow takes place through a trough, dense water is trapped in the downstream side of the trough unless the tilt angle (Hubbert, 1953, p. 1994) exceeds the dip of the bottom of the trough. The tilt angle has been calculated for various combinations of densities of water, using representative values of gradient in head. For example, for a gradient of 3 ft/mi, relative density differences of 0.01 and 0.03 yield tilt angles of 3°16' and 1°5', respectively. Therefore, troughs with little slope can trap water whose density is not greatly different from the density of the displacing water.

Flow through a series of troughs is illustrated in Figure 3. The tilt angles θ_1, θ_1', θ_2, θ_2', etc., depend on the local gradient, which is difficult to predict. However, the local gradient should be less than the overall average gradient and, therefore, actual tilt angles should be less than those calculated from the average gradient.

A trough (or a series of such troughs) containing dense water acts like a valve; no flow occurs through a series of troughs unless the head difference across the series is greater than the sum of the head differences across the individual troughs:

$$\left[\left(\rho_{D_1} - \rho_{L_1}\right)\Delta Z_1 + \left(\rho_{D_2} - \rho_{L_2}\right)\Delta Z_2 + \ldots + \left(\rho_{D_n} - \rho_{L_n}\right)\Delta Z_n\right],$$

where ρ_{D_1} and ρ_{L_1} are the relative densities of the dense and the light waters in the first trough and ΔZ_1 is the depth of the first trough, etc., and n is the number of troughs. The same principle applies if the direction of flow is reversed. Dense water is trapped at the bottom of any trough; it cannot be displaced completely, even if the direction of flow is reversed, as long as the tilt angle does not exceed the dip of the bottom of the trough. Even if water does flow through the series of troughs, the head that is available to cause flow is diminished by the amount:

$$\left[\left(\rho_{D_1} - \rho_{L_1}\right)\Delta Z_1 + \left(\rho_{D_2} - \rho_{L_2}\right)\Delta Z_2 + \ldots + \left(\rho_{D_n} - \rho_{L_n}\right)\Delta Z_n\right].$$

Structures such as saddles and anticlines also can change the head available for flow (Bond, 1972, p. 17). Any attempt to use observed head differences to predict flow must take into account all structural influences.

Thus, as light water displaces a denser water in part of an aquifer, gravitational effects can cause a head difference. If the cumulative effects of troughs and various structures build up enough head difference to balance the imposed head difference, water ceases to flow in that part of the aquifer. Therefore, a difference in $H^{1.00}$ does not necessarily cause flow. Likewise, a difference in potential is not necessarily associated with flow.

EXAMPLES OF AQUIFER ROCK TYPES IN WHICH APPRECIABLE GRAVITATIONAL EFFECTS CAN BE EXPECTED

The gravitational effects that are caused by long-range, gradual changes in water density--type 3(a)--pose no serious problem; these can be handled in the manner outlined. The primary concern is with the local effects of structure and of permeability barriers within the aquifer--that is, types 3(b) and 4--which appear to have been ignored in most flow studies. That is, we are concerned with gravitational effects in aquifers in which $|\Psi_{\rho} - \Psi_{\rho_{ave.}}|$ for pairs of points along possible flow lines is appreciable in comparison to observed differences in head.

Homogeneous rock does not commonly extend over any considerable part of a natural aquifer. Heterogeneity is the rule rather than the exception. Therefore, in many variable-density aquifers, gravitational effects of types 3(b) and 4 should occur. However, we need to know the magnitude of the effects that can be reasonably expected. Are the gravitational effects that are caused by structure and by permeability barriers appreciable in comparison to observed potentials and heads, or are they negligible and therefore of only academic interest?

Examples of types of aquifer rock in which appreciable gravitational effects can be expected if the density of the water is variable are considered in the following paragraphs. The types of rock discussed are blanket sandstones that contain shale beds and other barriers, deltaic deposits, and carbonate rocks that contain solution channels.

Blanket Sandstones

The Mount Simon Sandstone is an example of a blanket sandstone. The upper part of the Mount Simon aquifer (Suter et al., 1959) is a transitional zone consisting of the lower part of the Eau Claire Formation and the upper part of the Mount Simon Sandstone. In this transitional zone, sandstones and shales are distributed in an irregular fashion, up to a total thickness of 400 ft or more. Data from pumping tests and gas-injection tests indicate that the sandstones are generally hydraulically interconnected.

Troughs within aquifer--Evidence exists for the presence in the Mount Simon aquifer of tilted impermeable strata that may extend laterally for a considerable distance. These tilted strata, more or less randomly distributed, are barriers that prevent the horizontal flow of water. They cause water to flow downward and then up, over and over again, as the water is transported from one part of the aquifer to another. The assemblages of shale beds and other impermeable strata constitute troughs or U-tubes that can cause differences in head if the density of the water is variable.

One may argue that, even though barriers to horizontal flow are present, tortuous horizontal paths around these barriers can be foun

which permit some flow. This may be true for some parts of the aquifer, but, even so, the barriers decrease the overall, gross rate of transport of water from one part to another part of the aquifer in two ways. First, the barriers restrict the size of the paths available for flow; second, since the water must follow a tortuous, meandering path, the effective gradient in head is decreased. The net effect, a decrease in the overall rate of water transport, is the same as that which would be caused by one or more troughs.

In the central part of northern Illinois, the water in the Mount Simon aquifer is stratified. On the average, the relative density of the water changes about 0.03 units per thousand feet of change in elevation; locally, the change in density may be greater than this (Bond, 1972).

Estimates have been made of the head differences that could be caused by troughs in an aquifer like the Mount Simon aquifer when reasonable assumptions are made about: (1) the dips of the barriers that restrict horizontal flow, (2) the size of these barriers, and (3) the contrast between the densities of the native water and the displacing water as the aquifer is invaded by relatively fresh water. Also, calculations of the tilt angle were made for the average gradient in $H^{1.00}$ observed in the northern part of the Illinois basin (3 ft/mi); these calculations were made in order to determine whether the denser water would be trapped or flushed out of the trough under the assumed conditions.

The results of these calculations showed that one could reasonably expect the presence of troughs to cause a gradient in $H^{1.00}$ equal to a few feet per mile in an aquifer like the Mount Simon. This estimate is conservative. For all we know, troughs that contain quite dense, residual, unflushed brine may be scattered throughout the aquifer. In an aquifer with water that contains, say, 200,000 mg/l total dissolved solids, a 20 percent difference in salinity gives a density contrast of 0.026 units; with such a contrast, a single trough that is only 100 ft deep will produce a head difference of 2.6 ft.

Structure at top and bottom of aquifer--In addition to the influence of troughs within the aquifer, the influence of structure on $H^{1.00}$ should be evaluated. Along the northern part of the La Salle anticlinal belt, in the area considered above, several anticlines are present; in the Mount Simon aquifer, these anticlines have closures up to 250 ft. From one anticline to another the relative density of the water appears to change about 0.01 to 0.02 units. Thus, at the same elevation, at points near the tops of two adjacent anticlines, the values of $H^{1.00}$ can differ by about 2.5-5.0 ft, even if the water is not flowing. Since these anticlines are about 3-5 mi across, this difference is equivalent to a gradient in $H^{1.00}$ of roughly 1 ft/mi. Of course, larger, more sharply folded anticlines, spaced more closely and exhibiting larger differences in water density, can generate larger gradients in $H^{1.00}$.

The total effect of troughs within the Mount Simon aquifer and of structure at the top of the aquifer can easily result in gradients of a few feet per mile in $H^{1.00}$. These gradients are comparable in magnitude with the average gradients that are actually observed. Density differences in the water in troughs and in various structures can cause much, if not all, of the observed variation in $H^{1.00}$.

The Mount Simon is a relatively simple blanket sandstone. Aquifers that are more complex in structure and lithology, and that exhibit grosser variations in water density, can be expected to show even greater gravitational effects than does the Mount Simon.

Deltaic Deposits

"The delta system is one of the most complex of depositional systems" (Scott and Fisher, 1969, p. 10). The publication by Fisher et al. (1969) contains numerous illustrations that demonstrate the complexity of the delta system. In the illustration adapted here as Figure 4, the sections that are shown are parts of systems that can be thousands of feet thick.

Delta systems are complex not only in their lithology but also in their hydrology. "Deltas result in juxtaposition of facies with contrasting formation waters...." (Fisher and Brown, 1969, p. 54).

Such systems present ideal conditions for gravitational phenomena of the kind discussed above. The complexity of the lithology prevents horizontal flow over any considerable distance. When water does flow, it travels up and down in irregular, "corrugated" paths through a series of troughs of various sizes and configurations. Because the interstitial waters have different salinities in different parts of the system, gravitational differences in head are produced; these head differences can influence substantially the possibilities for flow. A single 100-ft deviation from horizontal flow, along with a 20 percent variation in salinity, can produce a head difference of 2 or 3 ft.

Therefore, in a deltaic deposit a gradient in head is not necessarily associated with flow. Where flow does occur, the flow rate is less than might be expected on the basis of the observed heads. On the other hand, even though the water in a given deltaic region is static, head differences

that are caused by gravitational effects can result in a gradient.

Carbonate Rocks Containing Solution Channels

Solution channels that are associated with karst topography in carbonate rocks result in irregular, random flow paths. Although we never know in detail where these channels are located, we can be sure that only rarely are they perfectly horizontal. For example, they can be expected to resemble the channels shown diagrammatically by Brown (1966; see Fig. 5, this paper).

Where rocks containing such channels are buried and become part of a variable-density aquifer, many possibilities for gravitational effects should exist. Generally we do not have enough information to enable us to evaluate these effects. However, if the channelized rock has considerable thickness, the head differences due to gravitational effects can become appreciable. One can easily visualize troughs several hundred feet or more deep; where moderate variations in salinity occur, these troughs can result in gravitational head differences of several feet per mile.

The rock types that have been cited are only a few of the types in which appreciable gravitational effects should be expected. The imaginative engineer or geologist will be able to add other examples taken from his own experience.

CONCLUSIONS REGARDING GRAVITATIONAL EFFECTS

In moderately complex aquifers, for example, sandstones that contain tilted shale stringers, gradients in head up to about 5 ft/mi can be expected to be caused by gravitational effects. In aquifers that have more complex lithology and structure or that show rapid changes in water density, gravitational gradients up to 10 ft/mi or more can occur. Only in aquifers where the observed gradient is large, on the order of 50-100 ft/mi, can the gravitational effects of troughs and structure be neglected; gradients of this magnitude are found only in intermontane basins (Hubbert, 1953, p. 1992). Thus, the results of many flow studies based on potential measurements are open to question. The conclusions that are derived from such studies are valid only if all of the gravitational effects have been evaluated and taken into account or if they can be proved to be negligible.

APPARENT ANOMALIES EXPLAINED BY GRAVITATIONAL EFFECTS

As stated, when fresh water invades a brine-filled aquifer, inhomogeneities in the rock cause the water to flow in a "corrugated" flow path which is the equivalent of a number of U-tubes, or troughs, in series. Each trough, containing light water in the upstream side and a more dense water in the downstream side, introduces a certain amount of head difference in opposition to flow. When the head difference due to the troughs in a given section of the aquifer is equal to the total head imposed across that section, water ceases to flow. Thus, an observed gradient in head can be a "fossilized" or relict gradient that was caused by partial flushing of brine from that section of the aquifer many years ago. A gradient can be evidence that flow is occurring, or it can be evidence that flow has occurred at some time in the past.

These conclusions can be used to explain several anomalous situations that have been encountered in hydrodynamic studies:

1. Tilted water-oil interfaces common in the Mid-Continent, where the water is salty and evidence for regional flow is absent (Dickey and Hunt, 1972; Russell, 1956);

2. Areas where brine remains unflushed in aquifers where potentiometric maps indicate flow of relatively fresh waters (Harms, 1966);

3. What appears on some potentiometric maps to be an impossible situation in which water flows from all directions toward a low-pressure area that has no exit (Dickey and Hunt, 1972; Hitchon, 1969b);

4. The local flow directions which appear to vary in almost random manner in the Mount Simon aquifer in some areas in northern Illinois (based on observed variations in head; Bond, 1972);

5. Failure of the thickness of the gas bubble to increase with time in the manner anticipated in certain gas-storage reservoirs (Evrenos and Comer, 1972; Chaumet et al., 1966; Katz and Coats, 1968).

As an example of the use of gravitational effects in explaining these anomalies, consider the first anomaly listed above. The gradient in head is a measure of the force that is exerted on the fluids in an aquifer; the cause of this force is immaterial. Therefore, a "fossil" gradient due to gravitational effects in a static aquifer can cause a tilt in an oil-water interface just like the tilt that is caused by a gradient that is produced by the flow of water through the aquifer.

Space does not permit a discussion of the other anomalies listed. However, the explanations for some of them should be apparent to the reader who understands the principles presented herein.

UNDERGROUND INJECTION OF WASTE LIQUIDS

In planning for the underground injection of waste liquids, we need to consider the density of the injected liquid relative to the density of the native interstitial water. If the density of the injected liquid is less than that of the native water, the injected liquid will rise under the influence of gravity. Therefore, the presence of an impermeable confining bed above the injection zone will be required; furthermore, the injected liquid will flow along the bottom of this confining bed until it reaches a structural high. If the density of the injected liquid is greater than that of the native water, however, the injected liquid will sink and will flow along the bottom of the aquifer until it reaches a structural low (Bond, 1972, p. 46).

If we want to insure that an injected waste liquid will not rise toward the surface, we must make the liquid as dense as possible. For example, we might mix the waste with a saturated brine, or we might add to the waste a solute that would increase its density.

We may be able to minimize the possibilities for flow under the influence of existing natural head differences. For example, it may be possible to inject the waste liquid into an aquifer where, because of gravitational effects due to water-density variations in inhomogeneous rock, little or no flow is possible.

CONCLUSIONS

1. In a variable-density aquifer, the head that is available to cause flow is influenced by the gravitational effects that are caused by (a) structure of the top and bottom of the aquifer (troughs, saddles, anticlines, synclines), and, (b) troughs that are formed by nonhorizontal barriers within the aquifer. Usually these gravitational effects decrease the available head; they may even cause flow to cease.

2. In a variable-density aquifer, valid conclusions about flow can be made only if the gravitational effects can be evaluated or can be proved to be negligible in comparison to the observed variations in head.

3. Where $|\Psi_\rho - \Psi_{\rho_{ave.}}|$ is appreciable in comparison to observed head differences, even if correcitons are made for the average rate of change of density from one observation point to another, a gradient in potential is not necessarily associated with flow. A gradient can be evidence that flow is occurring, or it can be evidence that flow has occurred at some time in the past.

4. In a variable-density aquifer, the interface between water and oil or gas deposit can be tilted even if the water under the deposit is static.

5. In a variable-density aquifer, gravitational effects can cause a potential sink--troughs in the region around the sink can cause a potential low--even if the water in the aquifer is static. Exotic explanations for the presence of potential sinks, such as those based on osmotic pressure, may not be needed.

6. In some aquifers, brine is in contact with relatively fresh water that apparently has been flowing for many years. Probably the reason this brine has not been flushed out is that the gravitational effects in the brine-saturated region have caused head differences that have opposed the impressed head difference; the brine has been static, or it has been flowing at a reduced rate in comparison to the rate in the less dense water in the rest of the aquifer.

7. In intermontane regions, gravitational effects that are caused by variations in water density probably can be neglected. In other regions, where gradients in head are on the order of 5-10 ft/mi, gravitational effects can be appreciable; in these regions, conclusions concerning flow based on potentiometric studies may be of doubtful validity.

8. If an injected waste liquid is more dense than the native water in the injection zone, or if it can be made more dense, possibilities for upward flow of the injected waste can be minimized.

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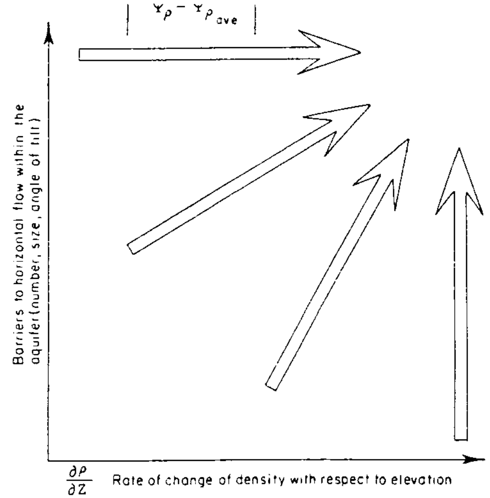

FIG. 1.

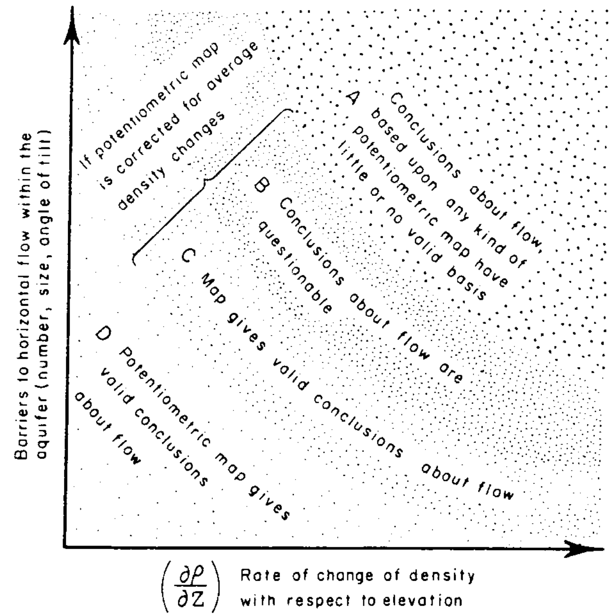

FIG. 2.

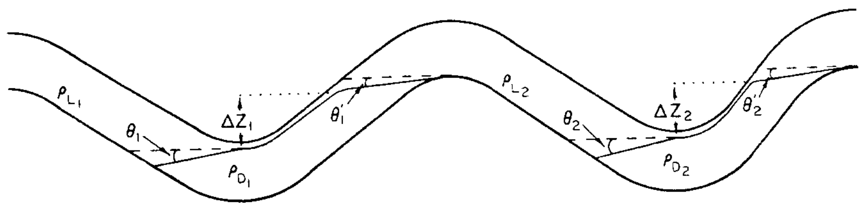

FIG. 3.

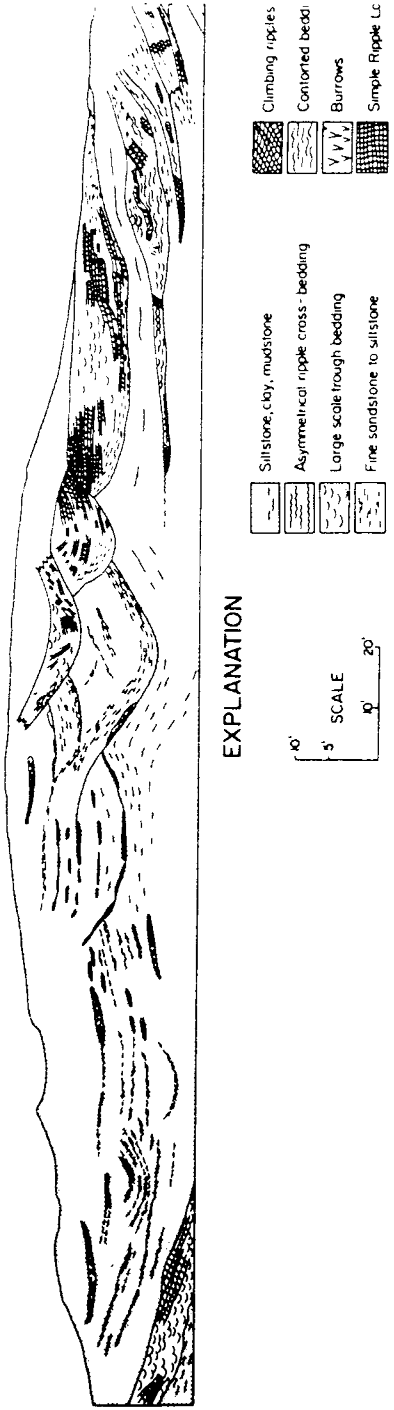

FIG. 4--Deltaic deposit (adapted from Kessler, 1969, Fig. 7).

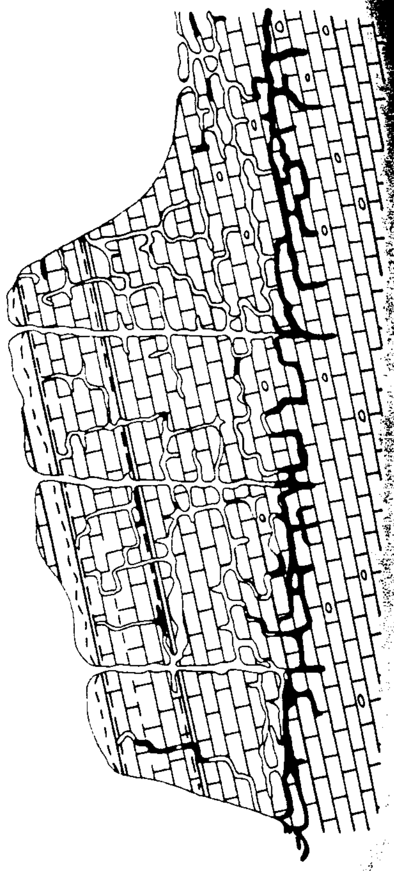

FIG. 5--Diagrammatic section through the Mammoth Cave plateau (adapted from Brown, 1966, ...

UNDERGROUND STORAGE AND RETRIEVAL OF FRESH WATER FROM A BRACKISH-WATER AQUIFER[1]

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ABSTRACT In 1967, the U. S. Geological Survey, in cooperation with the City of Norfolk, Virginia, began a study of injection of fresh water into a confined aquifer containing brackish water. The objectives of the study were (1) to determine whether the host formation would accept large volumes of fresh water, (2) to determine the degree of mixing of fresh and saline water, and (3) to determine the percentage of recoverable fresh water after long periods of storage.

During late 1971 and early 1972, three injection and withdrawal tests were conducted. In test 1, fresh water was injected at a rate of 400 gpm. The specific capacity of the well decreased from an initial value of 15.4 gpm/ft of drawdown to 9.3 gpm/ft after 260 minutes of injection. In test 2, the initial injection rate of 400 gpm decreased to 215 gpm after 7,900 minutes, and the specific capacity dropped from 14.2 to 3.7 gpm/ft. In test 3 the injection rate decreased from 290 gpm to 100 gpm within 150 minutes, and to a low of 70 gpm after approximately 1,300 minutes. The specific capacity decreased from 3.7 to 0.93 gpm/ft.

Specific capacity during redevelopment pumping was 19.7 gpm/ft after injection test 1, but only 6.7 gpm/ft after test 3. All attempts at redevelopment of the injection well failed to improve the specific capacity. Current-meter surveys conducted during injection and withdrawal indicated that the reduction in flow rate and specific capacity was due to a uniform reduction in permeability throughout the aquifer.

All the hydraulic data collected during the tests indicated a physical change of the formation materials. It was felt that the uniform loss of specific capacity was due to clay dispersion. Chemical data indicated that the sodium-rich clays were involved in cation exchange. During injection, calcium and magnesium replaced sodium on the clays. During withdrawal, this reaction was reversed. The net effect of the cation exchange was to decrease very slightly the tendency of the clays to disperse during injection. The exchange activity noted during all three tests did indicate that the clays would respond readily to chemical treatment to decrease or eliminate clay dispersion.

In injection test 4, a pre-flush of 3,000 gal of 0.2 normal calcium chloride was injected before the fresh water. The initial specific capacity was 4.3 gpm/ft as compared with the final specific capacity of 0.93 gpm/ft in test 3. Redevelopment pumping during injection improved the specific capacity to 5.3 gpm/ft. After 4 million gal had been injected, an additional 3,000 gal of 0.4 normal calcium chloride solution was injected. The injection specific capacity increased to 7.12 gpm/ft.

The injection specific capacity decreased with time because only the area around the borehole was treated to prevent clay dispersion, and partly because of clogging of the screen and gravel pack by sand and clay particles. The data from test 4 indicated that a maximum injection rate could be maintained by injecting for periods of 1,200 and 1,400 minutes, then withdrawing water for 30 minutes to clear the injection zones.

A total of 20 million gal of fresh water was injected in test 4. The water was left in the aquifer for about 8 weeks before beginning the withdrawal phase. It had been determined from the first three injection tests that about 85 percent of the injection water could be recovered and remain within the drinking standards of the U. S. Public Health Service.

INTRODUCTION

The water supply for the city of Norfolk, Virginia, comes from surface reservoirs in the independent cities of Nansemond, Norfolk, and Virginia Beach. During the winter months, when water demand is low and reservoirs are full, water must be diverted from the reservoirs and allowed to escape to the ocean. It has been estimated (Schweitzer, oral commun., 1970) that as much as 2.5 billion gal of water per winter quarter could be available for consumptive use, if sufficient storage areas were available.

The U. S. Geological Survey and the City of Norfolk entered into a cooperative program to determine if it would be possible to utilize the water presently flowing to waste, process it in the treatment plants, and store it underground in aquifers now containing saline water. The fresh water would be retrieved during the summer months when peak water demands and low water levels in reservoirs place strains on present water-system delivery capabilities.

Location

The Norfolk injection project is at Moore's Bridges Filter Plant, Norfolk, Virginia (Fig. 1). Excess water from the filter plant supplies the injection project. The water is taken from the treatment system after it has been chlorinated, settled, and filtered, but prior to the final chlorination and liming.

Previous Work

The first attempt at artificial recharge of fresh water into brackish-water aquifers in the Coastal Plain of Virginia was conducted by D. J. Cederstrom (Cederstrom, 1957) in 1946 at Camp Peary. During that experiment, water was injected into a brackish-water aquifer over a period of 85 days. The well into which the water was injected had screens in the intervals of 430-440 ft and 450-475 ft below land surface. The well clogged during injection, but the withdrawal phase was encouraging in support of the feasibility of the concept.

WELL FIELD

The well field (Fig. 2) consists of the injection well (IW-2), and four observation wells. The observation wells are designated as follows: annular-space well (ASW); observation well 3 (OW-3); observation well 2 (OW-2); and test well 1 (TW-1). All wells, with the exception of TW-1, were completed with fiberglass casing and fiberglass or stainless steel screens in order that they would be chemically inert.

Injection Well 2 (IW-2)

The design and construction of the injection well is shown in Figure 3. The wall thickness of the 18-in. casing is 0.5 in. and it has a theoretical ultimate collapse strength of 70 psi. The wall thickness of the 8-in. casing is 0.40 in. and the ultimate collapse strength is 300 psi. The lowest sections of both the 8-in. and the 1 1/2-in. fiberglass casings have stainless steel screens nipples wound into the fiberglass. The stainless steel screens are welded to the nipples with stainless steel welding rods in keeping with the use of noncorrosive materials in the well construction. Eighty feet of wire-wrapped, 40-slot, stainless steel screen is attached to both the 8-in. and 1 1/2-in. casings. Both screens have 10 ft of blank stainless steel pipe at the bottom to act as sand traps. The slot size of the screens was determined after mechanical analysis of the sand recovered from cores from the injection zone. The screens were set from 896 to 976 ft below land surface and they extend through the entire injection zone.

A 4.37-in. inner diameter (ID) fiberglass instrument-access pipe (Fig. 3) extends below the pump bowls and is used for insertion and withdrawal of the current meter and other monitoring equipment as needed. Water is injected into the aquifer through the 4.37-in. fiberglass line that enters the 18-in. casing below the pump bowls.

Injection Well 1 (IW-1)

IW-1 was the initial attempt at constructing an injection well. The design was the same as IW-2. The 18-in. casing collapsed at a depth of about 51 ft below land surface, prior to cementing the casing in place. Salvage attempts failed and the well was declared abandoned and filled with cement. The well is 25 ft northwest of IW-2 and is in the same plane as IW-2, OW-3, and OW-2.

Observation Wells

Annular-space well (ASW)--The purpose of the 1 1/2-in. annular-space well is to monitor head changes in the gravel pack of the injection well during injection and withdrawal tests. To accomplish this, the screen was attached parallel to, but separated from, the screen of the injection well (IW-2) by 1-in. wooden blocks. The specifications of the screen are: 1 1/4 in., heavy duty, wire wrapped, 40 slot, stainless steel. The entire injection zone is screened from 896 to 976 ft below land surface with a 10-ft fill-up pipe from 976 to 986 ft that is a sand trap.

Observation well 3 (OW-3)--OW-3 is 50 ft southwest of the injection well. The casing and screen are 4.37-in. ID epoxy-resin fiberglass with a wall thickness of 0.25 in. The screen was made by sawing horizontal slots 0.05 in. wide in a regular section of casing. The entire thickness of sand is screened from 900 to 981 ft below land surface and there is a 10-foot fill-up pipe from 981 to 991 ft below land surface. The well is gravel packed to a height of 60 ft above the screen and is cemented from that point upward to land surface.

Observation well 2 (OW-2)--OW-2 is 75 ft northeast of the injection well and lies in a plane through IW-2 and OW-3. The construction of OW-2 is identical to OW-3 with the exception that 90 ft of saw-slotted screen was installed. The entire thickness of sand is screened from 900 to 990 ft below land surface, and there is a 10-foot section of fill-up pipe from 990 to 1,000 ft below land surface.

Test well 1 (TW-1)--TW-1 is 415 ft northwest of the injection well. TW-1 was a test well originally drilled to a depth of 2,587 ft (Brown, 1971a). It was backfilled with cement to 1,000 ft below land surface and was completed as an observation well partly penetrating the injection zone. The hole was underreamed to a diameter of 24 in. from 890 to 970 ft below land surface. Sixty feet of 6-in. stainless steel shutter screen (0.055-in. openings) was set from 900 to 960 ft below land surface with a 10-foot fill-up pipe from 960 to 970 ft. Six-inch mild steel casing extends from the screen to land surface. The casing is cemented from the top of the gravel pack to the surface.

GEOLOGY AND HYDROLOGY OF THE INJECTION SAND

The aquifer chosen for the injection zone is a moderately sorted, angular to subangular, fine- to medium-grained, poorly cemented quartz sand of Cenomanian-Albian age (Brown, 1971b). Wood fragments, ostracods, and forams recovered in cores, as well as textural features of the sand, suggest that it is of marine origin and was deposited in a littoral environment, possibly a tidal flat. The size and angularity of the quartz grains indicate a lower energy level than would be expected in an open beach or bar environment, and probably indicate a semi-protected shoreline.

During coring attempts in TW-1, the sand in the injection zone was found to be so unconsolidated that standard wire-line coring techniques did not obtain satisfactory core recovery. For this reason, a special "rubber-sleeve" core barrel was employed to core the injection zone in OW-3. The tool is designed to take a core in unconsolidated material by encasing the core in a rubber sleeve as the core is cut. Selected intervals of the core recovered were submitted to the Geological Survey laboratory, Denver, Colorado, for determination of clay type, grain size, porosity, permeability, and identification of mineral content of the sand.

The injection zone is bounded above and below by laterally persistent silty-clay to clayey-silt facies that act as confining beds. The presence of confining beds simplifies analysis of the injection hydraulics to some extent, as the problem is effectively reduced to two dimensions. The clay in the confining beds was identified by X-ray analysis as a multilayer mixture of illite and montmorillonite, plus montmorillonite and minor amounts of kaolinite (Table 1).

Transmissivity values varied within the well field depending upon the sand development at the well site. Analysis of distance drawdown plots indicated that the transmissivity ranges from as low as 5,360 sq ft/day in the injection well to as high as 16,600 sq ft/day in areas of maximum sand thickness. The average hydraulic conductivity for the injection well is 0.0613 cu ft/minute/sq ft. The storage coefficient for the injection sand is 1.4×10^{-4}.

The formation water in the injection zone is brackish, with a dissolved-solid content of 3,010 mg/l. It is a sodium chloride-bicarbonate type water with a chloride content of 1,360 mg/l.

INJECTION SYSTEM

All pipes and valves are constructed of schedule 80 polyvinyl chloride. All waterways in the pumps are either rubber lined or epoxy coated so that at no place in the system is there iron in contact with the injection or withdrawal water, except as stainless steel. This was done to isolate iron in solution in the water to a specific source--the aquifer.

The city water is injected by a 10-horsepower centrifugal pump that is rated at 500 gpm (gallons per minute) against a total dynamic head of 30 ft. It operates at 800 rpm (revolutions per minute). The pump will deliver 800 gpm to free discharge at the well house, which is located 230 ft from the centrifugal pump vault. The pump vault is next to a 60-in. concrete line from which the injection water is supplied. The centrifugal pump has a rubber liner so that the city water is not in contact with any ferrous metal surfaces prior to injection.

The injected water is withdrawn by a 30-horsepower, 10-in., vertical hollow-shaft turbine pump rated at 800 gpm against a total dynamic head of 105 ft. It has 120 ft of 8-in. epoxy-coated pump column with a 4-stage stainless steel pump-bowl assembly. All waterways are epoxy coated.

INJECTION TESTS

It was originally planned to conduct successive injection tests with the emphasis on injection of large quantities of water and on long storage periods prior to withdrawal. Modification of these plans became necessary because the injection specific capacity of the aquifer became so low during injection test 3 that it was impractical to continue the test beyond 1,100 minutes of injection.

Injection Test 1

The purpose of injection test 1 was to test the flow system and sensing apparatus, to determine the head buildup that would occur during long-term tests, to detect errors in calibration limits set for the monitoring equipment, to determine chemical reactions occurring during injection and withdrawal, and to determine the number of people required to conduct the tests. In accordance with these objectives it was decided that the initial injection test would be limited to 8 hours.

Injection was begun at 0900 hours on Nov. 22, 1971, at a rate of 400 gpm. This injection rate was held for 270 minutes, after which the injection pump was shut down to allow insertion of the current meter into the screen of IW-2. After a shutdown of 35 minutes, injection continued at 400 gpm for an additional 205 minutes until shutdown occurred at 1730 hours on Nov. 22, 1971. No redevelopment pumping was attempted during the injection of the water. A total of 198,320 gal of city water was injected into the brackish-water aquifer. The water remained in place for 15.5 hours before withdrawal began.

Withdrawal of the injected water began at 0900 hours on Nov. 23, 1971, at a rate of 710 gpm. The withdrawal rate gradually increased to 730 gpm by the end of the withdrawal phase (330 minutes of withdrawal) of test 1. A total of 250,210 gal of water was withdrawn--26.16 percent more water was removed than was injected. Current-meter traverses of the screen in IW-2 were made during both injection and withdrawal, and conductivity profiles of OW-3 were made during the injection phase.

Injection Test 2

The purpose of injection test 2 was to verify the chemical and physical data obtained in test 1. Although the results of test 1 were encouraging, the time span was so small that valid conclusions about long-term testing could not be made. Injection test 2 was designed to inject approximately 10 times the quantity of water of test 1 and to leave it in the aquifer for 48 hours. Conductivity surveys of the screen of the nearest observation well (OW-3) were planned to determine the time of arrival of the injected water and to define the shape of the injection front at OW-3.

Injection test 2 began at 1400 hours on Feb. 14, 1972. The initial injection rate was 410 gpm, but it gradually decreased to 215 gpm after 7,800 minutes of injection. Injection was interrupted for 250 minutes after 5,340 minutes of continuous injection, because of repairs to the city filter plant. The injection phase of test 2 concluded after a total of 7,906 minutes (5.49 days) of pumping. A total of 2,445,530 gal of treated city water was injected into the brackish-water aquifer without any redevelopment pumping during the injection period. The water remained in the aquifer for 50 hours.

Withdrawal of the injection water began at 1100 hours on Feb. 22, 1972, and ended Feb. 26, 1972, at 1100 hours. A total of 3,504,100 gal of water was withdrawn--43.28 percent more than was injected. Current-meter traverses of IW-2 and conductivity profiles of OW-3 were made during injection and withdrawal.

Injection Test 3

The purpose of injection test 3 was to verify data obtained in tests 1 and 2 and to obtain additional chemical and physical data. The results of test 2, as well as those of test 1, were encouraging regarding the practical use of the aquifer for storage of fresh water. The decrease in the injection rate and excess head build-up near the end of test 2 were subjects of concern. Injection test 3 was made to inject about 10 times the quantity of water used in test 2 and to leave it in place a minimum of 2 weeks. Conductivity surveys of OW-3 were planned to compare the shape of the injection front and arrival time with the data of test 2.

Injection test 3 began at 1600 hours on April 13, 1972. The injection rate dropped from an initial rate of 400 gpm to 115 gpm within 80 minutes, and gradually decreased to a low of 70 gpm. Within 20 minutes, pressure on the discharge side of the injection pump rose from a normal pressure of 12 psi to the maximum value of 20 psi. After 1,174 minutes of constant injection it was apparent that the aquifer would not take water in sufficient quantities to continue the test.

A total of 132,700 gal of fresh water had been injected. In attempts to redevelop the well, water was alternately withdrawn and injected for a period of about 5 hours, resulting in an additional 13,300 gal of water injected, for a total of 146,000 gal of fresh water injected into the aquifer. The water remained in place for 76.5 hours.

Withdrawal of the injection water began at 1024 hours on April 17, 1972. A total of 318,000 gal of water was withdrawn--46 percent more than was injected. This total includes almost 95,000 gal recovered during redevelopment attempts during injection. Current-meter traverses of IW-2 were made during injection and withdrawal.

At this point, a total evaluation of what was occurring within the aquifer had to be made before any additional injection tests could be attempted.

AQUIFER ALTERATION

It became increasingly apparent during the injection phase of test 3 that aquifer alteration was occurring because of injection of the fresh water. Alteration to the aquifer and a resulting decrease in hydraulic conductivity was reflected by: (1) excessive head buildup in the injection well and nearby observation wells, (2) alterations in flow gradients between wells, and (3) low rates of injection into the aquifer.

It has been shown by many investigators (Baptist and Sweeny, 1955; 1957; White et al., 1962, 1964; Hewitt, 1963; Meade, 1964; Gray and Rex, 1966; Reed, 1972) that a reduction in permeability of some sand aquifers occurs when the salinity of the pore water is altered. The reduction is greatest if the salinity is greatly reduced. A sand that exhibits this tendency is described as "water sensitive."

The majority of alteration done to the water-sensitive aquifer is usually a result of physical movement of interstitial clay particles rather than clogging by chemical precipitation. Land and Baptist (1965) have demonstrated that most of the reduction in permeability is due to dispersion of clay particles rather than to in-situ swelling of the clays.

Evidence of clay-dispersion clogging versus in-situ swelling is mostly indirect, but there are two criteria that may be used to determine which process is involved: (1) reduction in permeability due to swelling would be mostly reversible when original conditions were restored, whereas alteration due to clay disperison is largely irreversible. As the clay particles are dispersed, they move until they become lodged in pore constrictions, causing clogging. Returning the water chemistry to original conditions will not repack the clay in its original position. (2) If a section of core of the aquifer is saturated with formation water, then flushed with fresh water, a milky, turbid effluent and decreasing permeability usually result if clay dispersion is occurring.

The mechanism for clay dispersion has been reported in detail by Meade (1964), Jones (1964), Gray and Rex (1964), and Reed (1972) and will be discussed here only in general terms. Clay dispersion is predominantly a result of electrokinetic properties. The electrostatic attraction between negatively charged clay particles and exchangeable cations is opposed by the tendency of the ions to diffuse and become uniformly distributed throughout an aqueous solution. When the electrostatic attraction is greater than the tendency to diffuse, flocculation occurs, and, conversely, dispersion occurs when diffusion forces are more powerful.

Meade, Gray, and Reed all agree that one of the most significant factors causing dispersion is a change in the double-layer thickness surrounding a clay particle. The double layer consists of a negative charge on the surface of the clay particle, and a second layer formed near its surface, consisting of exchangeable cations.

The thickness of the double layer is altered by increasing or decreasing the electrolyte concentration in the pore water. Decreasing the concentration by injecting fresh water into brackish water will in-

crease the thickness of the double layer. This increases the range and effectiveness of the forces of repulsion between clay particles and causes them to disperse and migrate.

The negative charge responsible for dispersion of clay particles is the same for a given particle regardless of the exchangeable cation. However, the tendency to disperse (zeta potential), which determines the effective charge repelling a second clay particle, is determined by the tightness with which the cation is held.

Cations that are held very closely to the clay-mineral surfaces because of their size (ionic radius) or hydration state tend to reduce the zeta potential and allow the particles to come together (flocculate). Thus, trivalent (Al^{+++}) and divalent (Ca^{++}) cations, both of which are more tightly held than monovalent (Na^{+}) cations, form a clay-cation system with a low zeta potential. Highly hydrated ions, such as sodium, result in a clay-cation system with high zeta potential. The water of hydration prevents the cation from being closely adsorbed. These facts may account for the tendency of sodium to cause dispersion of clay colloids and for calcium to cause flocculation.

Sand containing montmorillonite and mixed-layer clay is usually the most water-sensitive. As little as 0.4 percent montmorillonite has caused a 55 percent reduction in permeability (Hewitt, 1963, p. 817). The clay present in the injection-zone aquifer, as shown by X-ray studies (Table 1), has sufficient quantities of montmorillonite, illite, and mixed-layer clay to account for the reduction in hydraulic conductivity that occurred during injection tests 1, 2, and 3.

Head Buildup

The first evidence of formation alteration due to injection of the fresh water was the excess head buildup that occurred in the injection well during injection test 1. The expected head buildup in each well was determined by a preinjection aquifer test in which IW-2 was pumped at 400 gpm for 8 hours. The negative of the drawdown measured in each well was used to predict head buildup.

<u>Excess head buildup due to temperature and viscosity</u>--In the pumping well and annular-space well, the effects of density and temperature were taken into consideration in calculating the expected head buildup, using a method described by G. D. Bennett (written commun., 1969). The formulas for the radius of intrusion of the injection and for the excess head buildup are as follows:

$$r_i(t) = \sqrt{\frac{Q\ (t-t_c)}{\pi D\theta} + (r_w)^2}, \quad (1)$$

where

$r_i(t)$ = the radius in feet of the injection water (colder water) at time (t);

Q = the flow rate in cubic feet per minute;

$t-t_c$ = the time that the colder water has been in the aquifer, in minutes;

D = the length of screen, in feet;

θ = porosity, expressed as a decimal;

$(r_w)^2$ = radius of well, in feet;

and

$$S_{wTt}-S_{wt} = \left(\frac{1}{K_{1c}} - \frac{1}{K_1}\right) \left(\frac{2.3Q}{2\pi D}\right) \left(\log \frac{r_i(t)}{r_w}\right), \quad (2)$$

where

$S_{wTt}-S_{wt}$ = excess head buildup, in feet, due to colder water (head buildup in IW-2 for injection water of temperature T, at time t, minus head buildup in the well for the formation water at time t);

K_1 = the horizontal permeability of the formation to formation water, in cubic feet per minute per square foot;

K_{1c} = the horizontal permeability, in cubic feet per minute per square foot, of the formation to fresh water at the injection temperature. K_{1c} is defined as $K_1(u/u_c)$, where u_c is the viscosity of fresh cold water at injection temperature and u is the viscosity of the formation water.

The following assumptions are used in making the calculations: the water moves into the formation in a horizontal, radial pattern; it remains confined in the 80-ft zone in which the well screen is placed; accumulation of water in storage within the radius $r_i(t)$ is negligible for any time, t, during the test; and a definite interface exists between the colder, denser injection water and the formation water. If the average transmissivity based on aquifer-test data between well OW-3 and IW-2 is taken as 7,075 cu ft/day/ft, the following substitutions can be made in Equations 1 and 2.

K_1 = 0.0613 cu ft/minute/sq ft;

K_{1c} = 0.0409 cu ft/minute/sq ft, taking the water temperature of the injection water as 10°C, its viscosity as 1.3 centipoises, and the viscosity of the formation water as 0.87 centipoise;

Q = 400 gpm or 53.52 cu ft/minute;

D = 80 ft;

r_w = 0.33 ft;

θ = 0.30 (the porosity of the injection sand was obtained from core analysis and interpretation of compensated gamma-gamma density logs. Both sources indicated an effective porosity of 35-40 percent. A value of 30 percent, used here as a part of the effective porosity, is always occupied by essentially static water along the pore walls).

After 1 minute of injecting 10°C water, the radius of the injection water from the well would be:

$$r_i(t) = \frac{(53.52)(1)}{(3.14)(80)(0.3)} + (0.33)^2,$$

$$r_i(t) = 0.906 \text{ ft.}$$

Solving Equation 2 gives:

$$S_{wTt}-S_{wt} = \left(\frac{1}{0.0409} - \frac{1}{0.0613}\right) \cdot \frac{(2.3)(53.52)}{(2)(3.14)(80)} \cdot \log \frac{0.906}{0.33}$$

$S_{wTt}-S_{wt}$=0.875 ft of excess head buildup after 1 minute, owing to colder water.

The negative of the drawdown curve from a preinjection aquifer test was used as the basic head buildup to be expected in the wells. The calculations for excess head were added to the negative drawdown plot in order to approximate head buildup changes with time and temperature. The equations were solved for excess head buildup at various temperatures and times so that predicted head buildup could be approximated prior to any injection test. The differences in water temperature between the city water and formation water may vary as much as 20°C depending upon the season.

Figure 4 is a plot of the theoretical head buildup divided by discharge in IW-2 and OW-3, comparing it with actual head buildup divided by discharge recorded during the first 270 minutes of injection test 1.

The figure shows that the theoretical data approach the empirical data in the observation well but the head buildup divided by discharge in the injection well is far in excess of the predicted values. Plotting the values for tests 2 and 3 results in the same conclusions with the exception that the excess head buildup in IW-2 and ASW becomes greater with successive tests.

Excess head buildup due to permeability reduction--The field data indicate that only a part of the excess head buildup can be explained by temperature and density differences. The balance must be associated either with permeability reduction in the aquifer or with entrance losses due to clogging of the screen and gravel pack. As a first approach to estimating permeability change due to clay dispersion, Equation 2 may be solved for K_{1c}, the permeability to the injected water, and observed head differences may be inserted in place of the expression $(S_{wTt}-S_{wt})$. That is, the difference between the head buildup actually measured during an injection test and the theoretical head buildup taken from the drawdown measured during pumping at an equal rate may be substituted for $(S_{wTt}-S_{wt})$. This yields the equation,

$$K_{1c} = \frac{\frac{2.3Q}{2\pi D}\left(\log\frac{r_i(t)}{rw}\right)K_1}{K_1\left(S_{inj}-S_p\right)+\frac{2.3Q}{2\pi D}\log\frac{r_i(t)}{rw}}, \qquad (3)$$

where S_{inj} = the head buildup measured after a time (t) of injection at the rate Q;

S_p = the drawdown measured after the same time during pumping at a rate Q, prior to any injection;

$r_i(t)$ = value calculated from Equation 1;

and the remaining terms are as previously defined. If no alteration has occurred in the aquifer and if entrance losses are negligible, Equation 3 should yield a permeability value equal to that calculated for the injection water from the relation $K_{1c}=K_1(u/u_c)$. Alteration of the aquifer should be indicated by a correspondingly lower permeability. This approach assumes that all excess head is caused by permeability decrease rather than by clogging of the screen.

Application of Equation 2 for several times in injection test 1 yielded a permeability of about half that calculated from the relation $K_{1c}=K_1(u/u_c)$. Application of the equation in later tests yielded even lower permeability values, but it is believed that these later results

may reflect the results of screen clogging as well as permeability reduction.

Figure 5 shows distance-drawdown plots from two aquifer tests--one prior to any injection, and the one following test 3, after removal of all injection water and extensive redevelopment to remove screen clogging. Both tests were run at a discharge of 400 gpm. Calculation by the distance-drawdown method using the gradient between ASW and OW-3 shows that the average lateral permeability after the three injection tests was about 50 percent of the original value. This agrees with the estimates obtained from test 1 using Equation 3, and also serves to illustrate the irreversible nature of the permeability change, in that the redevelopment pumping did not restore the permeability to its original value.

Figure 5 shows that after injection of fresh water, the hydraulic gradient steepened greatly near the pumping well and has, in fact, been modified as far away as 50 ft from the pumping well. The hydraulic gradient steepened between OW-3 and OW-2 but not between OW-2 and TW-1. This is because fresh water was injected slightly beyond the radius of OW-3, but not as far as OW-2. Everywhere that the fresh water displaced the brackish water, deterioration of the aquifer permeability occurred. It can be shown, by plotting head buildup versus distance for each injection test, that the deterioration becomes more severe with each test and does not improve significantly with development pumping. The greatest reduction, as would be expected, is close to the injection well.

Current-Meter Traverses

Current-meter traverses were made in the well screen of IW-2 during preinjection aquifer tests in order to determine the water-bearing zones. Traverses made during the injection tests suggest that some minor physical clogging of the aquifer occurs as a result of injection of particulate matter. During the withdrawal pumping, the current-meter traverses suggest that redevelopment of the clogged zones takes place and the flow reverts to the preinjection pattern.

Nine current-meter traverses were made in IW-2 during the injection phase of test 2 (Fig. 6). It is significant to note that the plots of traverses 1 through 9 nearly parallel each other, even though the injection rate decreased from a high of 400 gpm in traverse 1 to 210 gpm in traverse 9. If the reduction in the ability of the aquifer to accept the water were caused by physical clogging with particulate matter, the flow pattern could be expected to change with time. Clogging would be most severe in the zones initially accepting the most water; deterioration of these zones would then force flow into the less permeable zones. Figure 6 supports the alternate theory that the reduction in acceptance of water is due to a rather uniform reduction in permeability to the fresh water of the entire screened zone of the aquifer.

Conductivity Stress

During the injection phase of tests 1 and 2, and the withdrawal phase of test 2, traverses were made of the screen in OW-3 using a down-hole conductivity probe to detect the arrival and movement of fresh water. The background conductivity along the traverse of OW-3 was 4,800 micromhos on a scale of 0-6,000. Changes of 50 micromhos were considered significant. No apparent freshening of the water in OW-3 occurred in test 1. The first definite detection of fresh water at OW-3 was during test 2, after 5,433 minutes of injection. Approximately 1.8 million gal of fresh water had been injected at that time. The fresh water first appeared at the depth interval 895-899 ft below sea level, near the top of the injection zone.

There may have been some internal flow within OW-3 during the injection test, caused by small vertical head differences in the aquifer; uncertainty regarding such internal movement complicates the interpretation of the breakthrough curves. Neverthess, there appear to be two additional zones in which breakthrough of fresh water occurred as the injection continued. These zones were from 908 to 916 ft and from 930 to 942 ft below sea level. No apparent freshening of the water in OW-2, located 75 ft from the injection well, occurred in test 2.

Figure 7 is a geologic section of a part of the well field showing the zones taking water in the injection well during injection test 4 and the zones of detection in the nearby observation wells. The section demonstrates the continuity of sand lenses within the injection zone. The clay and silt beds from about 880 to 890 ft and 975 to 985 ft below sea level in OW-2 are the confining beds of the aquifer.

During test 4, which will be described in a subsequent section, conductivity surveys were made in both OW-3 and OW-2. Fresh water was detected in OW-3 after 1.8 million gal had been injected, essentially the same volume as had been injected during test 2. The first arrival was again in the interval from 895 to 899 ft below sea level. This shows that the flow pattern between the injection well and OW-3 is repeatable.

Fresh water was detected in OW-2 after 6.67 million gal of fresh

water had been injected. There is good correlation between the intervals of high input in the injection well and the intervals of freshwater breakthrough in the observation wells, suggesting that the flow occurred in an essentially horizontal pattern.

If it is assumed that the injection front moves outward in the aquifer in the form of a circular cylinder, the radius of the freshwater zone, neglecting hydrodynamic dispersion, is given at any time by the equation,

$$r_i(t) = \sqrt{\frac{V}{\pi m \theta}}, \qquad (5)$$

where

V = the volume of water, in cubic feet, that has been injected up to time t;

m = the thickness of aquifer, in feet;

θ = the porosity.

Equation 5 is actually equivalent to Equation 1 except that r_w is considered negligible, V is used in place of $Q(t-t_c)$ and it is recognized that the thickness of aquifer, m, accepting flow may differ from the screen length D.

At the time of detection of fresh water in OW-3 during test 2, 1,816,000 gal of fresh water had been injected. Using this volume, expressed in cubic feet, in Equation 5, using m = 80 ft and using $\theta = 0.30$, $r_i(t)$ is calculated as 56.8 ft. However, the current-meter traverses show that only about 60 ft of the injection-well screen was taking water; if m is taken as 60 ft rather than 80 ft, $r_i(t)$ is calculated as 65 ft. Again, the current-meter traverses show that 40 percent of the total inflow occurred in the sand between 900 and 920 ft below sea level, which correlates well with the intervals of early breakthrough in OW-3. If Equation 5 is solved using 40 percent of the injected volume and using m = 20 ft, $r_i(t)$ is calculated to be about 72 ft. Thus, for each of these sets of assumptions, the injected water should have reached OW-3, at a radius of 50 ft from the injection well, considerably earlier than its time of detection. Arrival should have been from 1 day to more than 2 days prior to actual detection, depending upon the assumptions employed.

These calculations indicate that the injection front probably did not have the form of a cylinder. Subsequent calculations relating to the arrival time in OW-2, 75 ft from the injection well, confirm this interpretation. A possible alternative which comes to mind is that the front may have had an elliptical form, as should be expected if the aquifer were homogeneous but anisotropic. However, comparison of the arrival times in

OW-3 and OW-2 during test 4 rules out this possibility. It does seem clear, however, that the injection front was elongate in a direction roughly normal to the line through OW-2, OW-3, and the injection well. Trial calculations show that this elongation could not be due to superposition of a radial flow on the original hydraulic gradient in the aquifer, as this original gradient was very small.

Aquifer heterogeneity--A reasonable explanation of the arrival-time data can be offered on the basis of the geology. The zone of greatest hydraulic conductivity in the upper part of the aquifer is probably a channel-fill or a shoestring sand. A deposit of this type would have the coarsest material along the center of the channel, with transition to progressively finer material along the sides. The average permeability across the channel would accordingly be lower than that along the channel axis, and injected water would tend to follow the channel axis rather than to move laterally away from the channel. The channel would have a meandering orientation, but presumably its overall lineation would be at some high angle to a line through OW-2, OW-3, and the injection well. This interpretation is suported by pumping-test data, which indicate that the specific capacities of wells IW-2 and TW-1 are higher than those of OW-3 and OW-2.

WATER QUALITY

Comparison of Chemistry of City and Formation Water

The city water is a calcium sulfate-chloride type and the formation water is a sodium chloride-bicarbonate type water. The major constituents are listed in Table 2. The dissolved solids of the fresh water varied between 110 and 190 mg/l and the formation water contained about 3,000 mg/l.

Water Sampling

Prior to injection testing, it was believed that only simple dilution would occur when the two waters came in contact. However, there was a possibility of some reaction that might have a detrimental effect on the injection process. In order to observe any reactions, water samples were collected throughout the freshwater - withdrawal phase during tests 1 and 2. Samples were collected with respect to changes in specific conductance. Chemical analyses included the measurement of silica, calcium, magnesium, sodium, potassium, bicarbonate, carbonate, sulfate, chloride, nitrate, phosphate, fluoride, and boron.

Analytical Results

The analytical data thus collected indicated that, as the city and formation waters mixed, simple dilution of the major constituents in formation water did occur as shown in Table 3. Here it can be seen that specific conductance, sodium, chloride, and bicarbonate varied directly with the proportions of city and formation water. A plot of chloride concentration versus specific conductance indicates that a straight-line relation between them exists, and that, as the fresh water was injected, a uniform mixing zone was formed as the two waters moved away from the borehole (Fig. 8). However, the data in Table 3 also indicate that something other than simple dilution occurred with respect to calcium and magnesium.

Changes in Concentrations of Calcium, Magnesium, and Sodium

The concentration of calcium and magnesium in city water was 17 and 2.6 mg/l, respectively, and in formation water was 14 and 8.7 mg/l, respectively. An examination of the calcium and magnesium data shown in Table 3 indicates that the concentrations of these two constituents in the mixed water were at times much lower or higher than their concentrations in either city or formation water. The water containing the low concentrations, however, was still essentially fresh water. During the later periods of withdrawal, when formation water became dominant in the mixture, the calcium and magnesium concentrations began to increase. During the latter phase of withdrawal in test 1, the concentration of calcium was almost twice as great as in either city water or formation water. There appeared to be little doubt that calcium and magnesium were involved in some form of reaction as the fresh water entered the formation, and that the reaction was reversible (Fig. 9).

If calcium and magnesium concentrations were not decreasing as a result of calcite precipitation, then the decrease must have been due to simple cation exchange with the sodium-saturated clays within the formation. When calcium in the fresh water was exchanged onto the clays, exchangeable sodium should have been released into solution. Thus, there should be an excess of sodium in the water when calcium concentrations approach minimum values. An examination of Figure 9 shows that calcium was being lost (taken onto the clays) when the water was still fresh. As the percentage of formation water increased (based upon specific conductance) the calcium was released from the clays back into solution. The sodium data in Figure 8 do not show clearly that there was excess sodium during the periods when calcium was being lost from solution. At best, there is a slight change in the slope of the sodium curve at low concentrations, indicating that the concentration of sodium was increasing at a greater rate than the specific conductance of the mixed waters.

A mathematical demonstration that cation exchange was occurring was not possible because it was impossible to separate the sodium concentration due to fresh- and formation-water mixing from the sodium released by cation exchange. However, an indirect technique was attempted on the basis of the assumptions that (1) chloride ions do not enter into any reactions during either the injection or withdrawal of the fresh water (Fig. 8), and (2) sodium was involved in some reaction. The ratio of sodium to chloride in city water is 0.71 and in formation water is 1.30. If the above assumptions are correct, then the ratios of sodium to chloride in the mixed city and formation waters would be between the limits of 0.71 and 1.30. The ratios obtained from all of the analytical data indicated that the sodium-to-chloride ratio was never less than 0.71, but was frequently more than 1.30. Furthermore, when the ratio was more than 1.30, the calcium concentration was much less than in either city or formation water, as shown in Table 4 and Figure 10. If a comparison is made between the calcium-concentration data shown in Figure 8 and the sodium-to-chloride ratios shown in Figure 10, it can be seen that, with respect to specific conductance, excess sodium concentrations occurred during the withdrawal periods when calcium concentrations were approaching minimum values.

To test the validity of the assumption that chloride was a non-reactive constituent during all phases of injection and withdrawal, ratios of chlcride to bicarbonate concentrations were obtained in fresh and formation water. The ratio in fresh water is 1.0 and in formation water, 3.9. All other ratios calculated from mixed fresh and formation water produced a straight-line relation as shown in Figure 10. All of the data shown in Figures 9 and 10 indicate that, as the fresh water was injected into the formation, calcium was exchanging for sodium on the formation clays. Upon withdrawal, the calcium remains adsorbed until the concentration of sodium in the mixed fresh and formation water is high enough to reverse the process.

Effect of Cation Exchange on Formation Clays During Injection

It is unlikely that cation exchange during the injection phases had any other effect except to lower slightly the zeta potential (tendency to disperse) of the clays. If the sodium montmorillonite-illite clays

were going to disperse when subjected to fresh water, the exchange of calcium for sodium would only lower this tendency slightly. However, the fact that even low concentrations of calcium would exchange for sodium on the clays indicated that the clays might respond to chemical treatment.

LABORATORY DETERMINATION OF PERMEABILITY REDUCTION

In order to substantiate the theory that clay dispersion caused permeability reduction in the aquifer, core samples of the injection sand taken during the drilling of OW-3 were sent to the laboratory to determine if the aquifer was "water sensitive." Testing procedures were similar to the techniques described by Hewitt (1963). The cores were saturated with formation water for 24 hours prior to testing. Permeability was determined by running formation water through the core until the values stabilized. City water was then introduced into the core, displacing the formation water. Hydraulic conductivities were measured until the value stabilized (Table 5).

Laboratory results matched the field tests in that the hydraulic conductivity of the sand was irreversibly altered. Reductions in hydraulic conductivity ranged from 50 percent to over 70 percent. The aquifer, on the basis of Hewitt's classification (water permeability/Klinkenberg [gas] permeability--less than 0.3), would be classified as strongly sensitive to fresh water.

Various chemicals were added to the city water prior to injection in an attempt to overcome the dispersion problem (Table 5, Fig. 11). Sodium hydroxide and sodium carbonate were added to adjust the pH of the city water to values similar to or greater than that of the formation water. Deterioration of the hydraulic conductivity was not prevented by this treatment. Sodium hexametaphosphate, a compound used by drilling companies to clean wells that have had excessive invasion of drilling mud, was introduced into the core as a mixture in the city water. The compound did not prevent reduction in the hydraulic conductivity, and because it acts as a deflocculent, probably magnified the change.

The fact that a turbid effluent resulted when either untreated or chemically treated city water was introduced into the core saturated with formation water indicates that dispersion and particle plugging was occurring. When the city water was treated with calcium chloride, the double layer around the clay particles and the zeta potential were both reduced. No reduction in hydraulic conductivity occurred and the effluent was clear, indicating that dispersion and migration of clays did not occur to any significant extent.

When a calcium chloride preflush was used prior to injection of untreated city water, a reduction in hydraulic conductivity of only 11 percent occurred. Reed (personal commun.) has found that 11 to 12 percent reduction in hydraulic conductivity occurs in wells treated with polymeric hydroxy aluminum in water-flood projects, and should be expected if either the calcium or aluminum treatment is used on the clays. The core was resaturated with formation water, then injected with untreated city water. Clogging did occur and the effluent was slightly turbid, indicating that the calcium-for-sodium base exchange is a reversible reaction. The reduction in hydraulic conductivity was not as severe as in previous tests, suggesting that the base exchange may not be as complete when exchanging sodium for calcium. This is to be expected inasmuch as the calcium ion is held more tightly than the sodium ion.

INJECTION TEST 4

A fourth injection test was made to determine the effectiveness of chemical treatment of the clays to prevent dispersion under field conditions. Because the alteration of the aquifer from clay dispersion during the first three tests severely reduced the capacity of the well to accept water, it was decided to use an inexpensive, nonpermanent calcium chloride preflush treatment to stabilize the clay.

As the preflush moves away from the well screen, the calcium ions in the solution are adsorbed by the clay. The preflush becomes a sodium chloride solution with time as a result of addition of sodium ions released by the clays. When the calcium ions in the preflush solution are depleted, the effectiveness of the preflush is negated, and dispersion, migration, and particle plugging occur in the aquifer. Thus, a decrease in injection rate and increase in injection-head buildup will occur when the fresh water enters the untreated part of the aquifer.

It is neither practical nor necessary to treat the entire area that will come into contact with the injected water. In any problem of flow to or from a well, the cross-sectional area of flow decreases sharply as the well is approached. The highest head losses occur close to the well, in this region of restricted cross-sectional area. This can be shown by a simple application of Darcy's law, which states that, as the cross-sectional area of flow decreases, hydraulic gradient must increase, other factors remaining equal. In the present problem, this leads to the conclusion that, if the area directly around the well can be treated, most of the increased head losses due to permeability deterioration can be

avoided. The exact distance from the well to which the treatment should extend is open to debate; however, discussions with personnel from oil field service companies indicate that the preferred radius of treatment lies within the limits of 3-10 ft from the borehole.

Injection test 4 was begun Nov. 24, 1972. A preflush of 3,000 gal of 1.4 percent calcium chloride solution was injected in front of the city water. On the basis of current-meter data, this volume theoretically would treat the aquifer to a radius of 8 ft in the most permeable zones and up to 2 ft in the least permeable zone.

The injection rate stabilized at 185 gpm after 10 minutes and was maintained at that rate for 115 minutes. The hydraulic gradient declined throughout this time, indicating that the treatment was working effectively. After 115 minutes, the injection rate began to decline slowly and the injection-head pressure began to increase slowly. At this time, over 20,000 gal had been injected and the fresh water was beyond the area of treatment.

It was suspected that redevelopment pumping would increase the specific capacity of the well, but redevelopment pumping could not be attempted until a sufficient quantity of fresh water was injected to ensure that formation water was not brought back into the vicinity of the well bore. Because the calcium-for-sodium base exchange is reversible, if formation water were brought into contact with the "desensitized" clays, they would return to a water-sensitive condition.

Continuous injection was made for 2,580 minutes (398,000 gal injected) before any redevelopment pumping was attempted. The next injection phases consisted of continuous injection for periods of 11,380, 10,025, 2,495, 2,695, and 20,450 minutes between redevelopment pumpings. After the 20,450-minute injection phase, redevelopment pumping was conducted on a daily basis. Current-meter traverses of IW-2 and conductivity profiles of OW-2 and OW-3 were made throughout the injection of the fresh water.

Injection Specific Capacities

Figure 12 shows the injection specific capacity of IW-2 during the initial stages of test 4 as compared to the specific capacities measured during preinjection and tests 1, 2, and 3. In the first 1,000 minutes, the decrease in specific capacity was 51 percent in test 2, 75 percent in test 3, and only 32 percent in the initial phase of test 4.

After the initial injection phase, the decrease in specific capacity during the first 1,000 minutes of injection for each new injection period following redevelopment pumping ranged from 3 to 20 percent. The variation in percentage of decrease reflects the effectiveness of redevelopment pumping. A decrease of 11-12 percent per initial 1,000 minutes of injection is an average value for test 4. This value agrees with the decrease Reed (1972) found to occur in treated "water-flood wells" and with the decrease in laboratory hydraulic conductivity to fresh water of core treated with a calcium chloride preflush.

Each redevelopment-pumping period increased the specific capacity of the injection well. During redevelopment pumping, the water first entering the screen contained sand, clay, and mica in excess of 360 mg/l. The water was heavily sediment-laden for 5-6 minutes in the first redevelopment pumping, but the heavy sediment flow decreased to 1-minute duration after the fourth redevelopment pumping. Sediment from the well contained microfossils and glauconite particles that are foreign to the aquifer. This material probably represents drilling-mud invasion into the gravel pack during construction of the well. The clay particles from the well were flocculated and probably represent material loosened during treatment by the calcium chloride.

After 4.04 million gal had been injected, an additional preflush of 3,000 gal of 2.8 percent calcium chloride solution was injected in front of untreated city water. The specific capacity again improved and remained in the range of 5 gpm per foot of head buildup and the flow of sand during redevelopment diminished significantly.

Plots of the injection specific capacity versus time showed that an optimum specific-capacity value can be maintained if the injection period is limited to about 1,400 minutes. For IW-2, considering the altered condition of the aquifer and the sand problem, the best method of injection was to inject for 1,300-1,400 minutes, withdraw for 30 minutes to clear the screen area of sediment choking, then wait about 1 hour to approach static conditions before beginning the next injection phase.

A total of 20,146,100 gal of fresh water was injected in test 4. The injection specific capacity did not vary significantly until 16.35 million gal had been injected; then the specific capacity deteriorated and redevelopment pumping did not restore it.

Current-Meter Traverses

There was little clogging of the aquifer during the injection of the first 16 million gal of fresh water. Current-meter traverses showed that only 4 ft of aquifer that was taking water at the start of the test was

not accepting water after nearly 63 days of injection.

Prior to treatment of the aquifer with the calcium chloride, the zone from 900 to 917 ft below sea level accepted a maximum of 55 percent of the flow rate. After treatment with calcium chloride solutions, the zone took between 80 and 90 percent of the total injection rate. This suggests that the treatment may have operated preferentially in this zone, at the expense of some others. This is the most permeable zone within the aquifer and therefore would get the most calcium chloride solution during treatment. As a result, it should show the greatest improvement in hydraulic conductivity relative to the surrounding zones.

Cause of Clogging of IW-2 During Injection Phase of Test 4

The quantity of sediment produced during the redevelopment phases increased with each redevelopment effort but usually diminished after about 2 minutes of redevelopment pumping. After injection of 16.35 million gal, the heavy flow of sediment obtained during the daily redevelopment lasted for 8 to 10 minutes before abating. Not only had the quantity of sediment changed, but the character of the sediment also changed. It now consisted predominantly of clay granules with some colloidal clay, and silt to fine-grained particles of quartz and mica. The granules consist of flocculated clay and silt. The colloidal clay would remain in suspension for several hours in undisturbed water, but not for days as it did before treatment.

A possible cause of heavy sediment flow and the subsequent decrease in specific capacity may be due to disturbance of the gravel pack so that its effectiveness as a filter was negated. During the redevelopment cycle a combination of injection and withdrawal pumping was employed; this practice tends to agitate the gravel pack of the well. It was after this vigorous redevelopment pumping that the sediment flow during withdrawal became noticeably heavier and lasted up to 15 minutes before diminishing.

The appearance of some dispersed clays and the overall decrease in sediment size indicate that the sediment may be coming from either the lower part of the aquifer that did not receive effective calcium chloride treatment, or from channelization in the gravel pack, which would allow younger sediment to filter down into the screen from above the aquifer. When redevelopment pumping was conducted, the higher pumping rates produced appreciable flow from the lower part of the aquifer with enough velocity to move clay and silt into the borehole. As pumping continues, and the rate decreases, the velocity of the water coming from the lower section decreases, and movement of sediment ceases.

The clogging should not be taken as evidence that the calcium chloride treatment failed to stabilize the clays. The injection specific capacity varied little over a period of nearly 70 days while 16 million gal was being injected, then decreased rapidly with excessive injection heads, indicating a physical problem due to well construction. Head buildup measured in IW-2 and ASW during the 16 million-to-20 million gal phase indicates that the majority of the problems are in the gravel pack area. The fact that glauconite was still being recovered at the end of the test 4 injection phase also indicates that the sediment is coming from somewhere other than the injection formation.

The injection phase of test 4 was completed Feb. 26, 1973, with a total of 20,146,100 gal of fresh water injected into the brackish-water aquifer. The water will remain in the aquifer for 2 months prior to withdrawal.

RECOVERY OF POTABLE WATER

In injection tests 1, 2, and 3, the proportion of recovered water that is within the limits of the Public Health Standards is between 85 and 90 percent on a composite basis. The recovery percentages are expected to be in the same range for test 4. On a practical basis, the ideal situation would be to inject a large enough quantity of water for storage so that at no time would the withdrawal exceed 65 percent of the volume injected. This would maintain the injection front and keep the total solids content of the withdrawn water at a minimum.

SUMMARY

Injection tests 1, 2, and 3 proved that when fresh water is injected into a brackish-water sand aquifer the change in electrolyte concentration in the pore water causes interstitial clays to disperse and reduce the hydraulic conductivity severely. By the end of the injection phase of test 3, the specific capacity of the injection well had been decreased by more than 80 percent of the preinjection value. Of this value, about 50-60 percent of the decrease was due to the reduction of the aquifer permeability, and clogging of IW-2 accounted for the rest. Subsequent laboratory permeameter tests on a core from the injection zone confirmed that the aquifer sand is strongly sensitive to fresh water.

The chemical and laboratory data indicated that chemical treatment of the aquifer could prevent dispersion of the clays in subsequent tests, but could not restore the hydraulic conductivity of the aquifer to the preinjection values. A permanent treatment of clays would be accomplished by injecting a solution of polymeric hydroxy aluminum into the aquifer; however, because the aquifer was already altered in the vicinity of the well, an inexpensive, temporary method was chosen. A solution of calcium chloride was injected as a preflush prior to injection test 4. The injection specific capacity stabilized, indicating that dispersion was not occurring in the treated area around the borehole. After nearly 16 million gal of fresh water had been injected, clogging occurred in the gravel pack of the well, but this problem appears to be independent of the clay-dispersion problems of tests 1 through 3.

The data are still being analyzed and the withdrawal phase of test 4 has not been accomplished; however, the results of stabilizing the clays during the injection phase of test 4 and the percentage of potable water recovered are very encouraging. Preliminary indications are that a well field could be constructed and operated economically to store surplus water. Each well should be treated with the permanent polymeric hydroxy aluminum treatment as soon as it is completed. This phase of the project is still under study and will be reported at a later time.

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Table 1. X-Ray Diffraction Analysis for Samples from OW-3 Injection-Zone Core, Norfolk, Virginia

Depth of Sample in feet	Description	Quartz (%)	Feldspar (%)	Kaolinite (%)	Illitic Mica (%)	Mixed-layer Clays (%)	Montmorillonite (%)	Chlorite (%)	Total (%)
892-912[1]	Clay	32	16	5	17	30	--	--	100
975-995[1]	Sand	71	6	4	6	13	--	--	100
975-995[1]	Sand	78	5	3	4	10	--	--	100
892-912[2]	Clay	20	12	13	27	--	18	--	90
892-912[2]	Sand	52	24	3	3	1	2	1	86
935-955[2]	Sand	67	23	1	1	--	1	--	93
955-975[2]	Sand	52	19	7	7	--	2	2	89

[1]Core analysis by Wayne Hower, of Halliburton Company, who reported that dye stainings of consolidated fragments of the core showed that the kaolinite was limited to isolated "clumps," whereas the mixed-layer clays were present in the waterways. In his opinion, the core would be "quite sensitive to fresh water" (Hower, written commun., 1972).

[2]Core analysis by Barbara Anderson, U. S. Geological Survey, Denver, Colorado.

Table 2. Concentration of Major Constituents in City Water and Formation Water[1]

Constituents	Formation Water	City Water
Silica (SiO_2)	13	3.8
Calcium (Ca)	14	17
Magnesium (Mg)	8.7	2.6
Sodium (Na)	1,140	9.5
Potassium (K)	25	1.6
Bicarbonate (HCO_3)	624	9.0
Sulfate (SO_4)	136	36
Chloride (Cl)	1,360	21
Nitrate (NO_3)	0.1	1.2
Phosphate (PO_4)	0.28	0.00
Boron (B)	3.4	0.04
Fluoride (F)	1.4	0.1
Dissolved Solids	3,010	111
pH (units)	7.9	5.8
Specific Conductance (micromhos at 25°C)	5,000	190

[1]Milligrams per liter unless otherwise noted.

Table 3. Variations in Water Chemistry of Mixed Fresh Water and Formation Water During Withdrawal, Injection Tests 1 and 2[1]

Specific Conductance (micromhos)	Formation Water (percent)	Ca^{+2}	Mg^{+2}	Na^{+}	K^{+}	HCO_3^{-}	SO_4^{-2}	Cl^{-}
			Fresh Water					
180	0.0	17	2.6	9.5	1.6	9	36	2.1
			Injection Test 1					
290	1.0	6.2	2.0	48	8.4	60	34	35
370	2.0	4.0	2.1	72	8.8	73	35	49
840	11	8.0	2.3	160	13	129	45	170
2,000	35	23	8.0	400	26	272	75	500
3,400	61	31	12	710	35	422	100	860
4,600	93	23	11	1,000	40	582	130	1,300
			Formation Water					
5,000	100	14	8.6	1,200	40	618	150	1,400
			Injection Test 2					
190	0.3	10	3.2	17	2.9	8.0	36	24
245	.5	20	3.8	20	3.7	31	42	27
360	1.8	5.6	1.3	63	8.6	67	40	46
460	3.3	5.3	1.1	85	8.9	81	40	67
1,400	14	10	2.5	220	13	256	52	220
1,800	28	16	4.8	360	15	228	48	410
2,300	36	16	5.5	460	17	274	66	520
2,600	43	16	6.0	530	19	312	71	620
2,900	51	17	6.4	590	21	352	74	720
3,200	58	19	7.2	680	23	400	69	820
3,800	68	24	8.8	800	25	456	81	960
4,900	96	22	10	1,000	29	620	63	1,200
			Formation Water					
5,000	100	15	8.8	1,100	29	608	69	1,300

[1]Concentrations expressed as milligrams per liter.

Table 4. Sodium-to-Chloride Ratios and Associated Concentrations of Calcium in Samples Collected During Injections Tests 1 and 2

Specific Conductance (micromhos)	Sodium/Chloride (milliequivalents per liter)	Calcium (milligrams per liter)
	Injection Test 1	
290	2.11	6.2
370	2.27	4.0
840	1.45	8.0
2,000	1.23	23
3,400	1.27	31
4,600	1.19	23
5,000	1.30	14
	Injection Test 2	
190	1.09	10
245	1.14	14
360	2.11	5.6
460	1.96	5.3
1,380	1.54	10
1,800	1.35	16
2,300	1.36	16
2,600	1.32	16
2,900	1.26	17
3,200	1.11	19
3,800	1.29	24
4,900	1.29	22
5,000	1.30	15

Table 5. Effect of Water Chemistry on Laboratory Hydraulic Conductivity for Core Samples of OW-3 Injection Zone

Laboratory Sample Number	Depth (feet)	Klinkenberg Permeability (millidarcys)	Water Type	Water Modification	Input pH	Hydraulic Conductivity (meters/day)	Effluent Condition
73VA2a	892-912	1050	formation	none	8.5	4.2×10^{-1}	clear
2a		--	city	none	6.5	2.15×10^{-1}	turbid
2a		--	formation	none	8.5	2.42×10^{-1}	clear
2b	892-912	--	formation	none	8.3	4.98×10^{-1}	clear
2b		--	city	NaOH 1/	8.3	2.25×10^{-1}	turbid
2c	892-912	--	formation	none	8.3	1.34	clear
2c		--	city	Na_2CO_3 2/	10.2	4.52×10^{-1}	turbid
73VA5	955-975	1320	formation	none	7.5	2.43×10^{-1}	clear
5		--	city	none	6.5	6.49×10^{-2}	turbid
5		--	formation	none	7.5	7.2×10^{-2}	clear
73VA6	955-975	2150	formation	none	7.5	2.56×10^{-1}	clear
6		--	city	none	6.5	7.76×10^{-2}	turbid
6		--	city	$Na(PO_3)_m$ 3/	-	7.48×10^{-2}	turbid
6		--	formation	none	7.5	7.77×10^{-2}	clear
73VA7b	955-975	--	formation	none	7.0	6.2×10^{-1}	clear
7b		--	city	NaOH 4/	7.3	2.7×10^{-1}	turbid
7b		--	formation	none	7.0	2.5×10^{-1}	clear
73VA9	955-975	--	formation	none	-	2.08×10^{-1}	clear
9		--	city	CaCl	-	2.08×10^{-1}	clear
9		--	city	none 5/	-	1.86×10^{-1}	clear
9		--	formation	none	-	2.20×10^{-1}	clear
9		--	city	none	-	1.74×10^{-1}	slightly turbid

[1] Enough added to bring pH equal to or greater than 8.3.
[2] 40 mg/l added.
[3] 100 mg/l added.
[4] Enough added to modify pH to between 7.0 and 8.0.
[5] 1.375 g/l (0.14 percent solution) added.

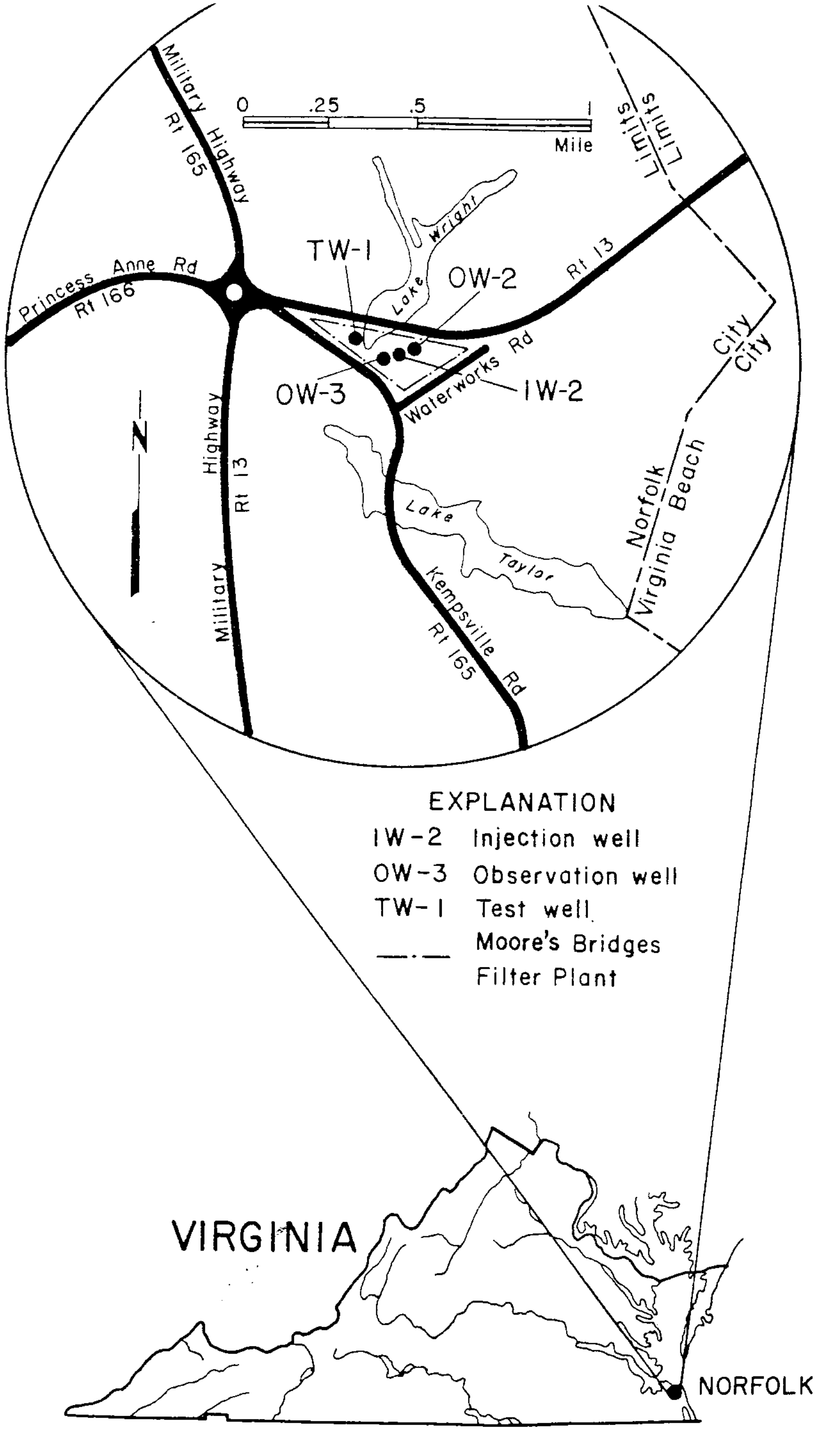

FIG. 1--Location of Norfolk, Va. test site.

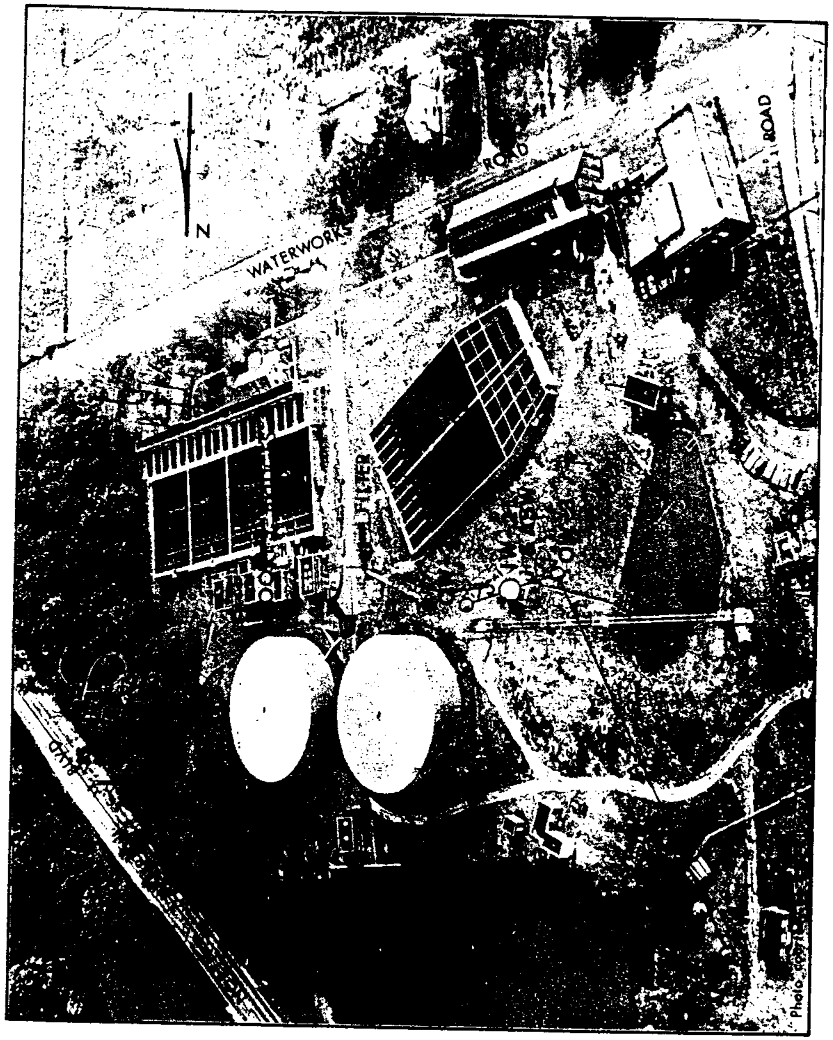

FIG. 2--Norfolk injection-project well field, Moore's Bridges Filter Plant, Norfolk, Va. Approximate scale 1 in. = 60 ft.

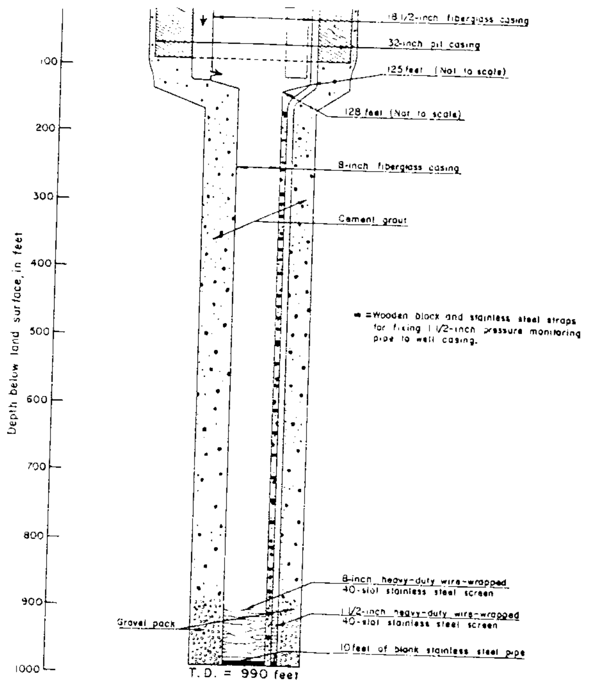

FIG. 3--Diagrammatic sketch of injection well (IW-2), Norfolk, Va.

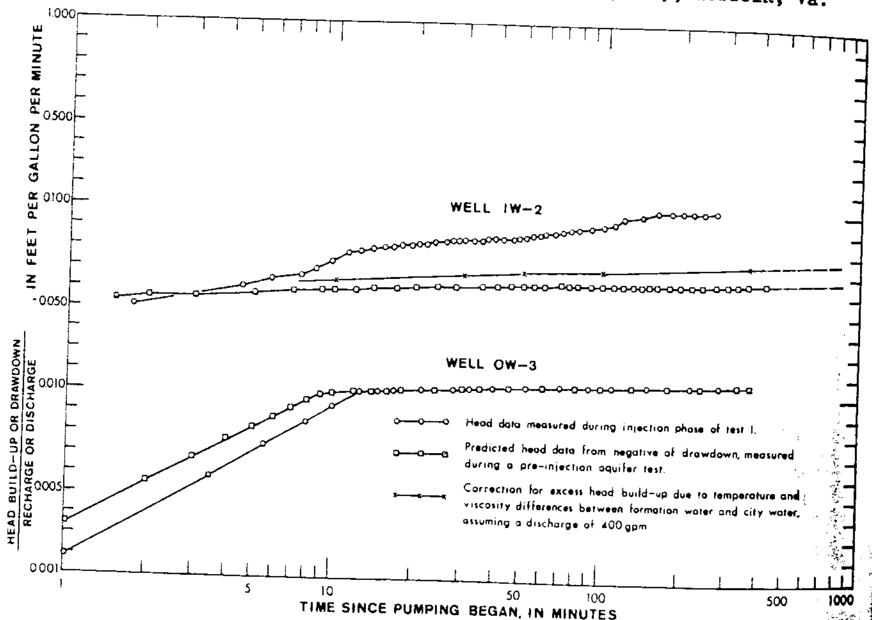

FIG. 4--Comparison of theoretical head data to measured head data, Norfolk injection test 1.

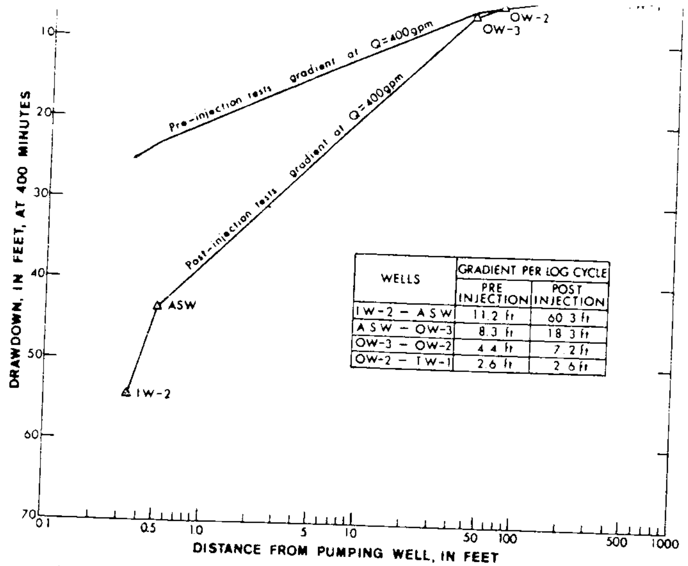

FIG. 5--Comparison between preinjection and postinjection hydraulic gradients, showing aquifer alteration caused by injection of fresh water.

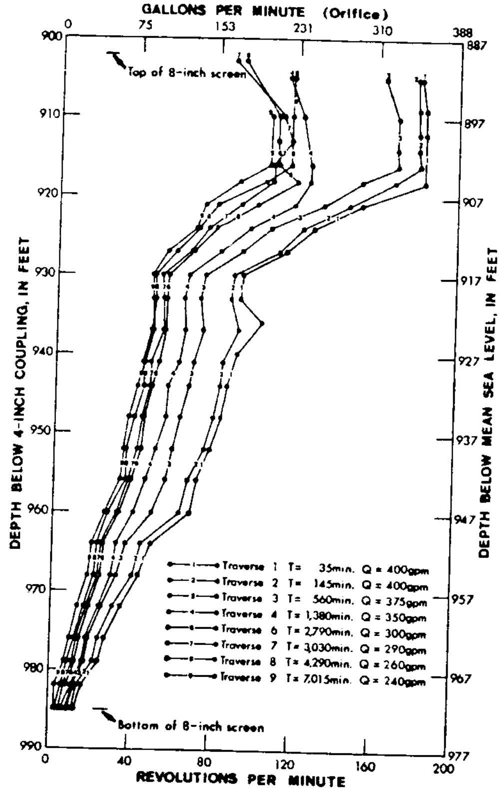

FIG. 6--Current-meter traverses of IW-2 during injection phase of test 2.

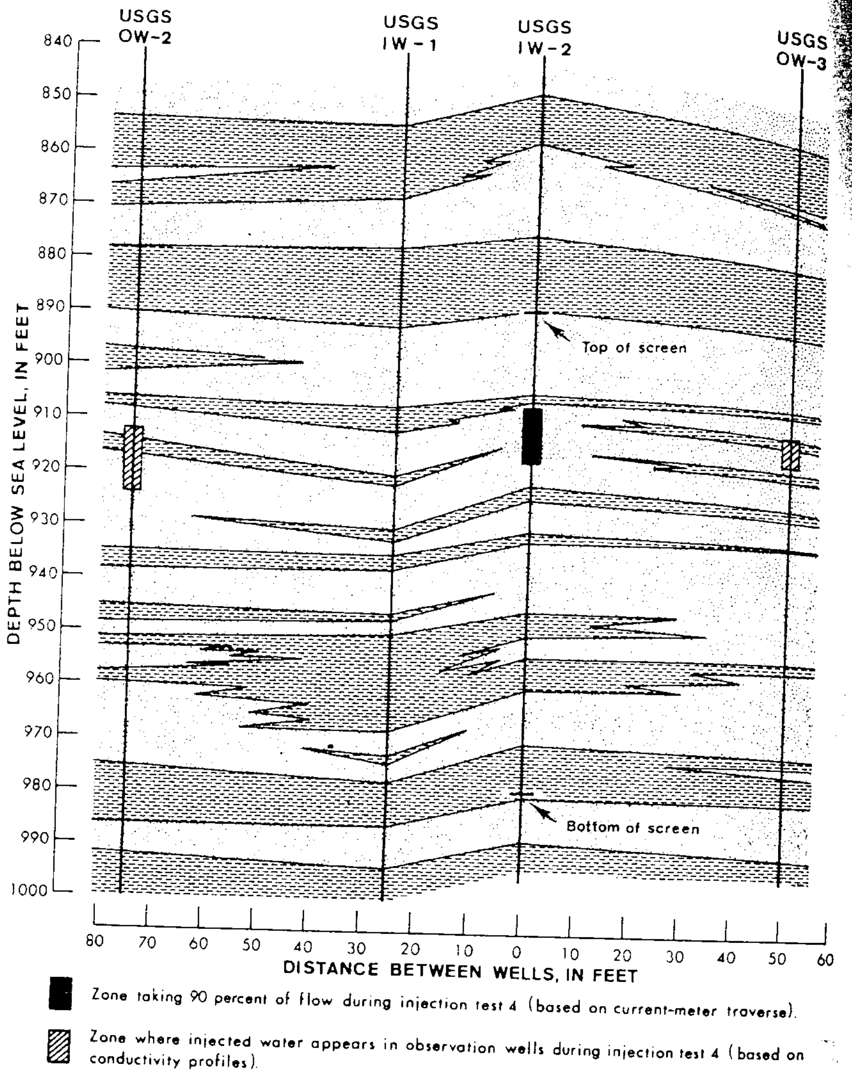

FIG. 7--Geologic cross section showing zone taking water in IW-2 and zones of detection of fresh water in observation wells 2 and 3.

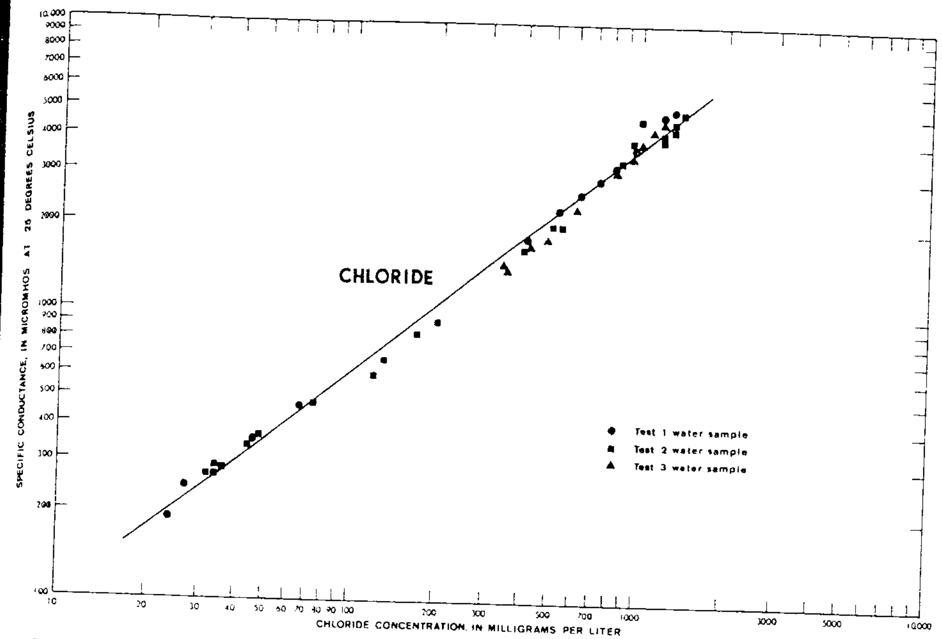

FIG. 8--Chloride concentration versus specific conductance for recovered injected water from tests 1, 2, and 3.

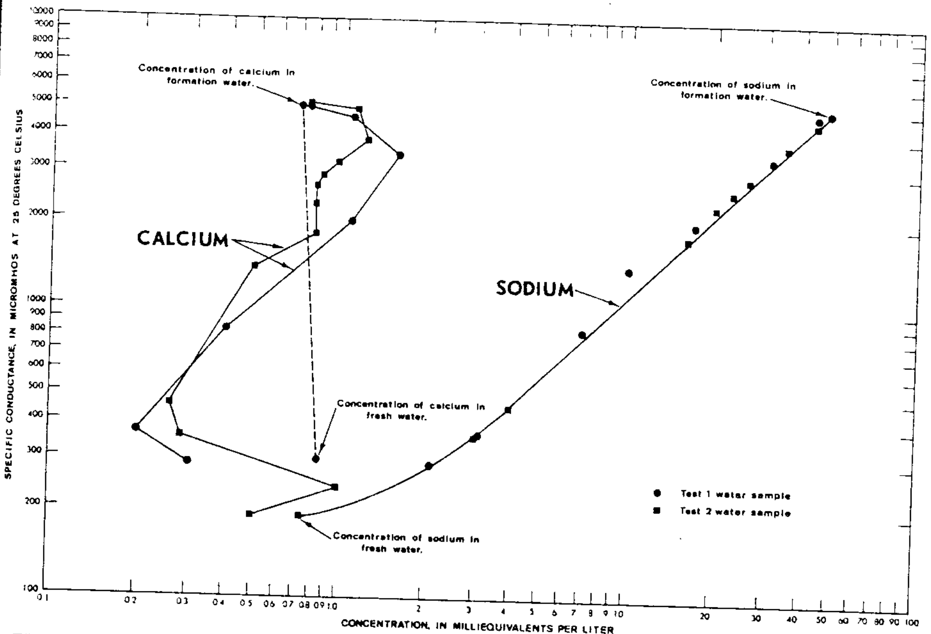

FIG. 9--Variations of calcium and sodium with respect to specific conductance during recovery of injected water during tests 1 and 2.

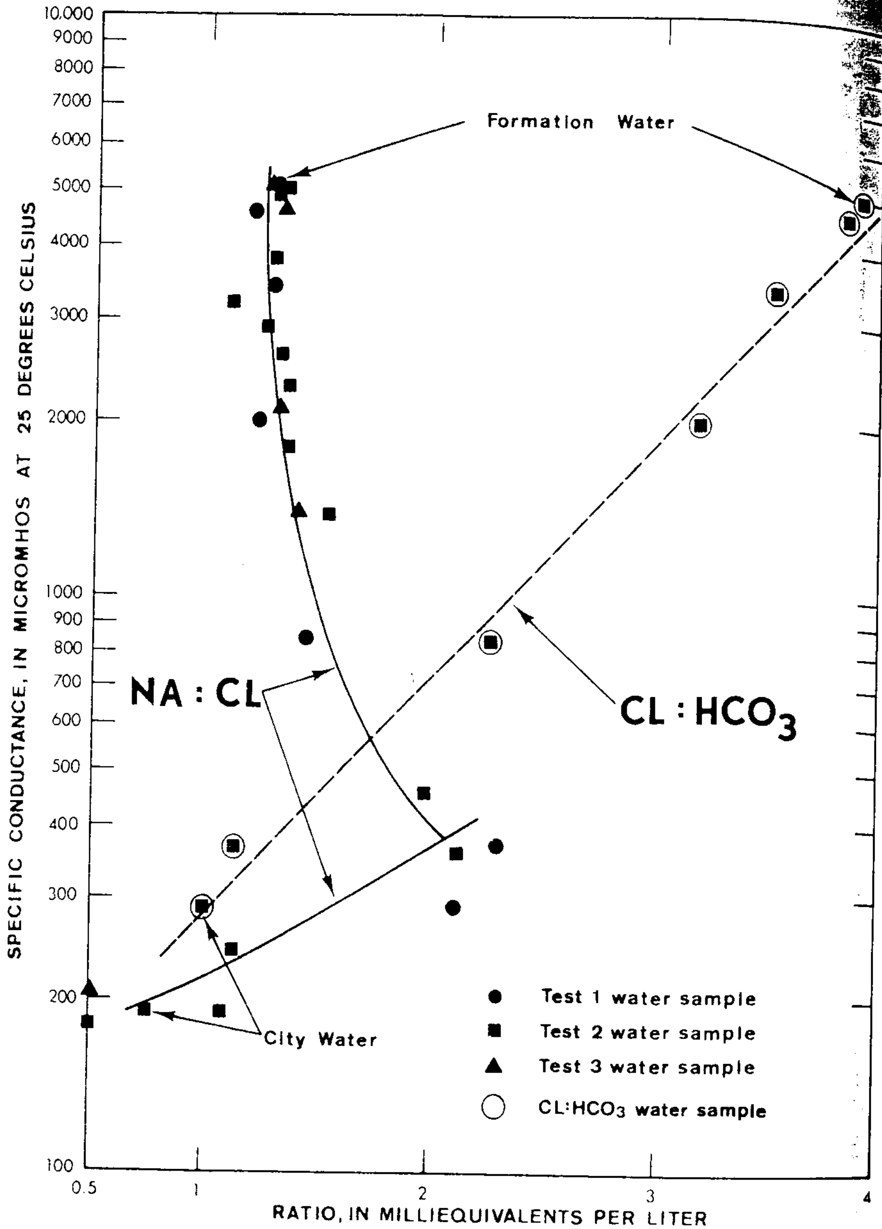

FIG. 10--Plot showing excess sodium during early phases of withdrawal of fresh water during tests 1, 2, and 3.

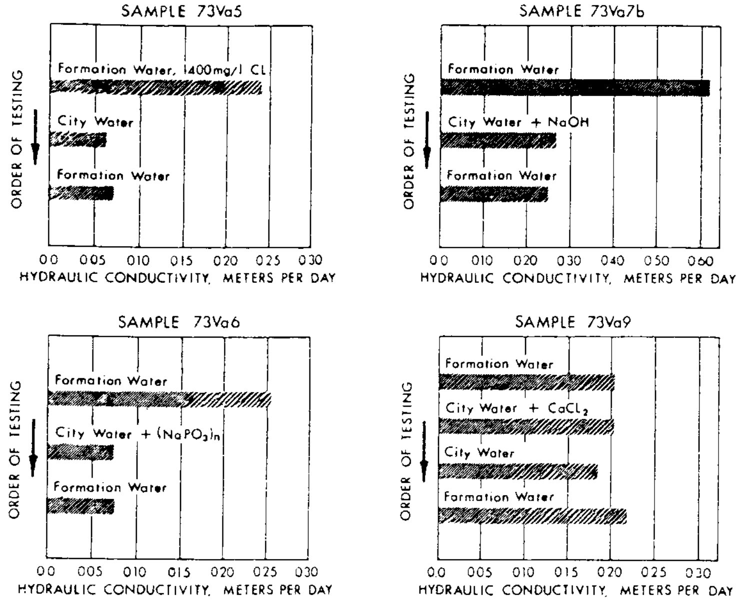

FIG. 11--Effects on hydraulic conductivity of core saturated with formation water by injecting city water with various chemical treatments.

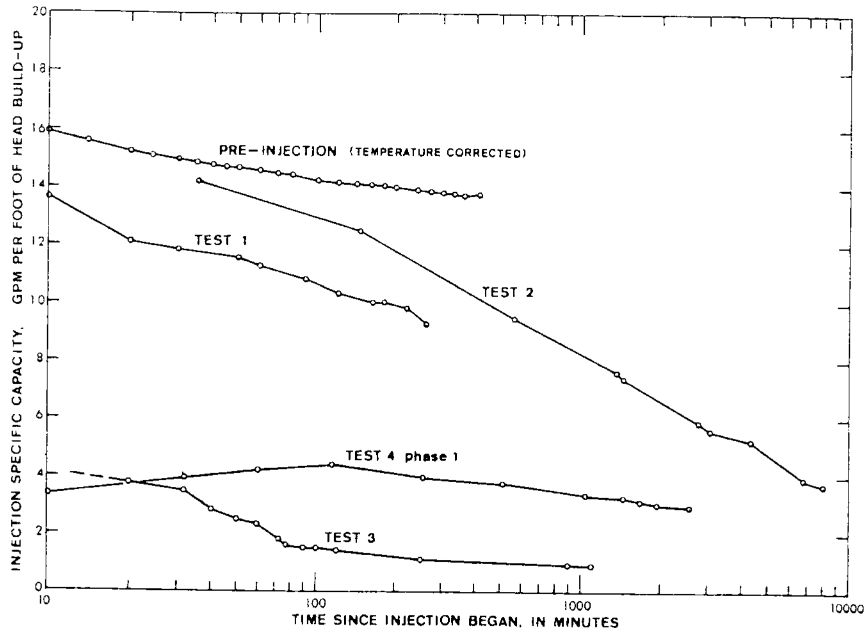

FIG. 12--Variations in specific capacity of IW-2 with time during injection tests, Norfolk, Va.

RETENTION OF DISSOLVED CONSTITUENTS OF WASTE BY GEOLOGIC MEMBRANES[1]

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ABSTRACT Clays and shales serve as semipermeable membranes, retarding by varying degrees the passage of the dissolved species with respect to water. The relative retardation by geologic membranes of cations and anions generally present in waste solutions has been investigated using a high-pressure and high-temperature filtration cell. The solutions were forced with varied hydraulic gradients through different clays and a disaggregated shale subjected to compaction pressures up to 10,000 psi and to temperatures from 20-70°C.

The membrane efficiencies measured in this and other studies increased with increase of exchange capacity of the material used and with decrease in concentration of the input solution. The efficiency of a given membrane increased with increasing compaction pressure but decreased at higher hydraulic gradients for solutions of the same ionic concentration.

The results further show that geologic membranes are specific in that the degree of retardation is different for different dissolved species. The retardation sequences obtained varied depending on the material used and on experimental conditions. The retardation sequences for monovalent and divalent cations were generally as follows: Li < Na < NH_3 < K < Rb < Cs and Mg < Ca < Sr < Ba. The retardation sequences for anions at room temperature were variable, but at higher temperatures the sequence was: HCO_3 < I < B < SO_4 < Cl < Br. Monovalent cations were generally retarded with respect to divalent cations at the higher hydraulic gradients. This trend was reversed, however, for Na and Ca at the lower hydraulic gradients.

The membrane behavior of natural materials is significant in waste management.

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INTRODUCTION

The voluminous experimental and field evidence available demonstrates that soils, clays, and shales (the geologic membranes) serve as semipermeable membranes (Bredehoeft et al., 1963; White, 1965; Berry, 1969; and Kharaka, 1971). Geologic membranes will retard, or restrict by varying degrees, the passage of the charged constituents of the liquid waste when the solution is subjected to flow by imposing hydraulic, chemical, thermal, or electrical gradients across the membrane material. Under such conditions, the effluent (throughput) solution will be lower in concentration compared to the hyperfiltrated solution remaining on the input side of the membrane. Also, because geologic membranes are selective in their retardation of the dissolved species, the increase in the concentration of the individual dissolved species in the hyperfiltrated solution will be different.

This paper summarizes previous experimental work on the retardation efficiencies obtained with geologic membranes and on the relative retardation by these natural materials of inorganic cations and anions generally present in waste solutions. New data on the effect on the retardation efficiency of varying the hydraulic-pressure gradient, and on the relative retardation of Ca and Na obtained with bentonite, are presented. The relative retardation of Ca and Na was investigated because the results of Kharaka (1971) and Kharaka and Berry (in press) showed Ca retardation with respect to Na, which is contrary to theoretical reasoning and some field data (White, 1965; Berry, 1969). The much higher hydraulic gradients used, of necessity, in the experiments (10^5-10^6 times higher than field gradients) were believed to account for this discrepancy.

Preliminary results comparing the relative retardation of alkali and alkaline-earth metals obtained with two bentonite samples maintained at ambient laboratory temperature and 55°C, respectively, are also presented. In the case of the higher temperature runs, precautions were taken to insure that the results reflected the filtration characteristics of the clays and not the changes in their exchange capacity (Grim, 1968) or changes in their ion-exchange selectivity (Bischoff et al., 1970) at these temperatures.

BACKGROUND

Previous Work

McKelvey and Milne (1962) were the first to conduct filtration experiments using geologic membranes. NaCl solutions of different normality were forced to flow through bentonite and shale pads compacted under 10,000-psi pressures. McKelvey and Milne reported a filtration ratio (concentration of input/concentration of effluent) as high as 8 with bentonite samples, and a ratio of 1.7 with shale samples; the filtration ratio decreased as the concentration of the input solution increased. They stated, but did not show, that efficiency of the membranes was minimal for compaction pressures less than 10,000 psi; such behavior suggests increased efficiency with increased compaction pressure. Using a modified apparatus, Milne et al. (1964) more than doubled the ratios obtained for bentonite samples. They further showed that the efficiency of a given membrane increased with the decrease of the input fluid pressure.

Hanshaw (1962) studied the filtration of NaCl solutions through compacted montmorillonite and illite membranes and reported a good agreement between the filtration ratios obtained experimentally and those predicted from a modified theory of Walton (1958). Hanshaw obtained a filtration ratio as high as 81 with montmorillonite when the input modality of the NaCl solution was 0.013N.

Kharaka (1971) and Kharaka and Berry (in press) investigated the flow of artificial seawater and chloride solutions through compacted bentonite, illite, and shale samples from Wyoming. Results of this investigation confirmed the trends reported by previous workers, which show that the efficiency of a membrane increases with the increase of the exchange capacity of materials used and with the decrease in the concentration of the input solution. In addition, this investigation showed that the efficiency of a given membrane increases with increased compaction pressure. Efficiency increased from 0 to 60 percent with compaction

pressures from 0 to 9,000 psi for the Wyoming bentonite and artificial seawater. The efficiency of a given membrane decreased slightly at higher temperatures for solutions of the same concentration. The efficiencies obtained, for example, with the Wyoming bentonite under 6,000-psi compaction pressure and with artificial seawater solution were 33 and 27 percent at laboratory temperature and 70°C, respectively. The results further showed that geologic membranes are specific for different dissolved species (Figs. 1, 2). The retardation sequences varied depending on the material used and on experimental conditions. The sequences for monovalent and divalent cations at 70°C were variable, but at laboratory temperatures they were generally as follows: Li < Na < NH_3 < K < Rb < Cs and Mg < Ca < Sr < Ba. The sequence for anions at room temperature were variable, but at 70°C the sequence was: HCO_3 < I < B < SO_4 < Cl < Br. Monovalent cations were generally retarded with respect to divalent cations.

Theory of Membrane Filtration

The membrane properties of shales result from charge deficiencies on the surfaces and edges of the clay particles. The negative electrical sites of the clays (Carroll, 1959; Grim, 1968; and others) are caused by (1) isomorphous replacement of high-valence cations with cations of lower valence, e.g., substitution of Al^{+3} for Si^{+4} in the tetrahedral layer; (2) broken bonds along the edges of clay platelets; (3) removal of the hydrogen of an exposed hydroxyl group and its replacement by an exchangeable cation; and (4) structural cations other than H^+ becoming exchangeable under certain conditions--e.g., at low pH values, Al^{+3} ions move to the exchange positions.

The membrane properties of the clays composing the shale are modified by the presence of an average of 2 percent by weight of organic material, mainly kerogen, in the shale (Berry, 1969). The organic particles which carry both anionic and cationic exchange sites (Helfferich, 1962) will interact with the clay exchange sites and also will modify the overall exchange properties of shales.

When a clay platelet with its associated exchangeable cations is placed in a solution, the cations are attracted to the charged surface, whereas diffusion tends to carry them away toward the equilibrium solution, where their concentration is lower. Anions are repelled by the surface, but diffusion counteracts the electric repulsion. Thus a "diffuse layer" is formed between the clay platelet and the equilibrium solution in which the concentrations of the cations and anions are higher and lower, respectively, than in the equilibrium solution (Fig. 3). Furthermore, in this "diffuse layer," the cation and anion concentrations increase and decrease, respectively, toward the surface of the clay particle. A double layer, known as the "Gouy-Chapman double layer," is formed by the "diffuse layer" and the charged surface of the platelet.

Increasing compaction pressures will eventually cause the double layers from two adjoining platelets to overlap, lowering the concentration of the anions in the pore solution (Fig. 4). Anion exclusion, whereby the concentration of the anions in the pore solution is lower than their concentration in the external solution, is responsible for the membrane properties of clays and shales.

The "fine-pore membrane" Teorell-Meyer-Siever model, with some modifications, is generally used for computing the concentration of solutions flowing through a membrane (Walton, 1958; Hanshaw, 1962; Kharaka, 1971; and others). This model takes into consideration the concentration of the ions in the pore solution, the velocity of the flowing water, and the electrical interaction of the ions with "streaming potential"--caused by the displacement of the double layer (composed predominantly of cations) by the flow of water. The effect of the "streaming potential" is the retardation of the flow of the cations and the acceleration of the flow of the anions within the membrane with respect to the flowing water.

The electrical interaction of the ions with the negative sites on the clay particles is ignored in this model. Diffusion in and out of the membrane and the interference (coupling) between the various species in solution are also ignored.

The concentration of a univalent salt in the effluent solution m^*_{ij} in this model is given by:

$$m^*_{ij} = \frac{m^2_{ij}}{\left[\frac{\overline{X}_-}{2}\left(\frac{\overline{u}_i - \overline{u}_j}{\overline{u}_i + \overline{u}_j}\right) + \left(\frac{\overline{X}_-^{\,2}}{4} + m^2_{ij}\right)\right]^{1/2}}, \qquad (1)$$

where m_{ij} is the concentration of the input solution, $\overline{X}_-$ is the concentration of the negative sites on the clay particles in moles per liter of pore space, and $\overline{u}_i$ and $\overline{u}_j$ are the ionic mobilities in the pore solution of cation i and anion j. For details of this equation, see Walton (1958), Hanshaw (1962), and Kharaka (1971).

The filtration ratios (m_{ij}/m^*_{ij}) predicted by Equation 1 are lower by up to one order of magnitude than the values obtained experimentally, even in the case of NaCl solutions (Hanshaw, 1962, Table 2; Kharaka and Berry, in press, Table 1). It is clear that the Teorell-Meyer-Siever model is inadequate for predicting the membrane efficiency of natural materials. More experimental work is needed to ascertain the complicating effect of the assumptions inherent in this model.

EXPERIMENTAL EQUIPMENT AND PROCEDURES

The filtration apparatus and the experimental procedures used in this study have been described in detail by Kharaka (1971) and Kharaka and Berry (in press). The filtration apparatus is illustrated in Figure 5. A clay sample (A), having a diameter of 10.16 cm and a thickness of approximately 0.25 cm, is confined in a filtration cell consisting of a plastic-lined K-monel cylinder (B) and pistons (C). Electrical insulation is provided by the plastic liner of the cylinder and the two plastic sheets between the pistons and the framework steel plates. Sintered K-monel discs (D) and millipore filters (E) separate the clay from the channels of the pistons. "O" rings (G) seal the passageway between the cylinder and pistons.

Wyoming bentonite provided by E. Jenne of the U.S. Geological Survey (USGS No. 65CM200) was used in this study. The sample consisted of 92 percent smectite, 6 percent quartz, 1 percent cristobalite, 1 percent plagioclase, and traces of K-feldspar and biotite. Pipette analysis of the sample showed that 100 percent of the material was <74μ, 99 percent <20μ, and 92 percent <0.5μ. The material had an exchange capacity of 88 meq/100 g and a specific gravity of 2.7.

The bentonite samples were equilibrated with the solutions by dispersing about 40 g of the clay in about 400 ml of the experimental solutions. The slurry was stirred for at least 4 hours before centrifuging at 9-11,000 rpm. The supernatant was discarded. This procedure was repeated 10 times. Analysis of Na, Ca, and Cs in the supernatant showed that the concentration of these elements was constant after the fifth mixing. The clay, dispersed in about 150 ml solution, was then transferred to the filtration cell. The desired compaction pressure, applied by a hydraulic press, was maintained for about 24 hours to allow the volume to reach equilibrium with respect to the applied pressure. The steel plates were then bolted together. An air pump was used for pumping the solution at elevated pressures from the reservoir (capacity, 4 l) through the compacted sample. Valves and a timer were used for flushing the concentrated solution on the input chamber back into the reservoir.

In the 55°C run, the equilibrated clay was mixed with about 300 ml solution and allowed to stand in the heated filtration cell for 24 hours before squeezing and subsequent compaction. Ten samples, 15-20 ml in volume, were collected under the same conditions. The chemical composition of the last three samples was the same, indicating that a steady state had been reached.

RESULTS AND DISCUSSION

Table 1 shows the compaction and the hydraulic pressures used, the flow rates obtained, the concentration of species in the reservoir and the effluent solutions, and the calculated filtration ratios (m_i/m_i^*) for the alkali and alkaline-earth metals obtained at room temperatures and at 55°C. The values reported for the concentration of species in the effluent solution for the 55°C run are the steady-state values that did not change with time.

The filtration ratios are a convenient way to express the efficiency of a given membrane and the relative retardation of the species by the membrane. The membrane efficiency is related to the filtration ratio such that a higher filtration ratio for a given ion translates to an increased efficiency of the membrane with respect to that ion. The formal relationship between efficiency and filtration ratios is given by:

$$\text{Efficiency} = 1 - \left(\frac{1}{\text{filtration ratio}}\right) \times 100. \qquad (2)$$

Stated differently, the species is retarded (concentrated in the hyperfiltrated solution) by the membrane with respect to all the ions with lower filtration ratios, but it passes preferentially with respect to the species with higher filtration ratios.

Figure 6 shows a plot of the filtration ratios obtained with Na-Ca chloride solution as a function of hydraulic-pressure gradients at three different compaction pressures. The plot shows that Na is retarded by the membrane with respect to Ca at hydraulic-pressure gradients higher than about 4.0×10^5 psi/ft; the relative retardation of Na increases with increasing gradients. This relation confirms the results reported by Kharaka (1971) and Kharaka and Berry (in press). For hydraulic-pressure gradients lower than about 4.0×10^5 psi/ft, however, Ca is retarded with respect to Na; the relative retardation of Ca with respect to Na increases as the

gradients decrease. Extrapolation of the trends shown in this plot to hydraulic-pressure gradients encountered in the subsurface (generally < 1 psi/ft) shows that Ca will be markedly retarded with respect to Na.

The reversal of the relative retardation of Ca with respect to Na noted above cannot be explained by Equation 1, derived from the Teorell-Meyer-Siever membrane model, because this model ignores the electrostatic interaction of the ions with the negative sites on the clay particles and the effect of the water drag on the ions attracted to the exchange sites. A fluid moving relative to a rigid boundary exerts a dynamic force on the boundary as a result of shear stresses and pressure-intensity variations along the surface. The component of this force in the direction of the relative velocity (V) past the body is the hydraulic drag (F_D). The hydraulic drag exerted on a spherical body by a flow with a low Reynolds number ($R < 1$) is given by the Stokes equation (Robinson and Stokes, 1970):

$$F_D = 6\pi r \eta V,$$

where r is the radius of the sphere and η is the dynamic viscosity of the flowing medium. Assuming that ions within the membrane are hydrated as they are in free solution, then the water drag on Ca ions will be larger than on Na ions, because the Stokes hydrated radii for Ca and Na ions in solution are 4.15 and 3.33 Å, respectively.

Electrostatic forces attracting the cations to the negative sites on the clay particles counteract the hydraulic drag and probably favor the retardation of Ca with respect to Na. This behavior is due to the fact that the intensity of the attraction forces will be higher for Ca ions because of their higher ionic potentials (charge/radius). The effect of interaction with the two poles of the "streaming potential" (negative on the inlet side and positive on the effluent side) will probably cancel out and be insignificant in determing the relative filtration of Ca and Na.

The relative retardation of Ca and Na for a specific clay cake thus will depend on the magnitude of the hydraulic drag which is opposed by the electrical forces of attraction mentioned above. In the absence of a significant drag component, Ca probably would be retarded with respect to Na. However, a high drag component resulting from a comparatively high velocity of flow causes, at a critical point, Na to be retarded with respect to Ca.

Figure 6 also shows that the filtration ratios obtained for both

Na and Ca, and hence the total membrane efficiency, increase with decreasing hydraulic-pressure gradients. Extrapolation of the trends shown in this plot to hydraulic-pressure gradients encountered in subsurface formations indicates that the average filtration ratios will be more than double the values obtained at the experimental hydraulic gradients of 3.0×10^5 psi/ft. This result is extremely important for subsurface waste injection because it shows that the efficiency of geologic membranes in subsurface conditions is even higher than the values reported experimentally.

The decrease in membrane efficiency with increasing hydraulic-pressure gradients, which was also reported by Milne et al. (1964), is most likely related to increased water drag. The higher flow rates obtained by increased pressure gradients probably cause more of the ions held by the negative sites on the clay particles to be carried along with the flowing solution.

The results shown in Table 1 for the samples D and E represent the first results obtained for a chloride solution of alkali and alkaline-earth metals flowing through bentonite at laboratory temperatures and at 55°C, respectively. The ultimate aim of the two runs is to investigate the effects of hydraulic-pressure gradients and temperature variations on the filtration characteristics of these cations. The results from these two runs confirm the findings of Kharaka (1971) and Kharaka and Berry (in press) and show that the retardation sequences for monovalent and divalent cations are: Li < Na < K < Rb < Cs and Mg < Ca < Sr < Ba. The overall membrane efficiency obtained at 55°C is lower than at laboratory temperatures. The filtration ratios obtained for Cs, Rb, and K are markedly lower at the higher temperature; the filtration ratios for divalent cations are generally higher.

The decrease in membrane efficiency observed at higher temperatures probably is due to higher flow rates, which in turn result from a decrease in water viscosity. Kharaka (1971) and Kharaka and Berry (in press) discussed the factors responsible for the observed retardation sequences.

IMPORTANCE OF MEMBRANE FILTRATION TO WASTE MANAGEMENT

When waste solutions are emplaced in a confined aquifer (e.g., a sandstone lens) surrounded by clay or shale aquitards, two alternative situations may arise. In the first situation, the emplacement of waste

results in a higher potentiometric surface (fluid head) in the aquifer compared to adjacent aquifers. The cross-formational flow of water through clays and shales serving as semipermeable membranes will result in the selective retention of various dissolved species in the injection aquifer. The degree of retention will be variable, depending on the physical and chemical properties of the system. Cs, Rb, and K will be retained preferentially to other cations; Br, Cl, and SO_4 will be retained preferentially to other anions. The concentration of dissolved species in the injection aquifer will continue to increase until one of the following steady states is attained: (1) the total and individual concentration of the dissolved species in the effluent solution (solution flowing to the adjacent aquifers) is equal to that in the injection solution; or (2) the fluid head in the injection aquifer is equal to the osmotic pressure caused by differences in the salinity between the injection aquifer and an adjacent aquifer. Osmotic pressure is defined as the hydrostatic pressure that must be applied to the more saline side of a membrane to equalize the flow of water in both directions. The value of this osmotic pressure in natural situations was discussed by Hanshaw (1972). This value can be calculated from the following equation based on thermodynamic arguments:

$$\pi = e \frac{RT}{V} \ln \frac{a^{I}_{H_2O}}{a^{II}_{H_2O}}, \tag{3}$$

where π is osmotic pressure, e is the membrane efficiency, R is the gas constant, T is temperature (°K), V is the mean molar volume of H_2O, and $a^{I}_{H_2O}$ and $a^{II}_{H_2O}$ are the activities of H_2O in solutions I and II, respectively.

The membrane behavior of shales and clays, useful as it is in the retention of undesirable species, may cause other chemical problems. Specifically, the salinity increases mentioned may cause the solution in the injection aquifer to become supersaturated with respect to several minerals whose precipitation may cause loss of porosity as well as other injection problems. A computer program that will test for the states of chemical reactions of a given solution with respect to about 160 minerals under different physical and chemical conditions has been developed by Kharaka and Barnes (1973). This program computes the distribution of species in solution and the Gibbs free-energy difference between the actual and the equilibrium states, ΔG_R, of the mineral reactions.

The equation (Barnes and Clarke, 1969), based on thermodynamic arguments, is:

$$\Delta G_R = RT\ln(Q/K), \quad (4)$$

where R is the gas constant, T is the temperature (°K), Q is the reaction quotient, and K is the equilibrium constant at temperature T. Precipitation of a given mineral will occur when the value of ΔG_R for a given mineral reaction is higher than the activation energy of precipitation for that reaction (Barnes, 1972; Kharaka and Barnes, 1973).

The second situation which may arise when solutions are emplaced in a confined aquifer is that the resulting salinity in the injection aquifer is much higher than in the adjacent aquifers. In this situation the osmotic pressure, given by Equation 3, may be higher than the fluid head in the injection aquifer, resulting in a net water flow from adjacent aquifers into the injection aquifer. This water flow will result in higher fluid pressures than anticipated in the injection aquifer. The higher fluid pressures may induce fracturing in the aquitard rocks and cause other injection problems. This situation was described by Hanshaw (1972).

CONCLUSIONS

The data from this and other investigations demonstrate that clays and shales behave as semipermeable membranes. This property of natural materials must be taken into account when large quantities of waste solutions are emplaced in aquifers confined by these materials. Samples of the confining rocks should be tested for their membrane properties; the testing must be conducted under conditions which simulate natural situations, within experimental limits.

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Table 1. Concentration (mg/l) of Reservoir Solutions (R) and Effluent Solutions Obtained with Bentonite at Stated Experimental Conditions

Sample No.	Compaction Pressure (psi)	Hydraulic Pressure (psi)	Flow rate ml/24hr	TDS[1]	Li	Na	K	Rb	Cs	Mg	Ca	Sr	Ba	Cl
ABe(R)				3860		550					890			2420
ABe3(1.5)	3000	1620	4.4	2540		410					540			1590
ABe7(1.5)	7000	1540	1.6	1480		270					290			930
ABe7(3)	7000	3020	3.3	1870		290					410			1170
ABe7(4.5)	7000	4420	5.4	2120		290					500			1330
ABe8.5(3)	8500	3030	2.6	1390		220					300			870
ABe8.5(5)	8500	4980	4.8	2050		260					500			1290
ABe8.5(6.5)	8500	6700	7.6	2320		270					590			1460
D & EBe(R)				6100	52	180	260	380	730	160	210	630	650	2810
DBe3(2)	3000	2020	5.3	4250	47	145	110	135	90	190	190	500	600	2240
				(1.44)	(1.11)	(1.24)	(2.36)	(2.81)	(8.10)	(0.84)	(1.11)	(1.26)	(1.08)	(1.25)
EBe3(2)H	3000	1970	8.9	4280	49	135	140	180	260	180	165	480	500	2190
				(1.43)	(1.06)	(1.33)	(1.86)	(2.11)	(2.81)	(0.89)	(1.27)	(1.31)	(1.30)	(1.28)

[1]TDS=calculated total dissolved solids; sample E is at 55°C; numbers in parentheses underneath the concentration values of samples D and E are the computed filtration ratios.

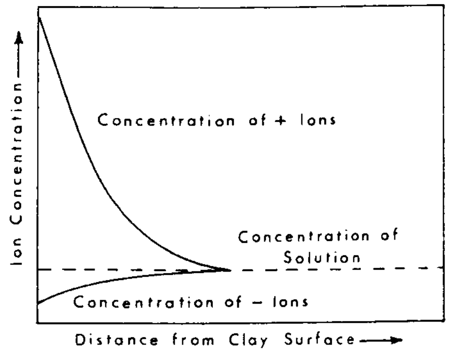

FIG. 1--Concentration of cations and anions in double layer. Concentrations of cations and anions increase and decrease, respectively, as surface of clay platelet is approached.

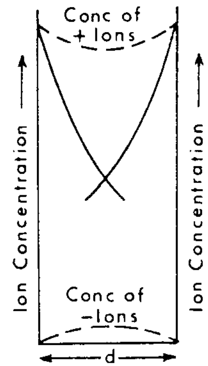

FIG. 2--Concentration of positive and negative ions between clay platelets at a distance (d) that is smaller than the total thickness of the two double layers.

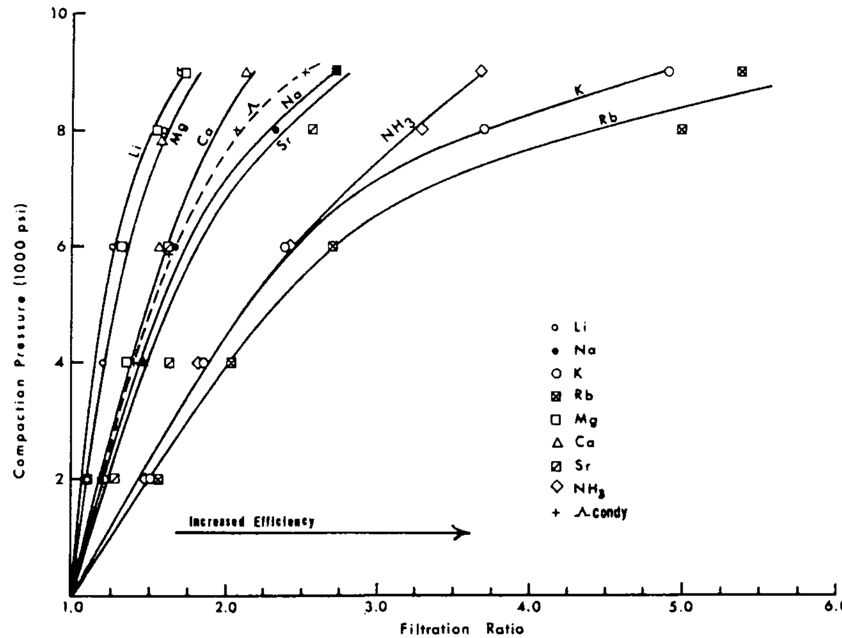

FIG. 3--Compaction pressure vs. filtration ratio for cations and conductivity of an artificial seawater solution flowing through bentonite. From Kharaka and Berry (in press).

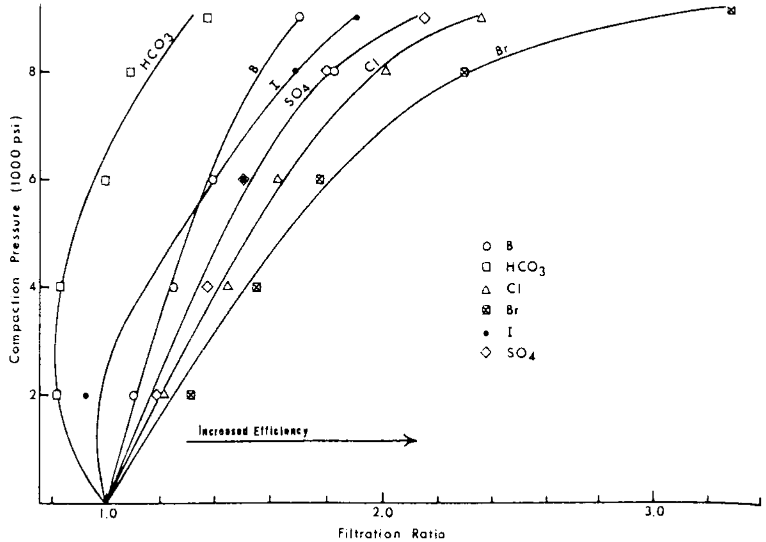

FIG. 4--Compaction pressure vs. filtration ratio for anions in an artificial seawater solution flowing through bentonite. From Kharaka and Berry (in press).

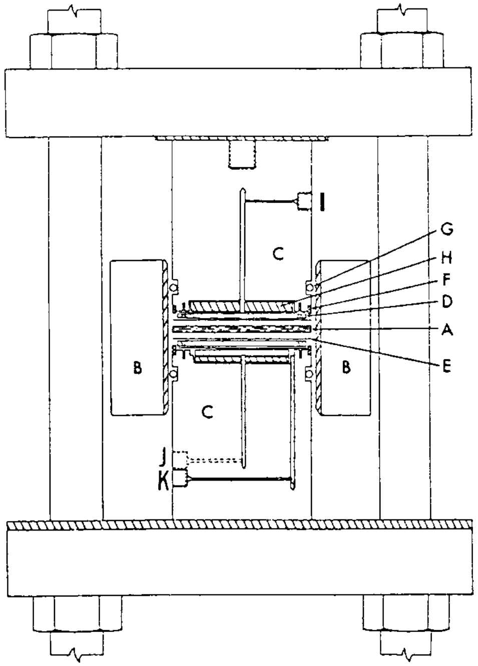

FIG. 5--Diagrammatic sketch of filtration cell. A=clay sample; B=plastic-lined K-Monel cylinder; C=K-Monel pistons; D=K-Monel sintered discs; E=Mill pore filters; F=Teflon rings; G="O" rings; H=plastic discs; I=outlet for effluent solution; J=inlet for input solution; K=outlet for flushing hyper-filtrated solution. Modified from Kharaka and Berry (in press).

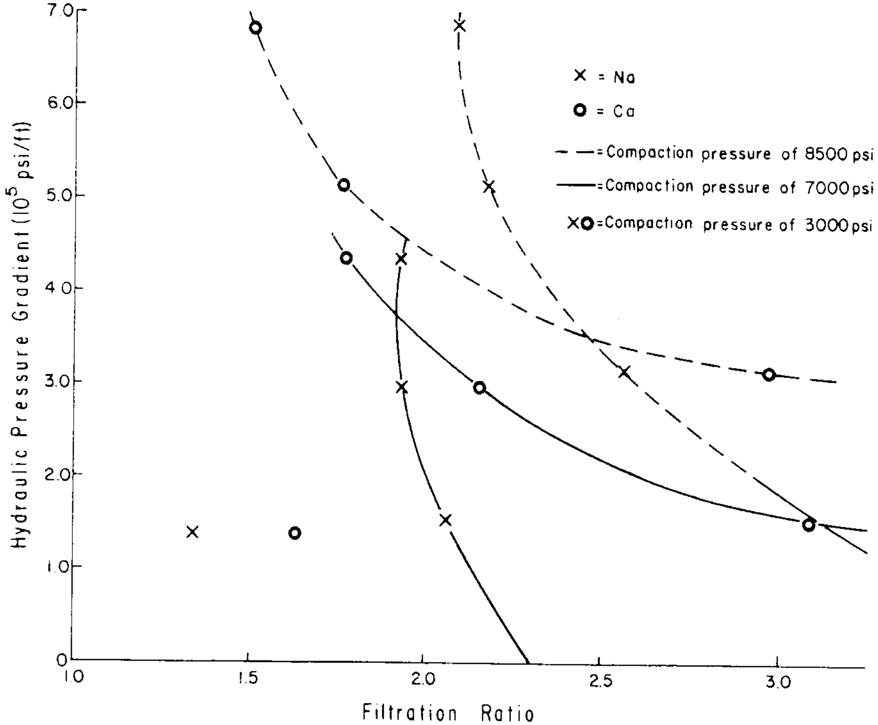

FIG. 6--Hydraulic-pressure gradient vs. filtration ratio for Na and Ca obtained at three different compaction pressures for a Na-Ca chloride solution flowing through bentonite.

HYDROGEOLOGIC STUDIES AT A SUBSURFACE RADIOACTIVE-WASTE-MANAGEMENT SITE IN WEST-CENTRAL CANADA[1]

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ABSTRACT One of Canada's two main subsurface radioactive-waste-management sites is located at the Whiteshell Nuclear Research Establishment in southeastern Manitoba. The area receives low-, medium-, and high-level solid wastes and small amounts of liquid waste. The wastes are buried at depths as great as 15 ft (5 m) below ground surface and are below the water table, which is normally within a few feet (about 1 m) of the ground surface. Only low-level solid wastes are not protected by metal and/or concrete containers. No significant groundwater contamination has occurred since use began in 1964.

Hydrogeologic studies of the area were conducted during a 5-year period. The methods of investigation included geologic test drilling, surface geophysics, installation and monitoring of an extensive piezometer and well network, short- and long-term pumping tests, single-well response tests, tracer-injection experiments, hydrochemical studies, and mathematical simulations of the groundwater flow pattern. This study resulted in an integrated interpretation of the groundwater flow system which was used to evaluate the waste-management properties of the existing site and adjacent terrain.

The waste-management operations are carried out in deposits of Pleistocene clay and clay-loam till. The deposits have significant secondary hydraulic conductivity resulting from numerous fractures. Because of high hydraulic head in a sandy deposit below the burial zone, the direction of natural groundwater flow is upward through the till and clay into the water-table zone.

The high water table is not a serious limitation at the site. Leaching of solid wastes eventually may produce contaminated groundwater, but the natural groundwater flow system will localize the contaminants in or near the soil zone. Monitoring and removal, if necessary, would be relatively simple. In the unlikely event of leakage of liquid wastes into the groundwater zone, the natural groundwater flow system will minimize the hazard by localizing the contamination at shallow depths in the waste-management area or by transporting the radionuclides at very slow rates in an underlying sandy deposit. The hydrogeologic studies show that contaminants in the sand could be effectively controlled or removed by a combination of natural sorption processes and well pumping.

INTRODUCTION

This paper summarizes the results of geologic and hydrologic studies that were conducted during the past five years at one of Canada's six subsurface radioactive-waste-management sites. The site is located at the Whiteshell Nuclear Research Establishment (WNRE) in southeastern Manitoba, about 75 mi (175 km) northeast of Winnipeg (Fig. 1). From an environmental viewpoint, the site is of special interest because the wastes are being buried below the water table. The hydrogeologic study described in this paper represents the most detailed subsurface investigation ever to be conducted at a waste-management site in Canada.

The waste-management area at WNRE occupies about 6 acres (2.5 ha.) and has been in operation since 1964. The area receives radioactive wastes from experimental reactor operations, and from the metallurgical, biological, and chemical laboratories at WNRE. Small amounts of wastes from universities and industries in west-central Canada are also buried at the site.

Before its operational use commenced, the waste-management area at WNRE was evaluated to confirm its suitability for this purpose, and it is now continuously monitored as a normal practice to insure that the waste remains confined to it. The study on which this paper is based was commissioned by Atomic Energy of Canada Limited to define more precisely the subsurface waste-management capacity of the delineated area and to appraise the waste-management characteristics of the adjacent terrain.

WNRE is situated on the western edge of the Canadian Precambrian shield. The Precambrian granite bedrock in the study area is overlain by 30-80 ft (10-27 m) of Pleistocene strata, mainly clay, clayey till, and sand. Mature soils have developed on the Pleistocene deposits. Terrain near the waste-management area was under cultivation prior to the establishment of WNRE. Mixed-forest vegetation such as aspen, willow, spruce, and pine is common in adjacent areas. The water table is normally within a few feet of ground surface.

The climate is mid-continental, characterized by extreme temperature variations. Recorded temperature extremes over the study period were a high of 93°F (35°C) and a low of -42°F (-23°C), with a mean of about 34°F. During the winter the ground usually freezes to depths of 1-3 ft (0.3-1 m).

The annual precipitation during the study period varied from 16 to 26 in. (41-66 cm); the greatest amount was measured between May and September.

WASTE-MANAGEMENT METHODS

Most of the radionuclides placed in the waste-management area consist of activation products from structural materials, impurities, cor-

rosion products, fission products from the reactor, and other wastes from research and development laboratories and decontamination facilities. The wastes contain a multiplicity of radionuclides such that complete analysis of all the radionuclides present in the waste has not been practical. The main constituents are radioisotopes of uranium, plutonium, strontium, and cesium. It is estimated that approximately 21,000 curies of Sr^{90} and Cs^{137} have been put into the area, almost entirely in the form of solid wastes. At least several hundred years of decay time would be required for the most hazardous of the radionuclides such as Sr^{90} and Cs^{137} to reach low activity levels.

On a volume basis most of the radioactive materials buried at the WNRE site are classified as low-level waste. These wastes are placed in 12-ft deep (4 m) unlined trenches about 50 ft (17 m) long and 20 ft (7 m) wide (Fig. 2). Filled trenches are covered and compacted with the excavated fill, mainly clay and till.

A series of concrete standpipes 2 ft (0.7 m) in diameter and 12 ft (4 m) deep are located in the southwest section of the waste-management area. These standpipes accept medium-level solid and liquid waste and some small flasks of high-level liquids. The first standpipes were asphalt-coated precast concrete placed in augered holes on wet cement pads and then backfilled. These standpipes were not watertight and some have subsequently been found to contain 6-8 ft (2-3 m) of water. The newest standpipes are poured in place and are steel lined, but still cannot be considered watertight over long periods of time. A 12-ft-deep (4 m) asphalt-coated concrete bunker receives the larger volumes of medium-level solid waste. A part of this bunker has been installed for approximately ten years with only a small amount of leakage evident to date.

Two 120-Am. gal (450 l) double-walled stainless steel tanks are buried in a concrete receptable in the north-central part of the waste-management area. The tanks are designed to receive high-level liquid waste and are equipped with monitoring facilities at the surface. The original excavation in which the concrete receptable was poured was about 19 ft (6 m) below ground surface.

METHODS OF INVESTIGATION

To evaluate the waste-management properties of shallow subsurface environments adequately, it is necessary to obtain a high degree of hydrogeologic predictability. This can be achieved best by using several types of investigative techniques so that independent results can be compared and evaluated. This approach was used in the study at WNRE. The main field and analytical methods are summarized in Table 1.

Geologic Characteristics of Pleistocene Deposits

The gross physical character and stratigraphic relations of the Pleistocene deposits overlying the Precambrian granite were defined by 76 test holes drilled by vehicle-mounted hydraulic drills equipped with solid and hollow-stem augers. The test holes were drilled to depths at which further penetration was not possible. A few of the holes were sampled using Shelby tubes.

Eight additional holes penetrating to the bedrock surface were drilled using a cable-tool drill and split-spoon sampler. These holes provided more detailed stratigraphic and lithologic data, particularly from sandy beds which were not adequately sampled using the auger drills. Six of the cable-tool holes were drilled near the waste-management area, and one was drilled in the center of the area (Fig. 2). The eighth hole was drilled 1/2 mi (0.8 km) west of the waste-management area.

The location of the bedrock surface was further defined by a geophysical survey which included siesmic refraction and resistivity profiles along east-west transects through the area (Chagarlamudi, 1971).

Distribution of Hydraulic Head

To define the hydraulic-head distribution in the Pleistocene deposits, 65 piezometer nests were installed using the auger drills (Fig. 3). Each nest comprises 3-5 individual piezometers set in separate drill holes a few feet apart. The piezometers were constructed of 1-in.-diameter (2.5 cm) semi-rigid plastic pipe with a slotted and fiberglass-wrapped interval at the bottom. At each site the piezometers were placed at various depths between the water table and the bottom of the Pleistocene deposits. The water levels in the piezometers were monitored routinely during the period of 1968-1973.

In 1971-1973, a network of observation wells was installed in and near the waste-management area (Fig. 2). The wells were constructed of 4-in.-diameter (10 cm) pipe with screened intervals at the bottom. Seven of the wells were set in sandy deposits near the bedrock. Five wells were set in sandy deposits near the bedrock. Five wells were placed in the water-table zone with slotted intervals between 6 and 10 ft (3 and 4 m) below ground surface. The wells were equipped with water-level

recorders soon after installation.

Field Permeability Tests

Nearly all of the piezometers were subjected to water-level response tests using the methods outlined by Hvorslev (1951). Hvorslev has shown that the rate of water-level response in a piezometer can be related to the average hydraulic conductivity of the porous medium adjacent to the piezometer intake.

Short- and long-term pumping tests were conducted using four deep observation wells near the waste-management area as pumping wells. Water-level changes caused by pumping were monitored in the other wells and piezometers in the area. Single-well response tests of the type described by Cooper et al. (1967) were also conducted in the deep wells.

Hydrochemical and Tracer Studies

Water samples were collected from the bottom few feet (about 1 m) of each piezometer using a manual suction pump. The electrical conductivity and pH of the samples were measured in the field. Concentrations of the major ions were determined in the laboratory using standard methods (Cherry et al., 1971).

Groundwater flow velocities were investigated by injecting tritiated water into the Pleistocene deposits at three locations. The locations of the injection sites are shown in Figures 2 and 3. Two of the tritium injections were made in a basal sand deposit, the third in glacial till. Each injection site comprises an injection well and between 17 and 35 piezometers located predominantly in the direction of anticipated flow. Injections were made in early summer, 1969. The flow paths of the tritium are being mapped by sampling the piezometers several times annually. The tritium content of the samples is determined in the laboratory using a liquid scintillation counter.

In late 1972 water samples were collected for age-dating analysis using natural C^{14} and tritium produced by thermonuclear testing. The results of these studies are not yet complete.

Flow-Pattern Simulations

A segment of the groundwater flow system was studied using a two-dimensional methematical model. The model is based on the solution of Richards' equation describing steady-state flow of water in nonhomogeneous, anisotropic porous media bounded by an irregular water table and flow barriers. Solutions were obtained using a computer program developed by Pinder and Frind (1972), using isoparametric finite elements. The program was adapted for use in the cross-sectional plane by Gillham (written commun., 1973).

GEOLOGY

The geology of the study area is summarized in Figures 4 and 5, which show stratigraphic relations between the silt, clay, till, and sandy deposits which overlie the Precambrian bedrock.

The silt and clay units were deposited in Glacial Lake Agassiz, which occupied much of southern Manitoba about 10,000 years ago (Elson, 1966). The silt unit includes thin laminated beds of fine sand, sandy silt, and clay. The clay unit is varved and includes numerous pebbles and clasts of till and silt which were probably dropped from floating ice masses on the lake during the main periods of bottom sedimentation.

The glacial till below the lacustrine clay is a dense overconsolidated clay-loam, pebbly till deposited about 12,000 to 15,000 years ago as a result of a glacial advance from the northwest. A layer of Precambrian cobbles and boulders is present at the base of the till. Detailed descriptions of the silt, clay, and till units are given by Beswick (1971) and Clister (1973).

Between the till and the Precambrian bedrock there is a sandy drift deposit referred to in this paper as the basal sand. It appears at the surface in the eastern part of the study area as a medium-grained, well-sorted lacustrine sand but grades laterally westward into an extremely heterogeneous unit with numerous lenses of sandy silt, silt, and clay. In some areas it includes gravels which contain various amounts of sand and silt. The sediments in the basal sand appear to have been deposited in close proximity to a glacier front. Although the eastern extension of the basal sand crops out on both sections A-A' and B-B', it is not vertically continuous in both sections. On section B-B' the continuity is interrupted by the clay and till units.

HYDROSTRATIGRAPHY

A hydrostratigraphic unit is a body of rock or unconsolidated sediment with considerable lateral extent and with distinctive hydrogeologic properties continuous throughout the unit (Maxey, 1964). As they exist

at the WNRE, the hydrostratigraphic units are layers of glacial drift. They are (1) the basal sand, (2) the clay-loam till and lacustrine clay, and (3) the surficial silty unit.

The bedrock does not crop out in the study area, but it appears at the surface nearby as small, rounded hills. The rock is a medium-grained granite laced with veins and microdikes. Fractures in the rock are widely spaced at unpredictable intervals.

An excavation made during construction of the reactor building at the WNRE plant site a mile (1.6 km) south of the study area exposed several thousand square feet of the bedrock surface. Only one fracture was present in the exposed area. Observations by R. A. Parsons (written commun., 1963) indicate that water discharged from the fracture at a rate of 5-10 gal (20-50 l) per minute. Consequently, although the bedrock can be regarded as a relatively impermeable unit when considered on a broad scale, there is no doubt that fractures in the rock can exert significant local influence on the hydrologic system.

Basal Sand

The most useful information on the hydraulic properties of the basal sand was obtained from the pumping tests, the results of which are summarized in Table 2. The average transmissivity of the basal sand in the vicinity of the waste-management area is 290 gal/day/ft (3.6 m^3/day/m). The average storage coefficient is 2.4×10^{-4}. Table 2 indicates that the transmissivity and storage coefficient as determined from pumping tests vary within one third of an order of magnitude. The variability of the pumping-test results reflects the textural heterogeneity of the deposit and variations in thickness. The single-well response tests gave transmissivities in the same order of magnitude as the pumping tests.

A mass-balance calculation using the assumption that one tenth of the annual precipitation recharges the groundwater flow system in the eastern part of the area suggests a bulk transmissivity of about 100 gal/day/ft (1.24 m^3/day/m). This calculation also required use of the hydraulic-head distribution.

Clay-Loam Till and Lacustrine Clay

The hydraulic properties of the clay-loam till and clay units are particularly important because all of the radioactive wastes at the waste-management site are buried in these deposits.

Shelby-tube samples and observations in trenches and pits in the waste-management area indicate that the clay and till are fractured. The sidewalls of a newly-opened pit examined in 1972 had fractures over the entire 15-ft (5 m) depth of the excavation. The fracture orientations were approximately north-south and east-west with vertical to 85° dips. The fractures were continuous across the contact between the till and lacustrine clay units. The geologic origin of the fracture system is discussed by Grisak and Cherry (1973).

Hydraulic-conductivity values obtained by Shawinigan Engineering Company (1960) using samples of the lacustrine clay were between 1.7 x 10^{-11} and 8.6 x 10^{-10} ft/sec (5.1 x 10^{-10} and 2.6 x 10^{-8} cm/sec). The samples were disturbed during sampling and repacked before testing. The results represent intergranular conductivity values. The order of magnitude obtained is typical of clayey deposits (Hvorslev, 1951).

The Hvorslev response tests conducted in piezometers in the till and clay yield hydraulic-conductivity values in the range of 10^{-6} to 10^{-10} ft/sec (3 x 10^{-5} to 3 x 10^{-9} cm/sec). These results presumably reflect the fact that the piezometer intake zones may or may not intersect various frequencies and types of fractures. The results may also be strongly affected by borewall disturbances during drilling, by irregularities in the sand pack around the piezometer screens, and by sediment clogging in the intake zones. These factors would tend to decrease the apparent hydraulic conductivities. The Hvorslev test results clearly establish that the fractures cause the till and clay deposits to have effective conductivities which are appreciably larger than would be the case if intergranular conductivities were the dominant influence.

During the pumping tests in the basal sand, water levels in the piezometers and wells located in the till and lacustrine clay were observed. The effects of the pumping were generally detected in these deposits within a few hours after pumping began. The times and magnitudes of the responses were used to estimate the bulk vertical hydraulic diffusivity of the till and clay units. The analytical methods developed by Hantush (1960) and Neuman and Witherspoon (1969a, 1969b, 1972) were used. Hydraulic diffusivity is defined as the hydraulic conductivity, in this case the bulk vertical hydraulic conductivity, divided by the specific storage of the beds.

To obtain vertical-hydraulic-conductivity values from the diffusivities, it was necessary to estimate the specific storage of the till and

clay. This was done using laboratory data on the void ratios, compression indices, and liquid limits of the clay and till. The laboratory tests were conducted by Shawinigan Engineering Company (1960).

The vertical-hydraulic-conductivity values obtained from the above analysis are in the range of 1 x 10^{-8} to 1 x 10^{-11} ft/sec (3 x 10^{-7} to 3 x 10^{-10} cm/sec). The analysis of the hydraulic properties of the till and clay is discussed in detail by Grisak and Cherry (1973). The fact that the values obtained were as much as two orders of magnitude greater than the laboratory hydraulic conductivities indicates that the fractures impart an effective bulk vertical hydraulic conductivity to the till and clay deposits. The significance of this factor with respect to radioactive-waste management at the stie is discussed below.

GROUNDWATER FLOW SYSTEM

Interpretations of the groundwater flow patterns in the study area are shown in Figure 6. The distribution of equipotential lines is based on average water levels obtained from the piezometer and well network during the period of May to July, 1971. The flow patterns are representative of the conditions that prevailed during the entire study period from 1968 to 1973. The potentiometric surface and approximate flow directions in the basal sand and lacustrine sand are shown in Figure 7. It is apparent from Figure 7 that the groundwater flow is generally parallel to cross-section A-A', whereas significant components of flow in the third dimension are common along cross-section B-B'.

The groundwater flow system has two distinct types of hydrologic environments. Using the terminology of Meyboom (1962) and Toth (1963), these environments are referred to as "recharge" and "discharge" areas. In recharge areas water moves downward from the water table into the flow system. In discharge areas water moves upward to the water table where it is removed from the groundwater system by evapotranspiration and in some areas by surface runoff. The recharge and discharge areas are separated by "transition zones" in which the flow is mainly lateral.

Water enters the groundwater zone in the upland recharge area and moves downward and then laterally toward the Winnipeg River (Fig. 6). As the water moves westward in the lacustrine sand and in the basal sand, upward hydraulic gradients in the overlying till and clay cause some of the water in the sands to flow upward into the central discharge area. Most of the water, however, continues to flow westward in the basal sand to a groundwater "sink" located less than 1/2 mi (0.8 km) east of the

Winnipeg River (Figs. 6, 7). The sink is an area of anomalously low hydraulic head into which water must flow from east and west and from above. It appears that the anomalous-head zone is caused by a large north-south-trending fracture or fracture zone which has sufficient hydraulic conductivity to channel water out of the basal sand. It is not known if the water in the fracture moves laterally at shallow depths or if it flows into a deeper extension of the fracture. The anomaly was not recognized during the first few years of the investigation (Cherry et al., 1971) because the piezometer network was incomplete in the central recharge area. Water recharging the groundwater zone west of the anomaly flows downward in the basal sand and then moves laterally to the Winnipeg River, where it discharges along the river bank and in the river bed.

The groundwater velocities associated with the flow patterns are important with respect to subsurface radioactive-waste management. Figure 6 indicates that groundwater in the waste-management area flows directly upward through the till and clay to the water-table zone. The lateral gradient components in the till and clay are very small compared to the upward components. Using the Darcy equation and an average vertical hydraulic gradient of 20 percent and an average vertical hydraulic conductivity of 1×10^{-9} ft/sec (3×10^{-8} cm/sec), the upward seepage flux in the till and clay is calculated to be 6×10^{-1} ft (2.1×10^{-1} m) per year. Assuming that the porosity of the fractured media is about 5 percent and using the Dupuit-Forchheimer assumption, which states that the average interstitial velocity is represented by the seepage flux divided by the effective porosity (Scheidegger, 1960), the rate of upward water flow is 3×10^{-2} ft (1×10^{-2} m) per year. Estimtes of groundwater velocities using average values of hydraulic conductivity in the till and clay may have little significance because the fracture conductivities are probably quite variable locally. In zones where one or more open fractures extend directly to the water table, the upward velocities could be many orders of magnitude larger than the calculated average values. Uncertainties of this magnitude cannot be avoided when dealing with complex conductivity networks such as occur in the till and clay.

The lateral groundwater velocities in the basal sand beneath and west of the waste-management area are also of special interest in this investigation. The velocities were estimated using two methods: (1) movement of the tritium tracer and (2) the Darcy equation with the Dupuit-Forchheimer assumption.

Monitoring of the tritium-injection site near the north boundary of

the waste-management area has indicated that the tritiated water has been moving west-northwestward at a rate of approximately 15-30 ft/yr (5-10 m/yr) during the past 3 years. Tritium is generally regarded as an excellent tracer because it travels at essentially the same rate as natural water.

Velocity estimates from the Darcy equation and the Dupuit-Forchheimer assumption require values of horizontal hydraulic gradient, effective porosity, and hydraulic conductivity. The hydraulic gradients along the flowlines were obtained from the piezometer measurements. An effective porosity of 0.35 ± 0.05 was considered to be a reasonable estimate. The hydraulic conductivities were obtained using three methods: (1) pumping tests, (2) single-well response tests, and (3) mass-balance calculations. The results are quite consistent and indicate horizontal velocities between 10 and 25 ft/yr (3 and 8 m/yr). These values are similar to the tritium-tracer results. By extending the velocity calculations westward along the flow path in the basal sand, it is estimated that it takes about 100 years for groundwater to flow from the vicinity of the waste-management area to the fracture anomaly.

Groundwater age-dating studies currently in progress using C^{14} and tritium from thermonuclear tests should provide further information on the groundwater flow rates in the basal sand.

GROUNDWATER CHEMISTRY

The concentrations of major ions in the groundwater zone of WNRE are the results of geologic, hydrologic, and chemical interactions. All of the major ions (Na^+, Ca^{+2}, Mg^{+2}, HCO_3^-, Cl^-, SO_4^{-2}) vary considerably in the flow system. The trends are illustrated by the distributions of Ca^{+2}, SO_4^{-2}, and electrical conductivity shown in Figure 8. The electrical conductivity is an approximate measure of the total dissolved solids. Comparison of this figure with the stratigraphy and flow patterns shown in Figures 5 and 6 indicates that water entering the groundwater flow system in the upland recharge area has relatively low major-ion concentrations. The concentrations remain low as the water moves westward in the basal sand beneath the central discharge area. Water flowing upward from the sand through the till and clay becomes saline over relatively short travel distances. The salinization of the groundwater occurs because soluble minerals in the till and clay, such as gypsum and halite, gradually dissolve. Ion exchange on clay minerals alters the cation ratios in the groundwater. Concentrations in the water-table zone also increase because of evapotranspiration. Figure 8 indicates that the burial zone at the waste-management site has relatively saline groundwater, with high concentrations of Ca^{+2} and SO_4^{-2}. Because of the upward-flowing groundwater, salts have been accumulating for a long time in the water-table zone of this area.

The chemical pattern below the central recharge area also strongly reflects the flow pattern. The basal sand in this area is fed water from the overlying till and clay and is therefore saline. The distinct lateral change in water chemistry in the basal sand coincides with the location of the fracture anomaly. It marks the termination of the lateral flux of fresh water from the upland recharge area. A more detailed discussion of the groundwater geochemistry at WNRE is given by Cherry et al. (1971) and Cherry (1972).

In conclusion it can be stated that the pattern of groundwater chemistry supports the interpretation of the groundwater flow pattern derived previously on the basis of hydraulic-head data.

FLOW-PATTERN SIMULATIONS

The flow pattern along cross-section A-A' was chosen for detailed digital-model simulation studies because the groundwater flow in the vicinity of this cross section is reasonably two-dimensional. The finite-element grid which was used in the simulation is shown in Figure 9. A constant water table was used as the upper boundary of the model. The base of the flow domain was assumed to be an impermeable barrier. The eastern part of the flow area was bounded by an impermeable barrier representing a groundwater flow divide. The groundwater sink in the presumed fracture zone was simulated as a fixed-head node in the model. Relative vertical and horizontal hydraulic conductivities were assigned to each of the finite elements.

Using these boundary and internal conditions, the model was used to simulate the hydraulic-head distribution in the flow domain. The simulated head distributions were then compared to the head values obtained from the piezometer and observation-well network. The relative hydraulic conductivities assigned to the elements were adjusted until the simulated head distribution was similar to the field head distribution.

The best simulation results were obtained using the relative hydraulic-conductivity values shown in Figure 9B. Use of moderate degrees of anisotropy in the basal sand has negligible effect on the simulated flow pattern. Figures 9C and 6 can be used to compare the simulated and real hydraulic-head distributions.

The simulation results suggest that the bulk hydraulic conductivity of the clay-loam till overlying the basal sand in the vicinity of the waste-management area is approximately four orders of magnitude less than the bulk hydraulic conductivity of the basal sand in this area. The average hydraulic conductivity in the basal sand as determined from the pumping tests is 2.7×10^{-5} ft/sec (8.2×10^{-4} cm/sec) (Table 2). Combining this value with the relative hydraulic conductivities from the simulation results indicates a bulk hydraulic conductivity of about 2.7×10^{-9} ft/sec (8.2×10^{-8} cm/sec) in the clay-loam till. This value is in the middle of the hydraulic-conductivity range obtained by analyzing the hydraulic-head response in the system during the aquifer pumping tests.

Use of a fixed-head node to represent the fracture anomaly produced a realistic hydraulic-head pattern in the flow-system segment beneath the central recharge area. The node permits water to be conducted out of the cross-sectional flow area and is analogous to flow downward into a fracture or lateral flow out of the cross-sectional plane.

The effect on the groundwater flow pattern of large changes of the water table was studied using the digital model. For example, during a long drought the water table will decline, possibly as much as 3-9 ft (1-3 m) below normal levels. The greatest drop occurs in the upland recharge area. Simulations using this type of water-table configuration indicated that the main characteristics of the flow pattern shown in Figure 6 are not altered appreciably, although of course the absolute values of hydraulic head changed considerably. The positions of the boundaries between the recharge, discharge, and transition areas remained relatively stable. The effect of abnormally high water-table levels on simulated flow patterns was also found to be small. It is concluded therefore that the flow pattern observed during the period of field observations is representative of the long-term groundwater conditions in the area.

IMPLICATIONS WITH RESPECT TO WASTE MANAGEMENT

Containment Capabilities of Natural Hydrogeologic Regime

Prior to our hydrogeologic studies of the waste-management area there was concern that radioactive contaminants might eventually be transported by groundwater to the Winnipeg River and thereby enter public waterways. The results of our investigation indicate that contaminants which may eventually enter the groundwater zone as a result of leaching of solid wastes have no possibility of moving through the groundwater to the

Winnipeg River. The natural groundwater flow in the till and clay of the waste-management area is upward. Radionuclides slowly leached from solid wastes would therefore be carried upward through fractures into the soil zone in the waste-management area. If vegetation and burrowing animals are totally excluded from the area and if soil erosion is prevented over long time periods, the radionuclides would tend to remain fixed by sorption on clays and organic matter in the upper few feet (1 m) of the soil profile. Sorption would probably not be extremely effective below the solum because the fractures are generally coated with iron oxides and with salt precipitates such as calcite and gypsum.

In the event that radionuclides are not totally sorbed in the soil and subsoil of the waste-management area, the dominant upward groundwater flow direction will insure that the contaminants can be monitored easily and removed easily by excavation if necessary. It is evident, therefore, that if groundwater contamination is caused by leaching of solid wastes the hydrogeologic system will act as an effective second line of defense.

The high-level liquid wastes intended for the double-walled stainless steel tanks will be produced mainly from experimental studies on reprocessing of spent fuel. This liquid waste is significantly more dense than the natural groundwater in the area. In the unlikely event that corrosion of the tanks causes direct leakage of this waste into the groundwater zone, the radionuclides possibly could move downward through the fractures surrounded by upward-flowing groundwater and eventually enter the basal sand. The distance from the bottom of the tanks to the top of the sand is about 10 ft (3 m). Because there are numerous horizontal silt and clay lenses within the basal sand, the radionuclides would not continue to sink very far downward in this deposit. Westward transport of the radionuclides in the sand would gradually occur.

The question arises as to the consequence of this type of contamination event. The radionuclides in the sand would be transported westward toward the fracture anomaly. If radionuclides were to enter the fracture zone, their travel paths would be impossible to predict on the basis of present data and would be extremely difficult to monitor, even with very expensive installations.

As indicated above, the groundwater travel time in the basal sand between the waste-management area and the fracture zone is calculated to be about 100 years. Inasmuch as the basal sand is a very heterogeneous deposit, the hydraulic conductivities along the entire flow path cannot be determined accurately. The 100-year travel-time estimate could conceivably

be in error by as much as half an order of magnitude. In other words, travel times in the range of 20-500 years would be within the realm of possibility.

It is probable that even relatively mobile radionuclides such as Sr^{90} would travel much slower than the groundwater. The main retardation process would be sorption. Ewing (1959) injected Sr^{89} into a sandy aquifer in California and observed that it traveled at a rate of only a few percent of the groundwater velocity. Parsons (1961) observed the movement of Sr^{90} in a shallow Pleistocene sand at Chalk River in east-central Ontario. The radionuclides were transported at a rate of about 3 percent of the groundwater velocity. The Chalk River sand is relatively clean compared to the basal sand at WNRE. The clay minerals at WNRE are of the montmorillonite group and have a characteristically high cation-exchange capacity when not influenced by excessive salt concentrations (Mills and Zwarich, 1970). Salt concentrations in the basal sand between the waste-management area and the fracture anomaly are relatively low compared to the concentrations in the other parts of the flow system.

It is probable, therefore, that radionuclides in the basal sand would travel at rates less than 10 percent of the groundwater velocity. The most hazardous radionuclides, Sr^{90} and Cs^{137}, would probably require at least several hundred years to reach the presumed fracture zone. This is sufficient time for most of the Sr-Cs radioactivity to decay to low levels. If only small amounts of contaminants were to enter the basal sand, it might be possible for all but negligible amounts of the radionuclides to be sorbed along the flow path in the basal sand and therefore prevent contamination from reaching the fracture zone. However, it must be recognized that, before accurate prediction can be made regarding the movement of radionuclides in the basal sand, intensive laboratory and field sorption experiments would have to be conducted.

Containment by Manipulating Hydrologic Regime

If contamination of groundwater should ever occur at the waste-management site, it might be desirable to contain or remove the radionuclides by hydrologic manipulation of the groundwater flow system. The hydrogeology of the site is well suited for manipulation procedures.

For example, if radionuclides leached from the solid wastes enter the groundwater in the till or clay, it may be considered desirable to prevent the radionuclides from moving upward into the soil zone. Inducement of downward movement of radionuclides would be one way of preventing radioactive contaminants from entering the ecosystem. This could be done by reversing the direction of groundwater flow in the till and clay. Downward flow can be produced simply by pumping one or more wells in the basal sand. Under natural conditions the flow is upward through the clay and till because the potentiometric surface in the basal sand is above the water table. This relation is shown schematically in Figure 10A. Pumping water from the basal sand causes a drawdown cone in the potentiometric surface, thereby reversing the hydraulic gradient. This causes downward flow. Figure 10B shows the water-table and potentiometric-surface configurations that were produced by 32 days of pumping a well (W-3) near the southwest corner of the waste-management area. The well was pumped at an average rate of 4 gal/day (1.8×10^{-2} m^3/day). By the end of the pumping period the drawdown cone had nearly stabilized and the gradients in the overlying till and clay clearly were reversed. The process is entirely reversible as indicated by the fact that the potentiometric surface returned to its original position within about 30 days after pumping stopped.

If pumping of one or more of the wells in the basal sand were continued for long periods of time, the radionuclides would move through the clay and till into the basal sand, subject to sorption en route. Sorption capacity in the clay and till would be expected to increase with depth because the natural groundwater is less saline and because the fracture surfaces have undergone less alteration by mineral precipitates. The greatest influence of radionuclide sorption would be achieved if the radionuclides were drawn into the basal sand by maintaining the drawdown cone by pumping.

Upon entry of radionuclides into the basal sand a variety of manipulation schemes would be possible. Choice of a manipulation scheme would depend on the waste-management objectives. For example, using the existing wells or new wells, it would be feasible to induce radionuclide movement in any desired lateral direction in the basal sand, even eastward, which is opposite to the direction of groundwater flow under natural conditions. Therefore, the possibility of migration of radionuclides to the fracture zone could be prevented with absolute certainty by altering the hydraulic gradients in the area. If it is observed that the basal sand has appreciable sorption capability, the migration paths of radionuclides could be controlled to achieve maximum sorption. Total sorption of the radionuclides within the basal sand might be achieved by producing appropriate migration paths using a network of control wells.

In the unlikely event that radionuclides enter the basal sand in

amounts that are considered unacceptable, it would be possible to remove all unsorbed contaminants by pumping at appropriate locations. The field experiments by de Laguna (1970) at Mol, Belgium, indicate that removal of radionuclides from shallow sandy aquifers using collector wells can be a feasible water-management option, although the pumping can produce relatively large volumes of slightly to moderately contaminated water which must then to dealt with at the surface.

The purpose of this discussion is not to recommend a particular manipulation procedure but rather to emphasize that the hydrogeologic environment at WNRE can be used to achieve various waste-management objectives when approached on an engineering basis.

Waste-Management Properties of Other Parts of Study Area

To illustrate further the influence of hydrogeologic factors on subsurface radioactive-waste management, it is useful to consider other segments of the groundwater flow system in the study area. For example, a waste-management site located in either of the recharge areas would have various advantages and disadvantages compared to the present area. The main difference is that radioactive contaminants would be transported deeper into the groundwater zone and thereby become isolated from the life zone. Permanent isolation will occur if the sorption capabilities of the groundwater zone are relatively large. In the case of the central recharge area, however, downward transport of radionuclides would represent a possible hazard because of proximity to the fracture anomaly and the Winnipeg River.

If a waste-management site were located in the middle of the upland recharge area along cross-section A-A', contaminants entering the groundwater would be partially or totally sorbed in the lacustrine sand which underlies the area. The sand has a significant silt and clay content and consequently would have appreciable sorption capability. This capability could be utilized without the necessity of operating physical manipulation schemes. The lacustrine sand has intergranular hydraulic conductivity and is not excessively heterogeneous. It would be relatively easy to monitor contamination zones, and, if contaminant migration were found to be unacceptable, it would be feasible to operate manipulation or scavenging schemes.

Another advantage of the upland recharge area in the vicinity of cross section A-A' is that it offers the possibility of burying solid radioactive wastes in a relatively dry environment. This could be achieved by combining the following two operations: (1) lowering the water table by installing gravity drains at depths of 10-15 ft (3-4 m) below ground surface and (2) burying the wastes in trenches or containers in areas covered with material which will prevent downward infiltration.

Impermeable covers constructed of man-made materials such as asphalt, concrete, or plastic sheeting would require considerable maintenance owing to the deleterious effects of ice and climatic variations. The use of natural materials to minimize infiltration over long periods of time would probably be the most reasonable approach. A compacted clay cover initially would prevent infiltration, but soon cracks or other secondary channels would develop, owing to desiccation, root penetration, or frost wedging.

An alternative method of impeding infiltration using natural materials has been developed by Horton and Hawkins (1965), Corey and Horton (1969), and Rancon (1972). These investigations have shown that coarse-grained sediments such as coarse sand or gravel within a much finer grained porous medium can act as a barrier to infiltration. Corey and Horton (1969) used column experiments, dye tracers, and digital simulations to establish that pore water in a partially saturated, fine-grained material overlying a clean gravel above the water table will not drain into the gravel. Figure 11 shows a schematic design for a dry container for solid radioactive wastes which makes use of the above soil-physics principle. Flow into the coarse-sand layer will not occur as long as the hydraulic head at the bottom of the fine-grained layer is less than the head in the upper part of the sand. This type of head distribution could be achieved by choosing geologic materials which have appropriate water-content and hydraulic-conductivity functions. Movement of water from the fine-grained layer into the underlying sand or gravel will only occur after the fine-grained medium becomes saturated at the contact. Vegetation in the surface layer of loam soil (Fig. 11) would remove some of the soil moisture by evapotranspiration, thereby reducing the water content and allowing at least a portion of subsequent rainfall to be retained.

During long periods of rainfall, and probably during the spring snow melt, the upper fine-grained layer would become saturated and cause infiltration into the underlying sand. The infiltration rate would be controlled by the saturated hydraulic conductivity of the fine-grained layer and therefore would be small. The minor amounts of water moving through the sand would be diverted laterally by the second interface between fine- and coarse-grained materials. A successful field structure of this type

would require careful placing of the appropriate stratified geologic materials. The structure would enable decay of the waste to proceed without leaching. Little or no maintenance would be required. A pilot structure would be necessary for design testing before proceeding with routine use of the method.

This approach would be suitable in the upland recharge area because it would be relatively easy to use buried drains to maintain a low water table. It would also be possible to use the method in the existing waste-management site in the central discharge area; however, to maintain the water-table at depths of 10-15 ft (3-4 m) below ground surface would be a much more expensive operation and would not be reliable over long time periods without considerable engineering costs.

The dominant hydrogeologic features in the major segments of the study area are summarized in Table 3. Both the upland recharge area and the central discharge area are suitable for burial of solid radioactive wastes and for storage of small quantities of liquid wastes. The environmental hazards associated with the use of the present site are negligible providing that proper maintenance and monitoring are continued indefinitely. If it is desired to prevent long-term leaching of solid wastes, several advantages with respect to solid-waste management possibly could be achieved in the sandy part of the upland recharge area. A waste-management site in this area could be constructed so that there would be less reliance on long-term maintenance and monitoring. The site probably would never require major remedial operations to cope with contamination problems. The main limiting factor with respect to this area is the lack of detailed information on the sorption capabilities of the sandy deposits. Appropriate sorption data could be obtained from laboratory tests using relatively undisturbed samples.

SUMMARY OF CONCLUSIONS

1. The use of field investigations based on geological, hydrological, and hydrochemical methods combined with quantitative methods of analysis including groundwater flow simulations has produced a relatively detailed knowledge of the waste-management area and adjacent terrain. The waste-management area is located in a groundwater-discharge area, caused primarily by the presence of the permeable basal sand beneath the till and clay and by its proximity to the upland recharge area. The chemistry of the groundwater and the presence of salts in the subsoil indicate that the discharging groundwater regime has existed for a long time, probably

many hundreds or even thousands of years. The computer-simulation study suggests that groundwater discharge will persist as long as the topography of the land surface remains in its present form.

2. The fact that the water table is close to the ground surface is not a major detrimental factor with respect to the safety or operation of the site, although it does cause some operational inconvenience. It can be stated with confidence that leaching of radionuclides from solid wastes at the site would, at worst, result in localized shallow groundwater contamination which can be easily monitored and removed if necessary. In this respect all solid waste buried at the site can be regarded as being in storage, as long as monitoring of the area is maintained.

3. In the unlikely event of leakage of high-level liquid wastes into the groundwater zone, the contaminants would either be localized at shallow depths or would move downward, because of density effects, into the basal sand. If radionuclides enter the sand, under existing conditions, they would be transported westward. They would not represent a hazard until they reach the fracture anomaly. This would take at least many decades and therefore provide adequate time for implementation of remedial measures.

4. The groundwater flow pattern can be effectively manipulated using simple techniques such as well pumping. Manipulation schemes could be used to control or remove contaminants if necessary.

5. Depending on the long-term objectives with respect to solid-waste management at WNRE, the upland recharge area may offer some technical advantages over the central discharge area. If it is desired to avoid long-term leaching problems, relatively dry containment structures could be operated in the unsaturated zone, providing that the water table is lowered by gravity drains and that the structures are appropriately covered with stratified geologic materials with appropriate water-content - conductivity relations.

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Table 1. Summary of Methods of Investigation

TOPIC	LITHOLOGY & STRATIGRAPHY	HYDRAULIC PROPERTIES [1,2]	GROUNDWATER FLOW PATTERN	INTERSTITIAL GROUND-WATER VELOCITY [2]
METHODS 1968-1973	Power augering, auger samples, shelby-tube samples	Response tests in piezometers	Water-level monitoring of piezometer network	Darcy equation & Dupuit-Forchheimer assumption
	Cable-tool drill, split-spoon samples	Response tests in wells in basal sand	Recording wells in water-table zone and basal sand	Tracer study using injected tritiated water
	Observations in excavations	Short and long-term pumping tests	Distribution of major ions and O^{18}, D in groundwater	Groundwater age dating using C^{14} and H3
	Hand augering	Mass-balance calculations using recharge estimates	Finite-element simulation model	

1 Shawinigan Engineering Limited (1960) conducted standard soil property tests in laboratory using borehole samples of Pleistocene deposits.

2 Mills and Zwarich (1970 and written communication) conducted hydraulic conductivity tests in the laboratory using disturbed borehole samples. Samples were packed to obtain intergranular conductivity values. Cation exchange capacities and strontium sorption properties were also measured. Mineralogy was studied using X-ray diffraction.

Table 2. Summary of Results of Short-Term Pumping Tests in Basal Sand

PUMPING WELL	PUMPING RATE (gal min⁻¹)[1]	PUMPING DURATION min	OBSERVATION WELL	ANALYSIS* 1	2	3	4	TRANSMISSIVITY (gal day⁻¹ ft⁻¹)[2]	CONDUCTIVITY ft sec⁻¹ [3]	COEFFICIENT of STORAGE
RW3 24/8/71	9.0	630	RW1		✓			540	5.0×10^{-5}	1.3×10^{-4}
			RW2		✓			450	4.3×10^{-5}	8.0×10^{-5}
			RW3				✓	250	2.3×10^{-5}	
			RW4		✓			630	5.9×10^{-5}	2.4×10^{-4}
RW1 26/8/71	1.0	1260	57-41		✓			280	2.5×10^{-5}	1.7×10^{-4}
			I-5-29		✓			220	2.0×10^{-5}	4.1×10^{-4}
RW3 18/4/72	8.7	525	RW1		✓			540	5.0×10^{-5}	1.6×10^{-4}
			RW3				✓	474	4.4×10^{-5}	
RW5 26/4/72	2.6	7207	RW1	✓				164	1.5×10^{-5}	5.4×10^{-4}
			RW1			✓		205	1.9×10^{-5}	4.9×10^{-4}
			RW1		✓			219	2.0×10^{-5}	3.8×10^{-4}
			RW3		✓			181	1.6×10^{-5}	1.5×10^{-4}
			RW3	✓				152	1.4×10^{-5}	1.9×10^{-4}
			RW4	✓				184	1.7×10^{-5}	1.6×10^{-4}
			RW4		✓			189	1.7×10^{-5}	1.8×10^{-4}
			RW5				✓	289	2.6×10^{-5}	
RW3 1/6/72	7.6	1492	RW1		✓			254	2.3×10^{-5}	1.3×10^{-4}
			RW1	✓				152	1.4×10^{-5}	1.8×10^{-4}
			RW4		✓			284	2.6×10^{-5}	3.5×10^{-4}
			RW4	✓				219	2.0×10^{-5}	1.9×10^{-4}
			RW5	✓				207	1.9×10^{-5}	3.7×10^{-4}
			RW5		✓			287	2.6×10^{-5}	2.5×10^{-4}
RW12 4/9/72	3.2	750	RW1	✓				159	1.4×10^{-5}	2.1×10^{-4}
			RW1		✓			256	2.3×10^{-5}	1.6×10^{-4}
			RW4	✓				367	3.4×10^{-5}	2.4×10^{-4}
			RW5	✓				378	3.5×10^{-5}	1.4×10^{-4}
Average								290	2.7×10^{-5}	2.4×10^{-4}

* 1. Drawdown in observation well, Theis type curve superposition method (Todd, 1959)
2. Drawdown in observation well, Jacob semilogarithmic method (Todd, 1959)
3. Recovery in observation well, semi-logarithmic recovery method (Davis and Deweist, 1966)
4. Recovery in pumping well, semi-logarithmic recovery method (Davis and Deweist, 1966)

1. 1 gal $min^{-1} = 4.55 \times 10^{-3}\ m^3\ min^{-1}$
   1 gal min^{-1} = 1.2 Am. gal min^{-1}
2. 1 gal $day^{-1} ft^{-1} = 1.242 \times 10^{-2}\ m^3\ day^{-1}\ m^{-1}$
   1 gal $day^{-1}\ ft^{-1}$ = 1.2 Am. gal $day^{-1}\ ft^{-1}$
3. Calculated by dividing transmissivity by total aquifer thickness of 20 feet.

Table 3. Summary of Dominant Hydrogeologic Features in Major Segments of Study Area

HYDROGEOLOGIC FACTORS	CENTRAL DISCHARGE AREA Existing WM Area	CENTRAL RECHARGE AREA	UPLAND RECHARGE AREA WEST SIDE OF FLOW DIVIDE Northern Segment	South Segment
STRATIGRAPHY AND LITHOLOGY	Lacustrine clay overlying clay-loam till	Lacustrine silt and clay overlying clay-loam till	Fine and medium grained sand, and silty sand	Sand and gravel up to 10 FT. thick on clay
DEPTH TO WATER TABLE, SEASONAL RANGE	2 TO 8 FT.	0 TO 10 FT.	3 TO 12 FT.	3 TO 8 FT.
FEASIBILITY OF WATER TABLE CONTROL	Difficult due to low relief, fine-grained deposits, and upward flowing groundwater	Possible with gravity drain system discharging to Winnipeg River	Well suited for gravity drain system	Well suited for gravity drain system
DIRECTION OF GROUNDWATER FLOW	Upward to water table from basal sand	Downward into basal sand and then to river or fracture zone	Downward and then laterally westward	lateral in sand and gravel; downward in clay
FEASIBILITY OF FLOW PATTERN MANIPULATION	Easy using one to several wells in basal sand	Feasible, but more difficult than other areas	Easy using wells and drains	Easy in sand and gravel, very difficult in clay
TYPE OF HYDRAULIC CONDUCTIVITY	Fractures, coated with iron oxides and salts	Fractures, with discontinuous iron oxide coatings	Intergranular	Intergranular in sand and gravel, fractures in clay
PROXIMITY TO UNDESIRABLE HYDROGEOLOGIC FEATURES	Moderate distance from bedrock fracture zone	Close to fracture zone and discharge zone at Winnipeg River	Far from river and fracture zone	Far from river and fracture zone
ESTIMATED* SORPTION CAPABILITIES	Generally low due to coated fractures and saline ground-water	Moderate to low, because of fractures	Moderate, depends on silt and clay content of sand	Low in sand and gravel because of very small silt and clay content

* Sorption data obtained by Mills and Zwarich (1970) in laboratory are difficult to apply to the field situation because remoulded borehole samples were used. Effects of fractures were not evaluated.

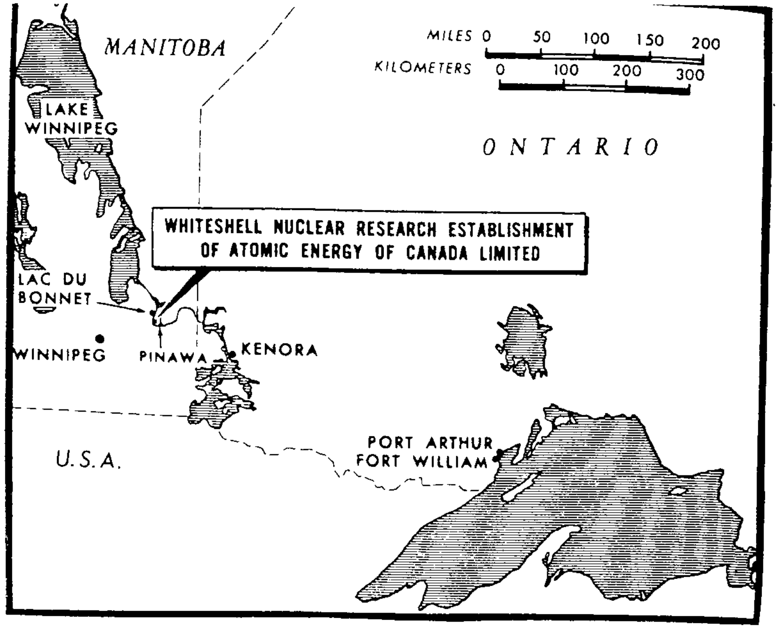

FIG. 1--Location of Whiteshell Nuclear Research Establishment.

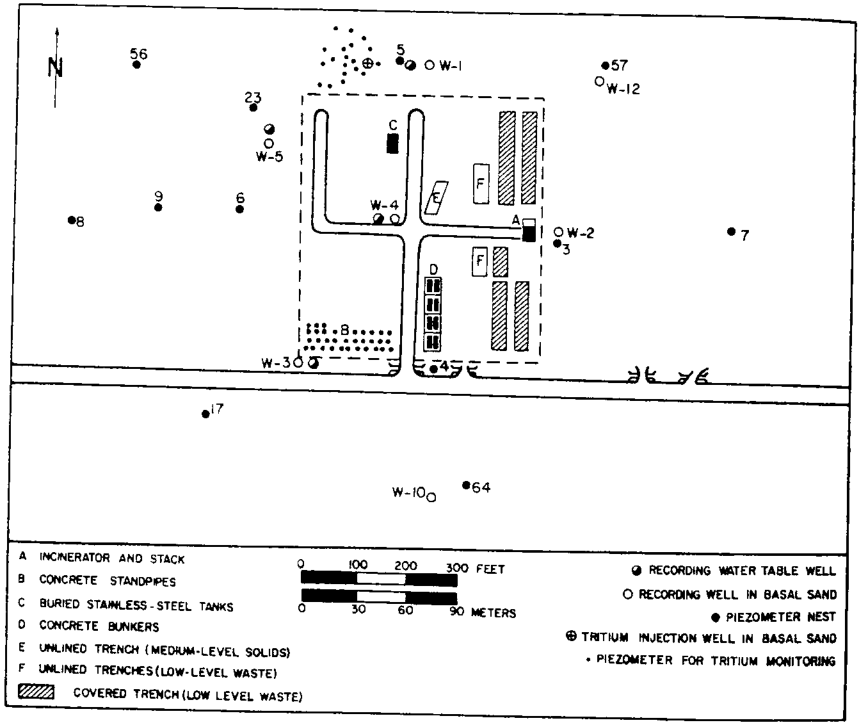

FIG. 2--Waste-management facilities and groundwater instrumentation at waste-management area.

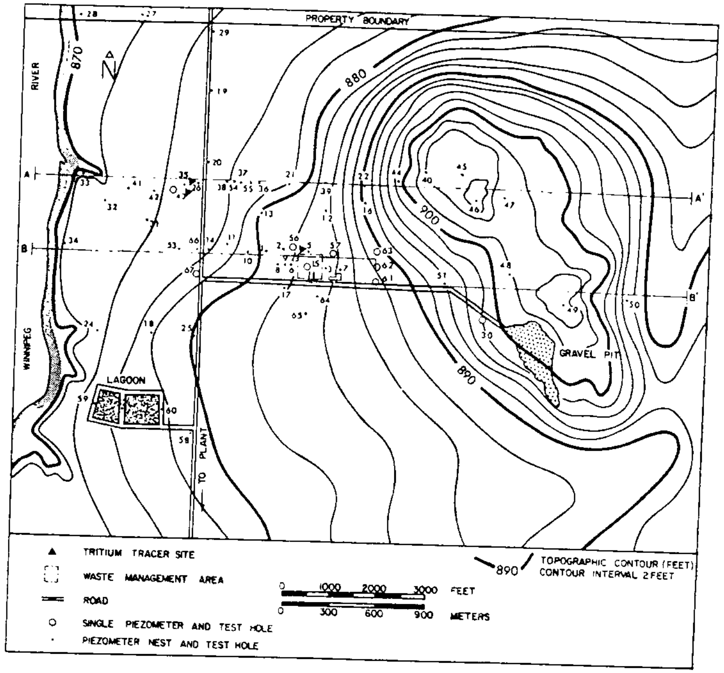

FIG. 3--Topography and instrumentation in study area.

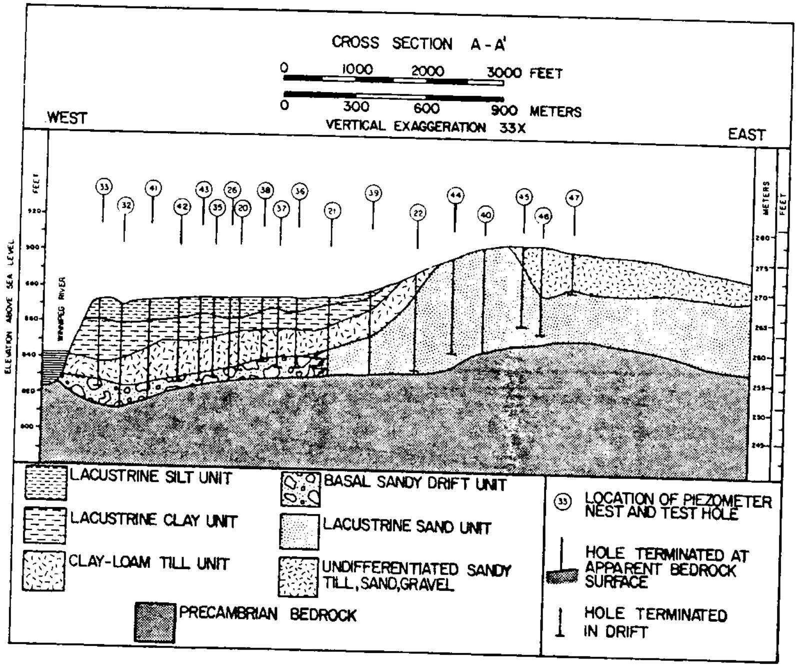

FIG. 4--Geologic cross section A-A'.

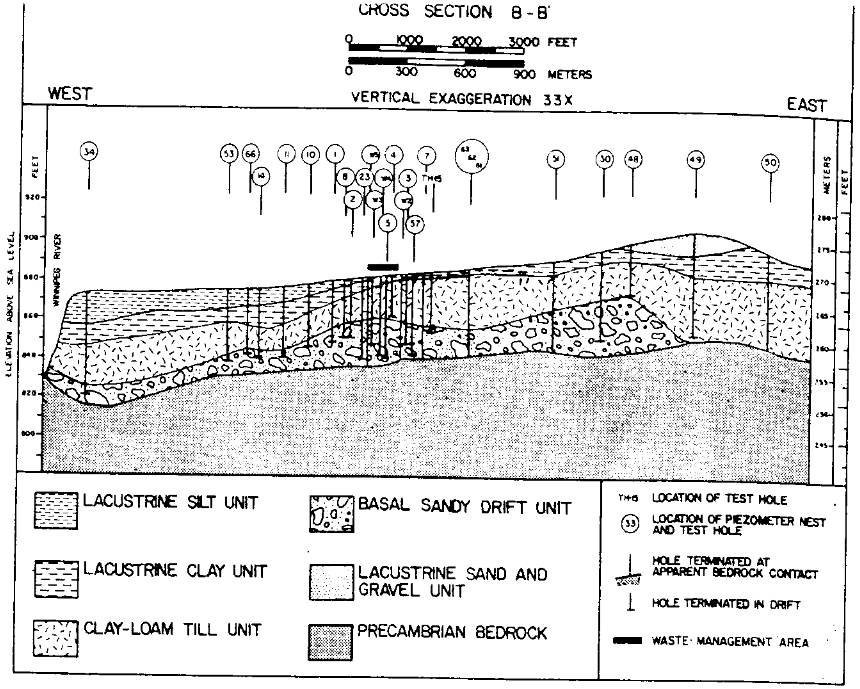

FIG. 5--Geologic cross section B-B'.

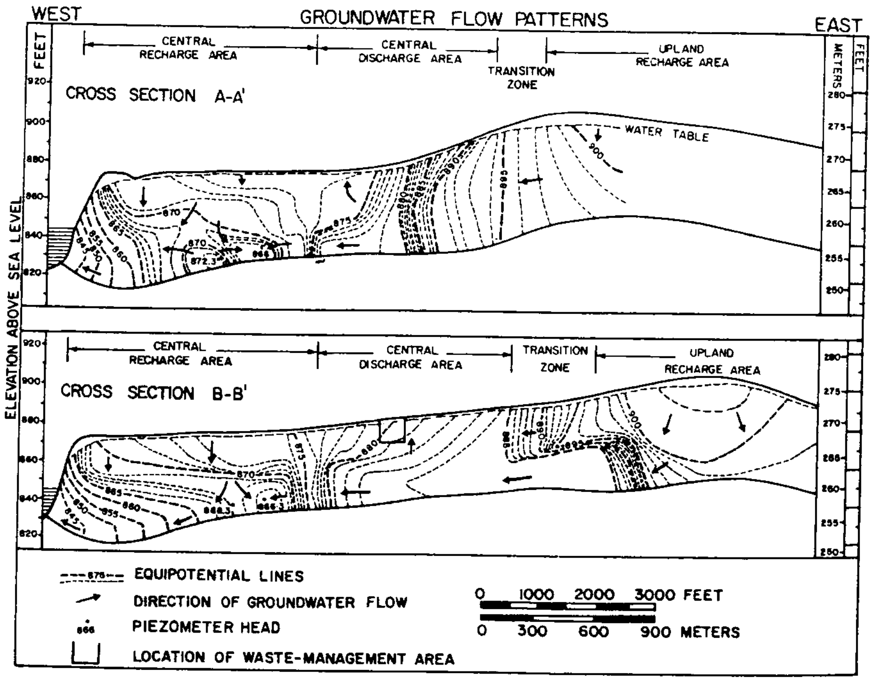

FIG. 6--Groundwater flow patterns interpreted from water-level data; cross section A-A' and cross section B-B'.

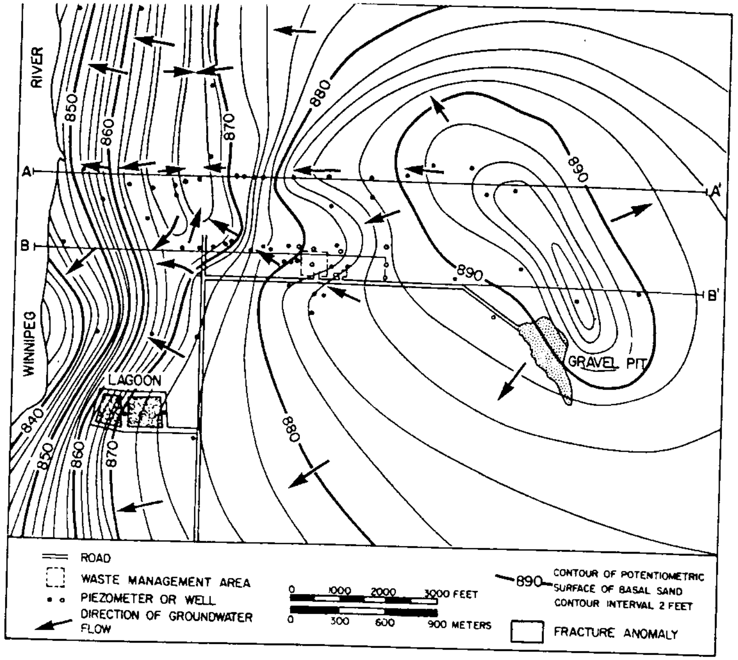

FIG. 7--Potentiometric surface and flow pattern in basal sand.

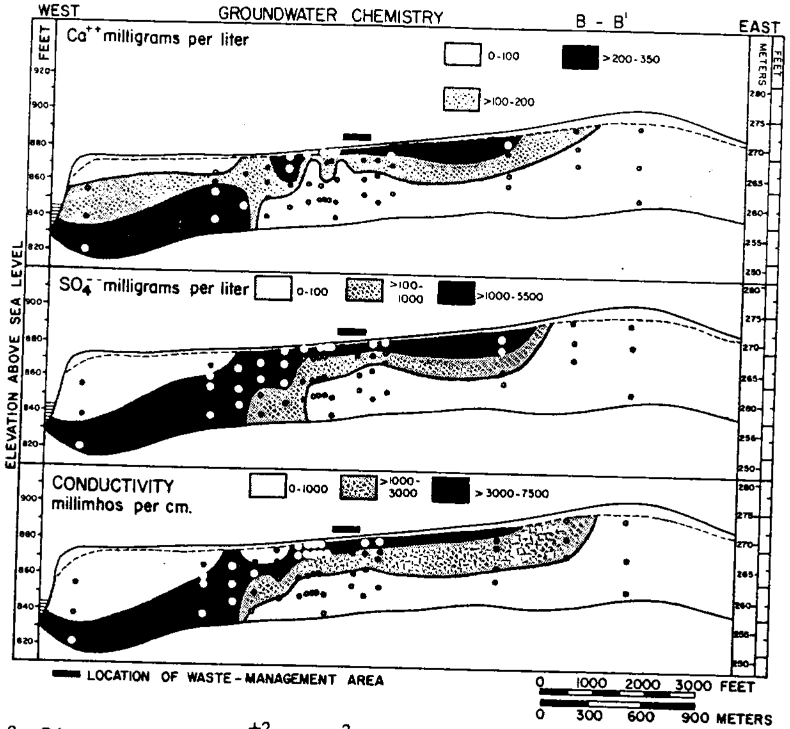

FIG. 8--Distribution of Ca^{+2}, SO_4^{-2}, and electrical conductivity along cross section B-B'.

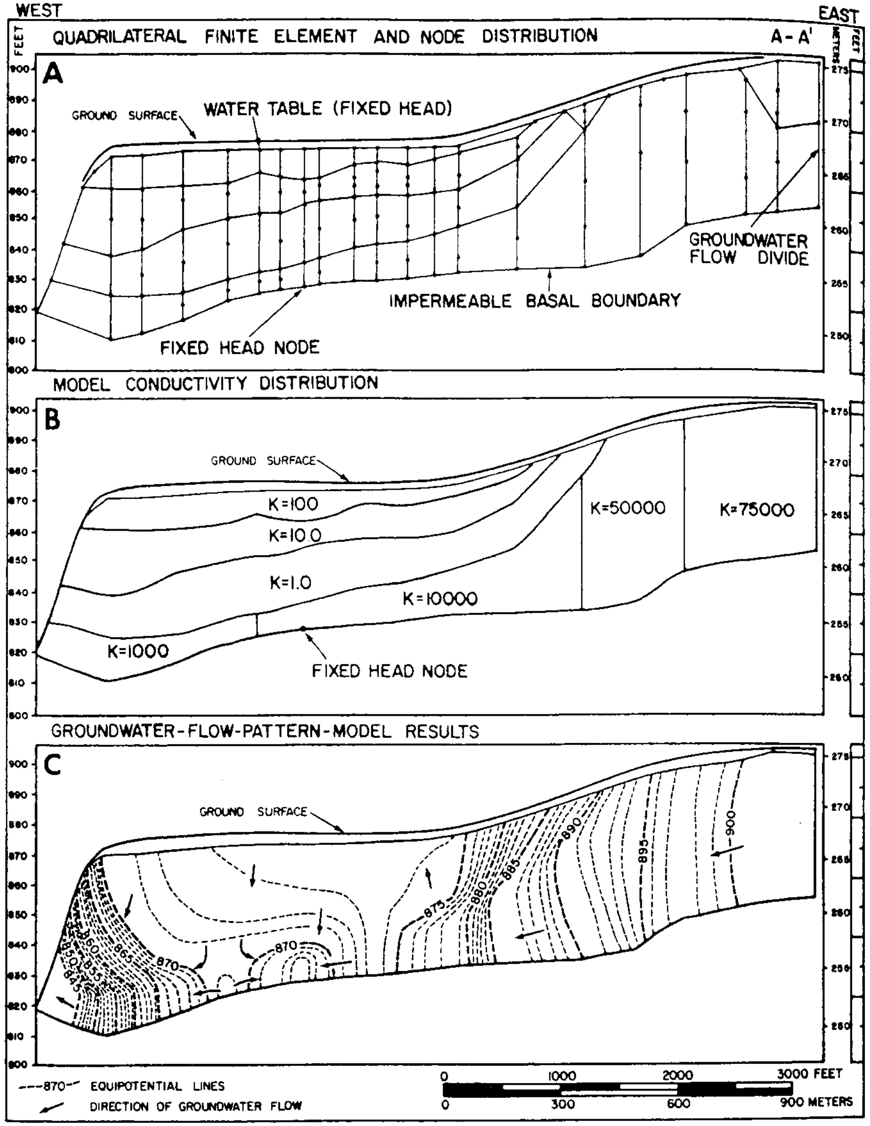

FIG. 9--Digital simulation of groundwater flow pattern along cross section A-A'. A. Boundary conditions and arrangement of elements. B. Distribution of relative permeabilities producing best results. C. Simulated flow pattern.

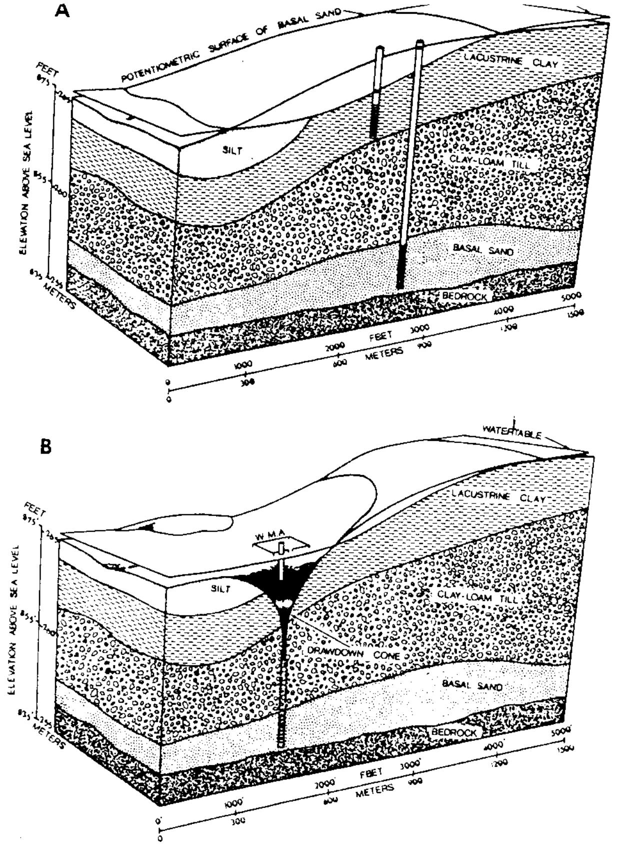

FIG. 10--Relations between water table and potentiometric surface. A. Under natural conditions. B. After pumping in basal sand.

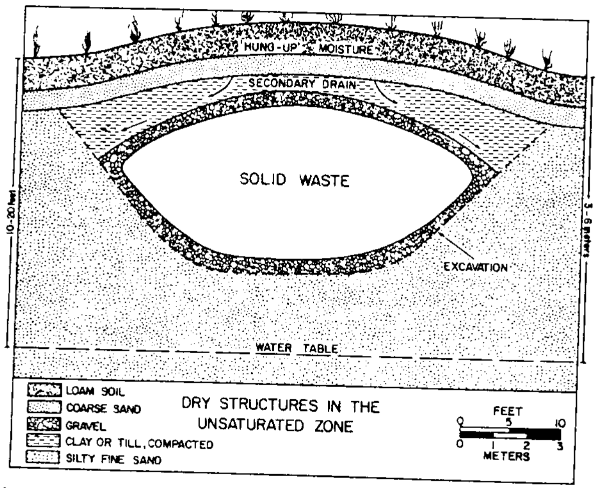

FIG. 11--Schematic diagram showing a hydrogeologic design for dry solid-waste burial zones.

MOVEMENT AND ACCUMULATION OF SUSPENDED SEDIMENT DURING BASIN RECHARGE[1]

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ABSTRACT The movement and accumulation of sediment suspended in water used for recharge were determined by microscopic examination of thin sections from material underlying a recharge basin. The basin had been infiltrated to the depth of 91 m by water averaging 191 ppm suspended sediment in six recharge cycles. The nature of the sediment accumulations permitted thin-section identification of their types, loci, and amounts. Three types of accumulations were observed: (1) flakelike structures on the upper 2.5 cm, (2) two horizontal sheetlike structures, 0.1 mm thick, between the depths of 5-8 and 8-12 cm, respectively, and (3) fillings in voids, mostly between 2.5 and 23 cm. The average pore volume lost between the depths of 2.5 and 5 cm was 1.5 percent, and, below 5 cm, less than 1 percent. The volume of accumulated sediment was 23 percent in the upper 2.5 cm, 5.5 percent between the depths of 2.5 and 5 cm, and less than 0.5 percent below 5 cm. Even though the material above 2.5 cm accumulated a large amount of sediment, porosity was maintained by freezing and thawing and by wetting and drying. Infiltration rates of the basin have not been noticeably reduced. The successful use of excavated basins for recharge of turbid water should not be limited by the movement and accumulation of suspended sediment in the basin material.

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INTRODUCTION

Recharge of aquifers by spreading runoff water containing up to 250 ppm suspended sediment in excavated basins was accomplished by Aronovici et al. (1970, 1972). Continuous recharge with turbid water was not possible owing to the plugging of the basin surface by suspended sediment. It was apparent that a large portion of the sediment suspended in the infiltrating waters was filtered at the surface. After plugging occurred, the thin coating of sediment at the surface was removed, permitting the initial infiltration rates to be reestablished. The period from initiation of recharge to the removal of the thin layer of surface sediment was termed a "recharge cycle."

Even though filter systems may be efficient in filtering suspended material at the surface, some of the material can move into the porous filter media (Thomas, 1968; Curry, 1966; Curry et al., 1965; Rausch and Curry, 1963; Stanley, 1955). If the sediment suspended in the infiltrating waters plugs the basin material at significant depths beneath the surface of a recharge basin, it may not be economical to reclaim the basin.

Goss et al. (1973b) used Cs^{134}-tagged sediment to trace the depth and amount of suspended-sediment movement into the material underlying recharge basins. The Cs^{134} study resulted in an estimate of the depth and amount of sediment penetration into the material underlying a recharge basin, but did not reveal the loci of the sediment accumulation.

The present study involved the examination of thin sections made from the material underlying a recharge basin that had recharged over 91 vertical meters of water averaging 191 ppm suspended sediment in six recharge cycles.

Examination of the basin material in thin section can accomplish two goals: (1) it can show the amount of sediment moving into the material over several recharge cycles, and (2) it can identify the loci of accumulation of the suspended sediment. Clay minerals deposited in voids in unconsolidated materials by infiltrating water are oriented on void and grain surfaces. When thin sections are viewed under the petrographic microscope, the oriented nature of these accumulated clay particles can readily be recognized, as reported by Brewer (1964). Goss et al. (1973a) found that over 85 percent of the material suspended in the infiltrating water was clay. It is probable that only the clay portion moved to any depth, because the coarser particles would have been filtered near the surface.

PROCEDURES

The basin and underlying materials have been described by Goss et al. (1973a) and Lotspeich and Coover (1962). In general, the material underlying the basins is calcareous unconsolidated sediment of clay-loam texture, containing many continuous, relatively large pores that range from 0.1 to 1.0 mm in diameter.

Continuous cores, 8 cm in diameter, were taken to a depth of 30 cm in the recharge basin that had been infiltrated with 91.1 m of water. These cores were impregnated with a plastic resin, and thin sections were cut from each core. A continuous set of vertical thin sections and horizontal thin sections, every 3 cm, was obtained from four cores. A total of 64 thin sections was made. A second set of eight thin sections was obtained from the same depth in the material just outside the basin. These thin sections were examined for oriented clay in the voids so that a distinction could be made between oriented indigenous clay and added clay. The volume of accumulated clay was determined by 12,800 point counts in thin sections made of material underlying the recharge basin. A 49-point grid was placed randomly over a portion of each thin section. As the thin section was viewed through the petrographic microscope, the type of material under each point was identified and counted. A total of 200 counts was made per thin section.

RESULTS AND DISCUSSION

The thin-section study revealed three major types of accumulations. First, in the upper 2.5 cm, the accumulations were in the form of flakes or aggregates resembling the crust formed at the surface at the end of a recharge cycle (Fig. 1). It was impossible to determine whether all the material in these flakes was added sediment, since some mixing with the original basin material may have occurred. Therefore, the accuracy of determination of the amount of accumulated sediment in the upper 2.5 cm may be questioned. These flakes had formed on the surface and had been worked into the material, or they had formed in cracks that developed in the basin during recharge.

The second type of accumulation was in the form of almost continuous sheets of sediment that occurred at two levels; the lower one was oriented, the upper one was not (Fig. 2). For the flakes in the upper 2.5 cm, it was impossible to determine if the non-oriented sheetlike accumulations were entirely added sediment. However, the marked orientation of the

lower sheetlike accumulations strongly indicated that they were added sediment. Field observations indicated that the two sheetlike accumulations occurred under most of the basin. The upper one was about 5-8 cm below the surface and the lower one was about 8-12 cm below the surface. The sheets were about 0.1 mm thick--generally in a continuous plane that could be traced over several square meters--but did not cover or block the larger pores and cracks in the basin material. No explanation can be given for the genesis of these structures.

The third type of accumulation occurred in voids. These accumulations could be observed only between 2.5 and 23 cm, and all were oriented. The degree of orientation generally increased with depth. Figure 3 shows an example of a pore containing accumulated sediment. These accumulations were formed when turbid water percolated through the voids and deposited part of its load as it passed.

Examination of the material outside the basin revealed no structures similar to the three discussed above. Therefore, it was assumed that these structures resulted from sediment accumulated during recharge.

The volume percent of accumulated sediment in the material underlying the basin, as determined from point counts in thin sections, is reported in Table 1. The material above 4 cm contained a large portion of accumulated sediment. However, the material to this depth is frequently subjected to wetting and drying, as well as freezing and thawing, which would maintain porosity. The percent of accumulated sediment was used to calculate the percent of the total sediment added by infiltrating water that moved deeper than a given depth (Table 1, col. 4). The total amount of sediment added during the six recharge cycles was 1.67 g/cm^2. The percent of sediment added to the basin that moved deeper than the indicated depth for this study, and for the study by Goss et al. (1973b), is shown in Figure 4. During the study by Goss et al., 0.18 g of Cs^{134}-tagged sediment was added per square centimeter. Both methods have some limitations. The Cs^{134} data were obtained from only one recharge cycle, and conditions may vary between cycles. The thin-section study has inherent inaccuracies due to the inability to distinguish added sediment from indigenous material in the upper 2.5 cm. However, both methods point out that very little of the sediment suspended in infiltrating water moved below 5 cm (Fig. 4).

The thickness of material removed at the end of each recharge cycle was usually between 0.10 and 0.16 cm. Approximately 24-40 percent of the sediment added by the infiltrating water was removed at the end of each recharge cycle, inasmuch as 60-76 percent of the total sediment was shown to move below the basin surface (Fig. 4).

The effect of accumulated sediment on infiltration rates can be estimated if the percent of pore-volume loss can be determined. The relation of hydraulic conductivity to pore size developed by Childs (1969) indicated that the percent of reduction in infiltration rate was equal to the percent of pore-volume loss squared. The percent of pore-volume loss is shown in Figure 5. If it is assumed that the accumulated sediment was distributed in all pores so as to reduce their volume by the same factor, the loss in the infiltration rate was about 2.5 percent at a depth of 4 cm and less than 1 percent below 10 cm. These percentages would be higher if the sediment were distributed only in the largest pores, and lower if distributed only in the smallest pores; however, examination of the thin sections did not indicate sediment distribution. The effect of accumulated sediment on infiltration rates could not be estimated for the upper 4 cm because the pore-volume loss could not be determined, inasmuch as freezing and thawing, as well as wetting and drying, tend to maintain porosity at this depth.

The recharge basin sampled for the thin-section study has undergone three additional recharge cycles for a total of 114.8 m of water in nine recharge cycles. Table 2 gives the average infiltration rates for the first day, the fifth day, and the day of maximum infiltration during each cycle. Infiltration rates have not been measurably reduced.

CONCLUSIONS

Three major types of sediment accumulations occurred in the basin material: (1) flakelike aggregates in the upper 2.5 cm, (2) two sheetlike structures approximately 0.1 mm thick between depths of 5-8 and 8-12 cm, and (3) accumulations in voids, mostly between depths of 2.5 and 23 cm. The amount of sediment that accumulated in voids below 5 cm was small and had no significant effect on the infiltration rates, even after considerable recharge was accomplished. A large amount of sediment accumulated in the upper 4 cm, but sufficient porosity was maintained to prevent measurable losses in the infiltration rates. Porosity in the upper 4 cm was probably maintained by freezing and thawing and wetting and drying during periods when recharge was not being conducted. If plugging eventually occurs, only the upper 5 cm will need to be removed to reclaim the basin. Therefore, the successful use of excavated basins for recharge of turbid water should not be limited by the movement and accumulation of suspended sediment in the basin material.

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Table 1. Data from Thin-Section Study[1]

Depth	Volume of Accumulated Sediment	Added Sediment Moving Deeper
cm	%	%
-0.2 to 0.0	--[2]	76
0.0 to 0.2	39	68
0.2 to 1.0	25	42
1.0 to 2.5	20	15
2.5 to 5.0	5.3	1.9
5.0 to 7.6	0.7	0.2
7.6 to 15.2	0.1	trace
15.2 to 22.9	trace	trace
22.9 to 30.0	trace	trace

[1]Volume percent of basin material that was accumulated sediment and percent of the total sediment added (1.67 g/cm^2) that moved deeper than indicated depth.

[2]Sediment accumulating at the surface was removed at the end of each recharge cycle.

Table 2. Average Infiltration Rates[1]

Recharge Cycle	First Day cm/hr	Fifth Day cm/hr	Maximum Day cm/hr
1	2.54	2.54	4.83
2	1.91	2.29	2.29
3	1.59	1.65	2.79
4	1.59	2.22	2.48
5	0.64	1.71	2.54
6	1.50	1.52	2.95
7	1.42	1.65	1.71
8	1.33	1.88	2.29
9	1.59	2.54	4.45

[1]Filtration rates for first day, fifth day, and day on which maximum infiltration was observed, during nine recharge cycles of an experimental basin with turbid water.

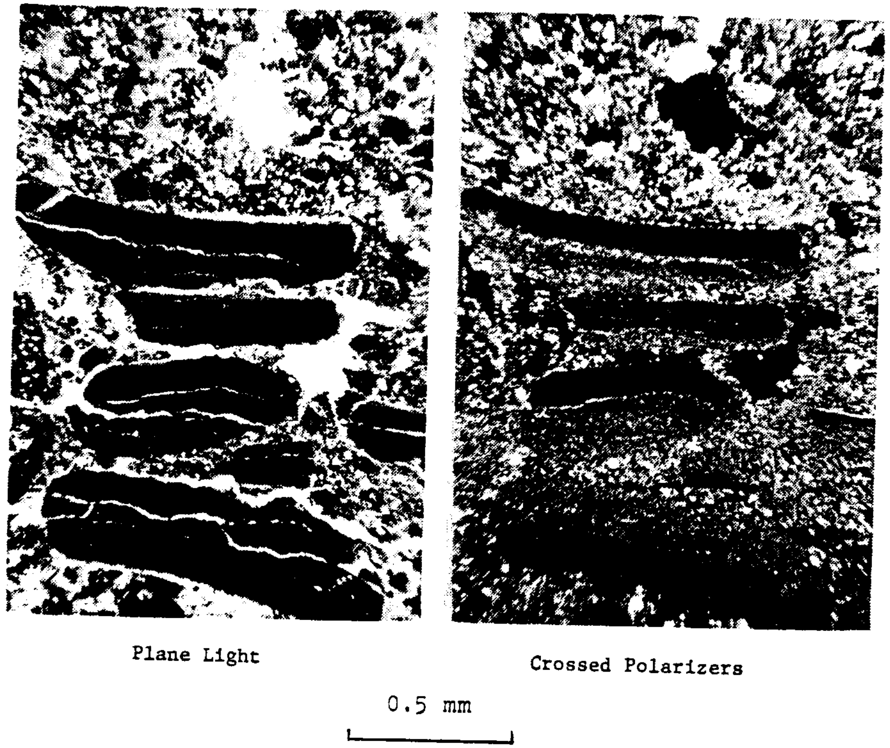
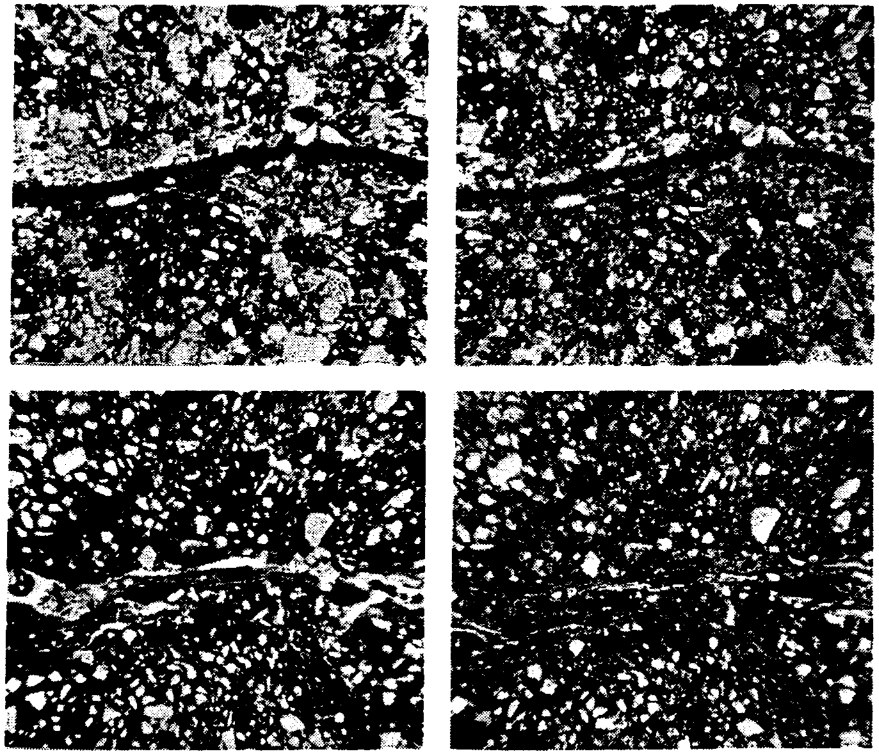


FIG. 1--Vertical thin section showing flakelike structures near surface. Top of photomicrographs is up, and at about 0.3 cm. If flakes had been oriented, light passing through them under crossed polarizers would be much more uniform.



Plane Light

Crossed Polarizers

0.5 mm

FIG. 2--Vertical thin sections showing two types of sheetlike structures. Top of photomicrographs is up. Upper photomicrograph shows non-oriented sheetlike structure at 6 cm, and lower photomicrograph shows oriented sheetlike structure at 9 cm.

Crossed Polarizers

Plane Light

0.5 mm

FIG. 3--Section through soil pore showing accumulated sediment. Thin section under plane light has original pore outlined by black or white line. Thin section was taken from 9 cm. Amount of sediment accumulation shown is greater than that normally observed.

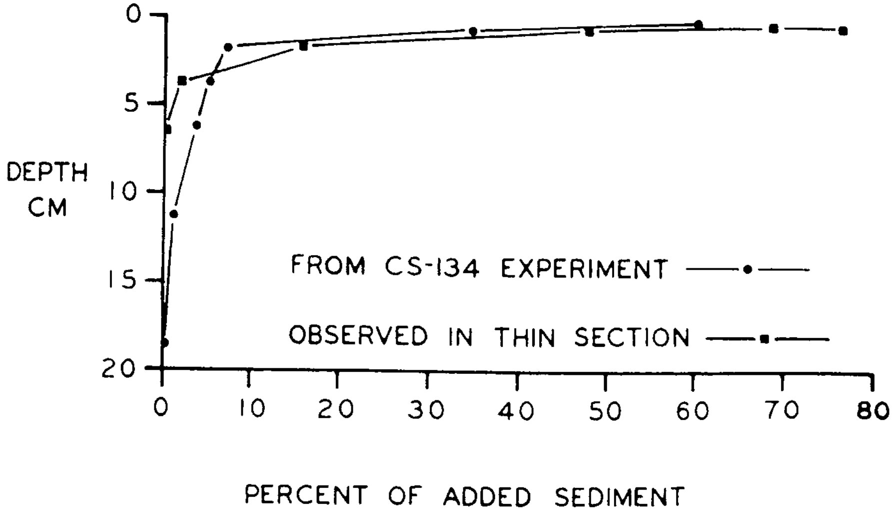


FIG. 4--Percent of sediment added to basin that moved deeper than indicated depth for our thin-section study and for study by Goss et al. (1973).

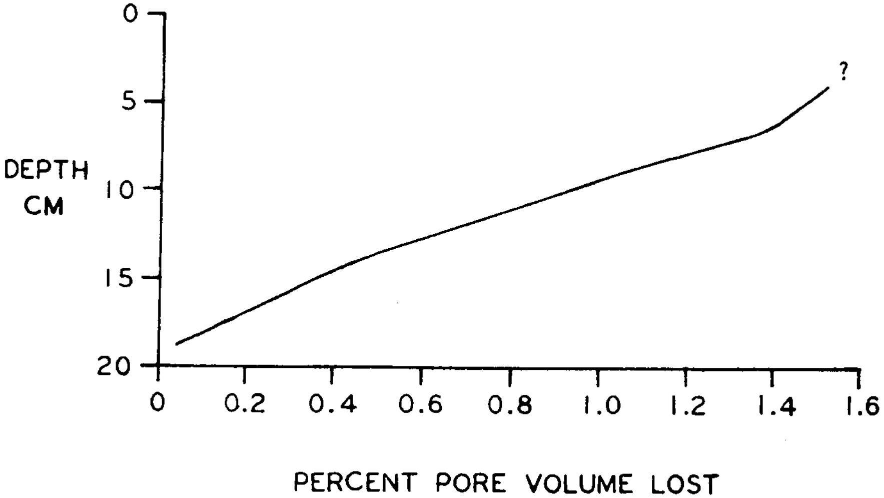


FIG. 5--Percent of pore volume lost as a result of accumulating sediment.

GEOHYDROLOGY OF BURIED TRIASSIC BASIN AT SAVANNAH RIVER PLANT, SOUTH CAROLINA[1]

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ABSTRACT At the Savannah River Plant near Aiken, South Carolina, as at other locations where there are chemical-separation plants for the processing of nuclear fuels, the high-level radioactive wastes are stored in concrete and steel tanks buried just beneath the ground surface. This waste is of such activity and longevity that it cannot be dispersed into the environment, but it must be contained for periods of time extending at least into hundreds and perhaps thousands of years. One concept for the terminal containment of this waste is to store it in excavated chambers within the bedrock, which is covered by about 1,000 ft of coastal-plain sedimentary beds at the plant site. As part of the safety evaluation of this concept, the geology and hydrology of both the coastal-plain units and the bedrock have been studied. However, intensive investigation of waste storage in bedrock has now been deferred pending more detailed evaluation of alternative concepts of waste storage and management.

In the course of these studies, a buried Triassic basin that might have potential for waste storage was discovered beneath the southern third of the plant site. Investigation into the characteristics of this basin was started in 1971. This was not an engineering or design study; rather, it was aimed at understanding the geohydrology of the Triassic basin to determine its compatibility with the safe storage of waste.

Seismic, gravity, and magnetic surveys, and the drilling of several exploratory wells, indicate that the Triassic basin is about 30 mi long,

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[2]Savannah River Laboratory, E. I. du Pont de Nemours and Company.

6 mi or more wide, and filled with sedimentary rocks perhaps 5,300 ft thick. One well penetrated the basin border, a second was in the center of the basin, and a third investigated an inferred intrabasin fault. The rock in the basin is predominantly mudstone of very low permeability, but a few lenses of poorly sorted gritty sandstone are present. The water yield of all the exploratory wells is extremely low, and water-transmitting fractures are virtually nonexistent.

In two wells within the basin, heads above land surface have been measured that cannot be explained by connection with a recharge area. Ten possible explanations have been evaluated: aquifer head, "fossil" head, tectonic compression, rapid loading and compaction of sediments, water derived from igneous intrusions, infiltration of gas, precipitation of minerals, phase changes, temperature increase, and osmotic-membrane phenomena. Systematic evaluation, particularly of the time for dissipation of an elevated head to the head of its surroundings, eliminates most of these explanations. Those that remain as possible explanations are: tectonic compression, temperature increase, and osmotic-membrane phenomena. It is not known at present whether the high head is general over the entire basin or only in segments of it.

INTRODUCTION

At the Savannah River Plant near Aiken, South Carolina, as at other locations where there are chemical-separation plants for the processing of nuclear fuels, the high-level radioactive wastes are stored in concrete and steel tanks buried just beneath the ground surface. This waste is of such activity and longevity that it cannot be dispersed into the environment, but must be contained for periods of time extending at least into hundreds and perhaps thousands of years. One concept for the terminal containment of this waste is to store it in excavated chambers within the bedrock, which is covered by about 1,000 ft of coastal-plain sedimentary beds at the plant site. As part of the safety evaluation of this concept, the geology and hydrology of both the coastal-plain units and the bedrock have been studied. However, intensive investigation of waste storage in bedrock has been deferred pending more detailed evaluation of alternative concepts of waste storage and management.

A buried Triassic basin that might have potential for waste storage was discovered beneath the southern third of the plant site. Active investigation into the characteristics of this basin was started in 1971. This was not an engineering or design study, but was aimed at understand-

ing the geohydrology of the Triassic basin to determine its compatibility with the safe storage of waste. In late 1972, before the investigation was complete, it was postponed indefinitely while major effort was turned toward alternative methods of waste storage, such as temporary, near-surface storage of waste in a solidified form. This paper reports on the status of the investigation when intensive work was stopped. Many questions require answers before a conclusion on this concept of waste storage is reached.

The concept of radioactive-waste storage involved here is shown in Figure 1; it would consist of:

1. Sinking a shaft about 15 ft in diameter through the coastal-plain sedimentary units and as far into the bedrock as investigations determine to be desirable.

2. Excavating several tunnels laterally to create storage space in the bedrock for about 80 million gal of waste. Conceptually, these tunnels should be about 28 ft high, 26 ft wide, and 3,500 ft long.

3. Sealing the storage chambers from the central access shaft by use of monolithic bulkheads.

4. Providing a service shaft containing fill and instrument lines to the storage chambers from the surface.

All storage space would be created by excavation. None of the natural porosity would be used. Storage would begin at subhydrostatic pressures.

The Savannah River Plant is located on the Atlantic Coastal Plain about 20 mi southeast of the outcrop of the crystalline metamorphic rock, at the Fall Line, which separates the Piedmont and Coastal Plain physiographic provinces (Fig. 2). There are three separate geologic and hydrologic systems under the Savannah River Plant (Fig. 3): (1) the coastal-plain sedimentary beds, of Cretaceous and Tertiary age, where water occurs in porous, unconsolidated to semiconsolidated sand and clay; (2) the buried crystalline metamorphic basement rock, consisting of chlorite-hornblende schist and hornblende gneiss with a lesser amount of quartzite, where water occurs in small fractures; and (3) a buried Triassic basin, consisting mostly of red consolidated mudstone with some poorly sorted sandstones, where water occurs in the intergranular space but is very restricted in movement because of the extremely low permeability.

GEOLOGIC CHARACTERISTICS

The general outlines of the buried Triassic basin (Siple, 1967, pl. 4) have been inferred from the aeromagnetic map of the area published by the U.S. Geological Survey (Petty et al., 1965). The basin appears to be about 30 mi long and at least 6 mi wide. This basin has been called the "Dunbarton basin"; greater detail on the location of its boundaries at the plant site was obtained from reflection seismic traverses (Fig. 4). In addition, gravity and ground magnetic surveys were made along many of the same lines. Several wells (P5R, DRB 9, and DRB 10) had been drilled in the Triassic basin prior to most of these geophysical surveys, and two wells (P12R, DRB 11) were drilled as a result of them.

Well P5R, drilled in 1962, was the "discovery" well for the buried Triassic sedimentary basin; it penetrated only about 95 ft of these sedimentary beds, consisting of maroon claystone and fine-grained sandstone. Well DRB 9 was drilled in 1969 at a location selected, on the basis of a short reflection seismic survey, to penetrate the edge of the Triassic basin and then pass into the crystalline metamorphic basement. At this location, the Triassic sedimentary rocks are 1,593 ft thick and consist of maroon fanglomerate made up of clasts of pink weathered gneiss set in a maroon siltstone matrix. The Triassic is underlain by augen gneiss with pink feldspar augen about 0.5 in. in diameter set in a matrix of fine-grained green hornblende. The well is about 0.4 mi from the edge of the basin; the contact of the Triassic with the augen gneiss dips about 35° to the southeast, as do a few faint bedding planes in the fanglomerate. Well DRB 10 was drilled in 1971 near what was thought to be the center of the basin. This well was drilled to a depth of 4,206 ft and penetrated about 3,035 ft of red siltstone and sandstone. Crystalline metamorphic rock was not expected in this well, and none was encountered. No well in the basin has penetrated any igneous rock or coal, even though they occur in other East Coast Triassic basins.

Geophysical work, consisting of reflection seismic and gravity-magnetic surveys, was done or analyzed after the information from these three wells was available. The reflection seismic surveys showed a sharp northwest boundary but did not indicate termination of the Triassic rocks where the southeast border was inferred from the aeromagnetic map. The contact of the coastal-plain beds and the Triassic rocks could be followed as a reflection but with scattered apparent discontinuities (labeled "inferred faults" in Fig. 4). The contact of the basement with the Triassic rocks could not be detected at all. The seismic velocities in the Triassic rocks generally increase with depth (Table 1) and approach basement-rock velocities near the bottom of the basin, thus giving a low seismic contrast between the two.

Gravity and magnetic surveys were made on the ground along many of the same traverse lines as the seismic survey in an effort to estimate the thickness of the Triassic rocks and to develop more information on the possible faults indicated by the reflection seismic surveys. Models were made by the geophysical contractor to fit the observed gravity and magnetic profiles. One such model is shown in Figure 5 for the section AA′ in Figure 4. This model is composed of blocklike units of different thicknesses. From the northwest Triassic border near well DRB 9, the basin generally becomes deeper to a point 11,500 ft southeast of well DRB 10, where its depth is estimated to be 6,500 ft. From that point, it shallows to about 4,200 ft at the southeast plant boundary. Southeast of the plant, depths are about 3,000 to 5,800 ft. These depths include the coastal-plain sedimentary section, which is about 1,200 ft thick over most of this area. Therefore, based on this model the Triassic section has a maximum thickness of about 5,300 ft.

One of the faults interpreted from the seismic data was selected for more intensive investigation by drilling. Two wells (DRB 11 and P12R) were drilled on either side of the fault. The elevation of the top of the Triassic was the same in both wells, showing that the last movement on the fault, if it exists, was before the development of the erosional surface. The age of the erosional surface is pre-Late Cretaceous or at least 100 m.y. Thus, there has been no movement on the fault in the last 100 m.y. This conclusion is substantiated by correlation of distinctive peaks on the gamma-ray and electrical logs of the coastal-plain beds.

Well P12R was cored from a depth of 984 ft, within the coastal-plain sedimentary beds, to 1,272 ft. The casing was set at 1,219 ft, which was 131 ft into Triassic rock. Well DRB 11 was cored from a depth of 1,020 ft to its total depth of 3,320 ft. Casing was set at a depth of 1,229 ft, which was 158 ft into Triassic rock. In both wells, casing was set far into the Triassic rock because the upper rock is weak. Cores of the upper section showed a high clay content, and the core samples tended to swell and crack when exposed to wetting and drying. Cores of the Triassic from both holes are predominantly fine-grained, dense maroon mudstone. A few small zones (1 ft or less) contain particles of gravel size (about 0.5 in. diameter) embedded in mudstone.

Well DRB 11 was designed to deviate about 15° from vertical in order to intercept the fault that had been indicated by the seismic and gravity-magnetic surveys. The well was drilled to a total depth of 3,320 ft, which represented a true vertical depth of 3,278 ft and a horizontal migration of 387 ft to the northwest. The well was cored completely, but the fault indicated by the geophysical surveys was not penetrated.

In well DRB 11, a zone about 5 ft thick was drilled at a depth of about 2,340 ft; numerous core fractures were present in this zone, and broken pieces of rock occasionally caved into the hole. This zone had numerous interlacing fractures, but neither slickensides nor fault gouge was present. This "caving zone" did not appear to be a major fault.

Cored material in well DRB 11 is 90 percent massive mudstone with no observed bedding; scattered layers of gritty, poorly sorted sandstone are present (10 percent). Larger fragments are schist, gneiss, or quartzite; these indicate a source region of crystalline rock similar to known rock northwest of well DRB 11.

HYDROLOGIC CHARACTERISTICS

The permeability or hydraulic conductivity of all of the tested Triassic rock is extremely low. Table 2 gives the yield, the transmissivity, and the calculated hydraulic conductivity for each of the five wells that penetrate the Triassic rocks; the entire open-hole protion of the well was used as the contributing aquifer. The wells that penetrate only the top of the Triassic (P5R and P12R) have the lowest permeability (10^{-7} to 10^{-9} m/day). These wells penetrate primarily claystone with very little sandstone. The permeabilities of the three deeper wells (DRB 9, 10, and 11), which penetrate sandstones, mudstones, and fanglomerate, are higher (10^{-6} to 5×10^{-8} m/day). These permeabilities are corroborated by 50 laboratory analyses of the permeability of about 25 representative core samples from well DRB 10 that included both vertical and horizontal permeability analyses; some samples were tested under low confining pressures and some under pressures that simulated the overburden pressure at the depth from which the sample was taken. The sandstones from well DRB 10 ranged in hydraulic conductivity from 10^{-5} to 10^{-6} m/day, whereas, conductivity of the mudstones ranged from 10^{-7} to 10^{-8} m/day. The hydraulic conductivity changed little with depth. For subsequent calculations, 10^{-6} m/day will be used for the overall average permeability for Triassic rocks.

Porosity tests were made on about 25 representative core samples from well DRB 10. These were mercury-injection tests for effective porosity of pores larger than 0.1 micron, and for total porosity. For sandstone cores, the effective porosity averaged about 7.0 percent and the total porosity, about 8.0 percent. For the mudstones, the average effective porosity was about 0.53 percent, whereas, the average total porosity was about 3.3 percent. The average effective porosity of all

samples tested was 3.0 percent, and the average total porosity was 6.3 percent. In subsequent calculations, 3.0 percent will be used as the average effective porosity of Triassic rock.

Of the five wells that penetrate the Triassic rock, two have heads above land surface and above the head in the overlying Coastal Plain aquifer. However, because of the very low permeability of the Triassic rock, no well that is open only to the Triassic has yet attained a water level that is in equilibrium with that of the formation. Figure 6 shows the recovery of well DRB 10 from the drawdown caused by the drilling of the well in May 1971. Only that part of the recovery curve above land surface is shown. As of March 15, 1973, the pressure at the well was 65 psi, which represents a head 222 ft above that in the overlying Coastal Plain aquifer at this location.

The pressure at well DRB 11 on April 13, 1973, was 115 psi, which represents a head of 365 ft above that of the overlying Coastal Plain aquifer (Fig. 7).

The geostatic ratio of well DRB 10 at its total depth is 0.453 psi/ft, and that of well DRB 11 is 0.473 psi/ft. The geostatic ratio of wells near the base of the coastal-plain section is 0.411 psi/ft. The difference of this value from the commonly used 0.433 psi/ft for freshwater wells is explained by the fact that the water level in the Coastal Plain aquifer does not rise to the land surface. Thus, these two wells in the Triassic basin are only slightly geopressured by most oil-field standards.

Of the other three wells, well DRB 9 penetrates the crystalline rock at the edge of the Triassic basin; however, a packer has been placed within 100 ft of the bottom of the Triassic section for 3 years, and the water levels in both the packer pipe (crystalline rock) and the annulus (Triassic) are at a static level about 90 ft below the surface (205 ft above mean sea level). This level is close to that of wells in crystalline rock, and packer leakage cannot be entirely ruled out. Indications at well P5R are that the head is normal, although the well yields so little water that many years will be required to be certain. Well P12R, adjacent to well DRB 11, also yields so little water that it will be years before any definitive statement can be made about its static water level. Thus, the distribution of the high head, both vertically and horizontally, has not been determined.

The chemical composition of formation water from wells DRB 9, 10, and 11 is given in Table 3. No reliable water samples have been obtained from wells P5R or P12R. The waters are highly mineralized (6,000 to

18,000 mg/l dissolved solids), although not as salty as seawater. The dominant ions are sodium, calcium, and chloride.

Dissolved gas effervesces from the water of wells DRB 10 and 11; however, these gases are quite different, as shown on Table 4. The gas from well DRB 10 is interesting in that it consists of about two-thirds hydrogen and one-third nitrogen. This is the same ratio as hydrogen to nitrogen in coal analyses from the Deep River Triassic basin in North Carolina (Reinemund, 1955, p. 105-107).

EXPLANATIONS OF HIGH HEAD

The possible causes of geopressured reservoirs were enumerated by Hanshaw and Zen (1965), and their list still appears to be reasonably complete. However, the recent literature has elaborated on many of these causes and the possible combinations of them (Jones, 1969a, b; Russell, 1973; Chapman, 1972; Dickey et al., 1972; Barker, 1972; Olsen, 1972; Perry and Hower, 1972). The causes listed by Hanshaw and Zen (1965) are: (1) aquifer head, (2) "fossil" head, (3) tectonic compression, (4) rapid loading and compaction of sediments, (5) water derived from an igneous intrusion, (6) infiltration of gas, (7) precipitation of minerals, (8) phase or chemical changes, (9) temperature increase, and (10) osmotic-membrane phenomena.

To evaluate several of the mechanisms listed, it is necessary to estimate the rate at which a pressure head in Triassic rock, higher than the head in surrounding formations, would dissipate after the source of pressure was removed. This rate was calculated as a function of depth (Fig. 8) after the method of Bredehoeft and Hanshaw (1968). The hydraulic conductivity used in these calculations was 10^{-6} m/day, obtained from field-test data (Table 2) and laboratory tests.

Specific storage, S_s, was calculated to be 1.3×10^{-7} ft^{-1} (4×10^{-7} m^{-1}) from the equation (Ferris et al., 1962, p. 88):

$$S_s = \gamma_0 \theta \left(\beta + \frac{\alpha}{\theta}\right),$$

where

γ_0 = specific weight of water, 0.036 lb/in.^{3},

θ = porosity, 0.03,

β = bulk modulus of compression of water, 3.3×10^{-6} in.^{2}/lb,

α = bulk modulus of compression of rock skeleton, 0.2×10^{-6} in.^{2}/lb,

$\frac{\alpha}{\theta}$ = reciprocal of bulk modulus of elasticity (obtained from three-dimensional sonic-logging data).

Figure 8 represents the rate of pressure dissipation in a formation that is laterally infinite and bounded below by an impermeable base and above by a formation in which hydrostatic head is normal. Because the Triassic basin is actually bounded below and laterally by crystalline rock having lower hydrostatic head than Triassic rocks and containing transmissive fracture zones, the rate of pressure relief is probably faster than shown in Figure 8. The Triassic basin is about 5,300 ft thick, according to models based on gravity and magnetic data (Fig. 5). Use of the 5,300-ft depth in Figure 8 gives the maximum remaining "excess" head in the Triassic section after the source of such head is removed.

The ten proposed mechanisms that could explain the high head are considered in order.

Aquifer Head

Aquifer head is not a possible explanation of the high head because there is no direct hydraulic connection from the Triassic rock to an area of normal groundwater recharge where the head is 555 ft above sea level (head at DRB 11, April 13, 1973). Nowhere is the head in the coastal-plain sedimentary beds this high, except perhaps in perched water tables. The head in the crystalline rock may reach this height in the Piedmont province many miles west of the Savannah River Plant, but there are many head measurements in both the coastal-plain beds and the crystalline metamorphic rock in the intervening area, none of which is this high.

"Fossil" Head

"Fossil" head (Watts, 1948, p. 196) corresponding to a previous greater depth of burial can be eliminated as a cause for the high head in these Triassic rocks. If the present head is a residual from some previously higher aquifer head, the magnitude of the previous head can be calculated from Figure 8. For example, because well DRB 11 vertically penetrated about 2,200 ft of Triassic rock, the present excess head of 365 ft (DRB 11) above the overlying aquifer would be 40 percent of that which existed 1,000 years ago, or 910 ft; 12 percent of that which existed 10,000 years ago, or 3,000 ft; 4 percent of that 100,000 years ago, or 9,100 ft; and 1 percent of that 1 m.y. ago, or 36,500 ft.

Cooke (1936) indicated that older Pleistocene (less than 10^6 years old) marine terraces are present in the vicinity of the Savannah River Plant; their presence indicates that there has been no denudation of material from the surface of the magnitude required to account for head now found in the Triassic basin. It is possible that in early and middle Tertiary or Cretaceous time (13-135 m.y. ago) the sedimentary thickness, and thus the water table, was somewhat greater than it is now--but not by the greater than 36,000 ft that would be required to account for the present head in the Triassic. It is a virtual certainty that the original Triassic section was thicker than it is now by several thousand feet, but it is not possible that even remnants of heads generated at that time (180 m.y. ago) persist today.

Tectonic Compression

Tectonic compression (Hubbert and Rubey, 1959, p. 153) cannot be completely ruled out as a cause of the head in the Triassic basin. Assessment of this mechanism is difficult without measuring the in-situ stress in Triassic rock. Even though there are some folds in the coastal-plain beds indicating compression, the general trend of movement since Late Cretaceous time (about 100 m.y. ago) has been the downwarping of the coastal plain that would result in tensional stresses. However, the general trend in the past is only of importance if it indicates what has been happening in the last 1 m.y., because only head changes in this period could possibly persist today (Fig. 8).

Earthquakes have occurred in historical time in the Coastal Plain region; however, these could result from the tensional forces caused by the continued downwarping of the coastal plain, as well as by tectonic compression. Local earthquakes have not caused a detectable fluctuation of water level in wells in crystalline rock. An earthquake of magnitude 4.5 that occurred on February 3, 1972, near Orangeburg, South Carolina, 50 mi away, showed no influence on the water level at well DRB 10; the water level was being continuously recorded at the time.

Rapid Loading and Compaction of Sediments

Rapid loading and compaction of sediments (Hubbert and Rubey, 1959, p. 152; Bredehoeft and Hanshaw, 1968, p. 1103) is an applicable explanation for high head in the Gulf Coast region, but not in the Savannah River Plant area. There are no thick sequences of Holocene, Pleistocene, or Pliocene rocks here. In fact, the surface is commonly formed on Miocene rocks, which means that in the last 13 m.y. there has been no gross addition of sediments, as would be required for the generated heads to persist. Rapid loading also implies that the Triassic is presently undergoing compaction. Because the Triassic section was originatly much

thicker, the remaining Triassic beds were probably buried more deeply at the end of Triassic time than they are now; thus they could not be compacting due to the present weight of sediment.

Water from An Igneous Intrusion

Water derived from an igneous intrusion (Platt, 1962) is not an applicable explanation, because the last igneous activity was in Late Triassic - Jurassic time (180 m.y. ago), and heads generated then could not persist to the present (Fig. 8).

Infiltration of Gas

Infiltration of gas is a rather vague suggestion for explaining high head. According to Tkhostov (1963, p. 5), A. K. Aliyev suggested that anomalously high rock pressures could be explained by "the penetration of gas from lower horizons to overlying ones along tectonic fissures or through the crater of a mud volcano." The source of the pressure in the "lower horizon" is not given. In any event, the reference appears to be to petroliferous gas. There are no known petroliferous deposits in this area.

Precipitation of Minerals

Precipitation of minerals (Levorsen, 1954, p. 403) with consequent changes in pore volume is an improbable explanation for the high head. There is no evidence of recent precipitation of minerals in the Triassic rock. Indications are that the rock-forming process (diagenesis) was completed before deposition of the coastal-plain sediments (100 m.y. ago), because the rock was subjected to weathering and deconsolidation at the "pre - coastal-plain" erosion surface. Heads generated before that time could not persist today (Fig. 8).

Phase or Chemical Changes

Phase or chemical changes as suggested by Bredehoeft and Hanshaw (1968, p. 1098) can be ruled out because the conditions required are not present. These changes include (1) gypsum to anhydrite plus water (Heard and Rubey, 1966), (2) montmorillonite to illite plus water (Powers, 1967; Perry and Hower, 1972), and (3) heavy petroliferous hydrocarbons to lighter hydrocarbons (Chaney, 1949). A fourth possibility, creation of a gas distillate from an igneous intrusion into a coal bed, is also highly improbable.

The first possibility is ruled out because no gypsum or anhydrite has been found. The Triassic water is not saturated with gypsum as would be expected if gypsum were present. Conversion of gypsum to anhydrite is an endothermic reaction (Hanshaw and Bredehoeft, 1968, p. 1113). The source of heat for this phase conversion on the Gulf Coast is the sinking of gypsum layers into regions of the earth that are hotter as a result of the geothermal gradient. There is no indication that the Triassic rock has become significantly deeper in the last 10 m.y., and if the conversion took place earlier the head would not persist (Fig. 8). There is evidence that Triassic rocks were more deeply buried in Late Triassic and Jurassic time than they are now. Thus, if gypsum were present in the basin, it probably would have converted to anhydrite at that time and not be available for conversion now.

Montmorillonite clay converts to illite clay and releases water starting at a temperature of 62°C or, on the Gulf Coast, at a depth of 3,600 ft (Hanshaw and Bredehoeft, 1968, p. 1117). This conversion continues in different stages over an interval of 4,000 to 10,000 ft (Perry and Hower, 1972). Mineral analyses of 25 representative core samples from well DRB 10 show that only three samples from the well contained montmorillonite, and these were all above a depth of 1,685 ft. Mineral analyses of 46 representative core samples from well DRB 11 showed no montmorillonite. The clay in both wells was mostly illite with some kaolinite. Thus, at present there is little montmorillonite in the Triassic rocks available to convert to illite. The conversion probably took place at the end of Triassic time and in Jurassic time when the present Triassic section was buried under 7,000-13,000 ft of additional material. (These thickness estimates of removed material are developed by analogy with the depth conditions in the Gulf Coast, without taking into consideration differing geochemical conditions.) Any pressure developed then would not persist today. The montmorillonite that now is restricted to near the top of the Triassic rock is probably not primary montmorillonite from original deposition, but has resulted from weathering and hydration of detrital minerals during the "pre-coastal-plain" erosion cycle (Late Jurassic - Early Cretaceous). Thus, this montmorillonite exists in a weathered layer of Triassic rocks, the equivalent of the saprolite over crystalline metamorphic rocks. In addition, the temperature in the Triassic deposits, as shown by the generalized temperature log (Fig. 9), is not sufficiently high for illite conversion to take place. The temperature is only about 32°C at the bottom of well DRB 11.

The conversion of heavy petroliferous hydrocarbons to lighter hydrocarbons is very unlikely because petroleum is not found in the known Triassic basins of the East Coast, and none would be expected in the Triassic at the Savannah River Plant.

The composition of gas that effervesces from well DRB 10 water (Table 4) suggested that the origin of the gas could have been compaction of, or igneous intrusion into, a coal bed. However, because the last igneous activity in the area was in Late Triassic - Jurassic time (180 m.y. ago), any pressure head created by this activity would not persist now (Fig. 8). The composition of the gas from well DRB 11, which also exhibits high pressure, is entirely different.

Temperature Increase

Temperature increase (Levorsen, 1954, p. 403; Barker, 1972) cannot be completely ruled out as a cause for the high head. A rise in temperature of 2°C would increase the pressure of pure, completely confined water by about as much as the difference between the presently o served head in the Triassic rocks and that in surrounding formations. The temperature log (Fig. 8) of well DRB 11 shows an overall geothermal gradient of 0.81°F/100 ft, or 14.7°C/km. This is the same as the geothermal gradient in wells in crystalline metamorphic rock (0.82°F/100 ft; Diment et al., 1965) at the Savannah River Plant, but is less than in many parts of the country and, in particular, much lower than the gradient in the Gulf Coast (1.3 to 1.7°F/100 ft).

Several mechanisms that might cause a change in water temperature are discussed below.

1. Injection of igneous material has been ruled out as having occurred too long ago to create a head that could persist. The low geothermal gradient does not indicate any residual heating at present from intrusive rocks.

2. Exothermic chemical reactions and phase changes are a possibility. Reactions that have been considered are endothermic and, furthermore, happened too long ago for their effects to persist. However, all possible geochemical changes may not have been considered.

3. Faulting creating frictional heat seems unlikely as a cause. Evidence from the drilling of wells DRB 11 and P12R has demonstrated that, along the indicated fault being investigated, if it exists, no movement has occurred since Cretaceous time. If this is a general condition, the movement was too long ago for the heating effects to persist. Well DRB 11,

adjacent to the fault, does not show a greater geothermal gradient than do other wells in the general region.

4. Long-term climatic changes can influence temperatures to great depths, as indicated by permafrost in polar regions to depths of 1,300 ft (Ferrians et al., 1969, p. 7). Presumably during the Pleistocene Epoch, the southern boundary of the permafrost region moved south. Although it did not reach South Carolina, it is probable that the ground was significantly cooler then and has been warming at least since the end of the Pleistocene. If the ground is transmitting a warming pulse downward from the surface, it should result in a decreased geothermal gradient. Birch (1948) estimated that the earth's surface has warmed about 10°C during the last 20-30 thousand years. Diment et al. (1970, p. 553) have estimated that this warming has altered the present geothermal gradient by about 2°C/km at depths of about 2,400 ft; the change is greater at shallower depths and less at deeper depths. This appears to be an adequate increase in temperature to account for an excess head of between 200 and 400 ft of water. The geothermal gradient of well DRB 11 (Fig. 9) is lower in the upper part of the Triassic section, being about 3°C/km less than in the deeper parts of the well. The geothermal gradient in the coastal-plain strata is higher than in the Triassic rocks; however, this region is dominated by the lateral movement of about 0.5 ft/day of large volumes of water. Thus, convection in the coastal-plain beds has probably brought the geothermal gradient into equilibrium with relatively recent surface temperature changes.

5. The global heat-flow pattern is not homogeneous with respect to space and probably is not constant in time. Thus, it is possible that the heat flux in this area is increasing, causing a rise in water pressure in rocks of extremely low permeability. However, no evidence exists to indicate that a change is occurring, or, if it is, to show that it results in heating or cooling of the area. The extended time scale required for such changes makes this explanation improbable.

Osmotic-Membrane Phenomena

Osmotic-membrane phenomena (Hanshaw and Zen, 1965; Jones, 1969a, b; Bredehoeft et al., 1963; Milne et al., 1964; Olsen, 1972) are considered as one of the more likely causes of the high Triassic head. On the basis of the dissolved-solids concentration of Triassic water (Table 3) and the fresh water in the overlying Coastal Plain aquifer (dissolved solids = 30 mg/l), and the temperature at the top of the Triassic section, the

equilibrium osmotic-pressure differential should be about 245 ft of fresh water at well DRB 10 and about 390 ft of fresh water at well DRB 11. These differentials convert to a pressure-gage reading at the wells (including corrections for fluid density) of 75 psi for well DRB 10 and 125 psi for well DRB 11. These pressures are shown in Figures 6 and 7, along with the recovery curves of the two wells. Neither well has yet exceeded the osmotic equilibrium pressure; thus, osmotic-membrane phenomena remain a viable explanation.

Two factors favor osmosis as an explanation for the high head in the Triassic basin. One is that the phenomenon has been shown in laboratory experiments to be operative on clays (Milne et al., 1964; Young and Low, 1965; Olsen, 1972). Thus, with the known clay content of the Triassic rocks and the known chemical difference of the Triassic waters and the waters of the Coastal Plain aquifer, osmosis should be operative--in addition to any other explanation for the high head that is offered. The second factor is that, as pointed out by Hanshaw and Zen (1965), osmosis is an equilibrium process rather than a single-occurrence phenomenon whose effect dissipates with time. Even though the hydraulic conductivity of the Triassic rocks is low, this may not be the only factor in preserving the high head. It may be preserved, and indeed generated, by the equal and opposite chemical drive of water toward the more saline Triassic basin.

CONCLUSIONS

The buried Triassic basin that underlies the southern third of the Savannah River Plant appears to be about 30 mi long and at least 6 mi wide. Geophysical investigations indicate that the Triassic section is about 5,300 ft thick. The Triassic strata consist predominantly of mudstone and poorly sorted sandstone, but poorly sorted fanglomerates are present near the basin margin. No coal or igneous rocks have been penetrated. Triassic and Jurassic faults may be present, but they have had no movement along them since Cretaceous time. The permeability of the Triassic rocks is extremely low, as is their porosity. A formation-water head well above that of the overlying Coastal Plain aquifer exists in at least some areas or depths of the Triassic basin, although this head difference is not nearly as great as is sometimes found in petroleum exploration. Most of the commonly offered explanations of high head are not applicable to this Triassic basin. On the basis of available information, three explanations appear to be possible: osmotic-membrane phenomena, current tectonic compression, and warming due to post-Pleistocene climatic changes.

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Table 1. Seismic Velocities in and Around Dunbarton Triassic Basin (ft/second)

Depth (ft)	Well DRB 9 Triassic and Metamorphic Rock	Well DRB 10 All Triassic Rock	Well DRB 11 All Triassic Rock	Well DRB 8 All Crystalline Metamorphic Rock
1,200	13,600	9,200	13,400	20,200
1,500	15,000	15,500	16,800	20,200
2,000	15,000	17,000	18,000	22,200 (1,950 ft)
2,500	15,000	18,500	19,000	
3,000		18,000	18,000	
3,500		19,000		
4,000		18,500		
Max. velocity in Triassic section	17,500	22,000	21,800	
Max. velocity in metamorphic section	18,000			23,000

NOTE: Refraction measurement of velocity in Triassic section at well P5R (Bonini and Woollard, 1960), 15,850.

Table 2. Yield, Transmissivity, and Calculated Hydraulic Conductivity

Well	Total Depth (ft)	Length of Open Hole (ft)	Yield	Transmissivity	Permeability (hydraulic conductivity)
DRB 9[a]	2694	1366[b]	1.7×10^{-2} gal/min 9×10^{-2} meter^3/day	0.4 gpd/ft[c] 5.0×10^{-3} meter^2/day	3×10^{-4} gpd/ft^2 1.6×10^{-2} millidarcy 1.2×10^{-6} meter/day
DRB 10	4206	2962	8.7×10^{-3} gal/min 4.7×10^{-2} meter^3/day	3.7×10^{-3} gpd/ft 4.5×10^{-5} meter^2/day	1.2×10^{-6} gpd/ft^2 6.6×10^{-5} millidarcy 5.0×10^{-8} meter/day
DRB 11	3320	2091	6.7×10^{-2} gal/min 3.6×10^{-1} meter^3/day	7.6×10^{-2} gpd/ft 9.4×10^{-4} meter^2/day	3.6×10^{-5} gpd/ft^2 2.0×10^{-3} millidarcy 1.5×10^{-6} meter/day
P5R	1313	60	1.4×10^{-4} gal/min 7.6×10^{-4} meter^3/day	3.7×10^{-4} gpd/ft 4.6×10^{-6} meter^2/day	6.2×10^{-6} gpd/ft^2 3.4×10^{-4} millidarcy 2.5×10^{-7} meter/day
P12R	1272	53	1.2×10^{-4} gal/min 6.5×10^{-4} meter^3/day	4.3×10^{-6} gpd/ft 5.3×10^{-8} meter^2/day	8.1×10^{-8} gpd/ft^2 4.5×10^{-6} millidarcy 3.3×10^{-9} meter/day

[a]Triassic only.
[b]Packed off interval.
[c]Possible packer leakage.

Table 3. Chemistry of Water from Triassic Rock at Savannah River Plant (constituents in mg/l)

Well	Depth (ft)	Date	SiO_2	Fe	Ca	Mg	Na	K	HCO_3	SO_4	Cl	Dissolved Solids (residue)	Specific Conductance (µmho)
DRB 9	2055	7/14/69	1.0	0.00	518	83	1120	30	72	420	2,620	5,950	6,770
DRB 10	4212	5/27/71	3.5	0.04	1990	53	2100	44	85	110	6,720	11,900	18,000
DRB 11	3320 (3278 vertical)	2/26/73	-	< 1	3845	8.5	2710	22	~1	~1	11,600	18,500	26,700

Table 4. Analyses of Gas Dissolved in Water from Triassic Rock (mole %)

Well	Date	CO_2	Ar	O_2	N_2	CH_4	He	H_2
DRB 10	2/29/72	0.023	0.47	0.008	32.7	0.098	0.28	66.2
DRB 11	2/27/73	0.005	0.70	0.46	92.7	0.021	<0.1	6.0

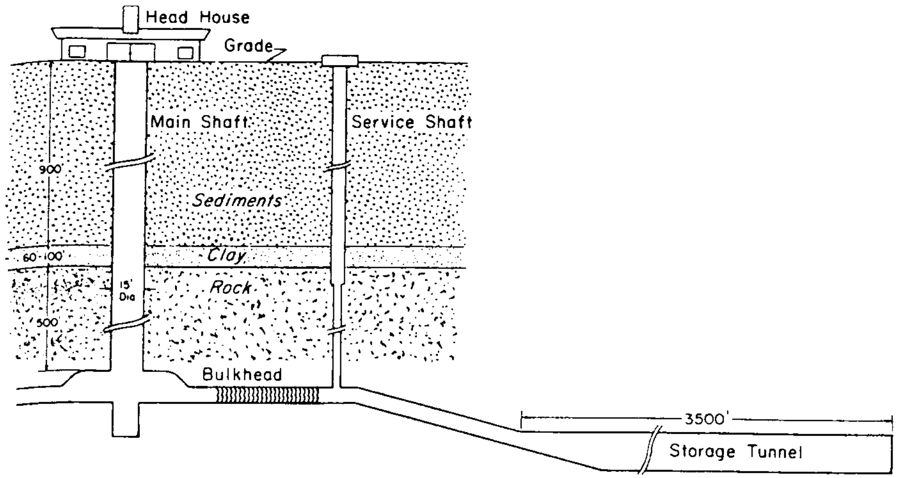

FIG. 1--Conceptual cross-sectional view of bedrock waste-storage facility.

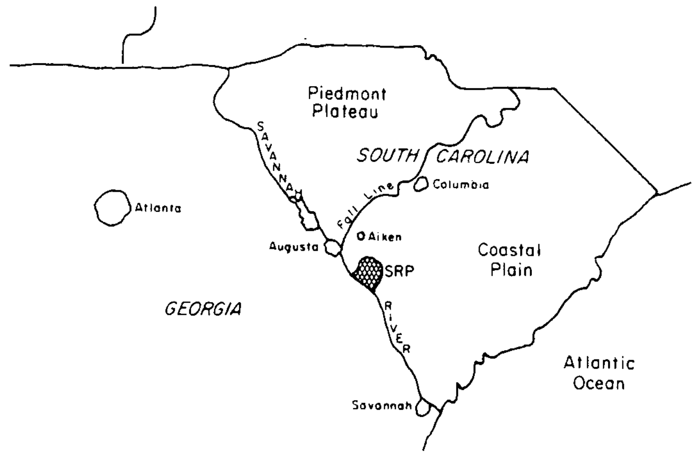

FIG. 2--Location of Savannah River Plant and nearby geologic provinces.

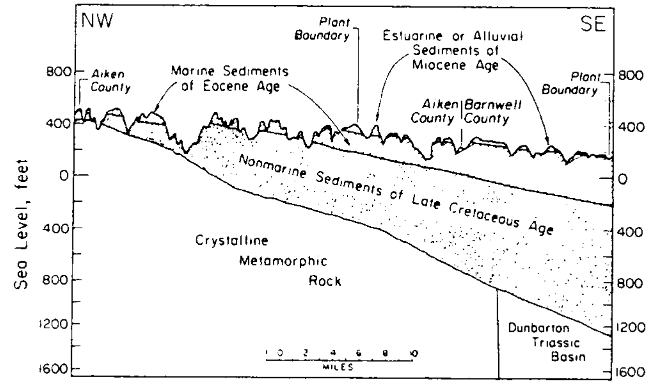

FIG. 3--Generalized NW-SE geologic profile across Savannah River Plant area.

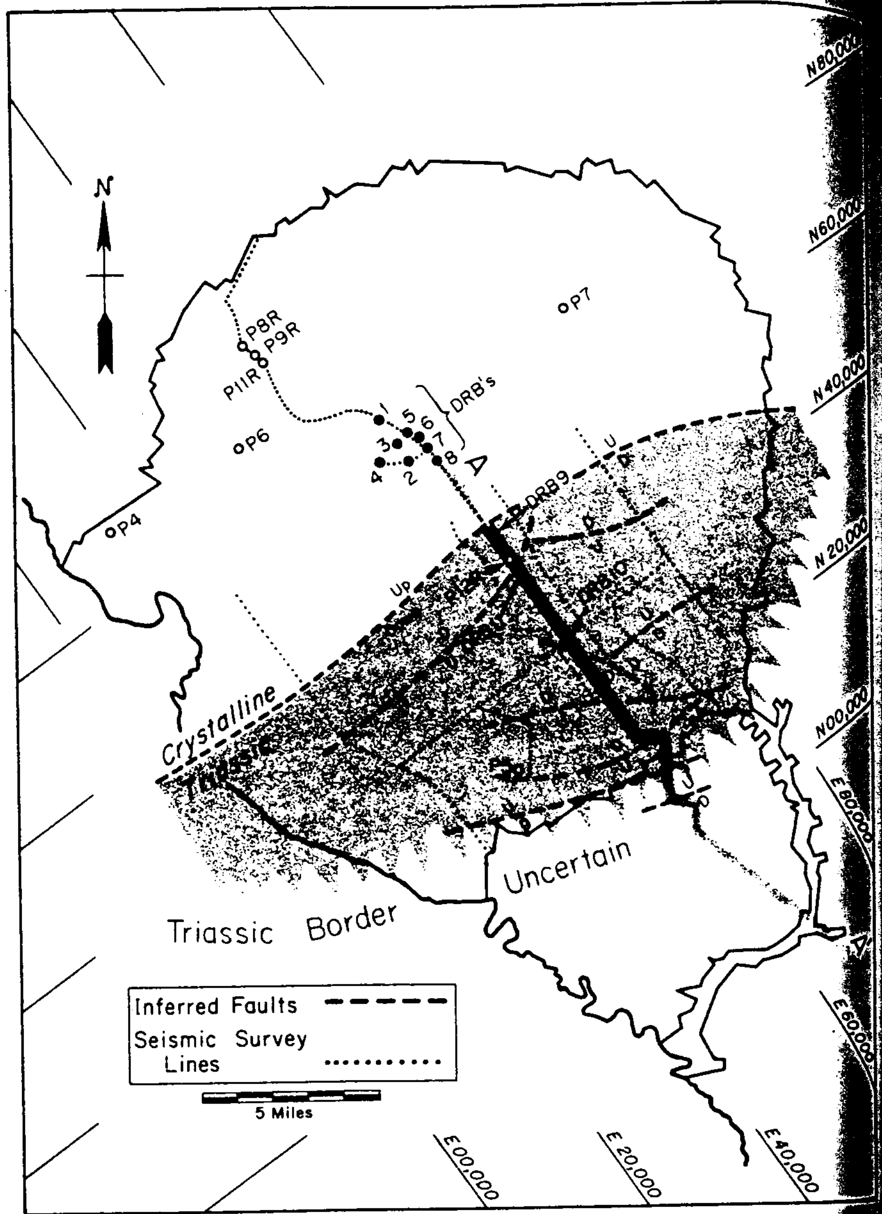

FIG. 4--Outline of buried Triassic basin at Savannah River Plant; reflect seismic traverses; and possible faults inferred from seismic data in Triassic basin.

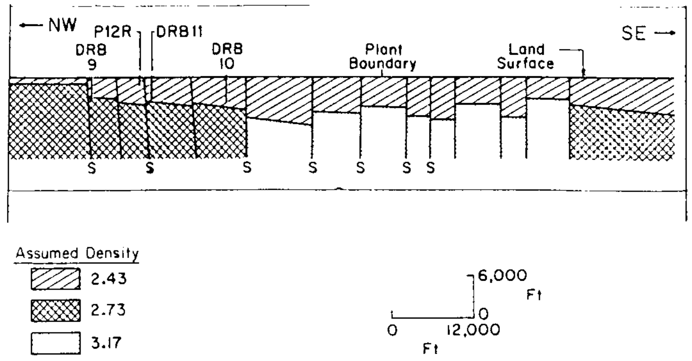

FIG. 5--Cross-sectional model of Triassic basin based on magnetic and gravity surveys.

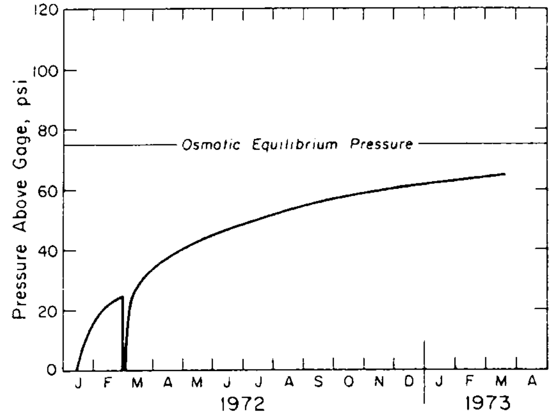

FIG. 6--Pressure recovery of well DRB 10.

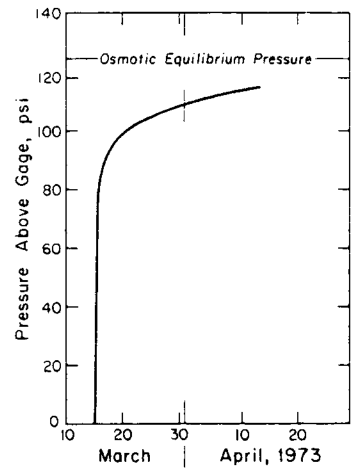

FIG. 7--Pressure recovery of well DRB 11.

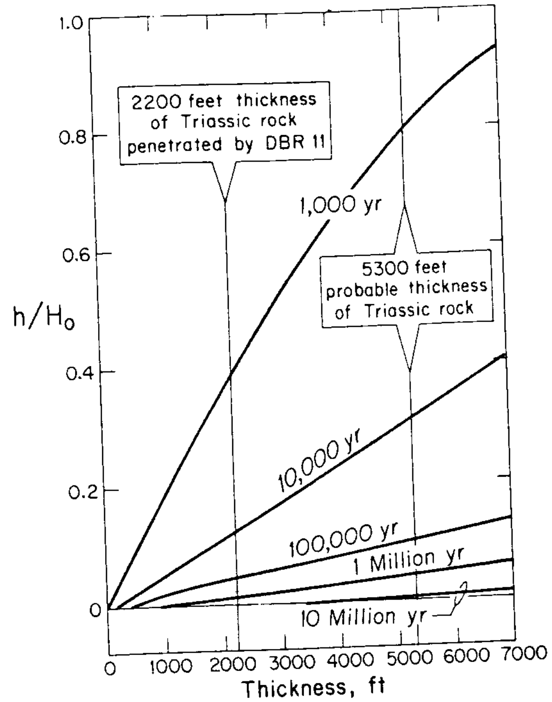

FIG. 8--Head dissipation in Triassic rock.

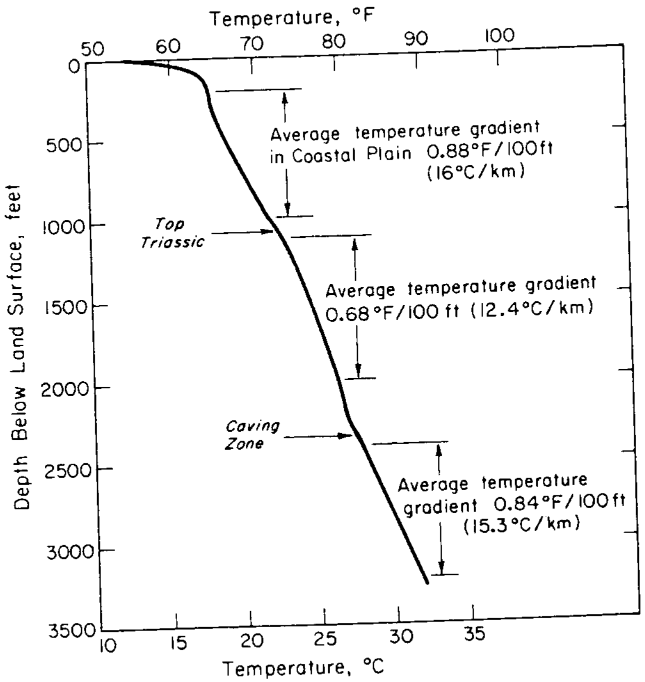

FIG. 9--Generalized temperature log and geothermal gradient, well DRB 11

ARTIFICIAL RECHARGE OF TREATED WASTE WATERS AND RAINFALL RUNOFF INTO DEEP SALINE AQUIFERS OF PENINSULA OF FLORIDA[1]

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ABSTRACT Fast-growing population centers of Florida, mainly on coastal beaches, have imposed large demands on the sources of fresh water. They also threaten to degrade the aesthetic and recreational value of the area with their waste waters. The largest of these population centers is on the southeastern end of the peninsula of Florida--the Miami area. The second largest is on the central west coast of the peninsula in the Tampa-St. Petersburg area.

The peninsula of Florida, at the southeastern extremity of the continental United States, is underlain by several thousand feet of carbonate rock and only minor amounts of siliceous clastic units. Cavernous limestone and dolomite aquifers at relatively shallow depths constitute the principal sources of fresh water in the area. Deeper cavernous zones, separated from the freshwater zones by practically impermeable limestone and dolomite, are uniquely suited for receiving injected fluids.

Deep-well disposal of waste waters into deep saline aquifers, after secondary biological treatment and disinfection, is feasible (1) if an aquifer exists that can accept treated waste waters without significant changes in its hydraulic and structural characteristics, and (2) if use of the water in that aquifer, in adjacent ones, or from surficial sources is not impaired.

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Two large disposal wells have been successfully constructed for a private utility in the Miami area; they are approximately 3,000 ft (915 m) deep. The disposal wells recharge an artesian aquifer which has chloride concentration near that of seawater (19,000 mg/1). The receiving aquifer is overlain by a thick aquiclude, above which is another aquifer (saline but of lower chloride concentration); a thick impervious section overlies that aquifer and separates the highly mineralized water from the shallow and fresh groundwater.

Three steel casings, cemented at the proper depths, permit injection into the deeper aquifer with protection of the upper strata. Monitoring of the upper saline-water-bearing stratum, where any possible leak from the deeper aquifer would normally be detected first, is performed through the annulus between the two inner well casings. An integrated water-quality aquisition system continuously monitors the injected waste and provides alarm and pump shutdown if established limitations are exceeded.

Operation of the first well for over a year has proved fully reliable and both economically and environmentally advantageous. Eight similar disposal wells are being considered in the area.

A new research program is being implemented to inject, store, and recover (when needed) rainfall runoff in the deep saline aquifers of southern Florida.

A test-prototype well is presently being constructed within St. Petersburg in order to determine (1) the characteristics of the deep underground formations, (2) the quality of the deep groundwaters, (3) the injection-rate capacity and associated increase in pressure, (4) the ratio of the amount of fresh water that could be subsequently recovered to that injected, and (5) the quality of the recovered water.

INTRODUCTION

The state of Florida, in the southeastern extremity of the United States, has one of the highest rates of population growth in the nation. The state has a total area of 58,000 sq mi (150,000 sq km), of which 4,400 sq mi (11,400 sq km) is covered by approximately 30,000 lakes, rivers, and swamps (Jones, 1972). It has 1,300 mi (2,100 km) of general coastline and about 8,700 mi (14,000 km) of detailed shoreline. The state population in 1973 was estimated at 7,400,000, an increase of almost 50 percent since 1960 (population 4,951,560).

Municipal or public water supply in 1970 was 884 million gal/day (3.35 million m^3/day) for the 5.42 million people served by public systems

(Pride, 1973), which gives an average of 164 gal (617 l) per day per person --one of the highest averages in the nation. (Of the water used as public supplies, 86 percent was groundwater.) An additional 3,300 million gal (12.7 million m^3) per day of fresh water was used in 1970 for self-supplied industry, irrigation, and rural population. Thus, total water usage in the state for 1970 was 4,200 million gal (16.0 million m^3), or 630 gal (2,363 l) per day per person. (State population in 1970 was 6,789,443; an additional 23,151,700 tourists visited the state that year.) Cooling water used in power production is excluded from these figures.

Population growth, mainly in urban centers near coastal beaches, has imposed large local demands on the sources of fresh water. Those population centers also threaten to degrade with their wastes the aesthetic and recreational value of the area. The largest of these population centers is in southeastern Florida. It extends from the city of Miami to West Palm Beach and is known as "Florida's Gold Coast." The second largest center is on the central west coast of the peninsula in the Tampa-St. Petersburg area.

The people of Florida are concerned with the threat such population centers present to the environment, and they demand solutions. In several parts of the state, criteria for municipal wastewater discharge into surface water bodies require effluents with not more than 5 milligrams per liter (mg/1) of biochemical oxygen demand (BOD) and suspended solids, 3 mg/1 of total nitrogen, and 1 mg/1 of total phosphorus (Florida Department of Pollution Control, 1972). The trend is to require those criteria for the entire state. The only practical method at present to achieve such a goal is advanced wastewater treatment. Such treatment requires plants which are expensive to construct and operate; skilled operators, presently very scarce; additional chemicals; and a considerable amount of power--already in short supply.

One alternative to the construction of those types of plants is the disposal into deep saline aquifers of effluents from existing biological plants which already reach 90 percent reduction in organic loading and suspended solids and provide bacterial disinfection by chlorination. Present hydrologic knowledge (Tibbals and Kimrey, 1970) indicates that the treated freshwater effluent would not mix readily with the saline waters of the aquifer but, instead, would create a large freshwater bubble in storage at the top of the aquifer which could be partially recovered at a later date for low-quality uses (irrigation) or for further specific treatment and reuse. The residual organic load in the effluent is expected to continue anaerobic biologic degradation at depth, and plating of the phosphate on the carbonate sediment of the aquifer and reduction of the dissolved nitrates into easily removable nitrogen gas are possible.

The concept of storing such effluents as described above has also brought about the strong possibility of storing rainfall runoff in a similar manner in the coastal and southern areas of the peninsula where very low-relief topography and very high evaporative losses reduce the feasibility of surface storage. An outline of these possible uses and a study of the extension throughout Florida of the Boulder Zone (deep zones of high transmissivities) have been made by Vernon (1970).

FUNDAMENTALS OF UNDERGROUND DISPOSAL

Underground wastewater disposal by wells can be achieved successfully if five general requirements are fulfilled (García-Bengochea, 1970a). These are:

1. A stratum or strata (aquifer) which can accept the waste are present.
2. The hydraulic and structural characteristics of the aquifer will not be changed significantly by the disposal of the waste.
3. The disposal of such waste will not impair present or future use of the water in such aquifer.
4. The disposal of such waste will not impair present or future use of the water in adjoining aquifers or surface water supplies.
5. The installation is designed properly, with consideration of the physical, chemical, and biological characteristics of the waste and the hydrogeologic characteristics of the receiving aquifer and confining strata.

Failure to meet some of these requirements has resulted in water pollution, deficiencies, and accidents, which have given this disposal method bad publicity and caused it to be controversial. Compliance with the above requirements was the key to successful completion of the disposal wells described herein.

HYDROGEOLOGY OF SOUTH FLORIDA

The hydrogeology of the southern peninsula of Florida was discussed by García-Bengochea and Vernon (1970). A brief summary of the hydrogeology follows.

The state of Florida is underlain by one of the most productive artesian aquifer systems in the world. The system is called the "Principal

Artesian aquifer" or "Floridan aquifer" (Parker et al., 1955), which might be a misnomer, because the Floridan aquifer actually consists of a series of groundwater-bearing strata of cavernous limestones and dolomites separated by thick and practically impervious layers of marl and dense limestone.

Limestones and dolomites within the Floridan aquifer range in age from middle Eocene through middle Miocene. These beds are overlain by a section composed mainly of clays, marls, and dense limestones of Miocene age which confine water under pressure. They also form the base of shallow groundwater, including the Biscayne aquifer, the principal source of water in southeastern Florida. The confining beds in this area extend from approximately 200 to 800 ft (60-250 m) or more in depth. The artesian aquifer is recharged by rainfall in the northern part of the peninsula where it is exposed, and through sinkholes, lakes, and permeable overlying beds in most of the central part.

Potentiometric maps of the Floridan aquifer within the state (Fig. 1) give a generalized idea of the artesian pressures, usually in the top part of the aquifer. Normal piezometric heads in the peninsula range from approximately 20-30 ft (6-9 m) above mean sea level near the southern shores to 50-60 ft (15-18 m) around Lake Okeechobee, and up to 130 ft (40 m) in the central highlands. The artesian pressure causes wells to flow in practically all the areas around and south of Lake Okeechobee, where ground elevation usually ranges from 10 to 20 ft (3-6 m) above mean sea level.

The first water-bearing zone of the Floridan aquifer in the southern peninsula extends from approximately 900 to 1,300 ft (270-400 m) in depth, and chloride concentrations range from 600 to 1,500 mg/1. The next water-bearing strata start at approximately 1,600 ft (500 m), and chloride concentrations are in excess of 2,000 mg/1; chloride concentrations increase abruptly to 10,000 mg/1 near 1,800 ft (550 m) depth. Cavernous sections of the Boulder Zone start at approximately 2,950 ft (900 m). These conditions suggested the development of disposal wells which penetrate this deeper zone (Boulder Zone).

The Florida Department of Pollution Control does not allow the discharge of hazardous or polluted waters into aquifers whose waters have chloride concentrations of 1,500 mg/1 or less. These zones are reserved for future demineralization. For the above reasons, injections were planned at greater depths.

Deep cavernous strata (usually below 2,950 ft in depth) are highly mineralized in parts of the central and all of the southern peninsula of

Florida. These deep beds have been penetrated mainly by oil-exploratory wells. Drilling through the highly cavernous zones breaks off large fragments of dolomite and limestone which collect, together with debris, in the caves. Such conditions present serious difficulties similar to those experienced where drilling through large boulders. For this reason, such strata are referred to as the "Boulder Zone" (Kohout, 1967).

Figure 2 shows the location of wells which penetrate cavernous sections of the Floridan aquifer and the Boulder Zone in central and southern Florida. This figure also shows the chloride concentration of the water in the upper part of the Floridan aquifer. Such concentration increases with depth until it reaches that of seawater (19,000 mg/l).

WELL CONSTRUCTION

Two wells to receive treated wastewater effluent have been completed in Dade County (Miami area). A third one is under construction at the writing of this paper, in the city of St. Petersburg (Tampa Bay area), to test the possibility of injecting rainfall runoff and to determine the ratio of the amount of fresh water that could be subsequently recovered to that injected. Locations of these wells are shown in Figure 2.

The first of the two wells in Dade County was completed in February 1970 to receive the effluent from the Sunset Park wastewater-treatment plant of the General Waterworks Corporation, an investor-owned utility. This plant, south of Miami and about 5 mi (8 km) from the coast, is a biologically activated sludge plant with a maximum capacity of 6 million gal/day (260 l/second). Until the completion of the well the treated effluent was discharged to an adjacent canal which, after flowing through one of the nicer residential areas south of Miami, discharges into the Biscayne Bay in the Atlantic Ocean. The plant provides 90 percent reduction of organic load and suspended solids, and also provides chlorine disinfection. The Dade County Pollution Control office required removal of nutrients (nitrates and phosphates) or discontinuation of discharge. This first well, after completion and thorough testing, has been successfully operated since early summer 1971 at an average injection rate of approximately 3 million gal/day (130 l/second). Peak injection rates have already reached maximum capacity.

The second well, completed in late summer 1972, is about to start operation from the Kendale Lakes plant. The installation, also owned and operated by General Waterworks Corporation, is practically identical to that at Sunset Park but is located 3.5 mi (5.6 km) due west.

Both wells have similar construction, and in both the top of the Boulder Zone is at a depth of approximately 2,950 ft (900 m). Figure 3 (Vilaret and García-Bengochea, 1972) shows some of the construction details and data from the second well. Information is shown in the same units recorded in the field: feet for depth, inches for diameter, and degrees Fahrenheit for temperature. Similar information for the first well was reported by García-Bengochea (1970b).

The third well, under construction in St. Petersburg, should be completed by spring or early summer of 1974.

DRILLING, CASING, AND LOGGING

Construction at each site begins by rotary-drilling a small-diameter (8 in. or 200 mm) testhole to determine depth and then by cementing each of the three casings provided. Testhole drilling is staged for each string of casing and allows also for the collection of formation and water samples at regular intervals. Formation (washed) samples are collected every 10 ft (3 m) and at every change in formation. Water samples are collected where possible every 20 ft (6 m) using air reverse rotary drilling. Field measurements are made immediately after water-sample collection for temperature, density, specific conductance, and chloride concentration. Geophysical logs for resistivity, spontaneous potential, gamma radiation, hole diameter, and temperature are also usually run in the testhole. Temperature logs have also been run within 12 hours after cementing the inner casing to determine the height reached by cement. This height has been double-checked later by cement-bond logging.

Because of the cavernous nature of the upper part of the Floridan aquifer, it is not possible to cement the inner casings in one single operation from the bottom to ground surface. This situation has been used to an advantage by not cementing the upper part of the annulus between the inner casing and the hole or middle casing; such annulus is then used for monitoring purposes as described below.

Temperature gradients show an inverted geothermal gradient in both wells. Water temperature dropped in the first well from 74°F (23°C) at 968 ft (295 m) to 61°F (16°C) at 2,947 ft (898 m). This anomaly is explained by the cooling effect from the deep seawaters in the Straits of Florida (Uchupi, 1968), approximately 35 mi (60 km) east of the first well. Temperature in the Straits (through which the Gulf Stream flows) is reported as between 45° and 50°F at 2,000-ft depth (7°C and 10°C at 600 m)

(Kohout, 1967). The high transmissivity of the Boulder Zone seems to facilitate such thermal exchange. The inverted geothermal gradient was also confirmed in the second well, as shown on Figure 3. Gamma-ray correlation between the wells and the sequence of formations penetrated are shown on Figure 4 (Vilaret and García-Bengochea, 1972).

Table 1 summarizes total depth and casing data from both wells. Casings are black steel with a thickness of 9.5 mm (0.375 in.).

INJECTION TEST

At the completion of each well, a pumping (injection) test was planned to determine (1) injection capacity, (2) pressures at the wellhead while injecting at different flow rates, (3) pressures at the receiving aquifer while injecting at different flow rates, and (4) effect of the injection pressure on the annulus pressure (a measure of the head in the upper part of the Floridan aquifer).

The differences between the pressures at the wellhead and at the receiving aquifer (2 and 3, above) are caused by the friction losses through the casing and borehole. Pressures at the injection stratum were read through 2,900 ft (884 m) of 3-in.-diameter (76 mm) drill rods temporarily installed in the well during the test to serve as a piezometer. Pressure gage P_b (Fig. 5) was connected to the top of the 3-in. drill rod.

Flowmeter logging in the first well indicated that practically all the water produced (or received) by the well was below 2,920 ft (890 m). Increase in pressure at P_b due to pumping into the well indicates the net increase in pressure at the receiving stratum, independent of the friction losses in the casing and borehole.

Pressure gage P_w measures the pressure at the wellhead, which is the sum of the increase in pressure at the receiving stratum plus friction losses through casing and borehole.

Pressure gage P_a indicates the pressure in the annulus. Any significant leak through the inner casing, through the cement seal at its bottom or through natural channels would cause a change in the artesian pressure of the annulus.

Because of the specific gravity (1.027) of the cool salt water in the column formed by the inner casing, the water level in that casing stays slightly below ground level. The same phenomenon was observed in the 3-in. (76 mm) drill rod (piezometer). The artesian pressure then increases to 31.3 psi or 72 ft (2.2 kg/cm^2 or 22 m) above land surface when fresh water with a specific gravity of slightly less than 1,0, is slowly injected into

the 3-in. drilling rod to displace the salt water in the corresponding column.

Results of the injection tests on the two Dade County wells are summarized in Table 2. Those from the first well are also shown graphically on Figure 6. Units in Table 2 and on Figure 6 are the same units as recorded in the field: U.S. gallons per minute for flow rate and pounds per square inch for pressure.

After each increase in flow, pressure equilibrium was reached in both wells very rapidly. Annulus pressure remained constant for the duration of the injection test on the first well (Sunset Park), indicating that no leak exists between the bottom hole or the inner casing and the upper aquifer. During the first test on the second well (Kendale Lakes), a slight rise in annulus pressure from 15.1 to 15.4 psi (1.06 to 1.08 kg/cm^2) in 49 hours indicated the possibility of leakage of the injected water into the annulus. Upon investigation, a pinhole leak of approximately 1 liter per minute was located in the uppermost welded joint of the 16-in. (410 mm) inner casing. After repair, a second and final test was run in that well with the results shown in Table 2.

Although the static pressures in the wellheads or in the receiving aquifers were higher than in the annulus, a higher piezometric head is not indicated. The specific gravity of the water in the annulus is higher than 1.0 and varies with depth (see Fig. 3), whereas that of the fresh water being injected is slightly lower than 1.0 (ambient temperature, approximately 75°F or 24°C). In order to establish a comparison between piezometric heads, the specific gravity of the water in the annulus and in the inner casing must be identical for the same column length. When the conversion is made, the piezometric head of the annulus is approximately 2 ft (0.6 m) higher than that of the Boulder Zone.

Injection tests show that fluid can be injected into the Boulder Zone at high flow rates because of its unusual transmissivity. Independent tidal studies made by Meyer (1972) on diurnal changes of water level of the Boulder Zone at the Sunset Park well indicate a transmissivity of 16.8 X 10^6 U.S. gallons per day per foot (2.4 m^3/sec/m) and a storage coefficient of 1.05 X 10^{-5}.

MONITORING SYSTEM

A system has been designed to monitor the operation of each disposal well and any possible effect such operation may have on the groundwater quality of the upper aquifer (artesian) and of the shallow Biscayne aquifer (water table). It includes surveillance of (1) quality and pressure of the fluid being injected into the well; (2) quality and pressure of the brackish water in the upper aquifer (artesian); and (3) quality of the fresh water in the shallow Biscayne aquifer, under water-table conditions.

The best location to monitor the quality and pressure of the upper part of the Floridan aquifer in these wells is at the annulus between the inner and middle casings. This location provides the quickest detection of any leakage through the cement seal at the bottom of the inner casing or through the casing itself. In addition, any upward leakage through the 600 ft (185 m) or more of the dense limestone overlying the receiving aquifer would have a greater tendency to develop near the well itself, as this is the point of highest pressure increase caused by the injection.

Surveillance under conditions 1 and 2 is done with a Honeywell integrated water-quality data-acquisition system (W200; Honeywell, Inc., 1969) designed to provide for continuous multiple-pen recording of the following variables.

Effluent Flow to well:	
Flow rate	0 - 7 million U.S. gal/day (307 lps)
Injection pressure*	0 - 100 psi (7.03 kg/cm^2)
Chlorine residual*	0 - 5 mg/l
pH	6 - 9
Dissolved oxygen	0 - 5 mg/l
Specific conductance	10 - 1,000 micromhos/cm
Turbidity*	10 - 100 Jackson units
Sample Stream from Annulus:	
Pressure*	0 - 30 psi (2.12 kg/cm^2)
Specific conductance	200 - 20,000 micromhos/cm

This system (Fig. 7) provides a visual and audible alarm plus injection-pump shutdown for the variables marked with an asterisk (*) in the event any of them exceeds or falls below the limits established by the regulatory agencies. It covers the determinations to be made on a continuous and automatic basis as established by the regulatory agencies.

Surveillance of the quality of the fresh water in the shallow Biscayne aquifer is done monthly by complete chemical and bacteriologic analyses of samples from a shallow well within 300 ft (90 m) of the injection well.

CONCLUSIONS

Properly designed, constructed, and operated disposal-well systems

discharging into saline aquifers offer a solution for the disposal of treated waste waters in those areas where hydrogeologic conditions allow such a method. Furthermore, these systems are an asset to the freshwater resources of the area. They actually constitute a method of artificial recharge.

Two recently completed disposal wells south of Miami, Florida, are an example of such a "disposal-artificial recharge" scheme. One of these wells has been in successful operation since July 1971, receiving an average of approximately 3 million gal/day (130 l/second). Such a well made unnecessary the construction of an advanced wastewater-treatment plant, which not only is considerably more expensive to build but requires considerable more power, additional chemicals, and sophisticated operation. The second well at a similar plant is about to start operation.

Hydrogeologic conditions in the southern peninsula of Florida are ideal for this type of disposal, and plans are under way for a major disposal system with a maximum capacity of 50 million U.S. gal per day (2,200 l/second) for the regional wastewater-treatment plant to serve the southern part of Dade County (Black, Crow and Eidsness, 1972). Also, a third well is presently under construction in St. Petersburg. This third well is to determine (1) the characteristics of the deep underground formation at that site, (2) the quality of the deep groundwater, (3) the injection-rate capacity and associated increase in pressure, (4) ratio of the amount of fresh water that could be subsequently recovered to the amount injected, and (5) quality of the recovered water.

Not all areas offer favorable conditions. A good knowledge of the local hydrogeology is fundamental. The next step is drilling and testing, to be followed, if successful, by sound design of the installation and of a reliable monitoring system. Strict regulations and surveillance by a responsible agency are mandatory. This agency should also have the authority to plan and manage the total water resources of the region.

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Table 1. Total Depth and Casing Data,
General Waterworks Corporation--Dade County Disposal Wells

	Well 1--Sunset Park		Well 2--Kendale Lakes	
	m	ft(')/in.(")	m	ft(')/in.(")
Total Depth	898	2947'	977	3205'
Outside Casing:				
Diameter	0.660	26"	0.760	30"
Length, from-to	0-64	0-210'	0-75	0-246'
Cemented, from-to	0-64	0-210'	0-75	0-246'
Middle Casing:				
Diameter	0.560	22"	0.610	24"
Length, from-to	0-166	0-545'	0-231	0-758'
Cemented, from-to	0-166	0-545'	0-231	0-758'
Inner Casing:				
Diameter	0.410	16"	0.410	16"
Length, from-to	0-552	0-1810'	0-691	0-2266'
Cemented, from-to	511-552	1678-1810'	531-691	1742-2266'

Table 2. Summary of Injection Test Data,
General Waterworks Corporation--Dade County Disposal Wells

Flow Rate		Time Period	Pressure Readings (psi)		
lps[1]	gpm[2]	(hours)	P_a Annulus	P_w Wellhead	P_b Hole Bottom
Sunset Park:					
0	0	-	16.0	31.8[3]	31.8
66	1050	3	16.0	35.0	32.8
129	2050	3	16.0	37.0	32.9
195	3100	17	16.0	43.0	33.0
250	3970	3	16.0	51.0	33.5
Kendale Lakes (first test):					
0	0	-	15.1	33.6[3]	33.6
254	4025	29.25	15.2	50.0	33.8
252	4000	43.15	15.3	50.0	33.7
255	4050	48.75	15.3	50.0	33.6
0	0	1.5[4]	15.4	33.7	33.7
0	0	16[4]	15.4	33.2	33.6
Kendale Lakes (second test):					
0	0	-	13.9	-[5]	-[6]
276	4385	1.0	13.9	41.6[7]	-
254	4025	1.25	13.9	39.6	-
212	3360	1.50	13.9	37.6	-
181	2875	1.75	13.9	35.6	-
276	4385	2.0	13.9	41.6	-
275	4370	22.0	13.9	42.1	-
274	4345	48.0	13.9	41.6	-
277	4390	240.0	13.9	41.6	-
277	4390	504.0	13.9	42.1	-

[1]Liters per second.

[2]U.S. gallons per minute.

[3]From P_b under static conditions.

[4]Recovery measurements, hours since injection stopped.

[5]Water level below measuring point.

[6]No piezometer installed for this test.

[7]Wellheads are lower during this test because of less friction as a result of absence of piezometer.

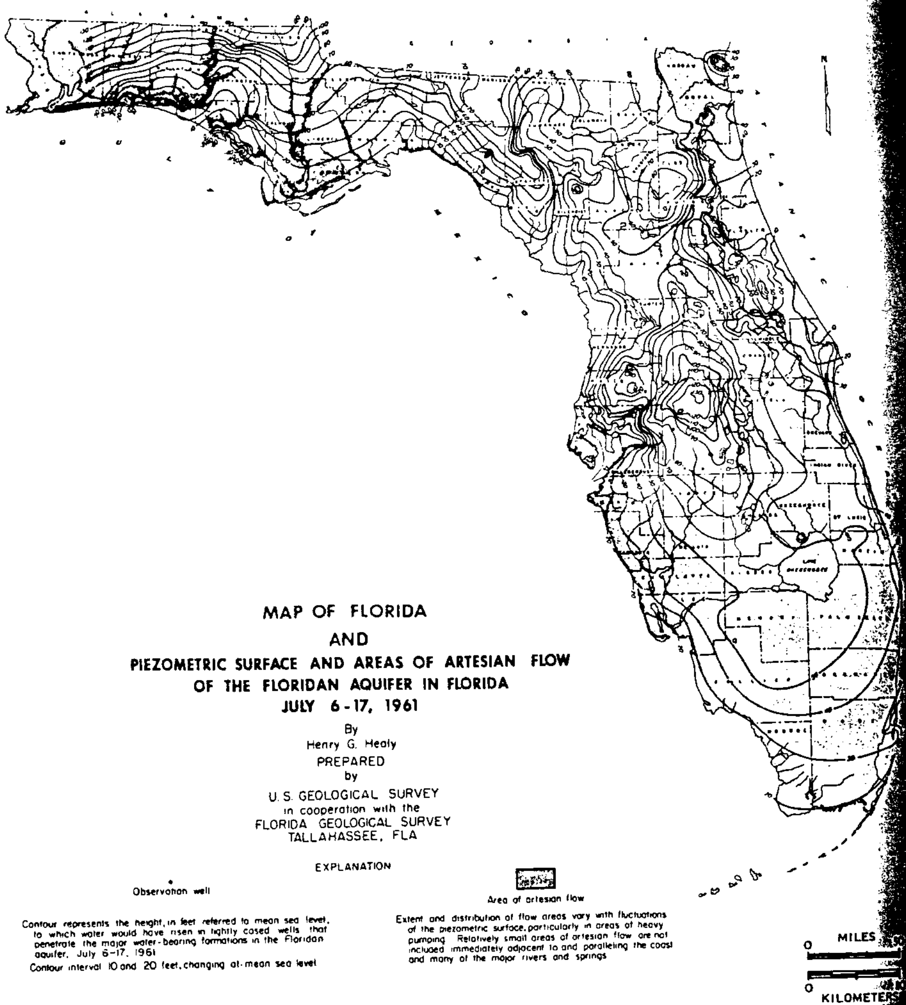


FIG. 1--Potentiometric map of Floridan aquifer within Florida.

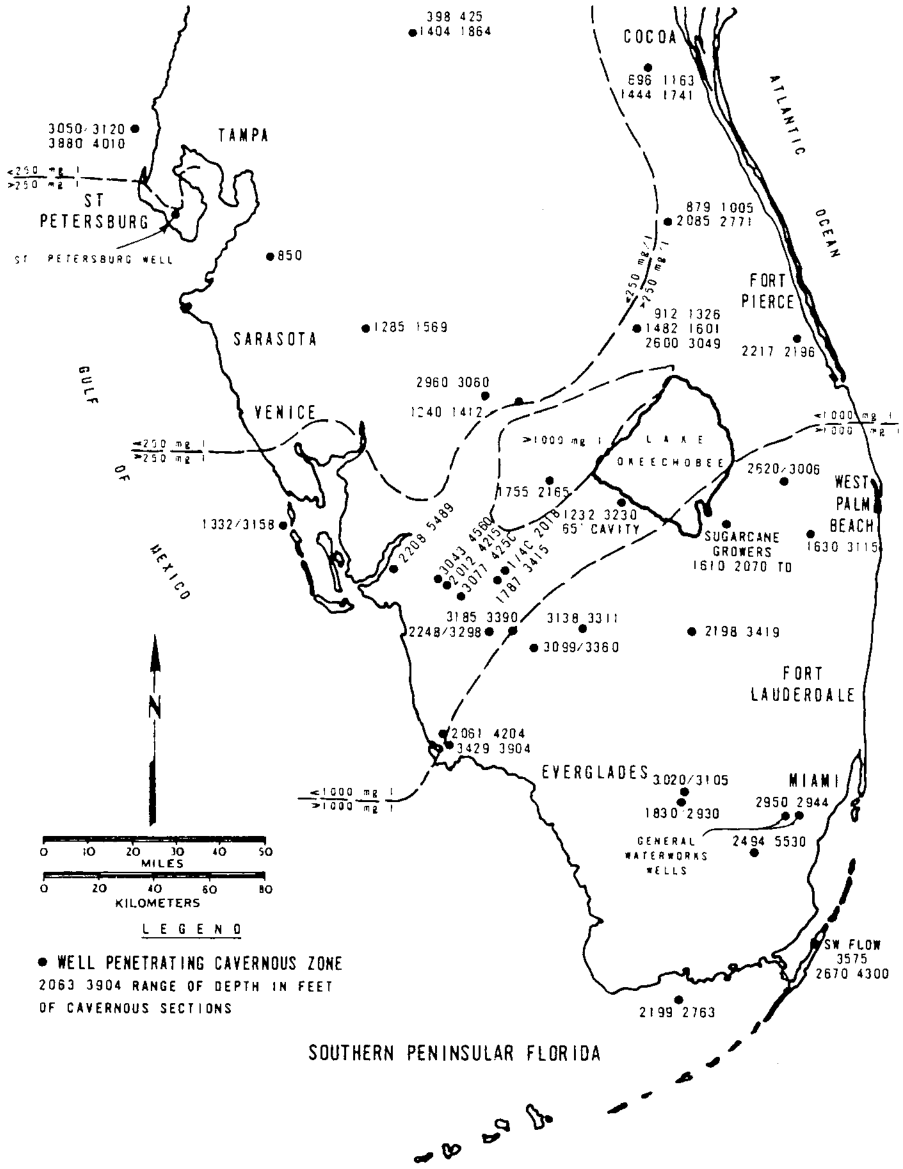
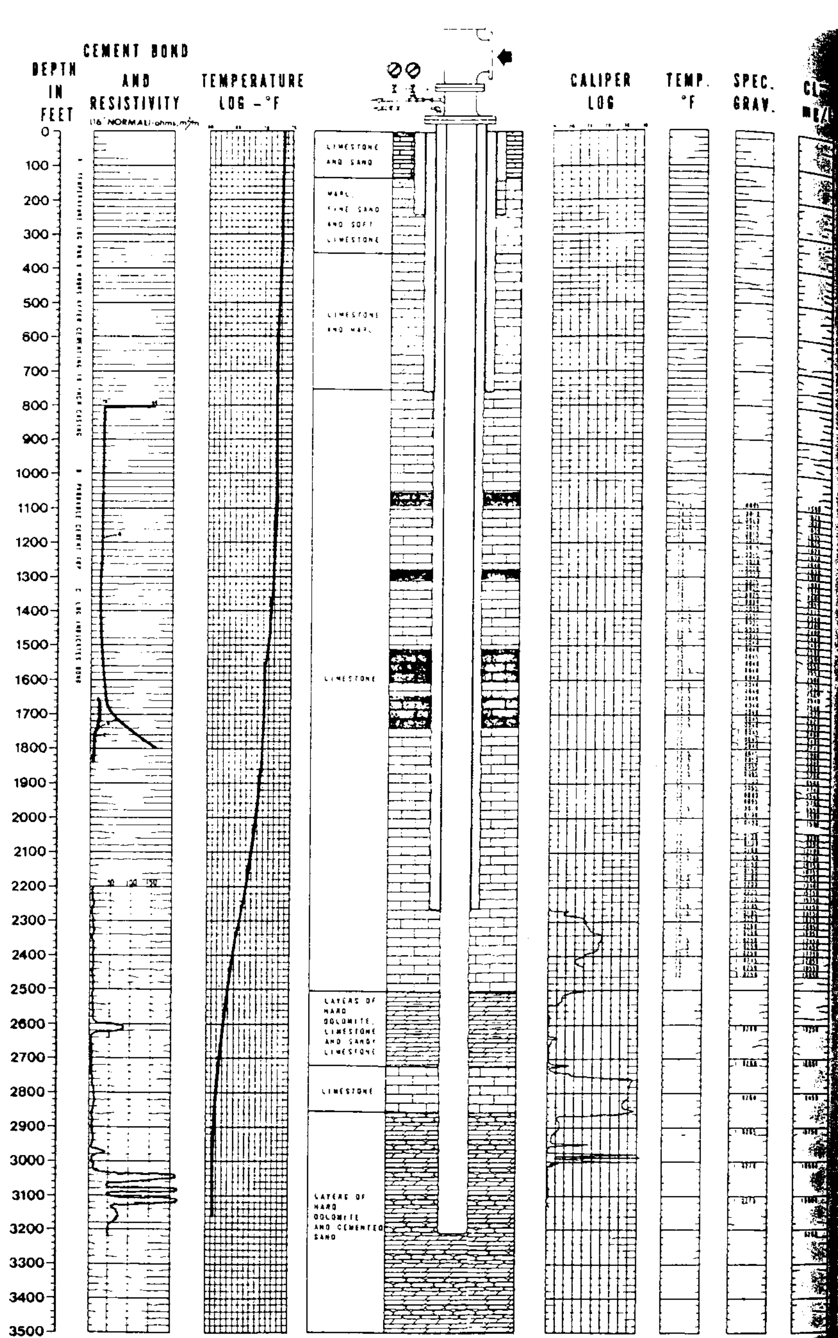


FIG. 2--Wells penetrating cavernous zones and chloride concentration in upper part of Floridan aquifer, central and southern peninsula of Florida. After Vernon and Garcia-Bengochea (1967).



Blue Shading Indicates Permeable Zones

FIG. 3--Data from Kendale Lakes injection well.

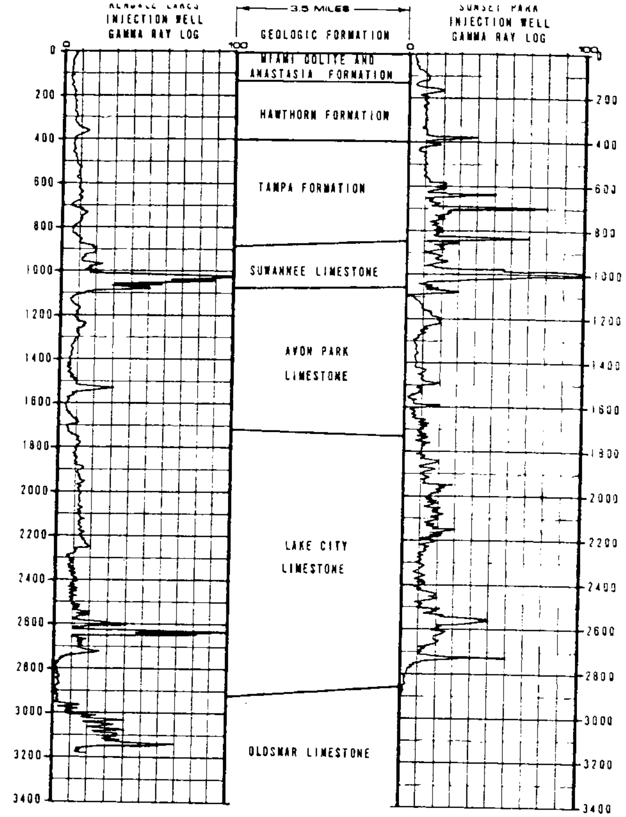


FIG. 4--Gamma-ray correlation of Kendale Lakes and Sunset Park disposal wells. After Vilaret and Garcia-Bengochea (1972).

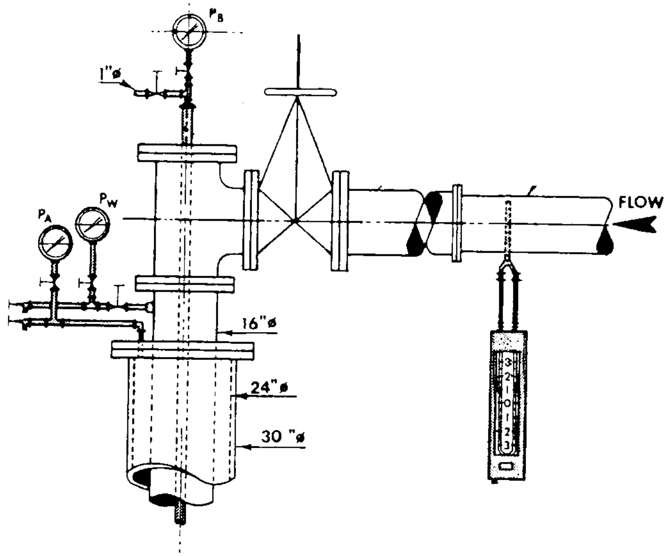


FIG. 5--Instrumentation layout for injection test.

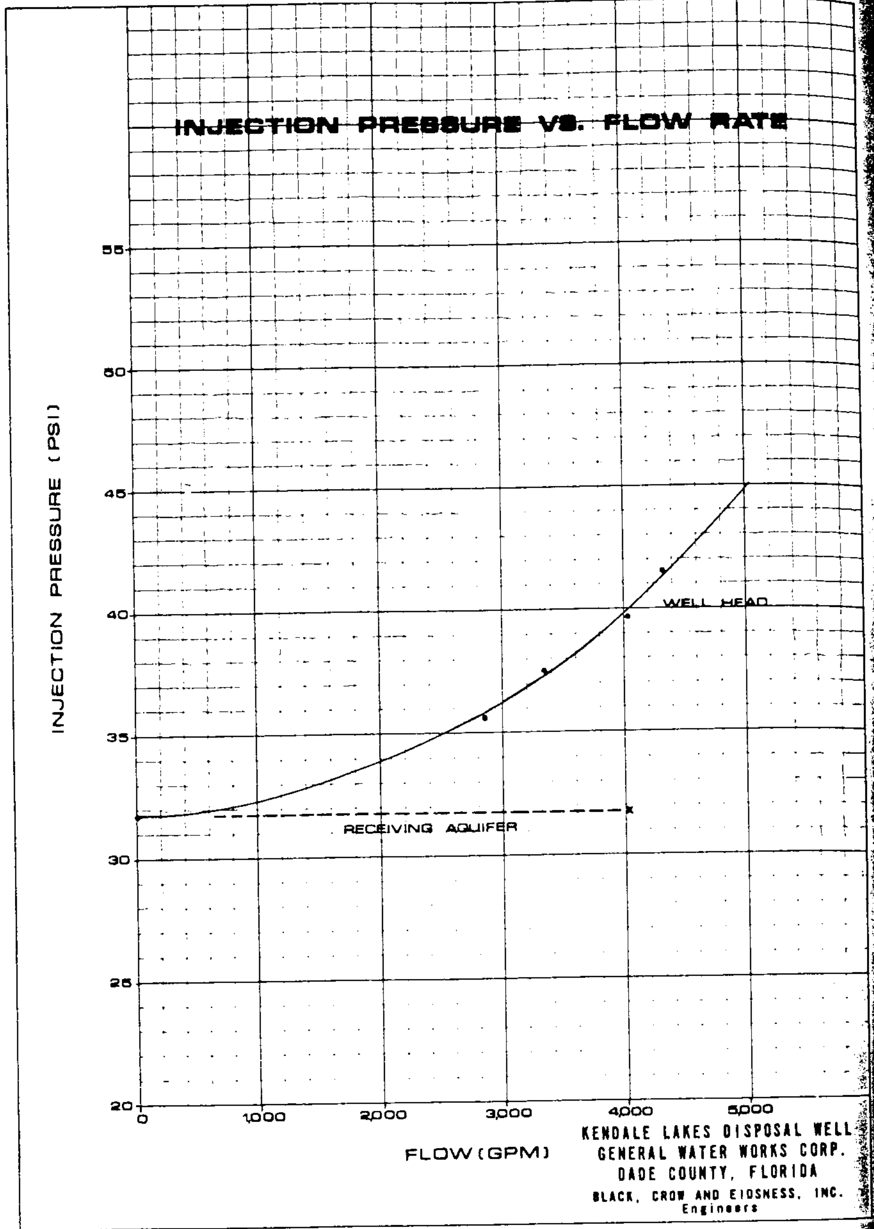


FIG. 6--Results of injection tests on Kendale Lakes disposal well.

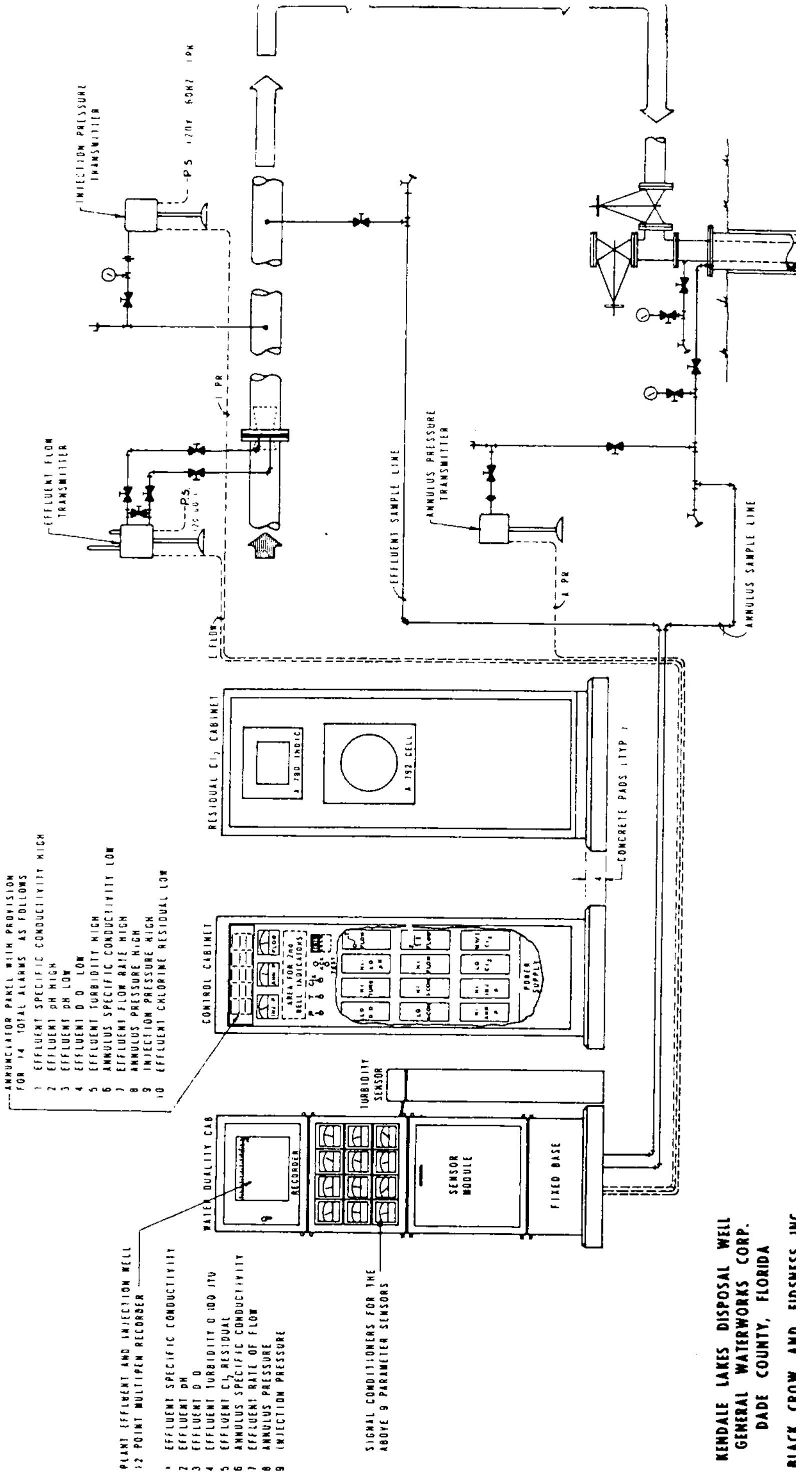


FIG. 7--Deep-well injection-parameter monitoring system.

INJECTION OF ACIDIC INDUSTRIAL WASTE INTO A SALINE CARBONATE AQUIFER: GEOCHEMICAL ASPECTS[1]

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ABSTRACT A section of carbonate rocks that includes several highly permeable cavernous zones filled with saline water underlies the south part of peninsular Florida at depths from about 1,500 to 4,500 ft. Because these cavernous zones are capable of yielding or accepting large quantities of fluids, they are used for storage of industrial and municipal liquid waste at several places. One such place is at the south end of Lake Okeechobee, near Belle Glade, Florida, where the effluent from a sugar mill and liquid waste from the production of furfural processed from sugar cane bagasse have been injected at depths between about 1,500 and 2,200 ft. The waste ranges in temperature from 71 to 103°C and in pH from 2.6 to 4.5; it is highly organic (chemical oxygen demand, 6,000-26,000 mg/l). Since 1966, more than 800 million gal of this waste has been injected. Injection rates range from 400 to 800 gal/minute at pressures of 30-60 lb/in.^{2} at the wellhead. The waste is partly neutralized almost immediately to a pH of 5.5 by dissolution of limestone. Caliper logs show localized patterns of dissolution of the carbonate aquifer. Anaerobic degradation of the organic waste begins near the injection well as indicated by the presence of hydrogen sulfide, methane, carbon dioxide, and nitrogen.

The waste has moved both upward and laterally in the aquifer system, as indicated by water-quality changes in monitor wells. Upward movement of altered waste into an overlying brackish-water zone was detected in a 1,400-ft monitor well 75 ft away from the injection well. Lateral movement was detected in a monitor well in the injection zone 1,000 ft from the injection well. When upward movement of waste was detected, injection was discontinued and the well was drilled and cased several hundred feet deeper. Data collected since the well was deepened are inconclusive regarding the effectiveness of the increased well depth in restricting the waste to the injection zone. Investigations to date (1973) at Belle Glade clearly point up the value and importance of a monitoring program.

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INTRODUCTION

The U.S. Geological Survey is engaged in an investigative and research program to develop a scientific basis for assessing the long-term environmental impact of subsurface waste injection. Prediction of movement, chemical interaction, and ultimate fate of injected liquid waste is difficult. As noted by Piper (1969, p. 2), "Uncritical acceptance [of deep-well injection] would be ill advised." The complexity of both the waste and the subsurface environment preclude making generalizations. The present (1973) state of the art and available data demand a thorough regional and localized study of each proposed waste-injection system.

Objectives and Approach

This paper presents the results of hydraulic and geochemical investigations of an industrial waste-injection system southeast of Lake Okeechobee, near Belle Glade, Florida (Fig. 1).

Objectives of the Survey's research program at the Belle Glade site were to develop a better understanding of liquid waste - aquifer rock - native fluid interactions and to help in determining the movement and ultimate fate of liquid wastes underground in a saline-water-filled carbonate environment of high transmissivity.

To help meet these objectives, samples of injected waste, native aquifer water, and fluids from the zone of active waste - rock - native fluid interaction were obtained periodically for comprehensive geochemical analyses and determination of dissolved gases and microbe content. Additionally, X-ray diffraction analyses of aquifer rock were made and geophysical logs of the injection well were obtained for interpretation.

Regional hydrology--A section of early Tertiary carbonate rocks that includes several highly permeable cavernous zones filled with saline water and separated from one another by relatively impermeable carbonate and/or evaporite beds underlies the south part of peninsular Florida at depths from about 1,500 to 4,500 ft. Approximately the upper 1,000 ft of this thick cavernous section is the lower part of the principal artesian aquifer of the southeastern United States, commonly called the "Floridan aquifer" in Florida (Stringfield, 1966). The Floridan aquifer, as defined by Parker et al. (1955, p. 188-189), consists mostly of middle Eocene to middle Miocene limestone and dolomite which act more or less as a hydrologic unit in most of Florida. The effective porosity of the aquifer and of those Tertiary carbonate rocks underlying it is variable, however; in many places the sequence consists of very cavernous intervals separated by zones of dense limestone and dolomite which act as confining beds.

From December 1966 through 1971, acid liquid waste was injected at the Belle Glade site into the lower part of the Floridan aquifer between depths of about 1,500 and 1,900 ft. The chloride content of the native groundwater at the time the injection well was drilled in 1966 ranged from 1,650 mg/l at 1,500 ft to more than 7,000 mg/l at 1,900 ft. The injection zone is separated from the overlying upper part of the Floridan aquifer by 150 ft of dense limestone. Native groundwater near the base of the upper Floridan aquifer in the Belle Glade area contains about 1,000 mg/l of chloride.

State regulations limit waste injection to zones containing native groundwater with at least 1,500 mg/l of chloride. After leakage of waste into the upper Floridan aquifer was detected in the fourth season of operation, efforts to confine the waste to the injection zone in the lower Floridan aquifer at Belle Glade resulted in modifications to the well. The well was deepened to 2,242 ft and cased down to 1,938 ft. Chloride content of the native groundwater in the new injection interval (1,938-2,242 ft) ranged from about 7,000 mg/l near the top of the interval to nearly 16,000 mg/l in the lower part of the interval.

The potentiometric surface of the upper part of the Floridan aquifer in most of south Florida (Fig. 1) is above land surface. At the Belle Glade site, the altitude of land surface is about 15 ft above mean sea level, and the altitude of the potentiometric surface is about 57 ft above mean sea level. The slope of the potentiometric surface indicates that the direction of regional flow in the Floridan aquifer is southeast

toward the Atlantic Ocean. Water moving through the aquifer from the Belle Glade area may ultimately discharge from submarine exposures of the aquifer into the Straits of Florida.

Figure 2 shows the general sequence of permeable and confining beds in the subsurface between Belle Glade and the Atlantic coast, as well as the stratigraphic relations between the Belle Glade injection-well site and the Straits of Florida.

DESCRIPTION OF INJECTION SYSTEM

The Belle Glade injection system consists of one injection well, one shallow monitor well, and one deep monitor well; the latter is also used as a standby injection well. Industrial waste is piped to the injection well, where it is injected into the brackish-saline water of the lower part of the Floridan aquifer. The two monitor wells are used to assess the effects of waste injections on the aquifer system (Fig. 3).

The injection well was originally cased to a depth of 1,495 ft with 12-in. casing, and an open-hole completion was made in the lower Floridan aquifer to a depth of 1,939 ft. An 8-in. stainless steel injection liner was run to the bottom of the casing and set with a packer. In the fall of 1971, the 8-in. injection liner was extended to 1,938 ft and cemented in place. The well was then deepened to 2,242 ft.

The shallow monitor well is 75 ft south of the injection well; it monitors hydraulic and geochemical effects within the upper Floridan aquifer above the confining beds. The well is cased to a depth of 648 ft and completed in the open hole in the upper part of the Floridan aquifer to a total depth of 1,400 ft.

The deep monitor well is 1,000 ft southeast of the injection well and is used to assess effects within the injection zone. It is cased to a depth of 1,490 ft with 12-in. casing and completed in the open hole to 2,067 ft in the lower part of the Floridan aquifer. This well also has an 8-in. stainless steel liner from the surface to the bottom of the casing, where it is set with a packer.

Data were also collected from a well which penetrates the upper part of the Floridan aquifer at the University of Florida's Everglades Experiment Station, 2 mi southeast of the Belle Glade site. This well is cased to 957 ft and completed in the open hole to 1,332 ft.

The injected waste fluids include the effluent from a sugar mill and the waste from the production of furfural, an aldehyde processed from sugar cane bagasse. The waste is hot, acidic, and highly organic. The temperature ranges from about 71 to 103°C and the pH ranges from 2.6 to 4.5; the chemical oxygen demand (COD) ranges from about 6,000 to 26,000 mg/l.

Information on this system for the first 3 years of operation has been reported by García-Bengochea and Vernon (1970).

Operating history--Injection began late in 1966, and it is seasonal. During the fall, winter, and spring, injection is more or less constant, but the system is inactive during the late summer. Injection rates range from 400 to 800 gpm (gallons per minute), and wellhead injection pressures range from 30 to 60 psi (pounds per square inch). During 1966-72, more than 800 million gal of waste was injected. The amount of waste injected each operating season gradually increased (Fig. 4); about 30 percent of the total waste was injected in the 1971-72 season. The injection index (injection rate in gallons per minute) divided by bottomhole-pressure increase above pre-injection pressure (in psi) has increased more than fivefold (Fig. 5), indicating that permeability near the well bore has increased substantially.

Although no pressure effects are evident in the two monitor wells, 75 and 1,000 ft away, geochemical effects are evident in both. Geochemical effects were detected at the deep monitor well in 1967, according to Vernon and García-Bengochea (written commun., 1967). An increase in COD and a decrease in pH were detected by company staff in the shallow monitor well in the fall of 1969, at the start of the fourth operating season.

The geochemical changes detected in the shallow monitor well indicated upward leakage of the waste, either around the casing or through the confining beds. To rectify this situation, the company's engineering consultants recommended that the injection well be deepened to about 2,200 ft and that the casing and liner be extended to about 1,900 ft to seal off the upper Floridan aquifer completely. Deepening the well and extending the casing were expected to reduce the chances that the waste would find its way up into the overlying upper part of the Floridan aquifer. In the fall of 1971, the well was deepened to a depth of 2,242 ft and cased and cemented to 1,938 ft.

During the above modification phase, from October 8, 1971, through January 12, 1972, the deep monitor well was used for waste injection. More than 75 million gal of waste under injection prssures of about 50 psi was injected into the monitor well. Upon completion of the recommended modifications, the company began injecting the waste into the deepened injection well.

Since the deepened injection well was returned to use, the deep monitor well has been allowed to backflow continuously at about 2-3 gpm. Results of the geochemical analyses of samples from this well will be discussed in a following section.

In October 1972, about 10 months after waste injection was resumed in the deepened well, a mechanical caliper log indicated that the hole was bridged or plugged at 1,945 ft and was open to the aquifer in the interval 1,923-1,945 ft. Indications were that all of the injected waste was exiting from the open borehole through two 8-ft cavities in the carbonate aquifer within the 22-ft interval below the bottom of the casing. Apparently, 15 ft of a 23-ft length of mild-steel casing extending from the bottom of the stainless steel casing, and originally set at 1,938 ft, had been removed by corrosion.

RESULTS

Chemistry of waste, native fluids, and aquifer rocks--Analyses of selected constituents in the waste being injected on October 20, 1971, and on March 28, 1972, are given in Table 1. The waste is hot, acidic, and highly organic. It also contains high concentrations of nitrogen and phosphorus. The organic carbon concentration exceeds 5,000 mg/l, and suspended solids exceed 1,800 mg/l. The chloride concentration is low compared to that in native groundwater in the injection zone. The specific gravity of the waste ranges from approximately 1.004 to 1.006. Owing to the high temperature at which the waste in injected, the waste is less dense than the native groundwater. Based on the relative density of water, the estimated density of the waste at 80°C is around 0.98 g/ml; native fluids have a density greater than 1.003 g/ml at normal aquifer temperatures.

Chemical analyses of native fluids from the injection zone and from the overlying upper part of the Floridan aquifer are presented in Table 2. Chloride content is about 1,000 mg/l in the upper Floridan aquifer and more than 15,000 mg/l in the lower Floridan aquifer (Fig. 2). Unfortunately, the chemical analyses of native fluids made before waste injection included only a very few parameters, making interpretation of subsequent geochemical reactions difficult. However, the native fluids are basically of sodium chloride type, but include appreciable quantities of sulfate, magnesium, and calcium. The chemistry and potential uses of these brackish to saline artesian waters in southeast Florida are discussed by Vernon (1970) and Meyer (1971).

It is apparent, both from the chemical analyses of samples from the injection and monitor wells (Table 2) and from the average chloride content by zone depicted on Figure 3, that the injection zone was within the transition zone from brackish to saline water before the well was deepened.

Samples from the Experiment Station well, 2 mi southeast of the injection site, reflect the chemical character of native fluid from the upper part of the Floridan aquifer. Comparison of recent (1972) analyses from this well (Table 2) with a partial chemical analysis reported by Stringfield (1933) indicates that there has been no change in the chemistry of native fluids in the upper Floridan aquifer near the well for the last 40 years.

To aid in assessing geochemical reactions, the mineralogical composition of the injection zone and confining beds was determined by X-ray diffraction analysis. Results are presented in Table 3. The interval from 1,495 to 1,600 ft is predominantly calcite, with about 2 percent quartz. The interval below 1,885 ft contains considerable dolomite and also about 2 percent quartz.

Geochemical effects: shallow monitor well and dissolution of aquifer rock--Geochemical effects associated with the upward migration of the hot acidic waste are shown especially well in the shallow monitor well (Table 4). With respect to native fluids, the effects include increased concentrations of calcium, magnesium, organic carbon, COD, and alkalinity; reduction in pH and sulfate concentrations; and generation of considerable hydrogen sulfide (83 to 98 mg/l). Dissolution of the carbonate rocks, anaerobic decomposition, and sulfate reduction within the subsurface environment have also occurred. Dissolved-gas analyses indicate the presence of methane, nitrogen, and carbon dioxide. No geochemical effects had been observed in the Experiment Station well (1,332 ft deep) as of December 1972.

Subtle chemical changes, as indicated by slight increases in calcium concentration and alkalinity, and by a slight decrease in the sulfate-to-chloride ration, suggest that the waste front arrived at the shallow monitor well in February-March 1969, about 27 months after waste injection began.

Samples from the shallow monitor well from March 1971 through March 1972 (Table 4) suggest a relative stabilization of the geochemical reactions, probably caused by the deepening of the injection well and injection of wastes into the deep monitor well during this period.

The percentage change for selected parameters in the shallow monitor well from March 1966 to March 1972 is listed below. The increase in

chloride reflects mixing with more saline water from the underlying injection zone. Significant positive or negative deviations from this value reflect subsurface geochemical interactions.

Parameter	Deviation	Percent
Calcium	+	150
Magnesium	+	85
Alkalinity	+	665
Sulfate	-	42
Chloride	+	10
SO_4/Cl ratio	-	50

The relation between sulfate and chloride for native fluids in southeast Florida and the results of sulfate reduction following waste emplacement are shown in Figure 6. Native fluids from the upper part of the Floridan aquifer are generally richer in sulfate relative to chloride than is the case for normal seawater. A reduction of several hundred milligrams per liter of sulfate is observed as a result of waste emplacement. The sulfate-to-chloride ratio in the shallow monitor well decreased from 0.43 in March 1966 to 0.22 in March 1971 and 1972 (Table 4). Interestingly, the sulfate concentration increased and the sulfate-to-chloride ratio increased to 0.42 in December 1972. The increases are probably associated with the deepening of the injection well and the injection of waste into the deep monitor well during the period October 1971 to January 1972.

A graph of COD and pH in the shallow monitor well from 1966 through early 1973 (Fig. 7) illustrates the time sequence of changes in these parameters. The graphs are based on weekly determinations furnished by the company, and a few analyses by the U.S. Geological Survey. Also shown for comparison are data from the Experiment Station well which reflect natural background values of the aquifer.

Changes in COD and pH since 1969 in the shallow monitor well (Fig. 7) are closely related to waste-injection schedules. The stable pH values from July to November 1970 coincide with seasonal plant shutdown; the decrease and relative stability of COD and pH values after July 1971 coincide with both a seasonal plant shutdown and injection of waste into the deep monitor well. Injection of waste into the deepened injection well commenced in mid-January 1972. Data collected through 1972 are insufficient to determine whether the modifications will prevent eventual upward migration of the injected waste. During July and November-December 1972, pH values were among the lowest reported; and the analysis for December 1972 indicated increased color, COD, and suspended solids.

Dissolution of the carbonate rocks is shown by comparing a borehole caliper log of the 100-ft section of the injection well just below the casing bottom (Fig. 8), made in June 1966, with one made in August 1971 during the modification phase. The logs show considerable enlargement of the borehole and localized channels in the carbonate rocks caused by solution.

Tests for presence of anaerobic sulfate-reducing bacteria--A reduction in sulfate concentration accompanied by increased concentrations of hydrogen sulfide (as much as 98 mg/l; Table 4) suggests the presence of anaerobic sulfate-reducing bacteria within the organic-rich subsurface environment.

To detect and estimate the concentration of these bacteria, a procedure of serial inoculation of water from the monitor wells and the waste into a series of 9-ml sterile vials containing Bacto-Sulfate API Broth (code 0500, Difco Laboratories)[3] was utilized in March 1972. The procedure was carried out in accordance with the American Petroleum Institute Recommended Practice 38 (May 1959, 1st ed.). Following inoculation, the vials were examined daily for 3 weeks for the occurrence of sulfate-reducing bacteria, as indicated by blackening of the broth. Results are given below. The shallow monitor well was positive for sulfate-reducing bacteria on 1.0 ml, 0.1 ml, and 0.01 ml volumes. Thus, bacterial concentrations are estimated to be about 100 cells/ml. The deep monitor well was positive on 1.0 ml only, indicating a concentration of about 1 cell/ml. Samples from both the injected waste and the Experiment Station well were negative. The latter sample serves as a control, since it represents the natural subsurface background in the upper Floridan aquifer within the study area.

	Volume of Sample (ml)					
Sampling Site	1.0	0.1	0.01	0.001	0.0001	0.00001
Industrial waste	-	-	-	-	-	-
Deep monitor well	+	-	-	-	-	-
Shallow monitor well	+	+	+	-	-	-
Experiment Station well	-	-	-	-	-	-

(+ Present, - Absent)

Geochemical effects: deep monitor well--The deep monitor well was used for waste injection from October 8, 1971, to January 12, 1972, while the injection well was being deepened. The COD of the waste during this period ranged from 9,330 to 23,690 mg/l, and the pH ranged from 2.8 to 3.5. After cessation of waste injection into the monitor well, the injected wastes were allowed to backflow continuously at about 2 to 3 gpm. Because of the low ratio of volume of waste withdrawn to volume of waste injected (less than 1:100), it was felt that continuous and/or periodic sampling of this fluid might provide significant information on (1) geochemical interactions of the liquid waste, native fluid, and aquifer rock, and (2) anaerobic degradation and ultimate fate of organic wastes with residence time within a saline carbonate aquifer.

A graph of the COD and pH of fluid backflowing from the deep monitor well for the period January 13, 1972, through December 1972 (Fig. 9) illustrates the time sequence of changes in these parameters. The graph is based on weekly determinations furnished by the company. As illustrated, the waste is apparently partly neutralized to a pH of 5.5 within the subsurface environment almost immediately. If minor fluctuations are ignored, the trends in the data suggest partial neutralization and decomposition of the waste with residence time. The increase in COD concentration during August 1972 may be due to arrival of the waste front from the injection well, where waste injection recommenced on January 13, 1972.

To explore this possibility further, the COD and pH of the injected waste were plotted for the period January 19 through August 9, 1972 (Fig. 10). No waste was injected from August 14 through October 31, 1972, as the plant was shut down. Comparison of Figures 9 and 10 shows some similarities between the two COD plots. The COD of the backflushed fluid (Fig. 9) appears to be a somewhat elongated and diluted version of the COD of the injected waste (Fig. 10). The backflushed fluid thus reflects, with at most a 2-3-month lag, the chemistry of the injected waste after mixing and dilution with native fluids and movement between the wells through 1,000 ft of solution channels in limestone.

The analyses of fluids from within the injection zone after residence times of 75 days (deep monitor well) and 160 days (injection well) are presented in Table 5. The injection well was sampled during the deepening phase. The fluid withdrawn from the injection well reflects a mixture of waste with a more saline native fluid from a deeper interval than that withdrawn from the deep monitor well.

The sample collected after 75 days residence time has been "normalized" to the composition of native aquifer fluids, using chloride as a conservative parameter. A conservative parameter is one which does not undergo

[3]The use of named products in this report is for identification only and does not imply endorsement by the U.S. Geological Survey.

any subsequent geochemical or biochemical reactions. The actual composition can then be compared with the normalized composition. Significant positive or negative deviations from the normalized composition indicate geochemical reactions and/or anaerobic degradation of the waste. The results are as follows (analysis in mg/l except as indicated):

	Normalized Composition	Actual Composition	Percent Change
Chloride	1,300	1,300	0
Alkalinity as $CaCO_3$	100	3,920	+ 3,800
Calcium	120	1,100	+ 820
Magnesium	125	700	+ 460
Silica	18	58	+ 220
Sulfate	420	228	- 45
Sulfate/Chloride ratio	0.32	0.18	- 44
Hydrogen sulfide (H_2S)	4	68	+ 1,600

Major increases occur in alkalinity (associated with the presence of weak acids in the injected waste) and in hydrogen sulfide, the latter due to sulfate reduction. Increases in calcium and magnesium are attributed to dissolution of carbonates within the injection zone.

Dissolved-gas analyses--Dissolved-gas samples were collected from the shallow monitor well and deep monitor well in October 1971 and March 1972, respectively. The samples were collected utilizing a Hamilton gas-tight syringe (#1020 LL, 20 cc capacity) combined with a Becton-Dickenson spring-loaded stopcock (#MS09, one-way male to female leur lock). The analyses were performed by John Rapp, U.S. Geological Survey, Menlo Park, California.

Although some air contamination was present in the sample from the shallow monitor well, the analysis indicated the presence of methane, nitrogen, and carbon dioxide. Results were much better (minimal air contamination) for the sample from the deep monitor well. This syringe was shipped inside a plastic container filled with native fluid from the well. Analysis of the sample collected from the deep monitor well on March 27, 1972, is given below. The gas sample was analyzed on two columns (Porapak Q at 20°C and LMS-13X at 21°C) and in duplicate. Results are in volume percent.

O_2+Ar	N_2	CH_4*	CO_2
0.20	2.1	23.2P, 23.7L	74.1
0.28	2.4	23.4P, 23.5L	74.4

*P=Porapak Q; L=Molecular Sieve 13X.

Discussion

A common misconception is that wastes injected underground do not decompose but remain in their original toxic state because of the lack of oxygen and the reduction in quantities of micro-organisms, which are needed to break down organic material. Decomposition may, however, proceed at a slower rate within the subsurface than at land surface.

Within the saline subsurface environment at the Belle Glade injection system, the oxidation (and thus decomposition) of organic matter continues by means of anaerobic bacterial processes in which sulfate and, perhaps, carbon dioxide serve as the energy source. The relatively high temperature of the waste contributes to increased bacterial metabolic activity, and the nutrient-rich, organic-rich waste is favorable for bacterial growth. One of the major changes in the composition of the saline water within the zone of waste emplacement is the appearance of high concentrations of hydrogen sulfide. The principal source of sulfide sulfur is the reduction of the sulfate present in the native groundwater.

If a carbon:nitrogen:phosphorus ratio of organic matter equal to 106:16:1 by atoms is assumed, a stoichiometric model for anaerobic organic decomposition by sulfate reduction is as follows (Richards, 1965):

$$(CH_2O)_{106}(NH_3)_{16}H_3PO_4 + 53\,SO_4^{-2}$$
$$53\,CO_2 + 53\,HCO_3^- + 53\,HS^- + 53\,H_2O + 16\,NH_3 + H_3PO_4.$$

Another step occurs when CO_2 acts as the energy source for the biochemical oxidation of organic matter, resulting in the formation of methane. Both processes are apparently occurring at the Belle Glade site.

Although it seems clear that anaerobic decomposition of the high-temperature, organic, nutrient-rich waste is taking place within the subsurface environment beneath the Belle Glade site, results of investigations to date are only qualitative. The lack of comprehensive chemical analyses of the native fluids from several zones before waste injection restricts quantitative interpretation. For example, the relative significance to be attached to mixing and dilution versus anaerobic decomposition as a presumed cause of decreases in waste concentration is uncertain.

The data indicate upward movement of the waste into the upper Floridan aquifer within about 27 months following the commencement of waste injection, and also lateral movement within the injection zone. However, the areal extent of the zone of contamination is unknown. No geochemical effects have been observed in a monitor well open to the upper Floridan aquifer and located about 2 mi southeast of the injection site.

Also, the spatial position of the waste front within the injection zone is unknown. The data indicate that the injected waste has long since moved beyond the one deep monitor well 1,000 ft southeast of the injection well. Whether the waste is moving predominantly southeast or in some other direction is uncertain. In view of a proposed hypothesis suggesting inland and upward circulation as part of a regional, geothermally heated, convective flow cycle (Kohout, 1965, 1967; Henry and Kohout, 1972), the possibility of inland movement cannot be discounted without adequate data to the contrary.

The lithology of the injection zone as reported by Vernon (1970) is confirmed by X-ray diffraction analysis. Thus, the confining beds above and below the injection zone are not capable of resisting acid attack; therefore, the legitimate question remains as to whether or not the hot acid wastes can be restricted to the injection zone. This question cannot be decided at this time. However, it is important that the question be asked and that adequate research be carried out to answer it.

A careful and conscientious, but limited, monitoring program has been conducted by the company since inception of waste injection; however, a more detailed and expanded monitoring program is needed. Additional monitor wells in both the injection zone and the upper Floridan aquifer, positioned both upgradient and farther downgradient, appear necessary.

These additional monitor wells would aid in (1) defining the spatial extent of the waste within the aquifer system, and the direction and rate of waste movement; (2) documenting liquid waste - aquifer rock - native fluid interaction in proximity to the waste front; (3) evaluating the potential degree of treatment and upgrading of the waste as it moves through the saline carbonate environment (ultimate fate of the waste); and (4) assessing the possibility of upward movement of the waste into the upper Floridan aquifer in areas other than that in immediate proximity to the injection site.

SUMMARY AND CONCLUSIONS

A section of early Tertiary carbonate rocks containing highly permeable zones filled with saline water, including extensive cavity systems, underlies southern peninsular Florida at depths ranging from about 1,500 to 4,500 ft. This section is capable of yielding or accepting large quan-

tities of fluids and is being used for storage of liquid industrial and domestic waste at several places in south Florida, one of which is Belle Glade.

The fluids being injected in the Belle Glade system include the effluent from a sugar mill and waste from the production of furfural, an aldehyde processed from sugar cane bagasse. The waste is hot (71-103°C), acidic (pH 2.6-4.5), and highly organic (COD 6,000-26,000 mg/l). Since 1966, more than 800 million gal of this waste has been injected into saline water in the lower Floridan aquifer. Injection rates range from 400 to 800 gpm, under injection pressures of 30-60 psi. Operation of the system is seasonal. Injection is more or less continuous from late fall through midsummer; the system is inactive during late summer.

Sampling of injected waste fluids allowed to backflow from an injection well after injection was stopped has provided useful information on geochemical interactions of the waste, native fluid, and aquifer rock. Partial neutralization of the waste to a pH of 5.5 by dissolution of limestone occurs almost immediately upon contact with the carbonate of the receiving zone. The localized patterns of dissolution of the carbonate rock are shown as enlargements of the borehole on caliper logs. Anaerobic decomposition of the organic waste occurs in the injection zone, as indicated by the presence of hydrogen sulfide, methane, carbon dioxide, and nitrogen gases.

The data show that the waste has moved both upward and laterally within the aquifer system. Upward movement of the waste through the carbonate confining beds into an overlying brackish-water aquifer occurred about 27 months after injection began. Upward movement of the waste was detected in a 1,400-ft-deep monitor well 75 ft from the injection well. Lateral movement was detected in a 2,067-ft-deep monitor well in the injection zone 1,000 ft from the injection well. No pressure effects were evident in these two monitor wells, and no geochemical changes nor pressure effects have been detected in a third monitor well which is 1,332 ft deep and open to the upper Floridan aquifer, and which is about 2 mi southeast of the injection site.

Because of upward migration of the waste, the injection well was drilled 503 ft deeper and cased 443 ft deeper in the fall of 1971. Data collected since the injection well was deepened are insufficient to determine the effectiveness of the changes in restricting the waste to the injection zone.

Observed geochemical effects associated with the movement of waste include increased concentrations of calcium, magnesium, organic carbon, COD, and alkalinity; reduction in pH and in sulfate concentration; and generation of considerable quantities of hydrogen sulfide (83 to 98 mg/l). Dissolution of the carbonate aquifer, anaerobic decomposition, and sulfate reduction within the subsurface environment have occurred. The presence of sulfate-reducing bacteria within the organic-rich subsurface environment was confirmed.

The nearly pure character of the carbonate rock is confirmed by X-ray diffraction analysis. Thus, the confining beds above and below the injection zone are not capable of resisting acid attack.

The problem of waste disposal is a difficult one, and much can be learned from the data obtained at the industrial deep-well waste-injection system at Belle Glade. Investigations clearly point up the value of a sound monitoring program. Parameters that seem to be of particular significance in evaluating the Belle Glade waste-injection system are the temperature, acidity, chemical oxygen demand, and density of the waste, and the carbonate lithology of the injection zone and confining layers.

A concerted effort by both industry and government, working together in spirit of harmony and cooperation, is required to provide the scientific information necessary to understand the subsurface environment and its response to waste injection.

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Table 1. Chemical Analyses--Injected Industrial Waste, Belle Glade, Florida[1]

Parameter	10-20-71	3-28-72
Acidity (me/1)[2]	137	36
pH (Units)	3.2	4.3
Calcium	114	30
Silica (SiO_2)	-	14
COD	10,900	13,300
Organic Carbon	9,300	5,670
Color (Units)	500	1,000
Organic Nitrogen (N)	72	59
Ammonium (NH_4 as N)	22	0.6
Total Phosphate (as P)	26	8.5
Suspended Solids	2,490	1,880
Fluoride	6.6	6.0
Chloride	114	90
Temperature (°C)	93	88.5
Specific Gravity (Dimensionless)	1.0061	1.0037

[1]Analyses in milligrams per liter except as indicated.

[2]Acidity in milliequivalents per liter.

Table 2. Chemical Analyses--Native-Aquifer Fluids, Belle Glade Area[1]

Parameter	Injection Well Depth 1495-1610 Feet 9-29-66	Deep Monitor Well Depth 1490-2067 Feet 2-14-67[2]	2-5-69[2]	Shallow Monitor Well Depth 648-1400 Feet 3-14-66[2]	5-17-33[3]	Exp't Station Well Depth 957-1332 Feet 10-19-71	3-28-72	12-15-72
Alkalinity (as $CaCO_3$)	-	122	114	118	-	125	136	131
pH (Units)	7.2	6.72	7.55	7.4	-	8.1	8.00	-
Calcium	70	477	113	99	144	130	140	140
Magnesium	129	625	96	97	138	140	140	140
Silica (SiO_2)	-	-	-	-	-	19	12	8.7
COD	-	-	25	-	-	-	6	36
Organic Carbon	-	-	-	-	-	2	0	-
Color (Units)	0	2	0	1	-	0	10	5
Organic Nitrogen (N)	-	-	-	-	-	0.39	0.05	0.04
Ammonium (NH_4 as N)	-	-	-	-	-	0.77	0.85	0.65
Total Phosphate (as P)	-	-	-	-	-	0.01	0.02	0.002
Fluoride	0.7	-	-	-	-	0.9	1.2	1.0
Temperature (°C)	26.5	-	-	-	-	-	24.5	26.0
Specific Gravity (Dimensionless)	-	-	-	-	-	-	1.0035	1.008
Sulfate	440	1,500	365	364	516	522	510	540
Chloride	1,650	10,350	950	855	1,650	1,620	1,600	1,680
Hydrogen Sulfide (H_2S)	-	-	-	-	-	-	3.8	-
SO_4/Cl Ratio	0.27	0.14	0.38	0.43	0.31	0.32	0.32	0.32

[1]Analyses in milligrams per liter except as indicated.

[2]Furnished courtesy of Black, Crow and Eidsness, Inc., Gainesville, Fla. 2-14-67, High Flow, 2,000 gpm; 2-5-69, Low Flow, 2 gpm.

[3]Stringfield (1933).

Table 3. X-Ray Diffraction Analysis

Sugar Cane Growers Coop. Well No. 2; Sec. 28, T43S, R37E, Palm Beach County, Florida
(USGS No. 264200N0803900.2). Analyses Performed November 1971[1]

Sample Number	Depth, Feet	Hydrologic Unit	Mineral Composition (percentages are approximate)
72FLA70	1495-1600	Avon Park Limestone (Upper Floridan Aquifer)	Calcite 85-90, dolomite 1, quartz 2, a very small amount of clay minerals is present, possibly montmorillonite.
72FLA71	1885-1925	Lake City Limestone (Base of Upper Floridan Aquifer?)	Dolomite 55-60, calcite 35-40, quartz 2.
72FLA72	1985-2080	Lake City Limestone (Base of Upper Floridan Aquifer?)	Calcite 55, dolomite 35, no other minerals appear to be present.

[1]U. S. Geological Survey, Denver, Colorado.

Table 4. Chemical Analyses--Shallow Monitor Well, Belle Glade, Florida[1]

	3-14-66[2]	2-5-69[2]	3-4-69[2]	3-18-71	10-20-71	3-28-72	9-28-72	12-15-72
Alkalinity (as $CaCO_3$)	118	132	157	902	898	902	845	853
pH (Units)	7.4	7.85	8.00	6.61	6.61	6.47	-	6.5
Calcium	99	105	118	258	252	250	270	260
Magnesium	97	92	101	186	182	180	190	170
Silica	-	-	-	21	20	23	20	18
COD	-	20	-	698	461	486	-	1,100
Organic Carbon	-	-	-	475	480	365	280	334
Color (Units)	1	0	8	50	60	20	40	120
Sulfate	364	350	363	186	218	210	310	380
Chloride	855	845	1,025	830	870	940	860	900
H_2S	-	-	-	85	98	83	-	-
SO_4/Cl Ratio	0.43	0.41	0.35	0.22	0.25	0.22	0.36	0.42
Suspended Solids	-	-	-	-	0	-	-	126

[1]Analyses in milligrams per liter except as indicated.

[2]Furnished courtesy of Black, Crow and Eidsness, Inc., Gainesville, Fla.

Table 5. Chemical Analyses--Injection-Zone Fluids Following Waste Emplacement, Belle Glade, Florida[1]

Parameter	Deep Monitor Well Depth 1490-2067 Feet 3-27-72 Residence Time = 75 days	Injection Well Depth 1938-2241 Feet 1-6-72 Residence Time ~160 days
Alkalinity (as $CaCO_3$)	3,920	3,477
pH (Units)	6.2	6.6
Calcium	1,100	1,100
Magnesium	700	700
Silica (SiO_2)	58	68
COD	6,610	4,166
Organic Carbon	3,870	2,430
Color (Units)	900	480
Organic Nitrogen (N)	7.9	3.9
Ammonium (NH_4 as N)	11	13
Total Phosphate (as P)	8	4
Fluoride	3.3	2.9
Sulfate	228	452
Chloride	1,300	8,000
Hydrogen Sulfide (H_2S)	68	79
SO_4/Cl Ratio	0.18	0.06
Temperature (°C)	40	28.8
Specific Gravity (Dimensionless)	1.0070	-
Eh (millivolts)	- 266	-

[1]Analyses in milligrams per liter except as indicated.

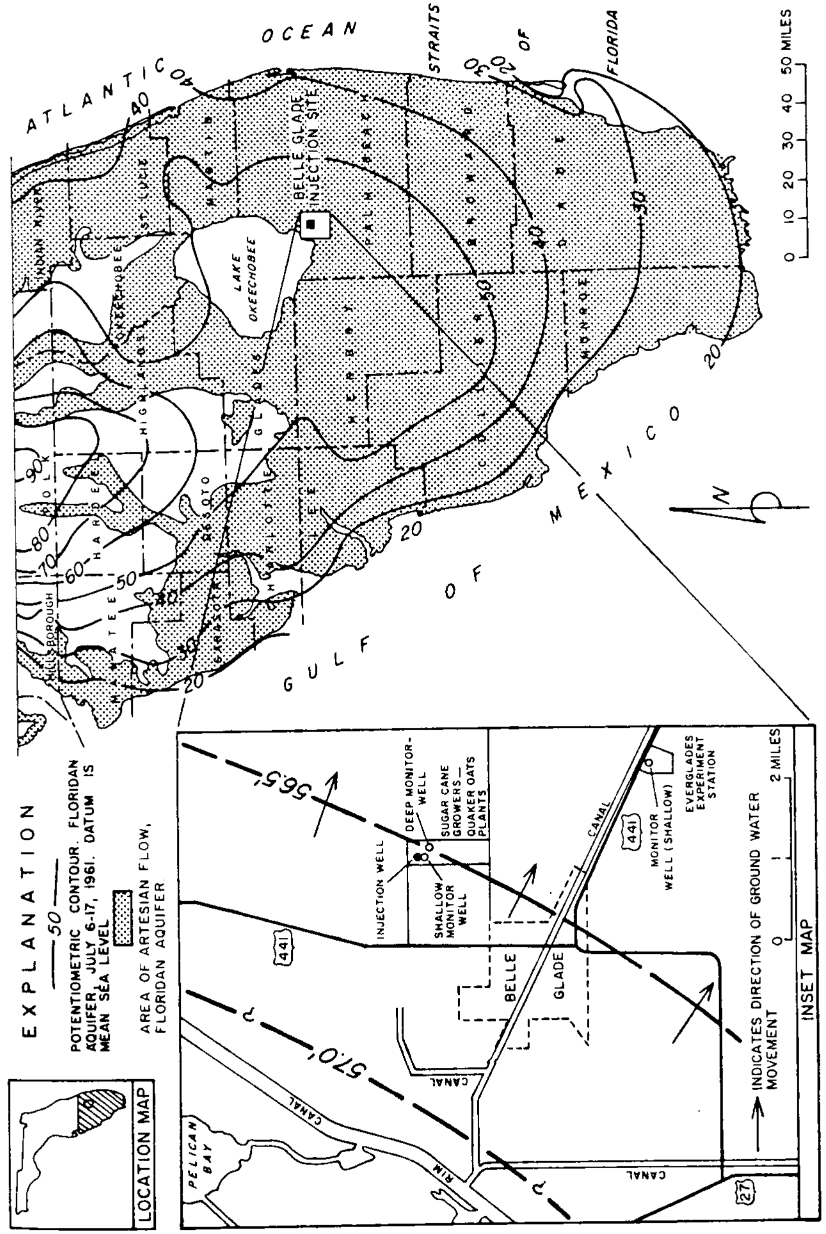

FIG. 1--Index map of Belle Glade area and potentiometric-surface map of Floridan aquifer in South Florida (after Healy, 19

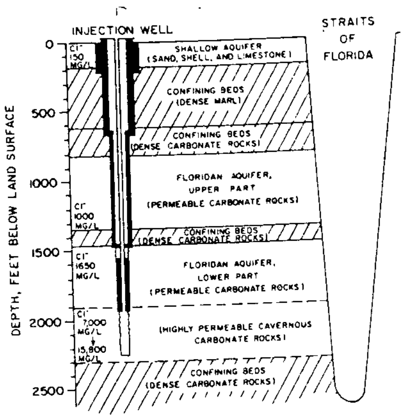

FIG. 2--Schematic hydrogeologic section between Belle Glade area and Straits of Florida (partial data sources include Garcia-Bengochea and Vernon, 1970; Kaufman, 1973).

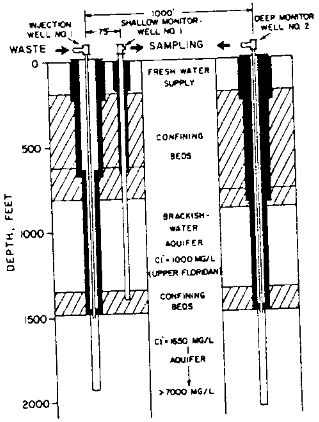

FIG. 3--Diagram of industrial waste injection and monitoring system, prior to deepening injection well (after Garcia-Bengochea and Vernon, 1970).

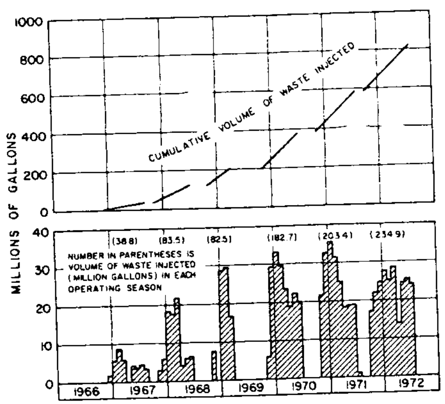

FIG. 4--Volume of waste injected versus time.

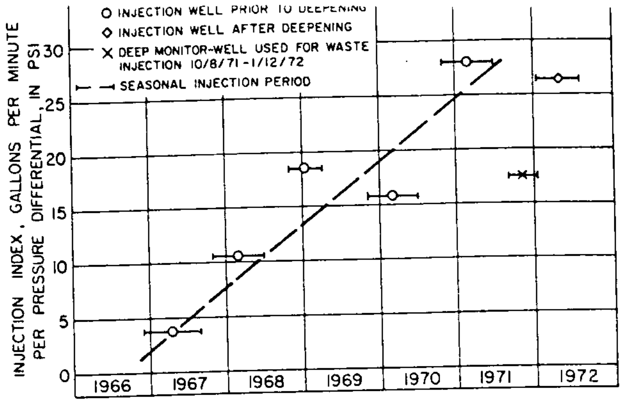

FIG. 5--Average injection index versus time.

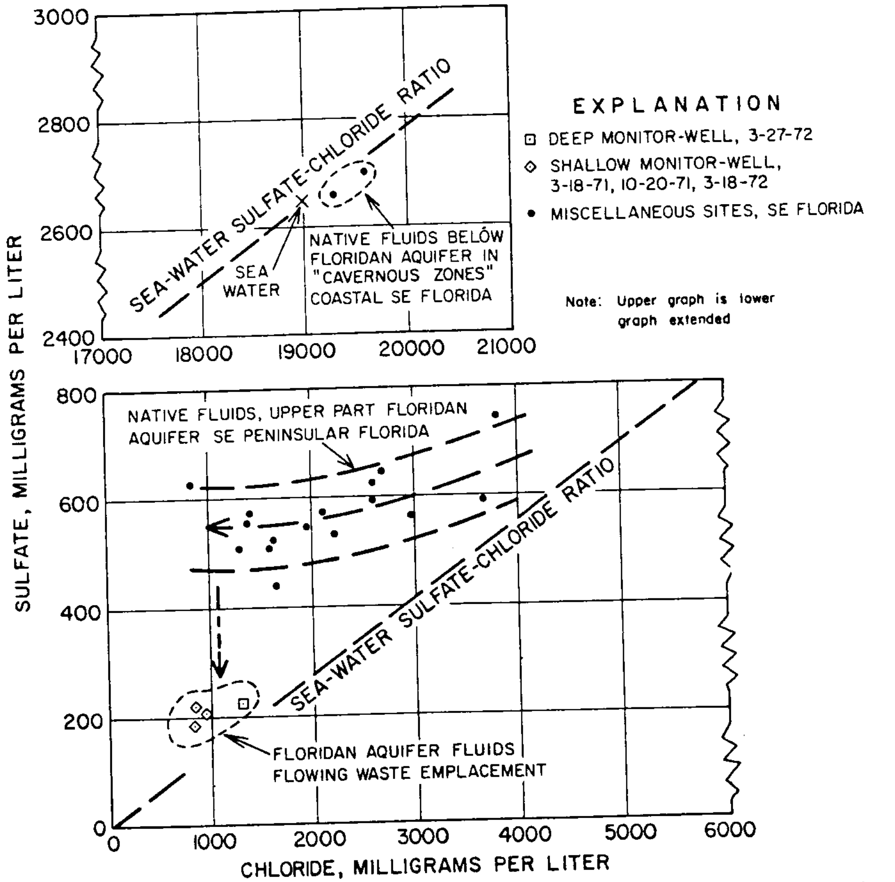

FIG. 6--Sulfate-chloride relationship for native fluids and sulfate reduction resulting from waste emplacement.

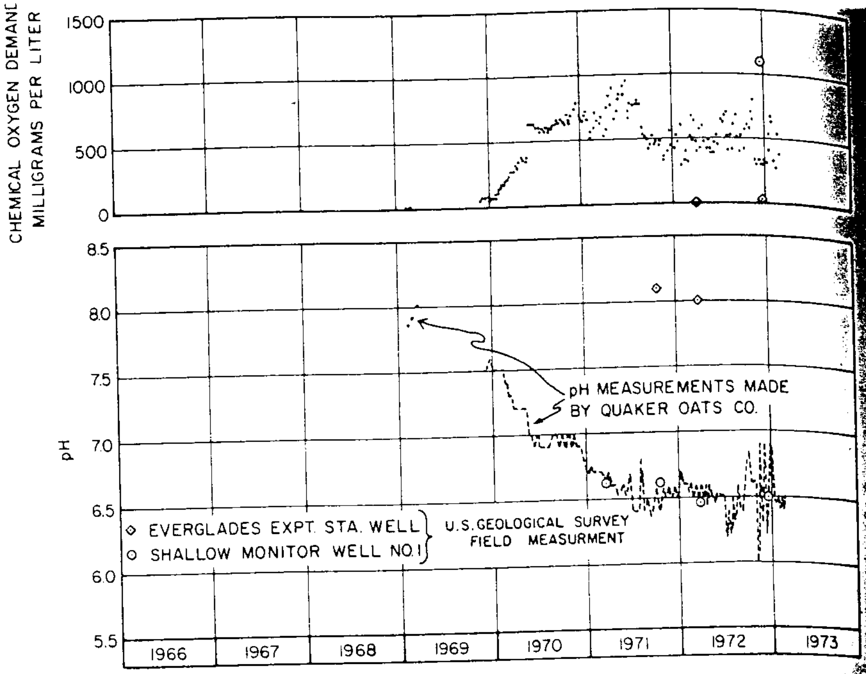

FIG. 7--COD and pH versus time, shallow monitor well.

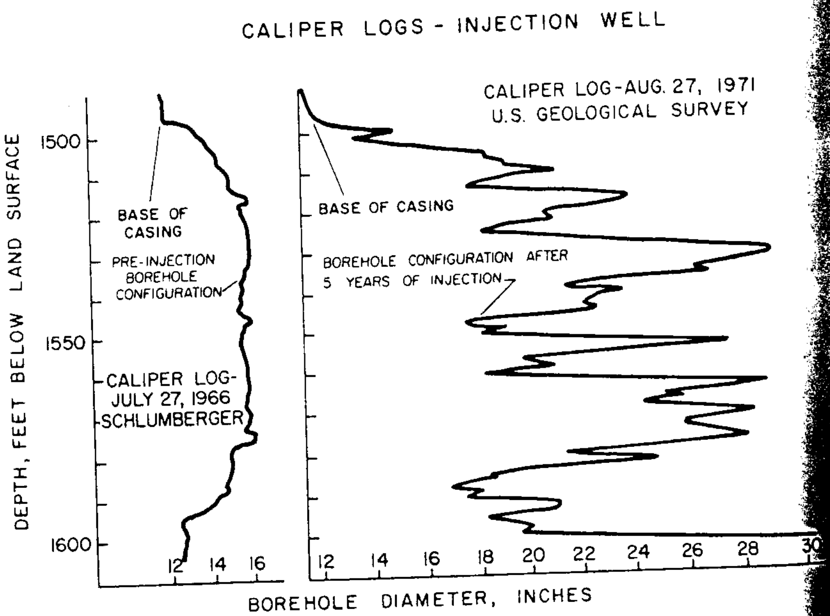

FIG. 8--Caliper logs of upper 100 ft of open-hole section of injection well (after Kaufman, 1973).

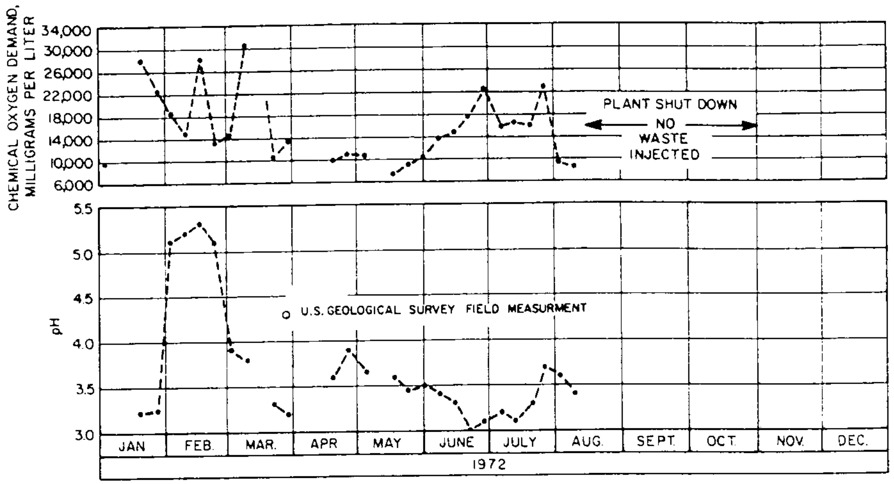

FIG. 9--COD and pH of backflushed fluid versus time for deep monitor well.

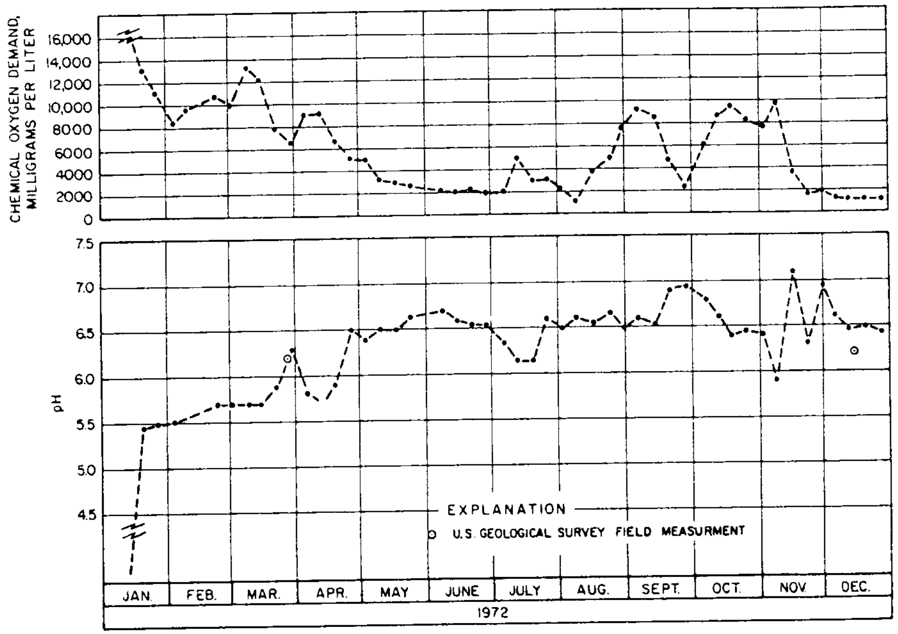

FIG. 10--COD and pH of waste versus time, before injection of waste into well.

HYDROLOGIC EVALUATION OF INDUSTRIAL-WASTE INJECTION AT MULBERRY, FLORIDA[1]

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ABSTRACT Florida's deepest waste-injection well, completed in 1972 at a chemical plant at Mulberry, will inject acid industrial waste into carbonate rock. The plant produces sodium fluosilicate from the reaction of sodium chloride with fluosilicic acid, a byproduce of nearby phosphate-processing plants. The resulting liquid waste, which is to be injected into the subsurface, has a high chloride content and a pH that is generally less than 2, and at times less than 1.

The cased injection well is finished as a 6 1/4-in. open hole in dolomite and limestone, from 4,040 to 4,984 ft below land surface. The injection-well annulus has two monitor wells, one open near the base of the Floridan aquifer from 1,254 to 1,264 ft, and the other open to the saline-water aquifer below the Floridan aquifer, from 2,755 to 2,788 ft. Before injection tests were made, geophysical logs and a tracer test were run on the well. The radioactive-tracer test indicated that several permeable zones are exposed in the open hole.

The native fluids, sampled prior to injection, and the waste fluid are markedly different in density, chloride content, and temperature.

Two injection tests were run in the fall of 1972 using the waste fluid. The injection rates during the first test were inadequately controlled. During the second test, waste fluid was injected for 118 hours

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The writers thank personnel of Kaiser Aluminum and Chemical Corporation, particularly S. F. Crawford and D. Baxter, for their cooperation and assistance in the collection of data. R. J. Funk, Subsurface Disposal Corporation, generously provided time and information during drilling and logging operations. G. Winston, Florida Bureau of Geology, examined well cuttings and provided a preliminary geologic log.

at 270 gpm and pressure readjustment after shutdown was observed for 5 days. Injection pressure measured at the surface increased for the first 31 hours, and then decreased for the remainder for the injection period. The decrease in pressure after 31 hours is attributed to the net effect of reactions that occurred in the injection zone during the tests. Dissolution of limestone by the low-pH waste substantially increased the permeability of the rock adjacent to the well bore and increased the density and temperature of the injection fluid as it moved into the injection zone. These changes, coupled with the unavailability of direct bottom-hole pressure measurements in the injection well and the lack of water-level measurements in an observation well finished in the injection zone, complicated the evaluation of test results. As a result, only a general estimate of injection-zone transmissivity (less than 800 sq ft/day) could be made.

Where acid wastes are to be injected into carbonate rocks, evaluation of aquifer coefficients would be facilitated by the use of a fluid other than a reactive waste as the injection fluid during initial tests, and by measuring bottom-hole pressures in the injection well and water levels in observation wells open to the injection zone.

INTRODUCTION

Florida's deepest waste-injection well was completed in 1972 at the Kaiser Aluminum and Chemical Corporation plant at Mulberry, about 30 mi east of Tampa (Fig. 1). As of 1972, the well was one of seven deep-well disposal systems in the State and was the first to be completed in the west-central part of the peninsula. When operational, the well will inject acidic, high-chloride industrial wastes into a carbonate zone about 4,000-5,000 ft below the land surface.

The U.S. Geological Survey participated in the collection and evaluation of data during the initial injection tests at the request of the Florida Department of Pollution Control. Evaluation of the results indicates some of the difficulties associated with the assessment of hydrologic conditions when acid wastes are injected into a carbonate environment, especially where monitoring facilities are limited. The study is part of the Geological Survey's research program to evaluate the effects of underground waste-injection on the nation's subsurface environment.

HYDROGEOLOGIC FRAMEWORK

The injection well at Mulberry penetrates nearly 5,000 ft of sedimentary rocks, chiefly limestone and dolomite, ranging in age from Late Cretaceous to Holocene (Fig. 2). The log shown in Figure 2 is based on a preliminary examination of well cuttings from the injection well by the Florida Bureau of Geology; lithologic descriptions and the positions of formation contacts may be subject to revision.

Freshwater supplies in the area are obtained principally from the Floridan aquifer. This aquifer is present throughout Florida and parts of other southeastern states and yields abundant supplies for industrial, agricultural, and municipal uses. In Polk County, groundwater withdrawals for these purposes averaged about 420 million gal/day in 1970 (Pride, 1973). As a result of long-term withdrawals of similar magnitudes, the potentiometric surface of the aquifer declined more than 40 ft in Polk County from 1949 to 1969 (Stewart et al., 1971).

At Mulberry, the Floridan aquifer consists of about 1,300 ft of limestone and dolomite (Fig. 2). As in most of Polk County, the uppermost unit of the aquifer is the Suwannee Limestone, and the aquifer base coincides with the base of the Avon Park Limestone (Stewart, 1966). At the well site, this base is about 1,545 ft below land surface. The underlying Lake City Limestone is chiefly dolomite containing 1-5 percent of intergranular anhydrite and gypsum. The Oldsmar Limestone and the upper part of the Cedar Keys Limestone, designated the saline-water aquifer, consist of limestone and dolomite with as much as 20 percent porosity.

The middle part of the Cedar Keys Limestone, 780 ft thick, consists of alternating beds of anhydrite and dolomite. Impermeable anhydrite makes up about 45 percent of the unit; the beds are mostly 10-50 ft thick, but the upper anhydrite bed is 135 ft thick. The interbedded dolomite has low permeability and a porosity of about 5-10 percent. This anhydrite-dolomite section has been recognized in oil test wells in central Florida and apparently is regionally extensive.

The lower part of the Cedar Keys Limestone of Paleocene age and the underlying Lawson Limestone of Late Cretaceous age and beds of Tayloran age, more than 880 ft thick, consist of poorly to moderately indurated vuggy dolomite and chalky limestone containing brine. Examination of well cuttings and analyses of sidewall cores indicate that this section generally has low permeability. Permeabilities of 8 sidewall cores of the Cretaceous rocks, taken from 4,500 to 4,950 ft below land surface, ranged from 5.0 to 28 md; the hydraulic conductivities ranged from 0.021 to 0.1

ft/day. Average values were 13 md, or 0.055 ft/day. However, radioactive-tracer tests indicate the presence of several zones with relatively high permeability; the most permeable of these is near the base of the lower part of the Cedar Keys Limestone, from about 4,340 to about 4,480 ft below land surface. The porosity of this section ranges from 5 to 10 percent.

The permeable zones of the lower part of the Cedar Keys and underlying units constitute potentially usable deep injection zones. Under undisturbed conditions, or before pumping or injection, fluid in this section was effectively confined by overlying low-permeability dolomite and anhydrite in the lower and middle parts of the Cedar Keys Limestone.

Head Relationships

Substantial differences exist among the hydraulic heads in the Floridan aquifer, saline-water aquifer, and injection zone. Measured heads in the lower part of the Floridan aquifer and in the saline-water aquifer before injection testing were about 76.5 and 60 ft below land surface, respectively. These water levels are equivalent to freshwater heads of 72 and 46 ft below land surface. In the 1950's, the head in the Floridan aquifer was about 32 ft below land surface, and the local vertical gradient in the upper 3,000 ft of rock was downward. Since that time, large-scale withdrawals of water from the Floridan aquifer for industrial and agricultural uses have resulted in a lowered head and a reversal in gradient between the Floridan aquifer and the saline-water aquifer. Before injection was started, the static level of the brine in the injection zone was about 125 ft below land surface, or equivalent to a freshwater head of about 150 ft above land surface. The injection zone is possibly hydraulically isolated from the overlying flow system, and little regional circulation of water occurs under natural gradients at this depth.

INJECTION FACILITIES

The Kaiser plant at Mulberry produces sodium fluosilicate from the reaction of sodium chloride with fluosilicic acid, a byproduct of nearby phosphate-processing plants. Prior to drilling of the injection well, this acidic, high-chloride waste was discharged into a 30-acre lake, where it overflowed and seeped into nearby streams.

When the injection well goes into operation, the plant effluent will flow by gravity to a lined storage pond from which it will be transferred to a 40,000-gal rubber-lined surge tank. From there the waste will be pumped into the well. Average injection rate is expected to be about 250 gpm (gallons per minute), with an anticipated range of 150 to 300 gpm.

The injection well is cased and grouted to 4,040 ft below land surface, and is a 6 1/4-in. open hole from 4,040 to 4,984 ft (Fig. 2). The 7 5/8-in. casing does not extend through the full thickness of the anhydrite-dolomite confining beds. One of the anhydrite beds is exposed in the upper part of the open hole, from 4,084 to 4,104 ft below land surface. A 4 1/2-in. fiberglass injection tubing extends 411 ft into the open hole; the bottom 30 ft of the tubing is slotted. The 140-ft zone of relatively high permeability in the lower part of the Cedar Keys Limestone is in the upper third of the open-hole section. About 80 ft of this zone is above the top of the slotted section of injection tubing.

The annulus between the fiberglass injection tubing and the steel casing is sealed off from the open hole by a noncorrodible packer assembly. The annulus contains sensors to monitor changes in pressure and conductivity that might reflect leaks in the casing, packer, or injection tubing.

Two monitor wells were installed in the grouted part of the injection-well annulus between the 10 3/4-in. casing and the rock wall. The wells consist of 2 3/8-in. tubing and are open to the surrounding rock formations by perforations that extend through the tubings and grout (Fig. 2). The shallower one is open to the lower part of the Floridan aquifer, from 1,254 to 1,264 ft, and the deeper one is open to the saline-water aquifer from 2,775 to 2,788 ft. The wells are equipped for collecting water samples and measuring water levels.

WATER QUALITY

Chemical analyses of samples from the two monitor wells, the injection zone, and the waste effluent are summarized in Table 1. The analyses show that groundwater increases in mineralization in the three successively deeper zones sampled. On the basis of dissolved-solids content (Hem, 1970, p. 219), the waters are classified as moderately saline in the basal part of the Avon Park Limestone and in the Oldsmar Limestone, and as brine in the injection zone. The waters also increase in temperature and density with depth, and water in the injection zone contains substantially higher concentrations of trace metals than those in the two shallower zones.

The waste effluent contains hydrochloric acid and sodium chloride and has a very low pH. In the sample analyzed, the pH was 1.5 (Table 1), and values less than 1.0 have been measured. Chloride content was 66,000 mg/l, similar to that of the native fluid in the injection zone (Table 1). Compared to the native fluid, the waste sample contained higher concentra-

tions of fluoride, arsenic, chromium, copper, mercury, nickel, and zinc. The waste effluent is cooler and less dense than the native fluid. These differences in temperature and density were considered in analyzing the response of the injection zone during injection tests.

The highly acid waste will be neutralized by reacting with the calcium carbonate in the injection zone. Under temperatures and pressures of the system, the carbon dioxide produced by the reaction is expected to remain in solution. Solution of calcium sulfate will also occur wherever the acid waste comes in contact with anhydrite; the 20-ft bed exposed in the open hole is directly subject to this solution activity.

INJECTION TESTS

Two tests were conducted on the injection well during the period from August 29 to October 14, 1972. In both tests, the injection fluid was waste effluent containing 4 percent by weight of hydrochloric acid. Pressure at the well head and water levels in the two monitor wells were observed during injection, and the decline in fluid level in the injection well was measured after injection ceased. The change in fluid level (partially adjusted for temperature and density differences) with time was used with the nonequilibrium-type curve of Theis, and Hantush's nonsteady-state-type curve for leaky artesian aquifers, to estimate the transmissivity of the injection zone (Ferris et al., 1962; Walton, 1970).

The first test, a preliminary one, began on August 29 and ended on September 1, 1972. The injection rate averaged 180 gpm for the 53-hour test and ranged from 100 to 300 gpm, with two periods of no injection lasting 2.8 and 0.8 hours. During the last 1.8 hours, injection was maintained at about 320 gpm. Pressure in the injection well at the surface rose to a maximum of 155 psi after 9 minutes (0.2 hours) and then declined. At the maximum pressure, the head of the brine in the injection zone (corrected for friction loss) was 46.9 ft above land surface. During the rest of the test, pressure and head varied markedly with variations in the injection rate. The head in the injection zone at the end of injection was 96.9 ft below land surface. The fluid level declined to 128.4 ft below land surface 12 hours after injection stopped. Measurement of fluid-level decline during the early part of recovery was hampered by foam in the injection well. A fit to the data for the first 200 minutes (3.3 hours) of recovery was obtained with the Theis curve. After 210 minutes (3.5 hours) of recovery, the data began to deviate from the Theis curve, and a fit of the data was obtained with the Hantush curve for r/B = 0.01. The apparent transmissivity of the injection zone using the type-curve match point and the average injection rate was computed to be 800 sq ft/day. The rate of pressure increase during the first few hours of injection indicates that the transmissivity of the injection zone is probably less than 800 sq ft/day and may be as small as 400 sq ft/day.

The second injection test consisted of a 118-hour injection phase and a 76-hour recovery phase ending October 14, 1972. Waste effluent with a pH of 1.5 was injected at a nearly constant rate for 118 hours. During the first 9 minutes of injection, the pressure in the well at the surface increased to 69 psi as the injection rate increased to 280 gpm. At this pressure, the head of the brine in the injection zone (corrected for friction loss) was 67 ft below land surface. The injection rate was adjusted to 270 gpm and remained constant for the rest of the test. The injection pressure declined to 62.5 psi at the surface and the adjusted head dropped to 77 ft below the surface. At the end of 31 hours, the pressure had gradually risen to 65.5 psi and the head to 70.1 ft. From 31 to 112.5 hours the pressure gradually declined to 40.5 psi and the head to 127.7 ft, where they remained until the end of the injection phase of the test.

After 76 hours of shutdown, the fluid level in the injection well had declined (recovered) from 127.7 to 151.7 ft below land surface. A match to the Theis curve was obtained using data from the first 330 minutes (5.5 hours) of recovery (Fig. 3). After about 330 minutes, the data deviate from the Theis curve, and a fit to a Hantush curve was obtained from 330 to 4,560 minutes (5.5 to 76 hours). These data fit the curve r/B = 0.01. The apparent transmissivity computed from this test is 2,000 sq ft/day.

Water-level fluctuations in the monitor wells during both tests showed no direct response to injection. For the second test this lack of response is shown by hydrographs in Figure 4. Semidiurnal fluctuations and broad trends during the tests are chiefly related to changes in barometric pressure; the water level in the shallow monitor well may also be affected by pumping of nearby wells.

HYDROLOGIC IMPLICATIONS OF TEST RESULTS

The different values of transmissivity obtained from the recovery phases of the two tests are attributable principally to the reaction of acid waste on the rock in the injection zone. With each injection test the permeability and porosity of the injection zone in the immediate

vicinity of the well apparently increased significantly. Although the test data were adjusted for temperature and density differences where possible, some of the difference in apparent transmissivity may be caused by undefined changes in conditions between the two tests.

The increase in permeability and porosity probably was concentrated along certain zones and occurred at a nonuniform rate. Although the interval from 4,040 to 4,984 ft in the injection well is completed as open hole, only part of the exposed rock is permeable. Radioactive-tracer tests conducted prior to waste injection indicated that the most permeable zone extends from about 4,340 to about 4,480 ft below land surface. Because most of the waste undoubtedly moved from the well into the rock along this permeable zone, the increase in porosity and permeability was probably also concentrated along this zone. However, it is possible that additional permeable zones were opened to the well by the acid waste. During the second test, the rate of increase of permeability and porosity, as a percent of the original values, was probably greatest from 6 to 80 hours and smallest during the later part of the test.

The injection zone apparently responded as though it were a leaky-layer system, as indicated by the deviation of test data from the Theis curve. Although this response may be attributed partly to the effects of rock dissolution in the injection zone, it also indicates possible leakage of fluid into the overlying and underlying confining layers.

The limited test facilities and complex test conditions at this site complicate the analysis of the data. Because of density differences between the waste and the native fluid in the injection zone, monitoring of pressure at the injection-well head provided only partial information on the changes occurring in the injection zone, and no reliable information on leakage through the confining layers. Using a fluid that reacted with the rock and with the confining layers for the initial testing posed major difficulties in data collection and analysis. Sufficient acid waste was injected during the second test to dissolve more than 5,000 cu ft of carbonate rock and raise the temperature of the injected fluid several degrees Celsius. This temperature increase further complicated the problem of data analysis, particularly because significant temperature and density differences already existed between the native fluid and the waste.

Despite these complications, the injection tests indicated that the permeability and porosity of the carbonate rock in the injection zone were altered by reaction with the acid waste and that there may be some leakage through the confining layers. However, the tests were inadequate to provide reliable estimates of the hydraulic characteristics of the injection zone or a leakage coefficient of the confining beds. The amount of leakage, if any, through confining beds has not been assessed.

The inconclusive results obtained from the analysis of the injection test emphasize the need to include at least one observation well in the injection zone and to obtain bottom-hole pressure measurements in systems designed to place a reactive waste fluid at depth. The results further indicate that leakage through confining beds may occur even at depths as great as 5,000 ft below land surface if a deep hydrologic system is stressed. Initially this leakage probably would represent chiefly movement of displaced native fluid. However, as the cone of influence of the injected fluid spreads, enough waste may leak and react with confining beds to reduce their effectiveness as confining units.

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Table 1. Chemical Analyses of Native and Injection Fluids[1]

	Floridan Aquifer (Shallow monitor)	Saline-Water Aquifer (Deep monitor)	Injection Zone	Waste Effluent
Date of collection	10-4-72	10-4-72	8-29-72	10-10-72
Calcium (Ca)	670	850	4,800	400
Magnesium (Mg)	200	200	1,600	400
Sodium (Na)	26	1,200	39,000	16,000
Potassium (K)	6.0	50	1,200	45
Bicarbonate (HCO_3)	230	144	340	0
Sulfate (SO_4)	1,800	2,400	3,200	55
Chloride (Cl)	220	1,800	72,000	66,000
Fluoride (F)	2.7	3.0	13	1,500
Dissolved solids[2]	3,100	6,600	120,000	84,000
Hardness as $CaCO_3$ (Ca, Mg)	2,500	3,000	19,000	2,700
Alkalinity as $CaCO_3$ total	189	118	279	0
Acidity (H^{+1})	--	--	--	1,530
pH	7.7	7.4	6.6	1.5
Specific conductance (micromhos @ 25°C)	3,440	8,750	131,000	364,000
Temperature (°C)	28.0	28.5	41.0	30.0
Density @ 20°C (grams/milliliter)	1.001	1.003	1.078	1.042
micrograms/liter				
Aluminum (Al), total	0	0	300	0
Arsenic (As)	10	10	10	7,800
Cadmium (Cd)	0	0	210	10
Chromium (Cr), total	0	20	150	230
Cobalt (Co)	1	1	750	7
Copper (Cu)	10	10	110	600
Iron (Fe), total	1,500	3,500	53,000	19,000
Lithium (Li)	20	80	1,700	10
Mercury (Hg), total	0	0	0.3	6.0
Nickel (Ni)	0	0	780	950
Strontium (Sr)	14,000	15,000	120,000	10,000
Zinc (Zn)	10	260	300	680

[1]Analyses made by U.S. Geological Survey. Chemical constituents in milligrams per liter except as noted.

[2]Calculated from determined constituents.

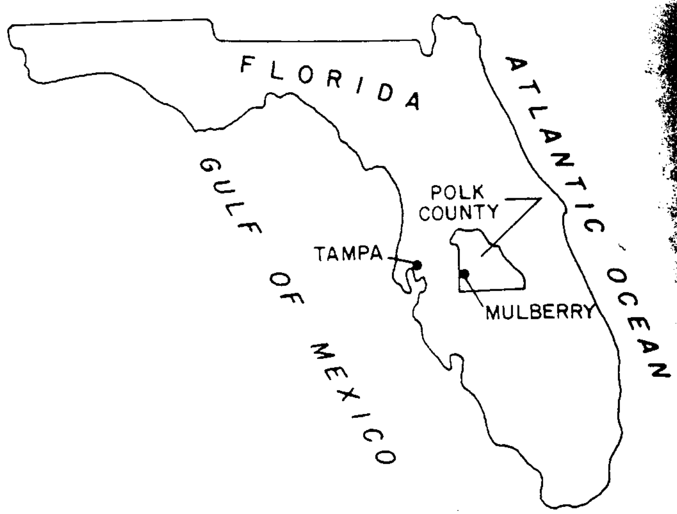

FIG. 1--Location of Mulberry, Florida.

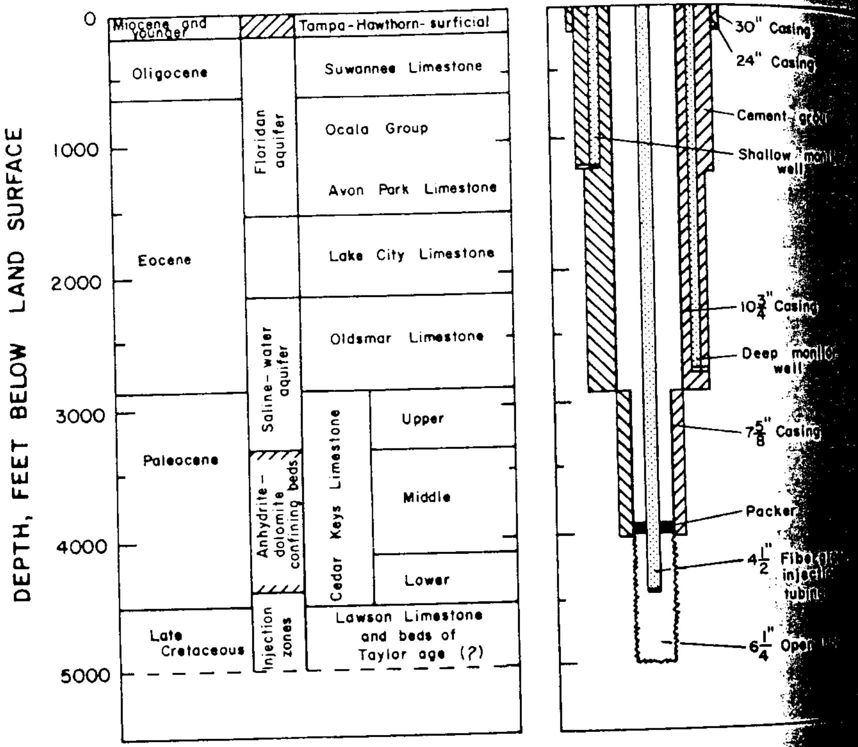

FIG. 2--Geologic column and injection-well construction.

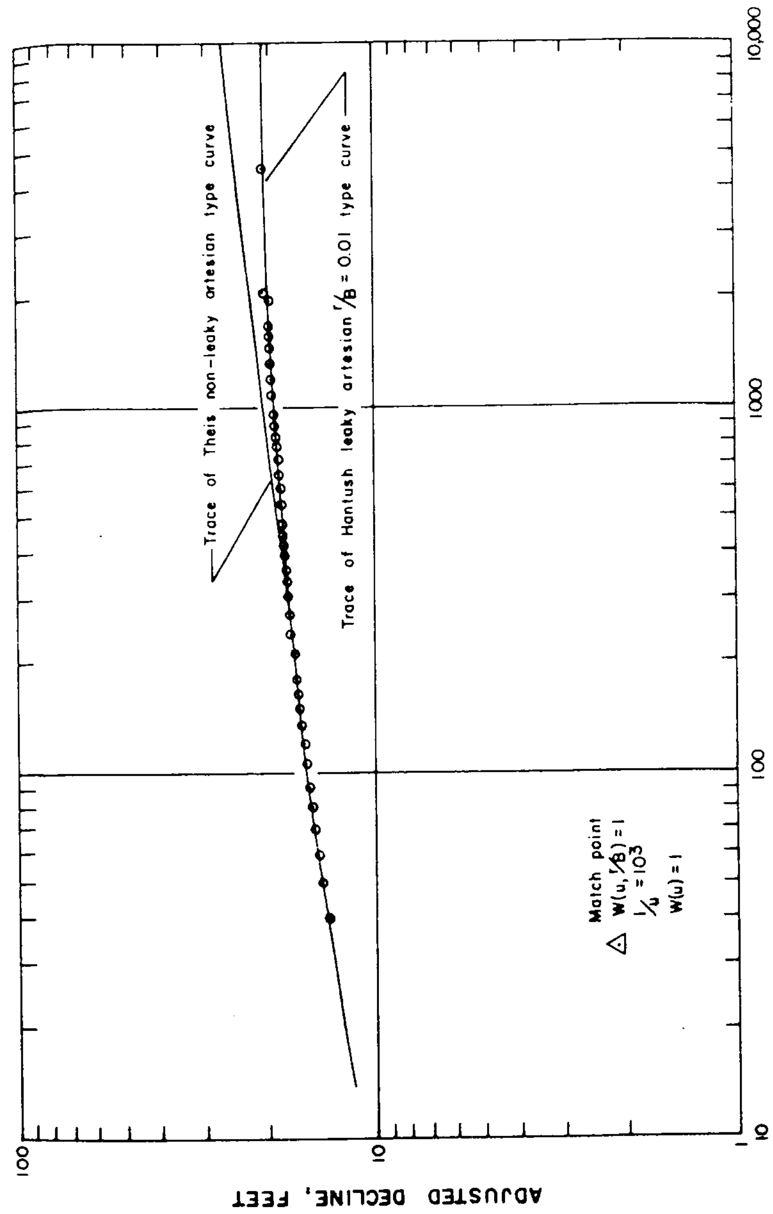

FIG. 3--Plot of recovery data and matching-type curves for second injection test.

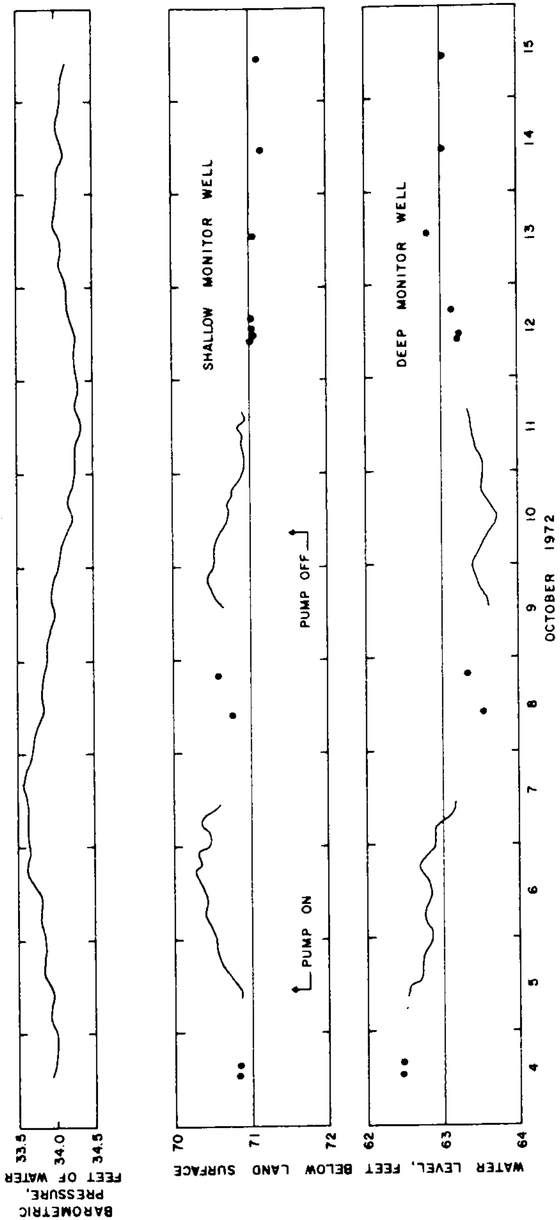

FIG. 4--Fluctuations of barometric pressure and water levels in monitor wells during second injection test.

CASE HISTORY OF SUBSURFACE WASTE INJECTION OF AN INDUSTRIAL ORGANIC WASTE[1]

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ABSTRACT From May 1968 to December 1972, an industrial organic waste was injected at rates of 100-200 gal/minute into an Upper Cretaceous sandstone, gravel, and limestone aquifer near Wilmington, North Carolina. The waste, an aqueous solution of formic, acetic, and phthalic acids, interacted with the aquifer to dissolve carbonate, aluminosilicate, and iron-containing minerals, and to produce carbon dioxide, methane, and hydrogen sulfide gases.

Water samples obtained from four observation wells that penetrate the aquifer near the injection well show a 3-fold increase in silica, a 5-fold increase in iron, and a 28-fold increase in aluminum over background data, indicating dissolution of aquifer aluminosilicate and iron-containing minerals. Gas that effervesced from these water samples contained up to 85 percent carbon dioxide by volume resulting from the reaction between carbonate minerals and the acidic waste.

Water samples obtained from an observation well 1,500 ft (457 m) north of the original injection wells gave evidence for biochemical waste transformations during passage of the waste front. Gas that effervesced from these water samples contained up to 54 percent methane by volume. Ferrous iron concentrations as high as 35 mg/l, hydrogen sulfide gas, and sulfide precipitates were additional indicators of biochemical reductive processes in the subsurface environment.

[1]Manuscript received, June 8, 1973. Publication authorized by the Director, U.S. Geological Survey.

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INTRODUCTION

Injection of an industrial organic waste into strata of Late Cretaceous age by Hercules Chemical, Inc., has been studied by a U.S. Geological Survey project (Organic Aspects of Subsurface Waste Storage) since January 1971. This study was funded as part of an effort by the U.S. Geological Survey to evaluate the environmental effects of subsurface waste injection at selected sites. This report is a case history of geochemical effects of waste injection at Hercules Inc. The specific objective was to investigate the interactions resulting from injection into the subsurface of a waste which contains organic compounds.

Hercules Chemical, Inc., injected aqueous process wastes from May 1968 to December 1972. At present (1973), the wastes are treated in a surface treatment facility with the subsurface-injection system serving as a backup. A map showing locations of injection and observation wells is given in Figure 1. The observation-well network resulted from waste-monitoring requirements set by the North Carolina Department of Natural and Economic Resources. The depths and completion dates of the wells are given in Table 1.

The waste-injection zones are in rocks of Late Cretaceous age. The lower zone is sandstone and gravel that ranges in thickness from 10 to 15 ft (3.0-4.6 m) and is 920 to 960 ft (280-293 m) below land surface. The injection zones have a low hydraulic conductivity and low productivity. These zones contain water high in content of dissolved solids. Aquifer-test data show an aggregate transmissivity value of less than 10,000 gal/day per foot for the injection zones. The artesian head of the zones before waste injection was approximately 65 ft (20 m) above land surface and about 90 ft (27 m) above mean sea level. This high artesian head indicates that the injection zones have low hydraulic conductivity seaward, and that discharge from the zones may be largely vertical through the confining layers. Water high in dissolved-solids content found in aquifers above the injection zones in this region tends to confirm vertical discharge of water from the injection zones. The generalized lithology at the injection site (Fig. 2) shows the high chloride concentrations in the injection zones and in aquifers near depths of 700 ft (213 m) and 300 ft (91 m). The artesian head is about 15 ft (6.5 m) above land surface for the 700-ft (213 m) aquifer, and is about 5 ft (1.5 m) above land surface for the aquifer near the 300-ft (91 m) depth. The freshwater aquifer at the site consists of unconsolidated sands that extend to a depth of about 75 to 100 ft (23-30 m). A more complete discussion of the geology and

groundwater hydrology at the Hercules waste-injection site is given in a report by Peek and Heath (1973) in Volume 2 of this meeting's proceedings. The geology and groundwater hydrology of the Wilmington region are given in a report by LeGrand (1960).

Hercules Chemical, Inc., was granted permission by the North Carolina Department of Natural and Economic Resources to inject up to 300,000 gal of waste per day. The maximum allowable injection pressure was 150 psi, or about 346 ft (105 m) of head above land surface.

Water and waste samples were obtained from the injection zones and adjacent aquifers from observation wells of construction similar to well 14 (Fig. 2). Twin sampling tubes are placed for the purpose of collecting water samples from both the center and the top of the injection zones. Methods of sampling and analysis were described in detail in a previous report on the Hercules waste-injection site (Leenheer and Malcolm, 1973).

CASE HISTORY OF WASTE INJECTION

The history of the Hercules injection system through 1970 is covered in a status report by the North Carolina Department of Water and Air Resources (1971). A chronology of significant events during waste injection is given in Table 1.

The original injection system was put into operation in May 1968, injecting waste at the rate of 200 gal/minute (gpm) through injection well I-6. Observation wells 1, 2, 4, and 5 (Fig. 1) were completed in the same zones as the injection well, and observation well 3 was completed in the first aquifer above the injection zones at about a 700-ft (213 m) depth. Observation well 13 was installed in November 1971 into an aquifer at a 300-ft (91 m) depth to permit sampling to determine possible waste leakage upward.

By September 1968, the waste was detected in samples collected from all the observation wells in the injection zone, which were located as far away as 150 ft (46 m) from the injection well I-6. Only pH and dissolved solids were determined on samples collected during the passage of the waste front through the observation wells. There were no determinations of any gas constituents evolved from subsurface waste reactions and no measurements of organic-waste content.

Waste injection continued in injection well I-6 until June 1969, at which time the injection pressure was up to 196 psi, which was greater than the 150 psi specified by the injection permit. In an attempt to reduce the wellhead pressure of the injection well, waste injection was shifted to observation wells 4 and 5. In November 1969, an attempt was made to reclaim the injection well by restoring permeability to a level to permit waste injection at rates up to 200 gpm without exceeding the 150 psi injection-pressure limit. The attempt to reclaim well I-6 was not successful, and injection through the observation wells continued until the second injection well (I-7A) was brought into service in May 1971.

Leakage of the waste into the aquifer at the 700-ft (213 m) depth, as a result of imperfections in well construction, was suspected when the pressure increased in observation well 3 in February 1971. In March 1971, formic acid was detected in water samples from observation well 3, confirming that waste had reached the 700-ft (213 m) zone. In May 1971, the casings of injection well I-6 and observation well 1 were filled with cement, and the pressure dropped in observation well 3, indicating that the leaks had been reduced or sealed. Additional leakage of waste from the injection zones to the aquifer at the 700-ft (213 m) depth was presumed, because the pressure dropped in the injection zone at observation well 5 in December 1971, and in observation well 2 in March 1972. These wells were reportedly plugged in October 1972.

A second injection well, I-7A, was drilled approximately 2,500 ft (762 m) northeast of injection well I-6. The second injection well was to be located at the present site of observation well 7, but the hydraulic conductivity of the injection zones was considered to be too low at this site; therefore, injection well 7 was converted to an observation well. During the development of well I-7A, waste and gas were pulled into the well. The gas was accidentally ignited by welding equipment, and later analysis by the company showed the gas to be predominantly methane. Identification of formic acid in water samples confirmed the presence of waste.

The spacing of the second observation-well network installed to monitor waste movement from both the old and new injection wells was 10-15 times the spacing used in the original observation-well network. Observation wells 11, 13, 14, 15, and 16 were installed from May 1971 to May 1972 to monitor pressure and waste movement in the injection zone. Observation wells 8 and 9, which were completed in the aquifer at the 700-ft (213 m) depth, were operational when the second injection well was completed.

Waste injection through well I-7A began in May 1971, at an average rate of 120 gpm. By October 1971, this new well was not accepting all the waste, as anticipated, and observation well 4 was converted to an injection well through which waste was injected at a rate of approximately

40 gpm. Waste injection continued through both well 4 and injection well I-7A up until the termination of injection in December 1972.

Leakage of waste into the aquifer at the 700-ft (213 m) depth also occurred at injection well I-7A. In December 1971, biological-oxygen-demand (BOD) analyses performed by the company on water samples obtained from observation well 9 indicated that waste was leaking through the confining beds or around the casing of injection well I-7A into the aquifer at the 700-ft (213 m) depth. By May 1972, the dissolved-organic-carbon (DOC) concentration in water samples obtained from well 9 had increased to 5,800 mg/l, which indicated high waste concentrations in the 700-ft aquifer.

After termination of waste injection in December 1972, fresh water has been injected at low rates in injection well I-7A and well 4 to prevent these wells from plugging. At present, the subsurface waste-injection system is serving as a backup to the surface waste-treatment plant.

WASTE COMPOSITION

The injected organic waste results from the manufacture of dimethyl terephthalate (DMT), which is used in the production of synthetic polyester fibers. DMT is synthesized by oxidizing p-xylene to terephthalic acid, which is esterified with methanol to give DMT. Prior to injection, the waste was moved through a settling basin, was passed through a filter to remove particles over 20 micrometers in size, and CaO was added to adjust to a pH of 4.

The average composition of the injected waste is shown in Table 2. The concentration of the organic constituents was determined before the addition of CaO. Inorganic constituents were determined after the addition of CaO. Dissolved organic carbon is defined as that part of total organic carbon which passes through a 0.45μ silver filter.

Almost three quarters of the dissolved organic carbon in the waste is comprised of acetic acid and formic acid. There is sufficient acidity remaining in the waste, after its partial neutralization with CaO to a pH of 4 and prior to injection, to produce 3.15 ml of carbon dioxide gas (at standard temperature and pressure) per milliliter of waste by reaction with calcium carbonate. The remaining one quarter of the dissolved organic carbon in the waste is mainly dicarboxylic aromatic acids. Phthalic acids and their salts are much less soluble than are acetic and formic acids and their salts.

CHEMISTRY OF NATIVE WATER

Chemical analyses of water samples obtained from uncontaminated observation wells were performed wherever possible. These analyses served as background data, so that small changes in water chemistry which occurred when dilute waste in the waste front entered the well could be determined. Background data were obtained for each observation well because of the variability in water chemistry in water samples obtained at different wells screened within the same aquifer. The chemical analyses of the native water obtained from aquifers at the waste-injection site are given in Table 3. Wells 11 and 12 were screened in the injection zones indicated in Figure 2, and wells 8 and 9 were screened in the aquifer near the 700-ft depth.

There were only minor variations in the water chemistry at different sampling sites (wells) in the injection zones. However, there were major differences in water chemistry in samples obtained from observation wells 8 and 9, which monitor the aquifer at the 700-ft (213 m) depth. The high iron content in the water from well 9 causes red iron oxide precipitates in the water after sampling, whereas the high sulfide content in water from well 8 causes black sulfide precipitates after sampling. These differences in groundwater quality serve to illustrate the variability in water chemistry at this waste-injection site.

DOC concentrations in the native groundwater ranged from 0.2 to 4.0 mg/l. Because the DOC concentrations of the injected waste ranged from 10,000 to 12,000 mg/l, only a small percentage of injected waste in the groundwater was necessary to raise the DOC concentration significantly. DOC was found to be a parameter for simply and conveniently measuring organic-waste occurrence in groundwater.

WASTE-AQUIFER INTERACTIONS

Evidence of waste-aquifer interactions was obtained during passage of the waste front through wells 1, 2, 4, and 5 in the first 4 months of waste injection. That interactions had occurred between the undiluted injected waste and the aquifer was observed in samples obtained from wells 2, 4, and 5 after passage of the waste front. Water samples obtained from well 14 from June 1972 to January 1973 gave evidence of major changes and interactions of the waste after it had moved a distance of 1,500 ft in the injection zone during a 4-year time period after injection. The data obtained from these samples led to the proposal of several waste-aquifer

interactions.

Concentration of residue on evaporation and pH measurements by Hercules Inc. during the first months of waste injection are shown in Figure 3 for samples obtained from observation wells 1 and 5. When the first samples were taken in July 1968, lower concentration of residue on evaporation indicated that waste was already present in observation well 1, which was only 50 ft (15 m) from injection well I-6, whereas observation well 5, at a distance of 150 ft (46 m), was waste-free.

Observation well 5 remained waste-free until the beginning of October, when both the pH and residue on evaporation started to decrease. The pH decreased because of the acidity in the waste and the dissolved solids decreased because the saline-aquifer water was replaced by the waste, which has a smaller residue on evaporation. The organic acids in the waste react with the carbonate minerals in the injection zones to form calcium and magnesium organic salts. These organic-salt solutions give neutral to alkaline pH values; therefore, the carbonate minerals act to neutralize the organic acids in the waste.

A waste-front zone in which the organic acids are neutralized is apparent in the data of observation well 1. From July 28 to October 15, the pH remained between pH 5 and pH 6, during which time the carbonates were reacting with the waste. The mixture of free organic acids and organic-acid salts from the neutralization reaction results in a pH between 5 and 6. When the waste-carbonate reaction stopped, the pH abruptly decreased to the pH of the injected waste, pH 4, on October 18, and has remained near this level to the present. Content of residue on evaporation in this well also decreased at this time to levels found in the injected waste.

Water samples collected while the waste was reacting with carbonate minerals contained large amounts of dissolved carbon dioxide, which is a product of the acid-carbonate reaction. A sample of gas which effervesced from a water sample collected from well 2 contained 85 percent carbon dioxide by volume. Observation wells 3 and 9 also yielded water samples which contained large amounts of dissolved carbon dioxide after the aquifer at the 700-ft (213 m) depth became contaminated with waste.

Reactions that occur between the undiluted, acidic waste and the aquifer in the region behind the waste front are believed to be mainly complexation and aluminosilicate dissolution reactions between organic waste acids and aquifer mineral constituents. W. F. Hower et al. (1972) mentioned that dilute solutions of complexing organic acids injected into the subsurface may dissolve significant amounts of clay minerals.

Table 4 compares the dissolved constituents in water samples from two uncontaminated observation wells with samples obtained from the interior of the waste-contaminated area in June 1971. High concentrations of silica, aluminum, and iron were found in water samples obtained from behind the waste front, which may indicate clay-mineral dissolution. Most of the calcium found in solution behind the waste front originates from the pre-injection treatment of waste with CaO.

To date, the leading edge of the waste front has been observed only at observation well 14. Organic waste was detected at a concentration of 20 mg/l DOC in the first sampling on June 20, 1972, of this well, which was completed in May 1972. The DOC was attributed to the injected waste rather than to contamination resulting from the drilling of the well, because 68 percent of the 20 mg/l DOC in the first water samples analyzed proved to be acetate and formate salts. In addition, wells 15 and 16, which were drilled, developed, and completed at the same time as well 14, did not show high DOC concentrations in water samples collected at this time.

Figure 4 shows variations in DOC, pressure difference between injection well I-7A and observation well 14, dissolved iron, and gas effervescence during the period that waste was present in this well. The waste concentrations never became sufficiently high to cause detectable effects on pH, alkalinity, specific conductance, and the major inorganic anions and cations. Pressure difference is plotted to establish a possible relation between (1) the decrease in pressure differential in the injection zone between the injection well and well 14 and (2) the decrease in DOC in well 14. Gas effervescence is defined as the volume of gas which evolves from an equivalent volume of water under atmospheric pressure at the sampling site.

From June through November 1972, the waste content in well 14 appeared to be increasing (Fig. 4). The injection rate decreased throughout November during conversion to surface treatment of the waste; after complete termination of injection early in December, the waste began to decrease until there was no sign of waste in the well by late January 1973. The waste content of the groundwater is a combination of both the DOC and gas content, which varied between 50 and 60 percent methane by volume. Although there were large pressure fluctuations caused by variations in the injection rate during conversion to surface treatment, the DOC tended to decrease as the pressure difference decreased. The major decrease in DOC occurred late in October just after there was a

large decrease in pressure difference. Because of the probable leakage of the waste into the adjacent overlying aquifer at the 700-ft (213 m) depth at injection well I-7A, and earlier at observation wells 2 and 5, it is the opinion of the writers that the waste front has virtually stopped its outward movement and is now moving upward into the 700-ft (213 m) aquifer, which is at a pressure of 35 psi less than the pressure in the injection zone. Pressure has increased about 3 psi in the 700-ft (213 m) aquifer at well 3, and 6.5 psi at well 9 since the leakage began.

The first period (June 20 to August 1) of increasing waste concentration showed only an increase in DOC as evidence of waste in the well. No gas was found in a sample collected on August 1, but gas appeared abruptly in a sample collected only 2 days later, and DOC rapidly decreased. For the period August 3 to October 31, the amount of gas continued to increase, whereas DOC did not increase at the rate that was observed prior to the appearance of the gas. A summary of the gas analysis during this period is given in Table 5.

It is suggested that the appearance of gas, which coincided with the abrupt decrease in DOC, marked the beginning of anaerobic microbial decomposition of the organic waste. The reason microbial degradation of the waste did not begin immediately with the appearance of the waste is because there is a lag time during which the microbes are building up numbers large enough to degrade the waste significantly. The waste concentration may also become too high to support microbial activity. DiTommaso and Elkan (1973) have evidence that undiluted injected waste will not support microbial growth: the waste must be diluted before microbial growth occurs.

Samples collected from August 3 to November 22 contained gas with methane concentrations up to 60 percent of the total gas volume. During this period, the concentration of dissolved iron fluctuated considerably, but tended to increase with gas evolution, suggesting microbial reduction of slightly soluble ferric iron to the more soluble ferrous iron. In samples collected up to September 4, 1972, there was considerable evidence of microbial sulfate reduction to sulfide in the form of black sulfide precipitates and the hydrogen sulfide gas found during the gas analysis. In later samples, sulfide precipitates and hydrogen sulfide gas were absent, and the level of dissolved iron increased, possibly because insoluble ferrous sulfide precipitates were no longer forming. Methane production, iron reduction, and sulfur reduction are believed to be indicators of anaerobic microbial activity indicators of anaerobic microbial activity induced by waste concentrations in the groundwater.

PROPOSED GEOCHEMICAL MODEL

A proposed geochemical model of waste movement and transformations after subsurface injection at Hercules Chemical, Inc., is given in Figure 5. The leading zone of the waste front prior to the appearance of methane gas is called the "dilution zone" because the waste appears as a dilute solution in the native groundwater. The zone following the dilution zone is called the "microbial-activity zone" because of indications of anaerobic microbial waste tranformations in this zone. A zone called the "transition zone" is postulated to follow the microbial-activity zone. After the waste attains a certain concentration in the injection zones, conditions will no longer favor continuing microbial activity, and there will be a die-off of waste-decomposing microorganisms in the transition zone. This zone is transitional between the zone of microbial waste transformations and the zone in which chemical reactions predominate. The last zone in the waste front, following the transition zone, is called the "neutralization zone." Organic acids of the injected waste are neutralized by the carbonate minerals contained in the aquifer to organic-acid salts. The neutralization zone may constitute the major area of physical change.

All four zones in the proposed model constitute parts of the waste-movement complex. The waste is diluted by the native groundwater and transformed by various chemical and microbial reactions in the waste front. The region where little dilution or transformation of the waste occurs is called the "waste interior"; however, the predominant complexation and dissolution reactions in the aquifer material caused by the organic acids in the waste occur here.

SUMMARY AND CONCLUSIONS

The case history of subsurface waste injection at Hercules Inc. has illustrated several of the possible geochemical and biochemical effects of introducing an industrial organic waste into the subsurface environment. After injection, the organic acids in the waste are first neutralized by the carbonate minerals in the aquifer. There is also evidence for dissolution of the aluminosilicate clay minerals by the complexing organic waste acids. As the neutralized waste is diluted by the native groundwater in the waste front, conditions become favorable for microbiologic degradation of the organic waste constituents. In the microbial-activity zone of the waste front there are indications that the injected organic waste is degraded to methane, with sulfate reduction and iron reduction occurring as byproduct reactions.

The geochemical model proposed in this report is strictly a qualitative model. Due to problems in waste injection and waste leakage, it was not possible to observe through an observation well the length of time of the passage of various zones in the waste front. This type of observation would give a semi-quantitative estimate of the volume and quantity of waste undergoing degradative reactions in the waste front. In a study of this type, it would be desirable to predict the rate at which the organic compounds in the waste are broken down to end products such as methane.

The Hercules waste-injection site has provided an opportunity for studying waste transformation at many observation points in the subsurface. Unfortunately, the problems encountered in waste injection and waste leakage prohibited quantitative studies of subsurface waste movement and transformations. Problems of aquifer plugging may be due in part to waste reactions which produce gaseous methane and carbon dioxide in the subsurface. Dissolution of aquifer solids by the complexing organic acids in the waste may be significant in the leakage problems at the injection and observation wells. These organic acids may dissolve the grout used to seal the annular space between the well casing and the confining beds and allow upward leakage of the waste into shallower aquifers. Ironically, the observation wells, which have provided much useful information concerning subsurface waste contamination, may provide the greatest threat of leakage of the waste into shallow aquifers containing water suitable for domestic and industrial use.

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May, 1968	wells 1, 2, 4, and 5 completed to 1,025 foot depth. Observation well 3 completed to 700-foot depth.
May, 1968	Waste injection begun through injection well I-6.
September, 1968	Waste was detected in wells 1, 2, 4, and 5.
June, 1969	Waste injection shifted from well I-6 to wells 4 and 5 because of excessive injection pressures in well I-6.
November, 1969	Injection well I-6 plugged during an attempt to reclaim. Waste injection continued through wells 4 and 5.
December, 1970	Observation well 8 completed to 700-foot depth.
January, 1971	Observation well 9 completed to 700-foot depth.
February, 1971	Leakage of waste into the 700-foot aquifer was detected at well 3.
April, 1971	Injection well I-7A was completed to 1,050-foot depth.
May, 1971	Waste injection shifted from wells 4 and 5 to well I-7A.
May, 1971	Observation wells 7 and 11 completed to 1,050-foot depth.
May, 1971	Well 1 and I-6 were cemented to stop waste-leakage into the 700-foot aquifer.
June, 1971	Observation well 12 completed to 1,050-foot depth.
October, 1971	Waste injection renewed through well 0-4 because well I-7A was not accepting all the waste at the specified injection pressure limit.
November, 1971	Observation well 13 completed to 300-foot depth.
December, 1971	Waste was detected in well 9 indicating waste leakage into the 700-foot aquifer in that area.
December, 1971	Pressure decrease in well 5 indicated possible waste leakage into an aquifer above the injection zone.
March, 1972	Pressure decrease in well 2 indicated possible waste leakage into an aquifer above the injection zone.
May, 1972	Observation wells 14, 15, and 16 were completed to 1,050-foot depth.
June, 1972	Waste was detected in well 14, and a weekly sampling program was instituted to monitor the passage of the waste front.
November, 1972	Waste injection was gradually phased over to surface treatment of the waste.
December, 1972	Waste injection terminated.
January, 1973	Waste disappeared in well 14 after injection termination.

Table 2. Partial Chemical Composition of Injected Waste

	Concentration (mg/l)	Dissolved Organic Carbon of Constituents (mg/l)	Percentage of Dissolved Organic Carbon
Dissolved Organic Carbon (DOC)		10,000	100
As:			
Acetic Acid	15,000	6,000	60
Formic Acid	5,000	1,300	13
Methanol	500	190	1.9
Phthalic, Isophthalic, and Terephthalic Acids	4,200	2,400	24
Calcium (Ca)	2,400		
Magnesium (Mg)	40		
Nitrate (NO_3)	3.9		
Orthophosphorus (PO_4)	1.3		
Residue on Evaporation at 180°C	5,100		
Specific conductance (μmhos/cm @ 25°C)	8,100		
pH	4.0		

Table 3. Chemical Analysis of Native Aquifer Water*

	Observation Well 11	Observation Well 12	Observation Well 8	Observation Well 9
Date of collection	11-03-71	11-03-71	06-15-71	06-15-71
Depth of sampling (feet below land surface datum)	935	945	695	725
Silica (SiO_2)	8.6	9.0	8.8	8.0
Aluminum (Al)	.19	.45	.50	.31
Iron (Fe)	1.70	5.30	2.20	12
Manganese (Mn)	.33	.27	.18	.18
Calcium (Ca)	537	417	265	321
Magnesium (Mg)	195	270	321	438
Sodium (Na)	6,600	6,650	5,900	6,750
Potassium (K)	330	320	212	230
Lithium (Li)	.28	.28	-	-
Bicarbonate (HCO_3)	232	233	358	244
Carbonate (CO_3)	0	0	0	0
Sulfate (SO_4)	210	250	610	740
Chloride (Cl)	12,000	11,900	9,650	11,400
Fluoride (F)	.6	.6	.5	.4
Nitrate (NO_3)	< .1	< .1	.8	.6
Orthophosphate (PO_4)	< .1	< .1	.02	.02
Hydrogen sulfide (H_2S)	.2	.3	2.1	.6
Copper (Cu)	.25	.26	.12	.27
Zinc (Zn)	.11	.05	.04	.04
Residue on evaporation at 180°C	19,500	20,000	17,100	20,000
Specific conductance (μmhos/cm @ 25°C)	32,000	32,000	27,100	31,100
pH	7.2	7.2	7.5	7.4
Dissolved organic carbon (DOC)	.6	.4	3.5	1.1

*Chemical constituents are reported in mg/l.

Table 4. Chemical Analysis of Waste-Free and Waste-Contaminated Aquifer Water.

Well	Specific cond. (micromhos/cm @ 25°C)	pH	Concentration of dissolved constituents, in milligrams per liter														
			SiO_2	Al	Fe	Mn	Ca	Mg	Na	K	HCO_3	SO_4	Cl	F	P	NO_3	DOC
7	32,500	7.3	11	0.3	1.8	0.2	710	107	6,800	330	230	280	12,000	0.9	<.1	<.1	1.2
11	32,000	7.2	8.6	0.2	1.8	0.3	540	195	6,600	330	230	210	12,000	0.6	<.1	<.1	0.7
4	8,280	4.0	23	6.8	8.3	0.3	2,500	34	2.9	2.2	0	8.0	230	1.3	1.3	1.3	10,600
5	8,080	4.0	34	7.1	8.0	0.3	2,400	29	3.2	2.3	0	19	140	1.3	1.1	3.9	11,200
Contaminated (4 + 5) / Uncontaminated (7 + 11)			2.9	28	5.1	1.0	3.9	0.27	.00046	.0069	0	0.055	0.02	1.7	∞	∞	11,500

Table 5. - Gas Analysis*

Well	Date of Sampling	Volume, in percent of total gas volume				
		H_2	N_2	CH_4	CO_2	H_2S
14	8-1-72	0.5	26	51	11	ND[1/]
14	8-7-72	ND	22	54	12	0.2
14	8-14-72	0.2	21	52	11	0.6
14	10-11-72	ND	29	48	11	ND
14	11-2-72	ND	62	33	3.8	ND
14	11-22-72	ND	68	12	1.5	ND

* Gas analysis made by Donald W. Fisher, U.S. Geological Survey, Washington, D.C.

1/ Not detectable.

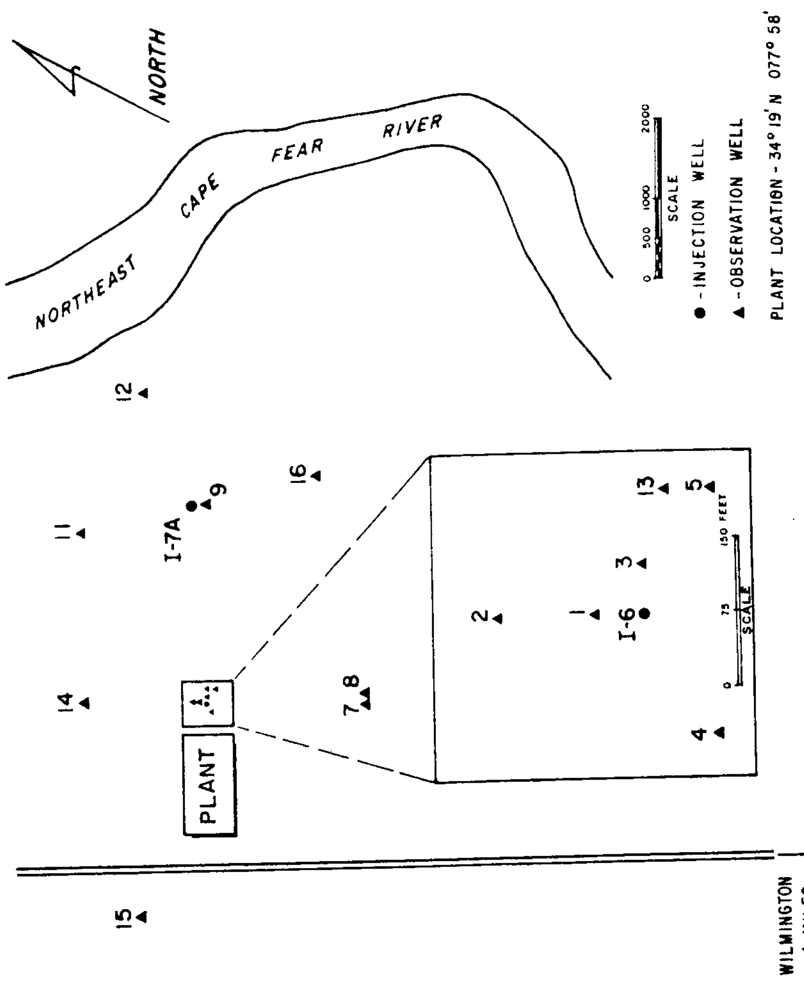

FIG. 1--Map of waste-injection and observation wells.

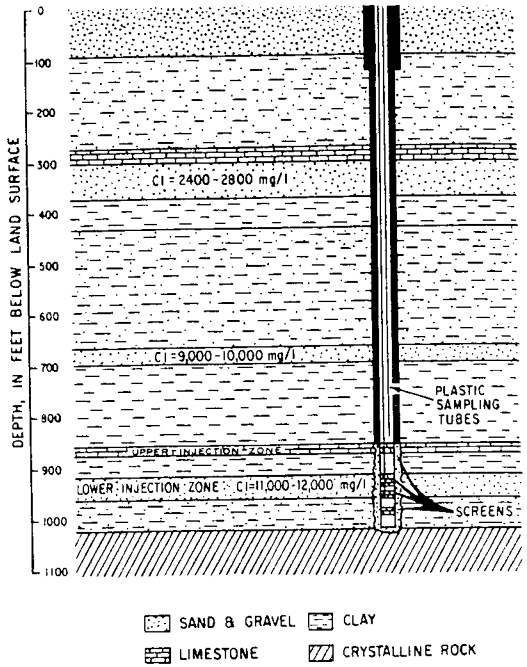

FIG. 2--Diagram showing construction features and lithologic log of well 14.

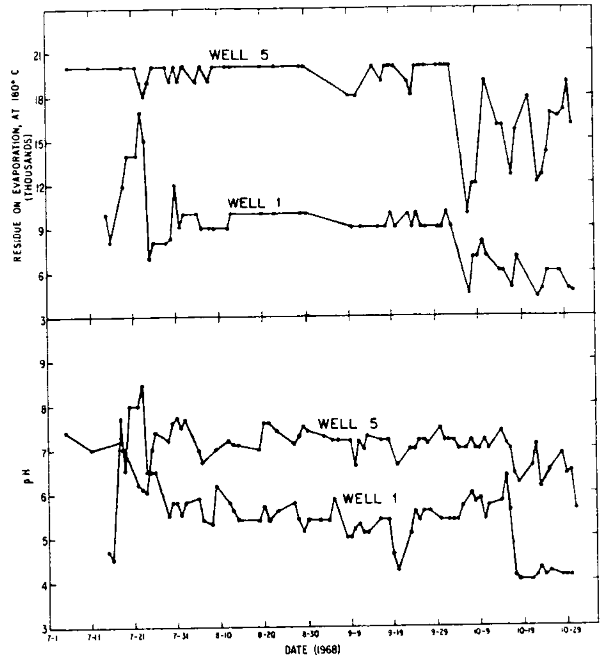

FIG. 3--Variations in pH and residue on evaporation from samples taken during observation of waste front in wells 1 and 5. (Data furnished by Hercules Inc.)

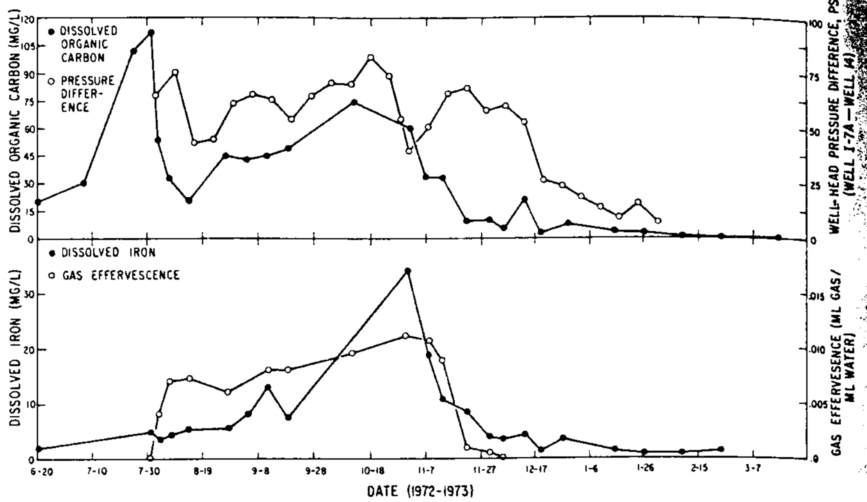

FIG. 4--Variables observed during passage of waste front in well 14.

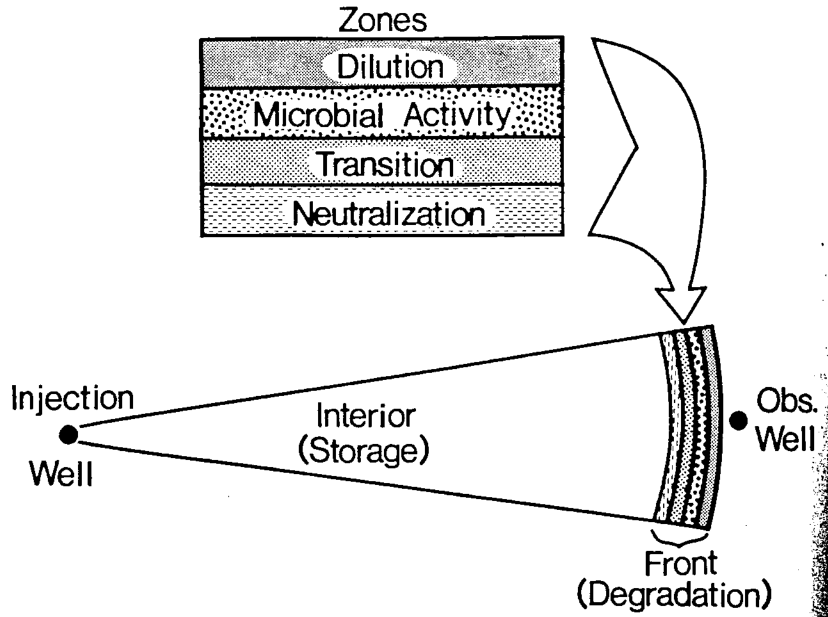

FIG. 5--Proposed geochemical model of waste after injection into subsurface

ROLE OF BACTERIA IN DECOMPOSITION OF INJECTED LIQUID WASTE AT WILMINGTON, NORTH CAROLINA[1]

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ABSTRACT In 1968, Hercules Chemical, Inc., Wilmington, North Carolina, began injecting organic waste through wells into a saline disposal zone at depths of 850 to 1,000 ft. The waste, which is a byproduct of dimethylterephthalate used in the production of synthetic fiber, is composed of water containing approximately 15,000 ppm of acetic acid, 5,000 ppm of formic acid, and 500 ppm of methanol, with a pH of about 4.0.

The movement and composition of the transformed waste have been monitored by a network of 14 observation wells of varying depths. The wastewater analysis and monitoring were performed by the United States Department of the Interior, Geological Survey.

The microorganisms in the unpolluted aquifer were isolated and identified. Genera including Agrobacterium, Pseudomonas, Bacillus, Arthobacter, Aerobacter, Corynebacter, and Staphylococcus have been observed. The viable microbial count in the unpolluted aquifer remained approximately 3,000/ml throughout the sampling period.

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In June 1972, waste was detected in observation well 14. The microbial population was periodically sampled as the dissolved-organic-carbon content increased. Appearance of the waste front was accompanied by a rapid increase in the bacterial population. These bacteria utilized the waste as a source of carbon and energy while receiving their nitrogen from the groundwater. The aquifer supplied various trace elements which the organisms need for growth.

One of the byproducts of this utilization is methane. The amount of methane produced increased with a corresponding decrease in dissolved organic carbon. Methane gas thus became an indication of microbial activity on the waste. The number of colony-forming units increased during the entire study. The number of bacteria showed a sharp increase corresponding to a decrease in dissolved organic carbon, thus giving in vivo evidence of waste decomposition.

The concentration of iron in the groundwater varied over the period of the study. It has been reported that 20-50 micrograms of iron per liter are required for the methane fermentation. Methane production increased as the iron concentration increased. Laboratory experiments showed the methanogenic bacteria to be capable of using the waste as a carbon and energy source. These organisms also produced methane from the waste under laboratory conditions. Though the aquifer is providing a natural medium for the bacteria, the latter become inhibited as the concentration of waste increases. Bacteria in the aquifer have the capability for causing decomposition, but cannot decompose the waste at the rate at which it is being injected.

INTRODUCTION

Deep-well disposal has been used for over 50 years as one solution to the storage of liquid wastes. Little, however, has been done to examine the natural flora which inhabits the well of the disposal aquifer. The effects of microorganisms on the injected waste need to be determined so that this type of disposal system can be properly evaluated.

In 1968, Hercules Chemical, Inc., Wilmington, North Carolina, began injecting organic waste through wells into a saline disposal zone at depths ranging from 850 to 1,000 ft. This waste, which is a byproduct of dimethylterephthalate used in the production of synthetic fiber, is composed of water containing approximately 15,000 parts per million of acetic acid, 5,000 parts per million of formic acid, and 500 parts per million of methanol, with a pH of about 4.0.

The movement and composition of the transformed waste have been monitored by a network of 14 observation wells, drilled to various depths. The wastewater analysis and monitoring were performed by the United States Geological Survey research division, Denver, Colorado.

This report summarizes preliminary data identifying genera of bacteria found in the aquifer and showing the role and relative efficiency of these organisms in decomposing the injected waste.

MATERIAL AND METHODS

Sampling Procedure

All data reported herein were obtained from water samples flushed from monitor wells as follows. After the well was flushed for thirty minutes, a 100-ml sample was collected in a sterile serum bottle and placed on ice. Triplicate 100-ml samples were obtained from the injected waste, a well containing waste, and an unpolluted control well. Samples were obtained weekly or biweekly, depending on the dissolved-organic-carbon levels. The samples were placed in a 32°C incubator overnight, after which serial dilutions, in triplicate, were performed on all samples. Anaerobic jars (with disposable gas-generator envelopes and anaerobic indicators; Bioquest, Cockeyville, Md.) were used to determine the number of facultative and anaerobic bacteria. All plates were incubated at 32°C for 3 days, and colony counts were then determined. Several growth media were screened, and the polypeptone-peptone (PP) medium shown in Table 1 was selected for use because it resulted in the best bacterial growth. Individual colonies were picked off the plates, checked for purity, restreaked on PP agar, and stored at 5°C.

Identification of Well Isolates

Bacteria which had been isolated as previously described, from polluted and unpolluted wells, were identified as to genus. Different colonies were picked off and streaked onto separate plates. The isolates were restreaked and tested for purity. Standard tests (Pelczar et al., 1957)--such as gram reaction, motility, sulfide production, reduction of nitrate, catalase production, starch hydrolysis, indole production, and carbohydrate utilization--were used to identify the isolates. Bergey's Manual (Breed, 1957) was used as the reference for the identifications.

Bacterial Nitrogen Source

The only apparent nitrogen source for the organisms appeared to be a filming amine added to the waste as a corrosion preventative. A growth medium (Table 2) with 5.0 g/l glucose as an energy source was supplemented with 10 ml/l of filming amine as the sole nitrogen source. This medium was inoculated, in triplicate, with the previously isolated bacteria. The plates were incubated for 3 days at 32°C and observed for growth.

Waste as a Carbon Source

In order to determine whether the bacterial isolates could utilize the waste as a carbon and energy source, the growth medium in Table 2 was modified by substituting for the glucose varying concentrations of the waste as the sole carbon and energy source. This medium was inoculated, in triplicate, with bacterial isolates, and the plates were incubated for 3 days at 32°C and observed for growth.

Gas Sampling

Appearance of the waste front resulted in a corresponding production of gas. When gas was present in the observation well, it was sampled in the following manner. The outflow from the tygon tubing was attached to the inlet of a 250-ml cylindrical glass gas-collecting tube, and the well water was allowed to flow through to displace the air in the collector tube, which was held in the vertical position with both inlet and outlet stopcocks open. After all the air was displaced, the gas-collector tube was placed in the horizontal position, and well water was allowed to flow through the tube until 5-10 ml of gas had been collected in the upper portion of the tube. The gas and water were sealed in the tube by simply closing the inlet and outlet stopcocks. Gas was analyzed by a gas chromatograph with thermal-conductivity detection after degassing the water and gas sample by exposing the sample in the gas-collection tube to a partial vacuum. By this method, dissolved gases as well as free gases could be included in the analysis. Hydrogen, argon, oxygen, nitrogen, methane, and carbon dioxide were included in the gas analysis. Gas analysis was performed using atomic absorption by the United States Geological Survey, Washington, D.C.

Assay for Methane

With the appearance of gas corresponding to a decrease in dissolved organic carbon, samples were taken to test for bacterial production of methane. After flushing the well for 30 minutes, 15-ml aliquots of waste

water were placed in 20-ml scintillation bottles to which 1 ml of cysteine hydrochloride, 1 ml of sodium bicarbonate, and 1 ml of sodium pyruvate had been added. Cysteine hydrochloride was used to maintain reduced conditions. Sodium bicarbonate was used as a buffer and the sodium pyruvate as a carbon source. Approximately 30 samples were obtained at each sampling date. The bottles were later transferred to anaerobic jars (Bioquest, Cockeyville, Md.). The jars were incubated at 32°C and sampled for methane after 1 week.

An F and M model-810 gas chromatograph (F and M Scientific Division, Avondale, Pa.) was used to detect methane. Gas from the laboratory gas jet was used as a standard. Samples of 10 cc were obtained using a 10-cc sterile luer-lok disposable syringe. The syringe was placed into the rubber hose of the jar just below the hose clamp. Fifteen percent Apiezon L (Apiezon Products, London, England) on 60/80 gas chromatograph Q (Applied Science Labs, Inc., State College, Pa.) was used, with helium at 30 cc per minute as the carrier gas. All gas analysis was performed at room temperature.

Enrichment for Methane Bacteria

Once it was determined that the methane was biological in origin, it became necessary to isolate the methanogenic bacteria so as to identify them according to genus. The original samples which were producing methane were used as the inoculum. A medium (Bryant et al., 1970) specifically designed for methanogenic bacteria was used (Table 3). Inocula of 1 ml were placed into 20-ml scintillation bottles which contained 15 ml of sterile medium. Twelve bottles were placed in each anaerobic jar and incubated at 32°C. After 1 week the jars were checked for the presence of methane. After an additional 1-week incubation, 1-ml samples of these cultures were used to inoculate fresh methanogenic media. This procedure was repeated three times, permitting enrichment of the methanogenic bacteria and selection against other species of microorganisms.

Utilization of Waste by Methanogenic Bacteria

It was important to insure that it was actually these bacteria which were causing the decrease in dissolved organic carbon. A medium consisting of filming amine (2.0 ml), organic waste (98.0 ml), and water (9.00 ml) was used under reduced conditions as the growth medium. The amine was used as the sole source of nitrogen. Inocula of 1 ml from the methane producing jars were used. After 1 week at 32°C the jars were tested for

Identification of Methane Bacteria

The isolates were streaked onto plates of methanogenic agar (Table 3). The plates were incubated at 32°C for 1 week. Colonies were picked off and restreaked. After 1 week of growth at 32°C, gram stains were performed and the bacteria were identified as to genus.

RESULTS AND DISCUSSION

The waste, prior to injection, was found to be void of any bacterial contamination. Likewise, samples obtained from monitor wells containing high levels of waste did not support a microbial flora. Laboratory experiments confirmed these observations. Approximately 3,000 organisms per milliliter were present in the unpolluted aquifer, and this count remained constant for the duration of the study. These native organisms were isolated and identified as shown in Table 4. Although anaerobic, methanogenic bacteria were found in the polluted wells, most of the organisms isolated from the unpolluted wells were facultative or aerobic genera rather representative of the normal microflora of aquatic environments.

The most common genera found included <u>Agrobacterium</u>, <u>Pseudomonas</u>, <u>Proteus</u>, <u>Bacillus</u>, <u>Aerobacter</u>, <u>Corynebacter</u>, <u>Arthrobacter</u>, and <u>Micrococcus</u>. In laboratory studies, isolates of these genera, either singly or in combination, were inoculated into a medium in which various dilutions of the waste served as the sole carbon and energy source. None of the well isolates were able to grow and decompose the waste under these conditions. In addition to these microorganisms, a very low number of obligate anaerobes was detected. Since there is little or no organic-energy substrate in the unpolluted aquifer, these organisms can be present only in limited number. When a readily available carbon and energy source was added in the form of the injected waste, these anaerobes increased in number and constituted the waste-decomposing microflora.

The waste front was first detected in well 14, on July 7, 1972. The effect of this addition of waste upon the microbial population, dissolved organic carbon (mg/l), methane production (%), and iron content (μg/l) are summarized numerically in Table 5. A rapid increase in the microbial population resulted from the waste injection. This increase is shown graphically in Figure 1. The colony-forming units (CFU) per milliliter in the control well (well 11) remained at an approximately constant 3,000 organisms per milliliter during the 20 weeks of the study, whereas, in the waste (well 14) the population increased to approximately 1,000,000 organisms per milliliter. When the ratio between bacterial counts and dissolved organic carbon (DOC) (CFU:DOC) are plotted against time as shown in Figure 2, a proportional increase in bacterial number as the waste content of the aquifer increases is apparent. Evidence that the bacteria are able to decompose the waste is shown in Figure 3. The DOC increases as the waste front reaches well 14. In response to this increase, the bacterial population (CFU) increases. When there was a sufficient CFU increase, a corresponding decrease in the DOC was observed, again indicating utilization of the waste by the bacteria. The population estimates in Table 5 and in Figures 1, 2, and 3 are based on the standard plate-count technique and appear low considering the rate at which the organic-carbon level changed. Therefore, the bacteria were counted comparatively using the direct microscopic-count technique, and the counts in well 14 were found to be approximately 1,000 times higher than was found using viable counts. Since anaerobic bacteria are difficult to culture, it is likely that the viable counts are indeed low. Another possible reason for the low bacterial count is the lack of anaerobic and/or pressurized sampling equipment. The bacteria live in the aquifer at a pressure of 200 psi. It is possible that some of these bacteria are barophilic--that is, they require pressures higher than atmospheric pressure in order to grow. Sampling by flushing the wells could kill these normally viable cells. A pressurized, anaerobic sampler would keep such bacterial losses to a minimum. However, the data indicate that a constant fraction of the bacterial population continues to be measured, resulting in valid comparisons.

Methane Production

On August 12, 1972, the DOC of well 14 reached 112 mg/l. Six days later the DOC content had decreased to 32 mg/l with the appearance of gas, of which 53 percent was methane. Methane production is of biological origin. The ability to produce methane is confined to a small group of strictly anaerobic bacteria. Unpolluted wells contained no gas, but since all the wells sampled were in the same aquifer it can be assumed that all the wells contained at least low numbers of methane bacteria. The polluted well, which contains carbon and energy sources for the bacteria, was the only well which had methane gas. The presence of the gas presumably is due to the increased methane bacterial population resulting from nutrient enrichment by the waste.

McCarthy and Spence (1964) reported that 20,000 to 50,000 µg/l of iron (ferric or ferrous) was required for methane fermentation. As shown in Table 5, only trace amounts of iron were present in the unpolluted aquifer and little methane was produced. A large amount (up to 35,000 µg/l) of iron was found in the aquifer after the waste front reached well 14. The ratio between the iron content of the water and methane production is graphically summarized in Figure 4. The data show an increase in methane fermentation when the iron content of the aquifer increased. The data in Table 5 also show that increases in methane production were accompanied by a corresponding decrease in DOC, again demonstrating *in vivo* utilization of the waste. The data demonstrate that there was a correlation between the decrease in DOC and increase in the microbial population, and that this was dependent on the iron content of the water. The production of methane also appears to be dependent on the amount of iron present.

High bacterial counts were obtained in the wells on the periphery of the waste front, whereas, in wells containing concentrated waste, bacterial growth was inhibited. On the periphery of the front, the waste was diluted with enough aquifer water so that it was not toxic to the microorganisms. Laboratory experiments performed using the waste as the sole carbon source confirmed that the bacteria were unable to utilize the concentrated waste. When the waste was sufficiently diluted (10 percent waste or less), the methanogenic bacteria grew and produced high levels of methane. Experiments confirmed that the microbe isolates from wells, including the methanogenic bacteria, were able to use the filming amine as the sole source of nitrogen. Later it was discovered that the groundwater did contain as much as 0.01 mg/l nitrate. This quantity of nitrogen would adequately sustain microbial growth.

Experiments were performed to ascertain which group of organisms was decomposing the waste. It was discovered that the well isolates, with the exception of the methane bacteria, could not grow on the waste no matter how dilute a concentration was used. The methanogenic bacteria did produce methane rapidly from a medium of dilute waste and filming amine. The amount of methane produced after a 1-week incubation was approximately 42 percent of the total atmosphere in the anaerobic jar. The waste, composed of acetate, formate, and methanol, could be an ideal carbon and energy source for the methanogenic bacteria which utilize one-carbon substrates for their growth. Thus the organic waste, along with the minerals in the

native groundwater, make an ideal "medium" for the methanogenic bacteria.

The methanogenic bacteria were isolated in pure culture, and gram stains were performed in an attempt to classify them according to genus. Two different morphologic types were observed. The first was a gram-negative, slightly curved rod, which was tentatively placed in the genus *Methanobacterium*. The other was a coccus which was gram positive and occurred in masses; it was tentatively placed in the genus *Methanococcus*.

Although the waste is decomposable by microorganisms, the system appears to have a low efficiency. Laboratory studies showed the waste to be toxic even in moderate concentrations. The major localization of decomposition in the wells is at the periphery of the waste where the waste is highly dilute. As the waste becomes more concentrated, the bacteria are affected by new external factors. Changes in pH, alkalinity, and concentration of heavy metals, and various chemical reactions brought about by the waste, cause inhibition of the microorganisms.

The microorganisms cannot decompose the waste at a rate anywhere near the rate at which it is being injected (200 gal/minute). The waste front would have to move at a much slower rate and the amount of waste being injected would have to be reduced drastically in order for efficient decomposition to occur.

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Table 1. Composition of Polypeptone-Peptone (PP) Medium used for Determination of Microbial Population from Well System[1]

Chemical Components	Composition (g/l)
Polypeptone peptone	2.0
Yeast extract	3.0
Succinic acid	8.0
Glucose	2.0
Bacto agar	16.0
KH_2PO_4	1.0
K_2HPO_4	1.0
NaCl	0.2
$MgSO_4$	0.2
$CaCl_2$	0.2
$FeCl_3$	0.04

[1]The pH of the resulting solution was between 70. and 7.2.

Table 2. Defined Media with Filming Amine as Sole Nitrogen Source[1]

Chemical Component	Composition (g/l)
Glucose	5.0
KH_2PO_4	1.0
K_2HPO_4	1.0
NaCl	0.2
$MgSO_4$	0.2
$CaCl_2$	0.2
$FeCl_3$	0.04
Filming amine	10 mls
Ion agar	16.0

¯pH adjusted to 7.0.

Table 3. Media Used for Growth and Isolation of Methanogenic Bacteria[1,2]

Chemical Component	Composition (g/l)
Na formate	2.0
Cysteine sulfide	5.0
Hemin solution	5.0 mls
$(NH_4)_2SO_4$	2.0 mls
Acetic acid	36.0 mls
Vitamin solution[3]	5.0 mls
Mineral solution[4]	50.0 mls

[1]Bryant, M., et al. (1970).

[2]The pH of the solution was adjusted to 6.9.

[3]This solution is composed of 20 mg each of thiamin HCl, Ca-d-pantothenate, nicotinamide, riboflavin, and pyrodoxine HCl, 0.5 mg biotin, 0.25 mg folic acid, and 0.2 mg Vitamin B_{12} in 100 ml of distilled water.

[4]This solution is composed of 18 g KH_2PO_4, 18 g NaCl, 0.53 g $CaCl_2 \cdot 2H_2O$, 0.4 g $MgCl_2 \cdot 6H_2O$, 0.2 g $MnCl_2 \cdot 4H_2O$, and 0.025 g $CaCl_2 \cdot 6H_2O$ in 1 liter of distilled water.

Table 4. Identification of Isolates from Unpolluted Deep Well (Well 11)

Organism Identified	Gram stain	Morphology	Motility	Sulfide Production	Reduction of Nitrate	Catalase Production	Starch Hydrolysis	Indole Production	Carbohydrate Utilization: Sucrose	Mannitol	Lactose	Glucose	Maltose
Agrobacterium	-	Rod	+	-	-	+	-	-	-	-	-	A	A*
Pseudomonas	-	Rod	+	-	+	+	-	-	A	Ag	Ag	Ag	Ag
Proteus	-	Rod	+	-	+	+	-	-	A	Ag	Ag	A	Ag
Bacillus	+	Rod	-	-	+	+	+	-	-	A*	A*	A	A
Aerobacter	-	Rod	-	-	+	+	-	-	A	Ag	Ag	Ag	Ag
Corynebacter	+	Rod	-	-	+	+	-		-	A	A	A	-
Arthrobacter	-	Rod	-	-	+	+	+	-	A	A	A	A	A
Micrococcus	+	Cocci	-	-	+	+	-	-	-	-	A	A	A
Pseudomonas fluorescens group	-	Rod	+	-	+	+	+	-	-	-	-	-	A*

Key: A - acid produced
g - gas produced
Ag - acid and gas produced
+ - positive reaction
- - negative reaction
A* - weak positive reaction

Table 5. Comparison of Microbial Counts, Dissolved Organic Carbon, Percent Methane, and Iron Content of Samples Obtained from Well 14

Sampling	Microbial Count Colony Forming units/ml	Dissolved Organic Carbon mg/l	Methane Percent	Iron Content µg/l
6-1-72	2.0×10^3	20	3	2.01
6-6-72	2.3×10^3	30	3	2.09
7-7-72	3.0×10^3	70	3	3.74
7-17-72	3.5×10^3	102	2	6.12
8-1-72	4.8×10^4	112	4	5100
8-7-72	5.2×10^4	32	50	4200
8-15-72	6.0×10^4	20	53	5300
8-28-72	5.8×10^4	44	40	5400
9-6-72	6.2×10^4	42	8	8100
9-13-72	7.0×10^4	44	2	13000
9-29-72	7.1×10^4	48	32	12000
10-11-72	7.2×10^4	74	7	35000
10-31-72	8.7×10^4	60	4	34000
11-2-72	1.0×10^5	18	30	34000
11-7-72	1.9×10^5	34	10	18000
11-13-72	4.1×10^5	33	5	11000
11-22-72	9.6×10^5	10	21	8600
12-5-72	9.5×10^5	6	3	3400
12-13-72	9.7×10^5	21	3	4200
12-19-72	9.6×10^5	3	24	1400
12-29-72	9.0×10^5	4	3	3600

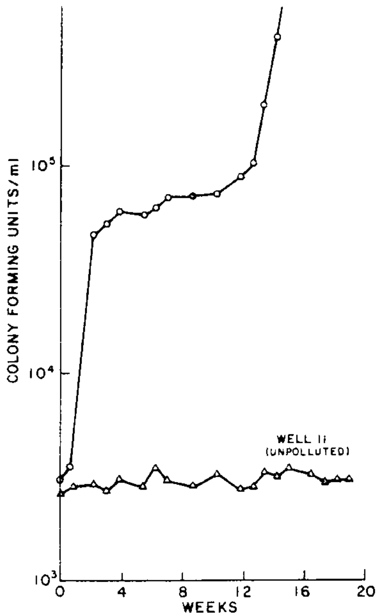

FIG. 1--Comparison of number of bacteria per milliliter (as colony-forming units) in waste front (well 14) and in unpolluted aquifer (well 11).

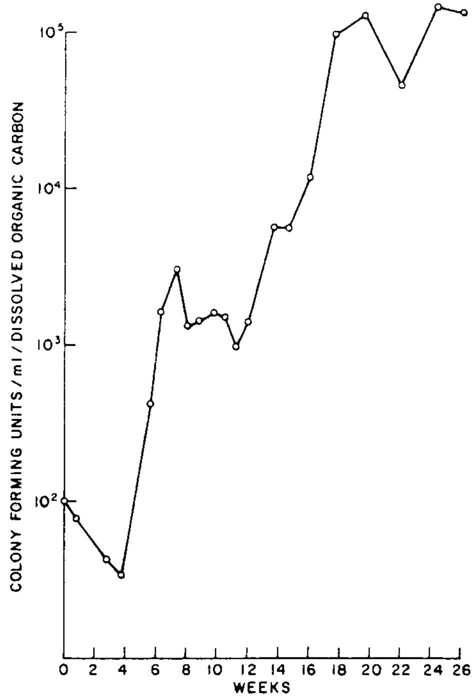

FIG. 2--Ratio of number of bacteria (as colony-forming units) to DOC (mg/ml) in waste front (well 14).

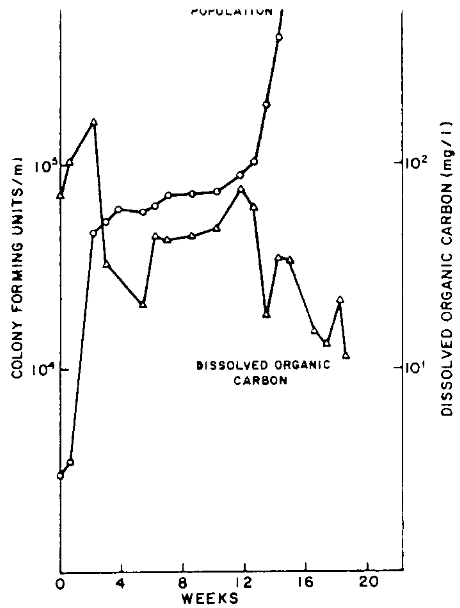

FIG. 3--Relation between microbial population (as colony-forming units) and DOC (mg/l) in waste front (well 14).

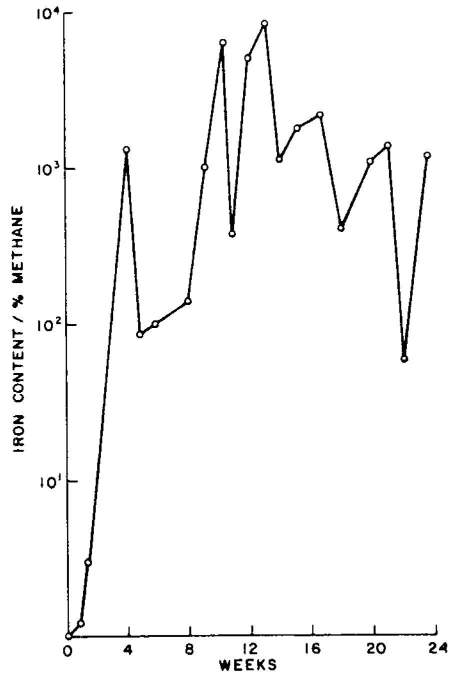

FIG. 4--Ratio of iron content (μg/l) to methane production (%) in waste front (well 14).

CASE HISTORIES--DECISION AND EVALUATION

HISTORY OF A TWO-WELL INDUSTRIAL-WASTE DISPOSAL SYSTEM[1]

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ABSTRACT A survey of underground injection conducted by Bureau of Mines engineers included a two-well injection system, at which operation began in 1964. The data include (1) the waste-disposal problem that led to selection of underground injection as the best solution, (2) origin and analyses of two separate waste streams, (3) surface equipment used for pre-injection treatment of the wastes, (4) well design and completion, (5) local and regional geology, (6) pre-injection tests of the wells and laboratory tests of the disposal formation, and (7) the operating history of the two wells.

The major constituents of the wastes are phenol, 1-butanol, butanal, and n-hexylamine. The adsorption characteristics of these compounds were determined under simulated reservoir conditions using an autoclave. The equilibrium amounts adsorbed and the effect of pH were determined as functions of concentration. The advance of the waste constituents is retarded by adsorption; thus, at the advancing front a zone develops that is depleted of waste constituents. The depth of this zone increases as more of the formation is invaded by the injected fluid. The radius from the wellbore of migration of the waste constituents was calculated with respect to the total volume of waste injected as an aid to the planning of waste-injection systems.

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INTRODUCTION

The problems associated with waste disposal have plagued the civilized world throughout recorded time. The Bible (Deut. 23:12-13) records Moses' instructions for waste disposal during the journey through the wilderness, ancient Rome had a major problem with her wastes, and even today cities like Tehran have open sewers and water-distribution systems. In the industrial Rhine Valley of Germany, a separate covered channel or canal eventually was built to keep wastes from polluting the Rhine River. However, such approaches are expedients that really only move the waste to another location where it is less hazardous and has less detrimental effects on the environment.

Industry utilizes the most economical means within the law to dispose of its wastes. Many of the former avenues of disposal are no longer open to the chemical-process industry; therefore, a reevaluation of the options open for waste disposal was made. Liquid chemical wastes, cooling-water wastes, spent process wastes, and many other liquid wastes, ranging from organic cleaning solvents to decomposed food stuff, compose the endless variety of industrial wastes. Because of the necessity of increasing the restrictions on surface disposal systems, more and more industries are turning to underground waste injection as a possible alternative.

The petroleum industry has a long history of using subsurface injection for the disposal of oil-field brines and for secondary recovery of petroleum. The technology has been transferred from the petroleum industry to many others having waste-disposal problems. Although similar, oil-field operations are sufficiently different from industrial-manufacturing waste-disposal requirements that special consideration must be given to the individual problems.

The thrust of the research on underground waste injection at the Bartlesville Energy Research Center is the determination of the fate of waste constituents injected into underground formations. However, the specific technology and operation of numerous industrial-waste injection systems also were studied in detail in preparation for the laboratory research. The data used for the discussion of the technology of the two-well injection system were obtained from a survey conducted by Bureau of Mines engineers. The laboratory data are part of the Bureau's larger study on the behavior and fate of waste constituents injected into a subsurface environment.

THE INDUSTRIAL-WASTE PROBLEM

The original waste-treating facility at a plant was underdesigned for the total volume of aqueous-waste production it was required to handle. The original waste-treatment process consisted of two algae lagoons. They were intended to accomplish natural bio-oxidative reduction of the biological oxygen demand (BOD) of the waste before discharge of the effluent into a nearby river. However, soon after the chemical plant was placed in full-scale operation in 1962, it became obvious that the lagoons were not large enough to provide sufficient holding time for bio-oxidation to accommodate the average daily volume of waste effluent. Efforts then were made to contain the waste on several acres of company-owned land, on the premise that evaporation from the large diked area would sufficiently reduce the volume of waste so that the plant could handle it. A new problem soon developed when a neighboring farmer complained about leakage of the waste through the dikes into his fields. At this point a consultant was employed to suggest alternate means for the waste disposal.

A feasibility study of underground injection of the waste was among the alternate methods of waste treatment presented to the company by the consultant. Underground injection was selected on the basis of adequate geologic conditions, economical operation, minimum space requirements, and lower environmental impact of the waste and surface treatment facilities.

WASTE ANALYSIS

The company produces acetic, adipic, and propionic acids, acetaldehyde, butanol, hexamethyldiamine, vinyl acetate, nylon, and other chemical products from petroleum-base stocks.

The plant waste effluent is collected at the waste-treatment facilities as two separate waste mixtures originating from the various processing units. The two waste-effluent streams are not mixed, because mixing produces considerable precipitation that leads to additional sludge-handling and disposal problems. Hence, the wastes are collected in separate sumps and then processed and injected separately. Thus, a two-well injection system is best suited for disposing of the total waste effluent generated by this chemical plant.

The composite waste injected into well 1 is a mixture of cooling-tower and boiler-water concentrates and wastes from chemical-processing units containing 1.0-2.0 percent organic materials. A specific analysis of the waste is listed in Table 1; however, the concentrations vary widely, depending on operating parameters of the plant. The amount of dissolved solids ranges from 0.3 to 1.0 percent and the pH ranges from 4 to 6. The waste at this point is a dark brown liquid with a strong pungent odor, and it enters the waste-treatment facilities at a temperature ranging between 65 and 80°C.

An analysis of the composite waste injected into well 2 is listed in Table 2. This waste contains amines and nitrates from nylon manufacture, hydrocarbon solvents used in processing, and other minor constituents. It has an unpleasant odor imparted by the amines and is basic, ranging in pH from 8 to 10. The waste enters the waste-disposal facilities at 45 to 55°C.

SURFACE EQUIPMENT

The pre-injection treatment facilities for the acid waste (Table 1) are shown in Figure 1. The waste from the process units enters an underground collecting sump by gravity drainage. It is then pumped to an automated neutralization tank where the pH is adjusted to a range of 4 to 6. Below this range, a flocculent precipitate forms; above this range, reactions occur that produce a tarlike solid that is difficult to handle. Hence, pH control is an important step. The next step involves sedimentation of suspended solids and oil removal, which are accomplished in an automated sedimentation tank equipped with a surface-skimming device and a sludge rake. The clarified waste is then filtered through two anthracite filters and held in a surge tank equipped with a liquid-level flow controller that governs the operation of the injection pump. High-pressure cartridge filters with 14-micron pore openings are used for final filtration before the waste enters the well. The cartridge filters are precoated with a volcanic-ash filter aid to prevent rapid plugging. The injection pump is a two-stage centrifugal pump with maximum ratings of 2 m^3 per minute ($m^3\ min^{-1}$) and 40 atm.

The treatment facilities used for processing the basic wastes (Table 2) are shown in Figure 2. The rainwater runoff from the plant is included in this facility. Hence, the collecting sump is equipped with trash screens to remove large objects. The screens are cleaned manually when required. Surface-skimming baffles are used to hold the oil, which is pumped to an incinerator. The waste then is processed by slow movement through a series of baffles in a sedimentation pond, where additional oil and sediment are collected and removed. The clarified waste is filtered with sand filters that discharge to a surge tank equippped with a float flow-controller to

govern the operation of the injection pump.

The waste-disposal facility for the basic waste is designed to process 3,000 m^3 per day ($m^3\ d^{-1}$). The overdesign is intended for future plant expansion.

LOCAL AND REGIONAL GEOLOGY

The plant is about 24 km inland from the Texas Gulf Coast. The topography of the area is generally flat, having a maximum relief of about 6 m. The geologic units in this area include the Beaumont Clay at the surface and the underlying Lissie Formation, at 30-60 m. Figure 3 shows the location of the two waste-disposal wells with respect to the local geology and the oil wells in the area. The Pleistocene Lissie Formation is composed of sand and sandstone with intercalated shale and clay overlying about 3,400 m of largely undifferentiated sand, sandstone, and shale beds. These formations dip in a southeasterly direction at about 6 m/km.

Groundwater from the Lissie Formation is used extensively for municipal and irrigation purposes. Water in zones directly below the Lissie Formation contains a high concentration of sodium bicarbonate.

The geologic interval used for the injection of wastes is a loosely consolidated, fine-grained Miocene sand ranging in depth from 1,021 to 1,082 m (Fig. 4). Some of the physical properties of side-wall cores from the two wells are listed in Table 3. The permeability measurements have been rounded to the nearest 10 md for ease of interpretation. The average permeabilities are 580 and 370 md, and porosities are 33 and 32 percent, for wells 1 and 2, respectively. The pressure gradient of well 1 was 0.098 atm per meter of depth; no abnormal pressures or temperatures were recorded. The reservoir temperature and pressure were 64°C and 112 atm. Under these conditions one would anticipate considerable reaction between the injected organic compounds on a silicate-clay surface, leading to formation of polymers as well as low-molecular-weight degradation products. Hence, in time, the chemical character of the injected waste probably will be altered.

The shallowest petroleum production in the area is obtained from the Frio Formation, which is about 2,200 m deep. There is an oil-field-brine disposal operation located about 11 km north of the plant that injects about 45,000 m^3 annually at a depth of 900-1,000 m. This is the same stratigraphic zone that is used for industrial-waste injection. Several other brine-injection wells west of the plant site are completed at a depth of 600 m; the closest of these shallow injection wells is 5 km from the plant.

DISPOSAL-WELL DESIGN AND COMPLETION

Well 1 was rotary-drilled to 466 m with a 38-cm-diameter bit. Casing (27 cm OD) was set and cemented to the surface to protect the freshwater zones of the Lissie Formation (Fig. 4). Drilling proceeded to the top of the proposed injection zone at 1,036 m. The interval 1,036-1,113 m was cored, the bottomhole pressure was recorded, and several cubic meters of brine were collected from the proposed injection zone for laboratory study. The hole then was reamed from 1,036 m to total depth of 1,151 m. An 18-cm-OD, 29.8-kg/m J-55 casing string was set at 1,151 m and cemented to the surface with acid-resistant cement. A heavy-wall epoxy-fiberglass tubing, 11 cm OD, was selected as the injection string, and a 76.2-m section of Hastelloy C[4] tubing was used as the bottom section adjacent to the disposal formation. The usual completion practice at this point is to use a packer to seal the annulus between the tubing and the 18-cm casing; however, this well was completed by cementing the tubing the total distance to the surface for added mechanical strength. A 13.7-m interval was perforated adjacent to the injection zone, using 26 perforations per meter. Injectivity tests after acidizing and backflow indicated a sustained injection rate of 1.0 m^3 min^{-1} was possible at the wellhead pressure of 14 atm, which was adequate to meet the requirements for injection of the acidic waste.

Waste-injection well 2 was drilled at a site 835 m north of well 1. The purpose of this well was twofold: disposal of the basic waste described in Table 2 and accommodation of future increases in plant waste from expansion or changes in unit operations. Therefore, larger diameter casing was used throughout, and an 18-cm casing was used in place of epoxy-fiberglass tubing. Briefly, the completion program was as follows: (1) a 35-cm surface casing was set at 414 m and cemented to the surface; (2) 24-cm, 53.6-kg/m J-55 casing was set at 1,147 m and cemented to the surface; (3) the casing was jet-perforated in the waste-injection zone from 1,073 to 1,082 m; (4) 18-cm, 29.8-kg/m J-55 injection casing was set in a packer at 1,006 m, and the annulus was filled with a corrosion-inhibiting fluid. After the well was acidized, an injectivity test indicated that

[4]Reference to specific trade names is made for identification only and does not imply endorsement by the Bureau of Mines.

injection would proceed at a flow rate of 2.6 m^3 min^{-1} at a wellhead pressure of 27 atm.

OPERATING HISTORY

During the first few months of operation, it was observed that whenever well 1 was shut in for more than 24 hours the wellhead pressure was considerably higher when injection was started again. This higher pressure was attributed to a combination of the precipitation of insolubles near the wellbore and backflow of sand into the well. The problem was solved by injecting a slug of brine after every period of interrupted flow.

The operating parameters of the entire two-well waste-injection system (Fig. 5) are constantly under surveillance in a unit control room where the data are recorded for future use. Flow rate, pressure, and pH are observed at several key locations in the injection system.

Changes in the composition of wastes resulting from process changes or the installation of new units are examined for compatibility with the usual waste effluent to determine which of the two systems is best suited to process the waste. Several new process wastes that were incompatible with either system as discharged from the new processing unit were made compatible by pH adjustment and dilution.

The interpretation of the pressure interference between the two wells is based on the radial form of the diffusion equation:

$$P_w - P_e = \frac{q\,\mu}{108.5\,k\,h}\,E_i\left[-\frac{r^2\,\mu\,c\,\phi}{34.7\,k\,t}\right], \qquad (1)$$

where $P_w - P_e$ = Pressure, atm,

q_1 = Injection rate, well 1 = 1,440.6 m^3 d^{-1},

q_2 = Injection rate, well 2 = 1,640 m^3 d^{-1},

μ = Subsurface viscosity = 0.45 centipoise,

k = Permeability, geometric mean of data = 0.380 darcy,

h = Formation thickness = 61 m,

r = Radius from wellbore, m,

c = Compressibility = 9 x 10^{-5} atm^{-1},

ϕ = Porosity = 32.5 percent,

t = Time, days, and

$E_i(-x)$ = Exponential integral = ln + 0.5772.

The equation describes the pressure distribution for unsteady-state flow in a homogeneous reservoir containing a single, slightly compressible fluid. The method used to determine the time of interference between two producing wells was illustrated by Stevens and Thodos (1959). The equation is also applicable to analysis of the interference between injection wells, as illustrated by Van Everdingen (1972).

Equation 1 was used to calculate the pressure profiles generated by injection into wells 1 and 2, independently, assuming an infinite homogeneous reservoir. The results of the calculations are presented in Figure 6. The flow rates used are averages over a 5-year period. The viscosity of water at 63°C was selected as the best approximation to the actual subsurface conditions inasmuch as the waste is composed primarily of water, and viscosity data for the waste are not available. Since specific data are not available for a complete description of the reservoir heterogeneity, a geometric mean of the permeability measurements for both wells listed in Table 3 was used in the calculations. In addition, the calculations were made on the basis of the 61-m average thickness of the formation.

When injection of waste begins, pressure waves emanate from each well operating independently as single wells in an infinite reservoir. However, after the pressure waves meet, mutual interference occurs, causing an increase in the wellhead pressures. Stevens and Thodos (1959) used a pressure decrease of 1.7 atm for the pressure waves of both wells as an arbitrary reference to define the time of interference of two producing wells. They used a 1.7-atm reference pressure because no appreciable interference exists until the pressure waves are significantly greater than the original reservoir pressure.

By November 1969, when the indication of pressure interference became obvious (Fig. 5), wells 1 and 2 had been in operation for approximately 2,270 and 2,000 days, respectively. At this time, the point of intersection of the pressure waves emanating from each well showed an increase of 2.4 atm above the original reservoir pressure. Figure 6 indicates the relative distance from the wellbore of each well to the point of pressure interference in November 1969 (323.7 m from well 1). At this time, the fluid front was only about 222.5 m away from the wellbore of well 1. The points of intersection of the pressure waves were closer to well 1 because of the higher injection rate into well 2.

From a purely physical point of view, the pressure waves emanating from the wells extend over the entire reservoir immediately after injection is started. However, the detectable interference does not occur until a significant increase in the original reservoir pressure is

established.

The pressure increase at the point of interference after 6,270 days of operation of well 1 (this corresponds to about November 1980) will be displaced further toward well 1. Figure 6 also indicates that the rate of increase of the original reservoir pressure will have decreased considerably, since each well exhibits an increase of only 2.7 atm at the point of intersection after 6,270 days of operation of well 1.

The graph of the injection rates and pressures (Fig. 5) to June 1972 indicates that the wellhead-pressure increase (from 5 to 7 atm) that became obvious in November 1969 is still relatively constant.

POTENTIAL POLLUTION PROBLEMS

Pollution of the freshwater aquifers in the area as a direct result of waste injection could result in several ways: (1) leakage from the disposal well because of corroded casing and faulty completion, (2) upward migration of the wastes through the overlying strata, and (3) communication with abandoned wells.

The method of well completion and operation of the disposal well should preclude leakage from the well. As to the second possibility, the strata between the base of the freshwater aquifers and the disposal formation include about 370 m of relatively impermeable shale and clay beds with individual zone thicknesses ranging from 3 to 75 m. Therefore, the probability of upward migration of the injected fluids is extremely remote.

The most obvious problem is related to the abandoned wells in the area. The increase in the reservoir pressure caused by waste injection could be great enough to the injection wells to produce surface flow in adjacent wells communicating with the disposal zone. Pollution of the freshwater aquifers could occur by the same route if the abandoned wells are not adequately sealed. Because of this possibility, all known abandoned wells within a radius of 5 km from the injection wells were located. They consisted of 26 abandoned dry holes, four abandoned oil-producing wells, and three producing oil wells. The total depths of these holes range from 2,000 to 3,000 m. Well records were located for all but four of the wells, and these show that surface casing was set at depths ranging from 500 to 800 m, which is below the freshwater aquifers in all wells. The records also indicate that in all but two of the abandoned wells, cement plugs were placed in the bottom of the surface casing upon abandonment, and the abandoned producing wells remain cased and cemented to the surface. If surface casing was set as reported in all of the wells, freshwater pollution should not occur as long as the casings remain in satisfactory condition. However, communication of fluids above and below the disposal zone could probably occur in some of the abandoned wells owing to increased reservoir pressure and inadequate seals between the disposal zone and the abandoned wells.

ADSORPTION

The migration patterns and the ultimate fate of organic wastes injected into underground formations are difficult to predict because of the many unknown variables associated with underground formations. However, subsurface formations have extremely large adsorption surfaces, and many are known to contain populations of anaerobic bacteria, both of which are beneficial in containing and decomposing the injected wastes. In addition, the salts in the subsurface brines and other chemical constituents of the rocks serve as buffering agents and chemical reactants to aid in the conversion and decomposition of the waste compounds. If the deleterious waste compounds are indeed irreversibly adsorbed and decomposed to harmless compounds, the environmental impact of underground waste storage is significantly less in magnitude than that of chemical waste disposal by burning or discharging to surface streams or the ocean.

The equilibrium-adsorption properties of several constituents of the wastes were determined at simulated subsurface conditions in an autoclave. Samples of the waste-disposal sand were not available; hence, samples from an outcrop of the Cottage Grove sandstone near Bartlesville, Oklahoma, were obtained. The test specimens were prepared for use by cutting them to size with diamond-edge equipment and then cleaning them in a steam extractor to remove organic contaminants. The surface area of the Cottage Grove sandstone, determined by nitrogen adsorption, is 0.89 $m^2\ g^{-1}$. The dry sandstone has a density of 1.93 g cm^{-3}, porosity of 26 percent, and permeability of 0.210 darcy. Mineral analysis of the sandstone by X-ray diffraction showed that it is composed primarily of alpha quartz (SiO_2) and calcium and sodium feldspars ($CaAl_2Si_2O_8$ and $NaAlSi_3O_8$). Clay minerals were separated and identified as kaolinite and illite. Concentration of kaolinite was much higher than that of illite.

The cleaned sandstone cylinders, 2.0 cm in diameter by 3.5 cm in length, were dried, evacuated, and saturated with aqueous solutions of the organic compounds and immediately subjected to 60°C and 200-atm pressure. At various time intervals the test samples were removed from the autoclave and centrifuged, and the fluids were analyzed by gas chromatography to detect the amount of organic constituent remaining in solution. The difference in concentration between the original and extracted sample represents the amount adsorbed.

The adsorption data collected from these experiments were plotted as a function of time. The procedure is illustrated in Figure 7, in which the total amount of n-hexylamine adsorbed is plotted as a function of time and the initial solution concentration. The equilibrium amounts adsorbed, as defined by the plateau of the curves in Figure 7, were used to construct the Freundlich isotherms shown in Figure 8.

The Freundlich isotherms provide a convenient mehtod for determining the amount of rock necessary to reduce the concentration of the contaminant to a particular level at equilibrium. These data may be used to estimate the radius of migration of the contaminant from the injection well with respect to the total volume of waste injected.

The influence of the adsorption characteristics of the contaminant on a waste-injection system may be estimated by material-balance calculation considering the properties of the geologic material. As a hypothetical case, consider the adsorption that would occur if a 0.1-percent solution of n-hexylamine were injected into a 100-m-thick zone of the Cottage Grove sandstone. After 20 years of continuous injection at 2,000 $m^3\ d^{-1}$, the injected fluid front (r_1) would be 423 m away from the wellbore. However, the extent of the migration of n-hexylamine (r_2) would be only 231 m from the wellbore. The same estimates for butanal, butanol, and phenol yield r_2 equal to 406, 413, and 418 m, respectively.

CONCLUSIONS

Injection of the industrial wastes in the two-well system solved a very difficult waste-disposal problem for the company in question within the confines of current legislative regulations. The two noncompatible wastes are handled with a minimum of surface treatment by using a two-well system.

An obvious advantage of a two-well injection system is that a single well may be able to handle the waste injection if one of the wells must be removed from service for maintenance. In this event, the combined wastes may require additional surface treatment. A second advantage of the two-well system is that it offers greater flexibility of operation and is more adaptable to plant expansion and changes in operations.

Pressure interference will occur when a two-well injection system is completed in the same formation. Therefore, the wells should be spaced as far apart as practical in order to minimize interference, which will create higher injection pressures at both wells.

Abandoned and improperly plugged wells in the vicinity of a waste-injection well present a serious source of potential pollution problems.

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Van Everdingen, A. F., 1972, Fluid mechanics of deep-well disposals, in John E. Galley, ed., Subsurface disposal in geologic basins--a study of reservoir strata: Am. Assoc. Petroleum Geologists Mem. 10, p. 32-42.

Table 1. Composition of Waste Injected into Well 1

	Concentration, ppm
Organic constituents:	
Acetaldehyde	1,000
Acetaldol	900
Acetic acid	5,000
Butanol-1	1,300
Butyraldehyde	1,100
Chloroacetaldehyde	800
Crotonaldehyde	400
Phenol	1,200
Propionic acid	2,200
Inorganic constituents:	
Calcium	100
Iron	10
Magnesium	50
Sodium	3,000
Chloride	2,000
Sulfate	100
General properties:	
Total hardness, ppm	350
Conductance, micromhos	10,000
pH	5
Temperature, °C	70
Color: Dark brown	
Odor: Strongly pungent	
Average daily composite rate, m^3	1,440

Table 2. Composition of Waste Injection into Well 2

Organic constituents:	Concentration, ppm
Amyl alcohol	600
Cyclohexane	200
Dodecane	100
Hexanol	1,000
1-Hexylamine	1,400
1,6-Hexylamine	300
Methanol	200
Valeric acid	900
Inorganic constituents:	
Ammonia	1,400
Copper	80
Manganese	50
Sodium	5,000
Vanadium	20
Bicarbonate	2,200
Carbonate	500
Nitrate	7,500
Nitrite	4,600
General properties:	
pH	9
Temperature, °C	50
Color: Pale yellow, cloudy	
Odor: Stale fish	
Average daily composite rate, m^3	1,640

Table 3. Side-wall Core Analysis of Waste-Injection Wells 1 and 2

Interval, meters	Permeability, millidarcys	Porosity, percent
	Well No. 1	
1,068.9 - 1,070.4	190	27
1,070.4 - 1,071.9	690	33
1,071.9 - 1,073.4	880	37
1,073.4 - 1,074.9	410	34
1,074.9 - 1,076.4	750	34
1,076.4 - 1,077.9	460	35
1,077.9 - 1,079.4	340	32
1,079.4 - 1,080.9	1,400	35
1,080.9 - 1,082.4	130	33
Averages	580	33
	Well No. 2	
1,046.7 - 1,048.2	270	32
1,048.2 - 1,049.7	210	31
1,049.7 - 1,051.2	440	32
1,051.2 - 1,052.7	150	26
1,052.7 - 1,054.2	540	33
1,054.2 - 1,055.7	700	33
1,055.7 - 1,057.2	220	31
1,057.2 - 1,058.7	480	33
1,058.7 - 1,060.2	650	34
1,060.2 - 1,061.7	170	30
1,061.7 - 1,063.2	260	32
Averages	370	32

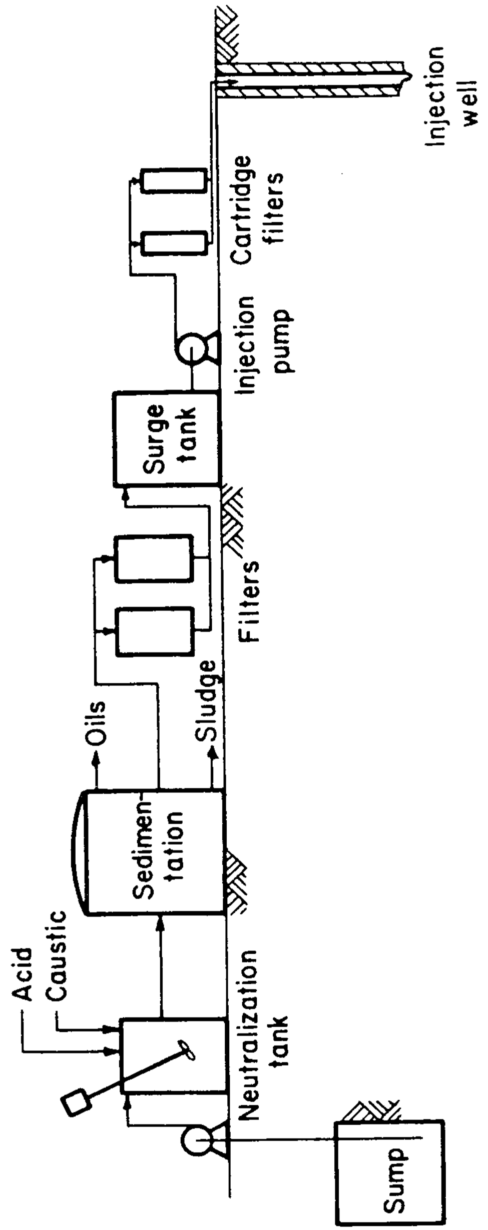

FIG. 1--Preinjection waste-treatment facilities for acid waste.

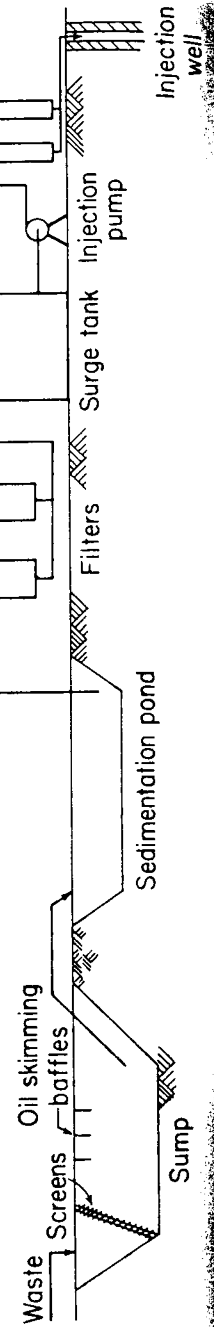

FIG. 2--Preinjection waste-treatment facilities for basic waste.

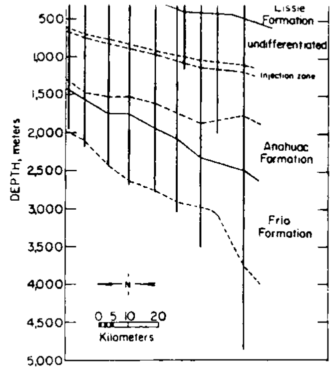

FIG. 3--North-south cross section showing oil wells and inclination of major formations.

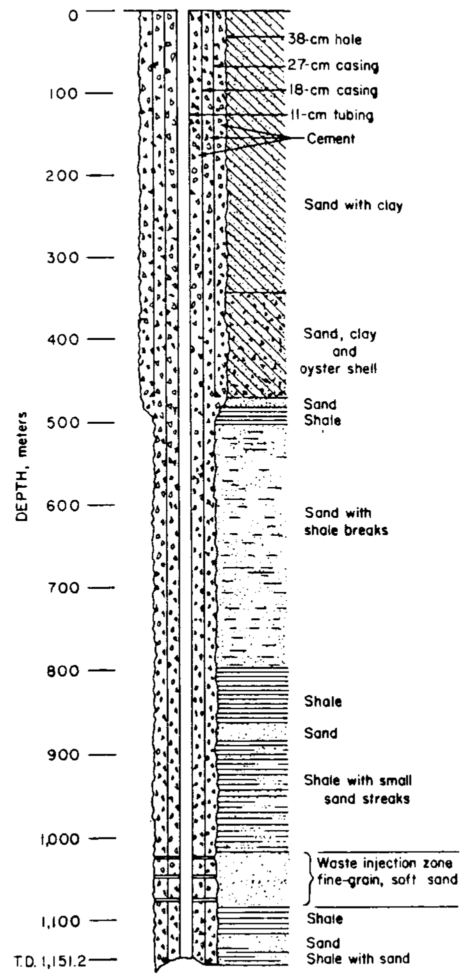

FIG. 4--Details of geologic strata and design of well no. 1.

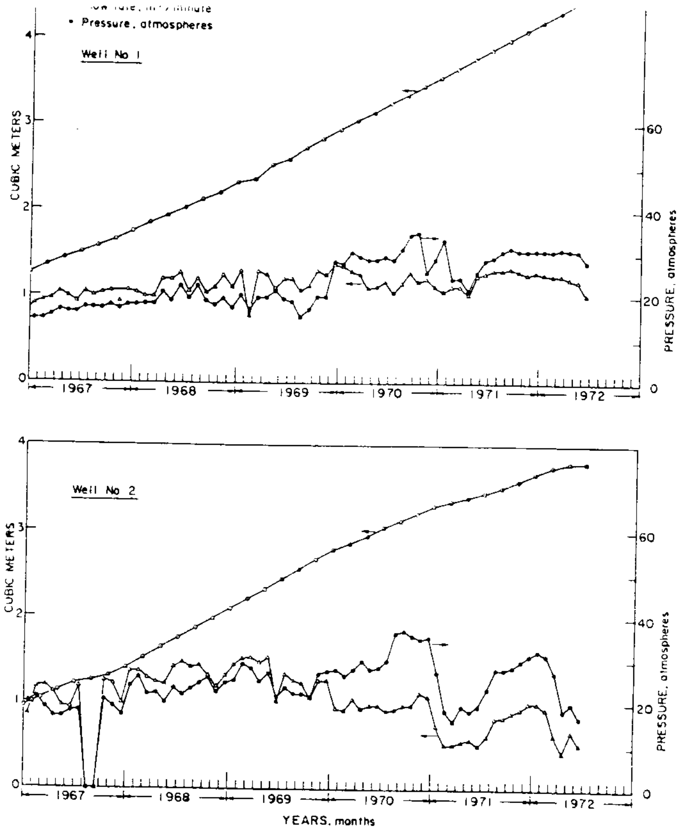

FIG. 5--Operating histories of waste injection wells no. 1 and 2.

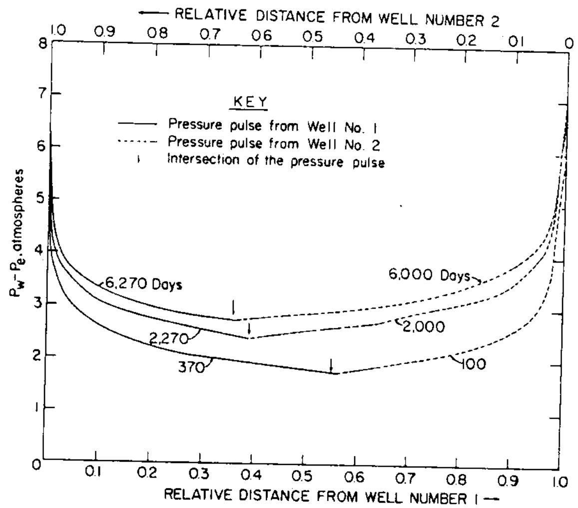

FIG. 6--Pressure interference between two injection wells spaced 835 m apart.

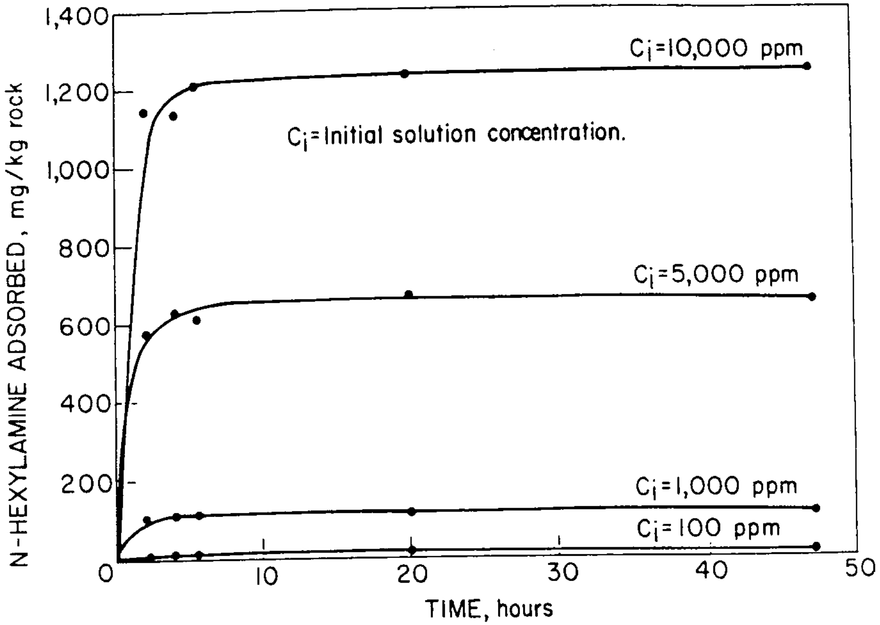

FIG. 7--Rate of adsorption of n-hexylamine on sandstone at 66° C and 200 atmospheres.

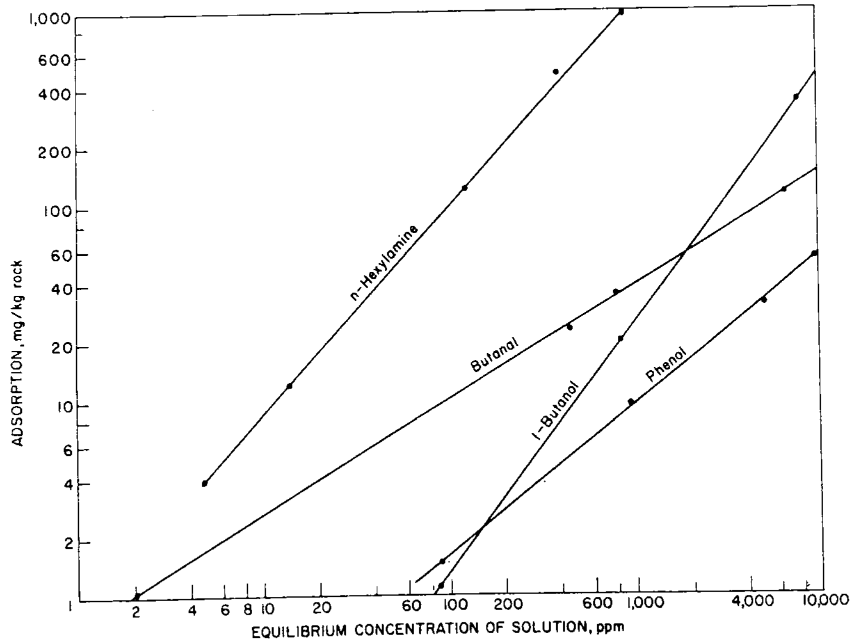

FIG. 8--Freundlich isotherms of several waste constituents on sandstone at 66° C and 200 atmospheres.

SUBSURFACE DISPOSAL OF WASTE IN KANSAS[1]

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ABSTRACT The use of wells for the subsurface disposal of wastes has been practiced in Kansas since 1935. All of the early waste-disposal wells were used to dispose of oil-field brine. Permits for the first industrial-waste disposal wells other than oil-field wells were issued in 1952.

Two state agencies have jurisdiction over the subsurface disposal of oil-field wastes in Kansas. Before using a well for the subsurface disposal of oil-field or gas-field brines, the operator must submit plans and specifications for each disposal well to the Kansas State Corporation Commission. These must be approved by both the State Corporation Commission and the Kansas State Department of Health. The Corporation Commission, through its Oil and Gas Conservation Division, determines that the use of the proposed well will not result in loss or waste of gas or petroleum resources. The Department of Health, through its Oil Field and Subsurface Disposal unit, Division of Environmental Health, determines that use of the proposed well will not result in pollution to the water resources of the state.

Before using a well for the subsurface disposal of industrial wastes other than oil-field or gas-field wastes, an application must be filed with, and a permit issued by, the Kansas State Department of Health. The maximum wellhead pressure that may be used to inject wastes into both classes of wells must be approved by the Department of Health.

At present, there are 3,200 approved oil-field and gas-field saltwater-disposal wells in use, receiving a total of about 3.5 million bbl of salt water per day. Not included are 2,550 saltwater-repressuring systems in the state which consist of from one to several hundred injection wells. The depths of the oil- and gas-field disposal wells range from less than 500 ft to about 6,000 ft. Thirty-eight zones ranging in age from Late Permian to Precambrian are being used. About 60 percent of the wells inject wastes into limestone or dolomitic rocks and 25 percent into sandstones. Lithology of other disposal formations is sandy or gypsiferous shale, salt, anhydrite, conglomerate, and "granite wash." Injection pressures being used range from gravity at the wellhead to 0.5 psi per foot of depth to the injection zone.

There are 30 industrial-waste disposal wells (other than oil- or gas-field wells) at 20 plants in the state which include 10 LPG underground-storage projects, two salt companies, two petroleum refineries, four natural-gas compressor stations, one chemical-manufacturing plant, and one fertilizer plant. Most industrial waste being disposed of in the subsurface consists of salt brine, which is introduced into Arbuckle rocks at depths ranging from about 3,000 to 6,000 ft. All wells are constructed so as to preclude any hazard to fresh water. The Department of Health's present policy concerning industrial wastes is that only those wastes that cannot be treated and disposed of by other practical methods will be considered for disposal in the subsurface.

Experience with both industrial and oil-field disposal wells shows that most operational problems are caused by (1) selection of an injection zone with inadequate permeability, (2) lack of preliminary waste treatment or inadequate treatment, or (3) failure to provide an effective maintenance program.

[1]Manuscript received, May 29, 1973.

[2]Kansas State Department of Health.

INTRODUCTION

The use of the subsurface for the disposal of oil-field brine was first permitted in Kansas about 38 years ago, but only after serious pollution had resulted to both surface water and groundwater from the uncontrolled discharge of brine to streams and ponds. Although subsurface disposal certainly has not been free of problems, the fact remains that the use of this method of disposal permitted oil production to be continued without causing the serious widespread pollution previously experienced. Surface waters and groundwaters that were once heavily polluted in the older oil-producing areas are now of satisfactory quality for most uses.

Subsurface disposal into approved formations was, and still is, considered the most feasible method for handling oil-field brines and certain other kinds of industrial liquid waste, where it is done with proper planning and controls. It is Kansas policy, however, that subsurface space be reserved for those wastes that cannot be treated practically and returned to the environment.

Although Kansas has an extensive program for controlling the disposal of solid wastes in sanitary landfills, the discussion in this paper will be limited to subsurface disposal of liquid wastes through wells.

As the result of statutory differentiation, disposal wells are classified as oil-field disposal wells or industrial-waste disposal wells. Oil-field disposal wells are those used on producing oil or gas leases for the disposal of the brine brought to the surface with oil or gas. All other wells are classed as industrial-waste disposal wells.

HISTORY OF WASTE DISPOSAL

The first waste-disposal wells in Kansas were used to dispose of oil-field brine. Although oil was discovered there prior to the Civil War, actual production did not start until around 1890. After 45 years of uncontrolled brine discharge to ditches, streams, or ponds, the Kansas Legislature, in a special session in 1934, enacted laws permitting the return of oil-field brine to any subsurface formation that already carried highly mineralized water and also allowing the oil operator to use his produced brine for repressuring oil zones. The first year after these laws were enacted, 24 disposal wells and 21 repressuring wells were constructed.

Permits for two industrial-waste disposal wells were issued in 1952--the first such permits to be issued in Kansas. Both of these wells were used to dispose of brine. One was at a natural-gas compressor station and was used for the disposal of the brine solution from the regeneration of a zeolite water softener. The other was used to dispose of a saturated brine solution resulting from the solutioning of salt to develop storage caverns at an LPG underground-storage project.

STATE CONTROLS

An act passed by the State Legislature in 1934, which became law in 1935, gave the Kansas State Corporation Commission control over the subsurface disposal of oil- and gas-field brine. This provision was repealed in 1945 and a more comprehensive law enacted.

The 1945 law gave regulatory authority over the disposal of waste produced in the oil and gas fields of Kansas to two state agencies--the

Kansas State Corporation Commission and the Kansas State Department of Health. The Corporation Commission, through its Conservation Division, although concerned primarily with the protection of the oil and gas resources, is also concerned with freshwater protection. The Department of Health, through the Oil Field and Subsurface Disposal unit, Water Quality Section, Division of Environmental Health, is concerned solely with the protection of fresh and usable water resources.

State law requires that an application for an oil- or gas-field disposal well, along with plans and specifications, shall be submitted to and be approved by the Corporation Commission and the Department of Health. The law states that:

> The state corporation commission, in giving its approval, shall determine that the proposed method of disposal will not result in the loss or waste of gas or petroleum resources. The state board of health, in giving its approval, shall determine that the proposed method is a feasible method to be employed in protecting the water resources of the state from preventable pollution.

In practice, applications for oil- and gas-field disposal wells are submitted to the Corporation Commission. The Corporation Commission transmits a copy of each application to and receives approval from the Department of Health before issuing a permit.

The maximum amount of pressure at which brine may be injected into disposal wells must be approved by the Department of Health and is indicated on the approval issued by the Corporation Commission. The maximum wellhead injection pressure that will be allowed for any saltwater-disposal well has been set at 0.5 pound per foot of depth to the injection zone. In some cases, less than 0.5 pound per foot, down to gravity pressure only, is allowed. Field personnel of the Department of Health check wells periodically for possible pressure violations. Most disposal wells operate under gravity pressure.

In 1957, the State Legislature passed a statute making it the duty of the State Department of Health, State Water Resources Board, and the State Geological Survey to determine the minimum safe depth for the disposal of salt water or other oil-field wastes for all producing areas of the state. A committee of representatives of these agencies, known as "Three-Agency Committee," was established to fulfill this duty. Minimum acceptable disposal depths were established for each county in the state. These are contained in a table as part of the rules and regulations of the Corporation Commission. Over the years these established minimum depths have been changed from time to time as new data have become available. These are absolute minimum depths that may be approved. In some cases, the Department of Health and the Corporation Commission may require the use of zones at greater depths.

Authority for controlling the discharge of municipal or industrial sewage into "waters of the state" was given to the State Board of Health by the State Legislature long before the first industrial-waste disposal well was put into use. Sewage, as defined in the statutes, includes chemical or other wastes from domestic activities as well as manufacturing or other forms of industry. In 1952, rules and regulations were adopted by the State Board of Health defining industrial-waste disposal wells, other than oil-field wells, and requiring permits for their use. Applications, including plans and specifications, are filed with the State Department of Health. The following administrative policy pertaining to industrial-waste disposal wells was adopted in 1970:

> The use of industrial-waste disposal wells will be considered only for those wastes that cannot be treated and disposed of by other methods. On this basis, we will require that each application for the disposal of substances other than salt water be accompanied by a report giving the results of studies of alternate methods of waste disposal and a justification of why subsurface disposal is considered the least hazardous method so far as environmental protection is concerned.
>
> The applicant will be expected to use the best and safest disposal formation available. In most areas, this will be one of the deeper formations such as the Arbuckle. Wellhead pressure will be limited to gravity in most cases.

STATUS OF WASTE DISPOSAL IN KANSAS

Since oil-field-brine disposal wells were first allowed, 4,960 such wells have been constructed and used in Kansas. Today, there are 3,200 oil-field disposal wells in use receiving approximately 3.5 million bbl of brine daily. Not included are about 2,550 saltwater-repressuring systems which have from one to several hundred injection wells. Well failure due to plugging, casing corrosion, or some other cause has been responsible for the abandonment of some of the disposal wells, but most were abandoned as the result of the depleted production of the oil wells that were the source of the brines.

Thirty-eight different zones ranging from the Upper Permian to the Precambrian are being used for brine disposal in the oil fields of Kansas. About 60 percent of the wells inject wastes into limestone or dolomitic rocks, 25 percent into sandstones, 5 percent into sandy or gypsiferous shale, 5 percent into salt or anhydrite zones, and 5 percent into conglomerate or "granite wash." Depths of the wells range from 200 ft to 6,315 ft. Oil and gas production in Kansas is found at widely varying depths

ranging from less than 200 ft in parts of southeast Kansas to 6,700 ft in the southwest part of the state.

It is estimated that only about 20 percent of the oil-field disposal wells were drilled specifically for disposal use. Most of them are dry exploratory wells or abandoned production wells that have been converted for disposal use. In a typical completion, surface casing is set and cemented, bottom to top, through all freshwater zones; conductor casing is set and cemented in the lower 200-500 ft or cemented bottom to top; and tubing and packer are run, with the packer being set just above the disposal zone and a noncorrosive fluid placed in the annulus between the tubing and the conductor casing.

Although most wells are being used only for disposal, a few are dually completed; that is, they are completed for both brine disposal and oil production from different zones by the use of one or more packers. In most such cases, disposal is in a zone below the producing zone. For example, several wells produce from the Lansing - Kansas City or Mississippian and dispose of the produced brine several hundred feet deeper into the Arbuckle Group via the same well.

In the past, a few wells were used in which the brine was disposed of down the annulus between the cemented surface casing and the production casing of a producing oil well. This type of completion is no longer permitted.

During the 20 years since the first industrial-waste disposal-well applications (other than oil- and gas-field disposal wells) were received, 44 permits for industrial-waste disposal wells have been issued. Of this number, four were never constructed, eight have been abandoned, and two have not yet been constructed or put into use. There are now 30 active wells. Table 1 lists these active wells by type of industry and character of waste received.

Of the 30 existing industrial-waste disposal wells, 25 are disposing of waste into Arbuckle dolomite (Upper Cambrian and Lower Ordovician) at depths ranging from 3,300 to 6,300 ft; two discharge into Pennsylvanian limestones at depths of 3,900 and 4,400 ft; one discharges into the Upper Permian Glorieta Sandstone at 1,065 to 1,100 ft; and two discharge into a lost-circulation zone in a salt bed of the Wellington Formation (Upper Permian) at depths between 220 ft and 420 ft.

There is no way to estimate the average volume of waste being disposed of in wells, because many of the wells are used intermittently and at irregular periods. Most of the 17 wells at the LPG underground-storage projects are used for disposal only while new storage wells are being "washed in salt," and are rarely used during the routine operation of the storage project. When they are used, they receive from several thousand to more than 40,000 bbl of sodium brine waste per day.

The other 13 disposal wells are used regularly and receive approximately 53,000 bbl of waste per day, of which about 22,000 bbl is sodium brine water and 31,000 bbl is industrial wastes.

Only two of the 30 industrial-waste disposal wells use wellhead injection pressure. Both are deep Arbuckle wells and require pressures of 150 psi or less at the wellhead.

All industrial-waste disposal wells are constructed to protect fresh and usable water. Most have surface casing set and cemented through all fresh and usable water zones. Conductor casing is set to or through the disposal zone, and cement is circulated from the bottom to the surface. Injection tubing is used in the conductor casing, and a packer is set just above the disposal zone. The annulus between the injection tubing and the casing is filled with a noncorrosive fluid.

DISPOSAL POTENTIAL IN KANSAS

Although several geologic zones in Kansas can be used satisfactorily for the disposal of small to moderate volumes of industrial waste, the most satisfactory for accepting large volumes of waste is the Arbuckle zone. Wherever possible, this is the zone recommended for disposal. In many parts of the state, Arbuckle rocks are several hundred feet thick, generally have good permeability, and generally will accept large volumes of waste under gravity conditions. As a result of its being an important oil-producing zone in the upper part and having been used extensively for over 35 years for oil-field brine disposal, much information is available concerning the potential of the Arbuckle for industrial-waste disposal.

Arbuckle rocks are found in the subsurface everywhere in Kansas except where they have been removed by erosion--i.e., at the crests of the Cambridge arch in northwestern Kansas, the Central Kansas uplift, and the Nemaha anticline in northeastern Kansas (Fig. 1). In the extreme southeastern part of the state in Cherokee, Crawford, and Labette counties, the Arbuckle is found at less than 1,000 ft below the surface and carries fresh or usable water; thus, over a large area of this part of the state, disposal in the Arbuckle is not permitted. In fact, in this area there are no acceptable zones for waste disposal.

The Arbuckle consists mainly of dolomite, sandy dolomite, and vuggy

limestone and dolomite; large amounts of chert are present in the upper part. The thickness ranges from a featheredge near where the deposits have been removed by erosion to more than 1,200 ft in parts of southwest Kansas. Depth to the Arbuckle is from about 400 ft to more than 7,500 ft. The depth ranges from about 400 to 2,000 ft in eastern Kansas; 3,000 to 4,000 ft in the central and south-central part; 4,000 to 5,000 ft in the northwest part; and 6,000 to 7,500 ft in the southwest part of the state.

The greatest porosity and permeability are found where the Arbuckle strata have undergone erosion on, and along the flanks of, uplifted areas. Input capacities of Arbuckle wells at gravity pressure have, in several wells, exceeded 40,000 bbl of fluid per day. During a 3-hour test, one well took fluid at the rate of 3,000 bbl/hour with 23 in. of vacuum at the wellhead. This well had an open hole in Arbuckle rocks from 3,974 ft to 4,371 ft, with a static fluid level of 600 ft below the surface.

It is anticipated that most subsurface disposal of industrial wastes in Kansas will be in Arbuckle rocks. All existing industrial-waste disposal wells are in the central, south-central, or southwest parts of the state.

OPERATIONAL PROBLEMS

Experience with both industrial and oil-field disposal wells in Kansas shows that most operational problems are caused by (1) selection of an injection zone with inadequate permeability; (2) absence of preliminary waste treatment or inadequate treatment, or (3) failure to provide an effective maintenance program.

The attempted use of poor injection zones has been more common with oil-field disposal wells than it has with industrial-waste disposal wells. Most industrial-waste disposal wells have been planned, and the drilling and construction supervised by, competent geologists or engineers experienced in the development of such wells, whereas many oil-field disposal wells belonging to individual operators or small independent companies have been constructed under the supervision of a local field man experienced in oil production but not in water disposal, and perhaps guided more by economy than by engineering principles.

In specific cases, the selection of a poor injection zone has been due to (1) use of a log of a nearby hole for picking the interval to be perforated rather than using a log of the well itself; (2) inadequate testing of the input capacity of the zone before the well is put into service; and (3) failure to obtain and use data on the hydrostatic head of the fluid in the proposed disposal formation. The success of a well over a long period of time depends on selecting a disposal formation that has wide areal distribution and good porosity and permeability, and one in which the existing fluids are under low hydrostatic pressure.

Although most of the oil-field brines in Kansas and the sodium chloride brine wastes from LPG underground-storage projects and salt-producing companies may be injected without treatment other than settling, some oil-field brines and industrial wastes must be treated prior to injection.

Probably the most common problem of this nature in the oil fields is the presence in some brines of dissolved iron and hydrogen sulfide. If not properly treated prior to injection, these constituents result in the build-up of iron and iron sulfide precipitates in the well casing and at the formation face, completely plugging or greatly reducing the input capacity of the well. Aeration and settling ponds or chemical treatment and filtration usually solve this problem. In some cases, a closed system is successful in preventing precipitation of the iron.

The treatment of some industrial wastes to make them compatible with fluids in the disposal formation is more complex and requires the services of chemists experienced in the field. One industry developed a disposal well in a limestone formation of Pennsylvanian age for cooling-tower blow-down and zeolite water-softener backwash wastes. Within a year, the formation was plugged, so the well was deepened to the more permeable Arbuckle. However, in a short time the input capacity of the well started decreasing as before. A study showed that chromates and phosphates in the waste water, used as corrosion inhibitors, resulted in the formation of precipitates when they came into contact with barium sulfate, hydrogen sulfide, and soluble iron in the Arbuckle water. A treatment program, the drilling of another well, and a regular program of acidizing the wells have solved the problem.

The kind and frequency of maintenance a well needs varies considerably depending on the volume and character of the wastes and on the character of the disposal formation itself. By maintaining accurate records of a well's input capacity and injection pressures, an effective maintenance program can be developed. Many oil companies have set up regular maintenance programs for their disposal wells. Most programs involve physically cleaning the well out and treating it with acid, detergent, or other chemicals at regularly scheduled periods. Such planned maintenance keeps the input capacity high and greatly extends the life of the well. In general, wells completed in limestone and dolomite require less frequent attention

than do wells in sandstone.

In reply to a questionnaire, 15 industrial-waste disposal-well owners, reported they had experienced no operational problems with their wells. Four reported they had experienced a decrease in the input capacity of their wells and needed to clean out and acidize at regular intervals. Chemical corrosion of the tubing and casing resulted in the failure of two wells at one plant. This problem has not recurred since new wells were constructed and equipped with a different kind of tubing.

CONCLUSION

It is believed that, if a satisfactory disposal zone is chosen, if injection pressure is limited, and if proper well construction is used, subsurface disposal is a practical and feasible method of handling certain types of industrial wastes to prevent pollution of fresh and usable water in Kansas. Because of the "if's," subsurface disposal must be regulated with strict controls over construction and operation.

Table 1. Number of Disposal Wells by Type of Industry

Type of Industry	Number of Wells	Character of Waste
LP Gas Underground Storage (10)[1]	17	Mostly sodium brine solution.[2]
Natural Gas Compressor Station (4)	4	Sodium brine solution.[3]
Natural Gasoline Plant (2)	3	Sodium brine solution[3] and cooling tower blowdown fluid.
Chemical Manufacturing (1)	3	Liquid chemical waste from chlor-alkali manufacture.
Salt Mining (Hydraulic) (2)	2	Sodium brine solution.
Ammonia Fertilizer Manufacturing (1)	1	Nitrate-Ammonia process fluid and cooling tower blowdown fluid.

[1]Number of industries using disposal wells.
[2]From solutioning of salt formation to develop gas storage "wells".
[3]From regeneration of zeolite water softener.

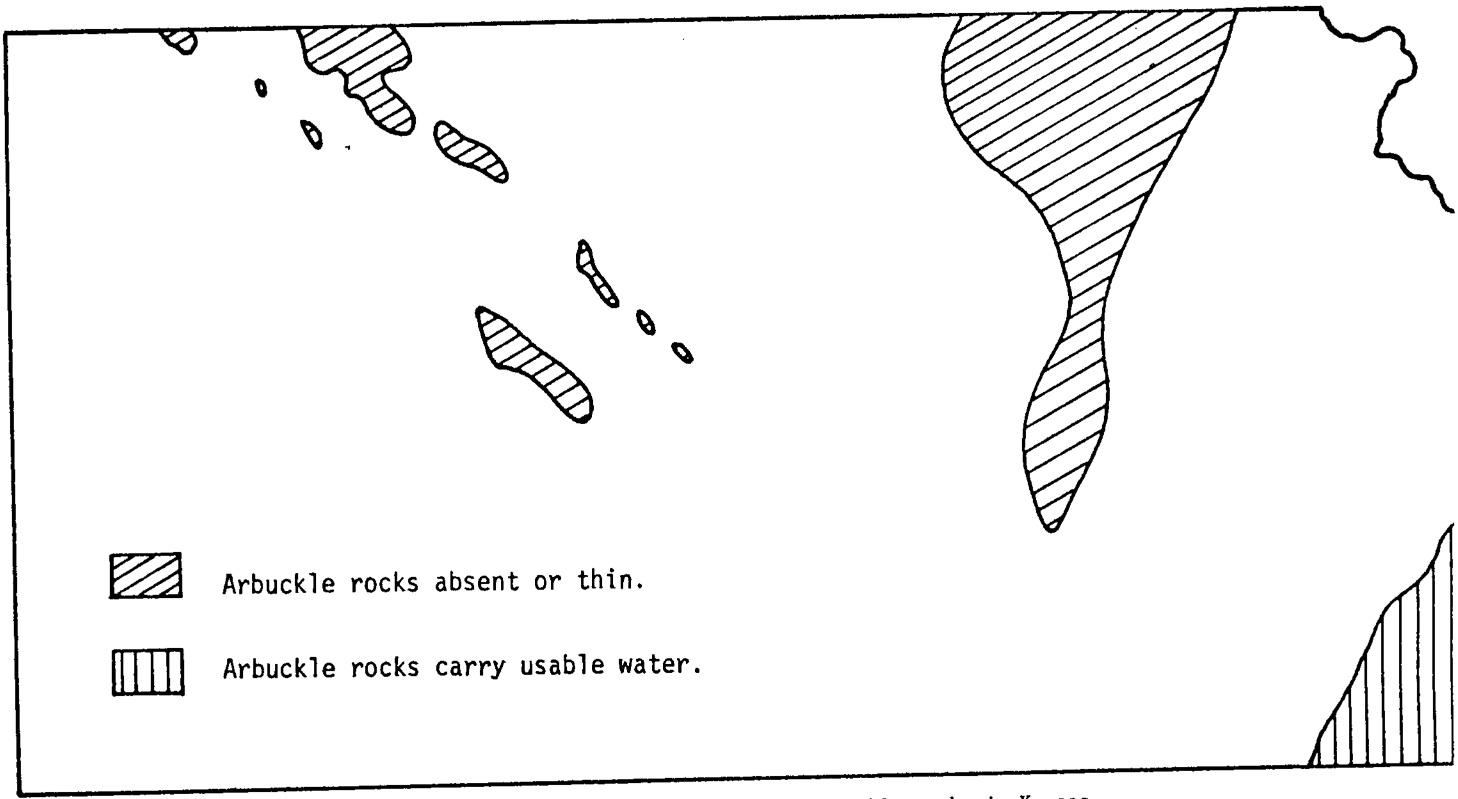

FIG. 1--Subsurface distribution of Arbuckle rocks in Kansas.

SITE INVESTIGATIONS FOR A BEDDED-SALT PILOT PLANT IN PERMIAN BASIN[1]

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ABSTRACT High-level radioactive waste contains long-lived nuclides that require complete confinement for long periods of geologic time. Rock salt long has been acclaimed as the preferred geologic medium for the ultimate disposal of these wastes as its unique self-healing properties make it impervious to the intraformational circulation of groundwaters. Data concerning the nature and extent of all major salt deposits in the conterminous United States have been compiled; however, specific site studies have been confined to the Permian basin and, in particular, to central Kansas and to a large tract of federally owned land in southeastern New Mexico. A series of coreholes has been drilled to provide the critical data for selecting appropriate disposal levels and for assessing the hydraulic characteristics of the overlying and underlying formations. Selective parts of the cores have been analyzed to determine the quantities and characteristics of various minerals in the evaporite section and, in particular, of any hydrated minerals and rocks that could dehydrate upon heating due to radioactive decay of the wastes. Measurements of the physical properties of these rocks also have been made to calculate their deformational behavior.

Because of the need for long-term waste confinement, several unique studies have been initiated. The long geologic history of relative quiescence coupled with data on historic earthquakes strongly suggests that the Permian basin will continue to be tectonically stable for the next few hundreds of thousands of years or for the effective lifetime of the wastes. In addition, studies of subsurface salt dissolution show that the rate of basinward migration of the relatively shallow edges of the salt has, during Quaternary time, averaged only a few miles per million years. Also, present rates of denudation in central Kansas have been found to average less than 1 ft per 1,000 years and stream incisions in the same area during Quaternary time have not exceeded several hundred feet. Finally, investigations have revealed that the buried wastes would not be adversely affected by the advance of a new continental ice sheet.

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INTRODUCTION

In 1955 a committee to consider the possibilities of disposal of high-level radioactive wastes in the United States was established by the National Academy of Sciences-National Research Council at the request of the U.S. Atomic Energy Commission. After some study, the NAS-NRC committee reported that "the most promising method of disposal of high-level wastes at the present time seems to be in salt deposits. The great advantage here is that no water can pass through salt" (Committee on Waste Disposal, 1957). To determine the feasibility of utilizing salt deposits for waste disposal, the committee recommended that investigations be conducted specifically to define the structural limitations of rock salt, to study the heat-transfer and thermal-effects problem, and to examine the economics of such a disposal scheme. It has since been demonstrated through extensive laboratory studies and field tests that salt deposits are practicable repositories for high-level wastes. The remaining task is to construct a pilot-plant repository in rock salt that will provide for the eventual permanent disposal of the anticipated large volumes of high-level wastes from the nuclear-power economy. This facility will be located in an area which is tectonically stable and which contains geographically extensive, thick, and relatively flat-lying beds of rock salt. The Permian salt basin, lying within the stable interior of the United States and underlain by thick deposits of flat-lying salt, has been studied specifically for potentially acceptable repository sites. Some of the more important geohydrologic characteristics of the salt basin that affect the long-term containment of radioactive waste at specific study areas are discussed herein.

CHARACTERISTICS OF RADIOACTIVE WASTES

Radioactive wastes are undesirable but inevitable products of the development and use of nuclear energy. In the past few decades significant quantities of radioactive wastes have been generated in building the nation's nuclear-weapons arsenal. In addition, much smaller quantities of wastes have accumulated from a myriad of nuclear-research activities and from various medical and industrial uses of radioactive materials. The most formidable wastes, however, are those anticipated from the rapidly expanding nuclear-power economy. Figure 1 is a simplified diagram of the fuel cycle for light-water, electricity-generating reactors. After mining, the uranium is converted to the hexafluoride for U^{235} enrichment at gaseous diffusion plants. The material is then prepared in oxide form suitable for use in fuels and shipped to plants for fabrication into reactor-fuel elements. Nuclear fission of the fuel in the reactor provides the thermal power for the generation of electricity, but the reaction also creates the fission products that comprise the bulk of the radioactive wastes generated in the fuel cycle. The lifespan of fuel elements in power reactors is about 3 years, after which time they are removed and transported to reprocessing facilities. There the spent fuel elements are dissolved and the "unburned" uranium and newly generated plutonium are extracted, leaving a residue of intensely radioactive fission-product wastes. At the present time only the U^{235} isotope is recycled; however, within the next few years it is anticipated that the recoverable plutonium (as well as fissionable U^{233}, which is not shown in Figure 1) will be utilized as fuel. In the fission-product-actinide separation process, small but significant quantities of Pu^{239} and other long-lived transuranium isotopes are unrecoverable and remain in the fission-product waste residues. The impact of these contaminants on the disposal of the fission-product wastes is indeed profound, as the time required for decay to innocuous levels for fission products alone is a few hundred years, whereas that of the fission-products-transuranium mixture is a few hundreds of thousands of years. Ultimate disposal schemes for these wastes, such as burial in rock-salt formations, must therefore provide complete containment for long periods of geologic time.

Based on current projections of the nuclear-power economy through the year 2000, estimates have been made of the expected quantities of the related radioactive wastes. In Table 1, which summarizes these power projections and waste estimates, it is observed that the installed nuclear electricity-generating capacity will increase from some 150,000 megawatts

(MW) in 1980 to about 940,000 megawatts by the end of the century. Note, too, the marked increases in the generation rates and accumulated volumes of wastes that occur during this same time span. The volumes of wastes generated annually will increase from 9.7×10^3 cu ft in 1980 to 58×10^3 cu ft by the year 2000. About 770×10^3 cu ft of wastes will have accumulated by the year 2000, representing about 1000 megawatts of thermal power and containing some 270,000 million curies of beta activity. Estimated quantities of some of the more significant isotopes are also given in the table. The 1.7 million curies of Pu^{239} with a half-life of 24,400 years is especially noteworthy because its presence as a contaminant in the high-level fission-product wastes increases the effective lifetime of the wastes to several hundreds of thousands of years and thereby affects greatly the conditions for disposal.

BEDDED-SALT PILOT-PLANT CONCEPT

Figure 2 is an illustrative drawing of a model waste repository in bedded salt. The wastes are to be transported in specially designed rail cars to the topside facilities at the repository. Here, after inspection and monitoring, the individual containers of waste, which range up to 1 ft in diameter and 10 ft in length, are lowered down the charging shaft to the desired disposal level within the salt formation at a depth of about 2,000 ft. A shielded carrier is then used to transport the containers through the underground workings to burial holes drilled in the mine floor. It is planned to operate the facility initially as a pilot plant with the capability of complete retrievability of all waste containers. Eventually, after all safety-related aspects of the scheme have been adequately demonstrated, a full-scale repository will be developed at the site of the pilot plant or possibly at some other location. The full-scale repository which covers an area of about 1,000 acres is designed to accommodate all of the nation's high-level radioactive wastes for about 30 years.

DISTRIBUTION OF ROCK-SALT DEPOSITS IN UNITED STATES

In selecting a site for the bedded-salt pilot plant, investigations have been made of the nature and extent of all the major salt deposits in the conterminous U.S. Figure 3 is a map of the country showing the occurrences of rock salt. In the illustration it is seen that salt is indeed an abundant commodity with deposits underlying some 24 of the 50 states. In the geologic past, these areas were downwarped desiccative basins where salt and other evaporites were laid down. In some places, such as the Gulf Coast embayment, the salt rocks subsequently have been buried at extremely great depths whereas, in other areas, such as the Permian basin, the evaporite sequences have remained relatively near to the land surface. The depth of these deposits is an important factor in site selection because it is impractical to make and maintain excavations for a waste repository in salt beds at depths greater than about 3,500 ft. For this reason the bedded deposits in the Gulf Coast embayment, which are all greater than 4,000 ft deep, were excluded from further consideration, as were all the salt beds in the Williston basin and large portions of the deposits in the Appalachian and Michigan basins (Salina salt basin) of the northeastern states. The salt deposits in southern Florida were also rejected for repository use because they lie at depths of 11,000-12,000 ft. In the western states, rock salt is known to exist at moderate depths in at least parts of the Paradox, Sevier, Supai, Green River, and Virgin River basins. Also, salt beds exist at favorable depths and thicknesses at the southern and northern tips of the Michigan basin and in a portion of the Appalachian basin in western New York state. Finally, some of the more than 300 salt domes in the Gulf Coast embayment area lie at depths suitable for repository use. Undoubtedly, favorable conditions for the bedded-salt pilot plant exist at many of these localities where the rock salt lies at moderate depths. However, owing primarily to the extensiveness of the thick and shallow salt beds in the Permian basin and to the tectonic stability of the region encompassing the basin, specific site investigations have been confined to the Permian basin and in particular to central Kansas, where a demonstration of disposal in salt was conducted recently in an abandoned mine, and to a large tract of federally owned land in southeastern New Mexico.

SITE-SELECTION INVESTIGATIONS IN PERMIAN SALT BASIN

Extent and Thickness of Salt

Salt deposits are present throughout the Permian basin, which stretches from central Kansas through the Oklahoma Panhandle and into western Texas and eastern New Mexico, covering an area of about 120,000 sq mi. In general, salt deposits within the basin become progressively younger, thicker, and deeper toward the southwest; however, in most areas the salt body is found at depths of less than 2,000 ft. In Figure 4, which shows the aggregate thicknesses of the principal salt-bearing formations in the Permian basin, it is seen that extremely thick beds of salt

occur in the Salado Formation in southeastern New Mexico in the study area. This formation dips gently eastward along with the overlying Rustler Formation and the underlying Castile Formation, which locally contain appreciable quantities of salt. Together they comprise the Ochoa Series of evaporites of Late Permian age. The Salado Formation is characterized by thick salt beds with thin intervals of anhydrite, shale, and polyhalite. Locally, near Carlsbad, New Mexico, the formation is also rich in potash minerals. In Kansas, the thickest and most widespread salt deposit is the Hutchinson Salt Member of the Wellington Formation, a part of the Sumner Group of westward-dipping Upper Permian rocks. Salt deposits have also been found in the younger Nippewalla Group of Permian rocks toward the southwest, but their distribution and thickness are not precisely known. The Hutchinson salt, as shown in Figure 4, underlies central and south-central Kansas and extends southward into Oklahoma. The eastern edge of the salt body lies approximately 400 ft below the land surface, but near its western edge in Kansas it is found at a depth of more than 1,500 ft. Thicknesses of the unit in the state range up to 700 ft. In general, the Hutchinson consists of a complex mixture of salt, anhydrite, and shale, with salt as the predominant fraction throughout most of its extent in Kansas. Figure 4 shows the salt to be thickest in the south-central part of the state. At Lyons, where a demonstration disposal in salt was completed recently in an abandoned mine which was subsequently evaluated for its potential as a waste repository, the salt is about 300 ft thick but is purer than in the thicker section to the southwest.

Tectonics and Seismicity

The Permian salt basin is located within the stable Mid-Continent area of North America, which is characterized by low topographic relief and flat-lying beds of sedimentary rocks (see Fig. 5). Even though the Permian salt basin and the stable interior of the country have not been subjected to diastrophic movements since Precambrian time, some areas have been structurally positive (rising) in the geologic past, and others have been structurally negative (subsiding). These ancient structural features, although largely masked by surface rocks today, are extremely important in the subsurface as they were instrumental in controlling the deposition of the salt rocks. Principal events in the geologic history of the region include, during the early Paleozoic, submergence and subsequent deposition of marine sediments on an irregular and long-eroded surface of Precambrian igneous and metamorphic rocks. During Mississippian and Pennsylvanian time, the region was alternately submerged and elevated before a period of aridity during the Late Permian which is evidenced by the extensive evaporite deposits of that age. Another major period of emergence and erosion occurred throughout most of the region during the Mesozoic Era before the deposition of marine and continental sediments mostly of Cretaceous age. Erosion was prominent during the early Tertiary, but later eastward- and southeastward-flowing streams left thick accumulations of gravel, sand, and silt over the area. Some slight tilting and warping along with wind and water erosion have since formed the present plains landscape of the region.

In addition to the long geologic history of tectonic stability of the Permian basin, as revealed by the generally flat-lying nature of the rocks and by the dearth of deep-seated faults and igneous intrusions, the seismicity of the region also suggests quiescence and stable tectonics. Figure 6 is a map of a part of the southwestern United States showing the epicentral locations of all major earthquakes recorded in that region. From this map it is observed that relatively few earthquakes have occurred within the Permian basin during historic times. More specifically, none have been recorded in the New Mexico portion of the Permian basin, and only two have occurred within the basin in central Kansas. The few earthquakes that have occurred within the Permian salt basin range up to a modified Mercalli Intensity Site rating of V1 (damage small) and are believed to represent minor adjustments in the underlying granitic crustal rocks. Outside the salt basin it is noted in Figure 6 that the frequencies and intensities of earthquakes are markedly higher in tectonically active areas such as the Rio Grande Valley in central New Mexico and the Rocky Mountain front region of central Colorado.

Figure 7 is the most recent seismic-risk map of the United States published by the Environmental Science Services Administration and the U.S. Coast and Geodetic Survey (1969). The Permian salt basin lies within zone 1 (expected minor damage), whereas most of the area to the west and a smaller area to the east are designated zone 2 (expected moderate damage). The areas of expected major damage (zone 3) in the United States are associated with major active fault zones and/or areas that have experienced great earthquakes in historic times. It is significant that none of these zones of expected major damage lie close to the Permian salt basin.

All rocks that lie relatively near the land surface undergo some leaching by circulating groundwater. However, rock salt is unique in that generally only the uppermost surface is vulnerable to the leaching action of water as its self-healing properties preclude the development of open fractures, fissures, faults, etc., that provide the avenues for deeper and intraformational circulation in all other rock types. In the Permian salt basin, dissolution of rock salt at shallow depths by circulating groundwater is a common phenomenon. In central Kansas at least a part of the eastern edge of the salt basin has been dissolved in the subsurface by groundwater, as has the western edge of the salt in southeastern New Mexico. Active solution of shallow salt beds in other parts of the basin is also occurring as evidenced by the high salinities of rivers, such as the Arkansas, Red, Canadian, Brazos, and Pecos, that drain the region. Figure 8 is a generalized cross section of a part of central Kansas that shows the prominent stratigraphic and structural features of the Hutchinson Salt Member. It is observed that the gently westward-dipping salt body does not extend to the land surface but is dissolved along its eastern edge to a depth of several hundred feet. The abrupt end of the deposit coupled with an overlying series of subsidence ponds and saltwater springs strongly suggests that the original limits of the salt extended somewhat farther eastward. Channel fillings of Pleistocene and older sediments near the edge of the salt show a distinct westward progression of younger sediments that are presumed to have been laid down as the underlying salt was removed in that direction. Precise dating of these sediments indicates that during the last few million years the edge of the salt body has retreated westward at the rate of a few miles per million years. Therefore, only the first few miles of the salt body along its eastern edge would be vulnerable to dissolution by this phenomenon for the lifetime of the wastes or for the next few hundreds of thousands of years.

In southeastern New Mexico, the western boundary of the Permian salt basin is believed to have migrated eastward at the rate of a few miles per million years during the recent geologic past. On Figure 9, which depicts the present state of the salt and overlying beds in the region near Carlsbad and the study area, it is apparent that the uppermost beds of salt once extended farther west but were subsequently eroded with the development of the Pecos River Valley east of the Guadalupe Mountains. Indeed, Bachman and Johnson (1973) have suggested that at the close of "Ogallala" time (4 million years ago) the Salado salt may have extended as far westward as the base of the Guadalupe Mountains, or another 25-35 mi. With this premise they concluded that the rate of retreat averaged about 6-8 mi/m.y. during this span of time. Furthermore, Bachman has found that the quantities of salt presently being removed from watersheds within the Permian salt basin range up to about 0.5 ft/1,000 years. With these rates of vertical and horizontal removal of salt, it is seen that the study area, which is some 20-30 mi from the western edge of salt and is more than 1,000 ft deep into the salt beds, would not be affected by dissolution for the next few hundreds of thousands of years or for the duration of the wastes' radioactivity.

Mineral Resources

Significant quantities of potash ore and extensive deposits of oil and gas occur in selected localities of southeast New Mexico. To preclude conflicts of interest in the economic development of the region, the rocks underlying the study area of the pilot plant preferably should have a low potential for oil or gas development and should not contain extensive high-grade potash ores. Potash mines in the southeast New Mexico region produce more than 80 percent of all the potassium minerals mined in the U.S. However, most of the higher grade commercial ore within the area is nearing depletion. As seen in Figure 10, which indicates the locations and extent of the underground workings within the region, most of the mining has centered along a northward-trending belt about 10-12 mi east of Carlsbad, although two mines have been opened 6-8 mi farther to the east. The study area is more than 5 mi from the nearest workings and is far removed from the important mineralized bodies of ore. Indeed, the site is located outside the potash mining district as designated by the Secretary of the Interior in May 1965.

Figure 11 is a generalized oil and gas map of a part of southeast New Mexico. On this map it is seen that major oil and gas fields have not been discovered in the vicinity of the study area, although some localized accumulations of oil and gas are known to be present in nearby rocks. Exploration for gas has intensified recently in the deeper rocks of southeastern New Mexico and will undoubtedly lead to a renewed interest in the deeper rocks in the vicinity of the study area.

Hydraulic Testing in Open Boreholes

Because circulating groundwater is perhaps the only mechanism for dispersing and transporting radioactive wastes placed in rock salt, it is

essential to determine the hydrologic characteristics of the rocks that lie in close proximity to the salt. In the Permian salt basin fresh water is generally confined to the first few hundred feet of surface rocks, and the rocks below the salt beds commonly contain saline water. The hydraulic characteristics of water-bearing rock formations can best be identified by testing in open boreholes. In Kansas, hydraulic tests have been made in boreholes near the Lyons site as well as in the more northern part of the basin. These tests showed that the rocks immediately above and below the salt beds were extremely tight and incapable of transporting significant quantities of groundwater; however, at greater vertical distances from the salt some prolific water-bearing zones were found. Thus, the Hutchinson salt is intact and not undergoing dissolution at the test sites.

For the study area in southeast New Mexico hydraulic tests will be conducted in a series of boreholes as illustrated in Figure 12. As shown, the boreholes will penetrate all of the rocks overlying the salt and will extend some 2,000 ft into the salt, or to a depth of about 1,000 ft below the disposal level. As rock salt is known to be impermeable, all water zones penetrated by the holes will lie above the salt rocks. To test specific water-bearing zones, inflatable packers will be set in the holes above and below the water zones. The isolated intervals will then be subjected to slug and/or swabbing tests. For the higher yield aquifer (Santa Rosa Sandstone), pumping tests will be made. These tests will be used to determine such parameters as formation transmissivities, yields, and hydraulic conductivities. Pumping tests will then be made over the combined water-bearing zones to ascertain the adequacy of the individual tests and to give assurance that all water-bearing zones have been identified.

Erosion and Denudation

In extreme cases the natural geologic processes of erosion and denudation have the potential, over long periods of geologic time, for stripping away significant quantities of overburden and subjecting wastes that are buried at shallow depths to circulating groundwater and surface water. On the basis of general knowledge of the Great Plains landscape and on the sediment loads of streams that drain the province, it appears that the rates of denudation and stream incision within the Permian salt basin range up to only a few hundreds of feet per million years. This general conclusion is substantiated in part by the recent work on erosion and denudation in the Lyons, Kansas, area (Stewart, 1973). On the basis of the precept that glacial and interglacial episodes similar to those of the Pleistocene will continue throughout the next 1 million years, Stewart concludes that the probability that stream erosion will breach the salt formation in the Lyons area is so small that it is inconsequential. By extrapolating and adjusting present-day rates of denudation in central Kansas to accommodate glacial as well as interglacial conditions, Stewart also finds that, for all practical purposes, the probability that denudation will exceed 25 ft during the next 1 million years in the Lyons area is zero. Should continental glaciers advance into central Kansas, Stewart judges that the flow patterns of the major streams will not change appreciably, nor will the rise and fall of sea level accompanying glaciations and interglaciations affect stream entrenchment and valley alluviation in the Mid-Continent. The effects of new ice sheets on the underlying rocks in central Kansas have also been evaluated and it is concluded that deep scouring is unlikely and only minor fracturing and flexuring would be expected in the near-surface rocks due to glacial loading and unloading (Stewart et al., 1972).

Rock Properties

In order to establish the stratigraphic levels for waste disposal and to provide sample specimens for mineralogic determinations and rock-property testing, series of coreholes are drilled at the study areas. In central Kansas, mineralogic studies of cores indicate that hydrated minerals and water-bearing rocks are present throughout the evaporite section, with a general decrease in water content with depth (Kopp and Fallis, 1973). Upon heating sample specimens to 100°C it was found that gypsum is the major mineral constituent for rocks that lose more than 10 percent water. Similarly, for rocks that lose from 2 to 10 percent water upon heating, shales are dominant or the rocks have high clay content. The relatively pure halite rocks were found to lose less than 2 percent water upon heating. Thus, to avoid extensive dewatering, the containers of heat-generating waste should be placed only within the relatively pure halite beds of the evaporite sequences. The cores also provide samples for establishing thermal and mechanical properties including conductivity, specific heat, density, elastic moduli, Poisson's ratio, etc., for the several rock types at the study areas for use in thermal-analysis calculations and the rock-deformation analyses.

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Table 1. High-Level-Wastes Estimates from Projected Nuclear-Power Economy*

	Calendar Year Ending		
	1980	1990	2000
Installed Nuclear Electric Capacity, MW (electric)	150,000	450,000	940,000
Field Reprocessed, metric tons/year	3,000	9,000	19,000
Solidified High-Level Waste[a]			
Annual Volume, 10^3 ft^3	9.7	33	58
Accumulated Volume, 10^3 ft^3	44	290	770
Total Accumulated Activity, MCi	19,000	110,000	270,000
Total Thermal Power, MW	80	410	1,040
Significant Isotopes Accumulated, MCi			
28.9-y ^{90}Sr	960	5,700	12,000
30-y ^{137}Cs	1,300	8,000	20,000
10.8-y ^{85}Kr	120	690	1,500
12.3-y ^{3}H	7.3	44	110
24,400-y ^{239}Pu[b]	0.022	0.3	1.7
Number of Shipments to Repositories[c]	23	240	590

[a] Assumes 1 ft^3 of solidified waste per 10,000 MWd (th).

[b] Assumes 0.5% of plutonium in fuel is lost to waste.

[c] Each shipment consists of 57.6 cu ft of waste in 36 6-in.-diameter cylinders. Half of the waste is aged 5 years and half is aged 10 years at the time of its shipment.

*Adapted from Culler et al. (1971).

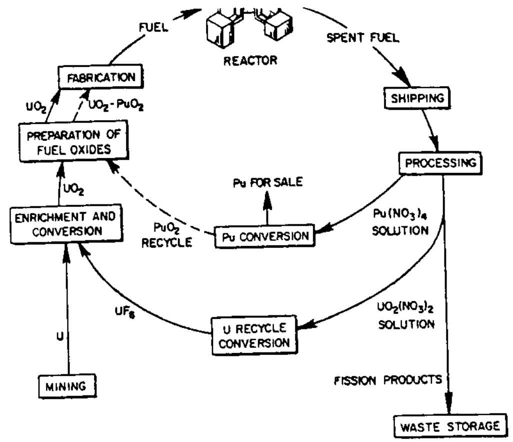

FIG. 1--Nuclear fuel cycle for light-water reactors.

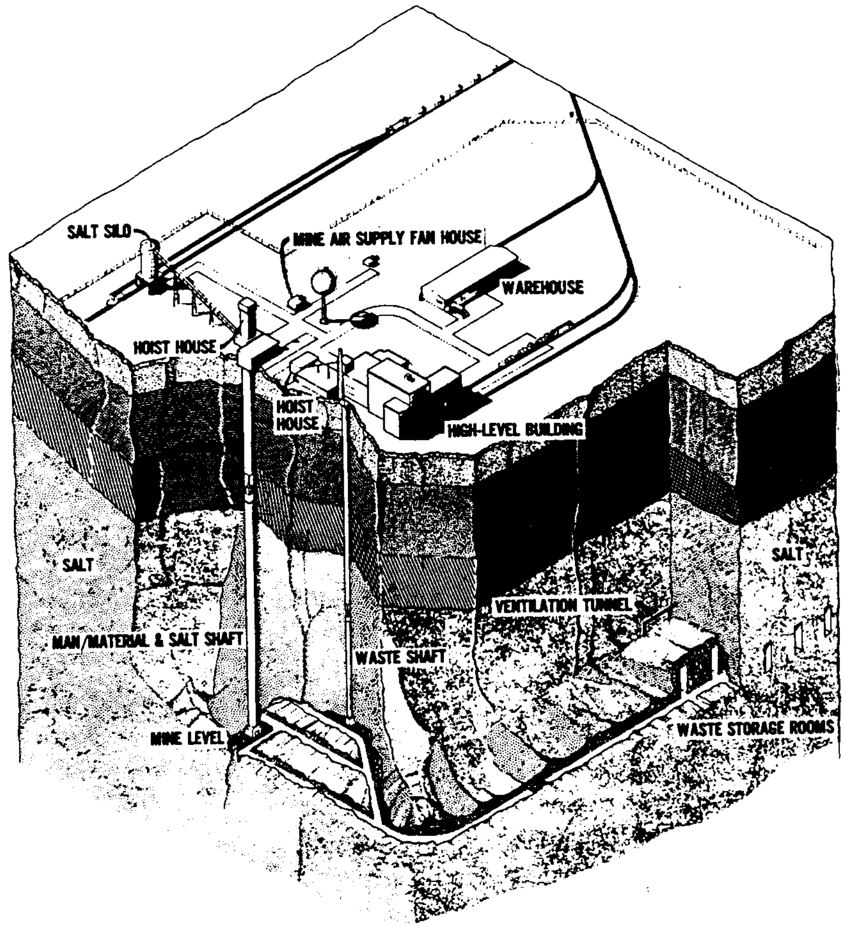

FIG. 2--Bedded-salt pilot plant.

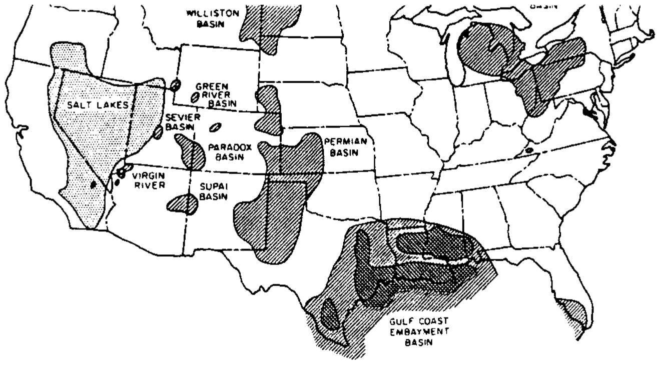

FIG. 3--Map of conterminous United States showing areas underlain by rock salt--double cross-hatched portions of Gulf Coast embayment represent salt dome areas.

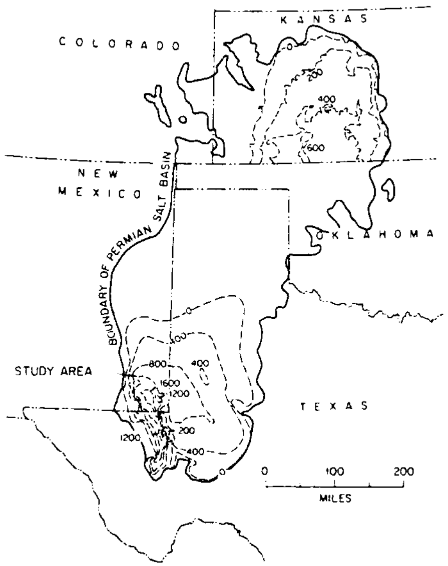

FIG. 4--Outline of Permian salt basin showing aggregate thicknesses of salt in Salado and Wellington Formations. After Hayes (1958) and Kulstad (1959).

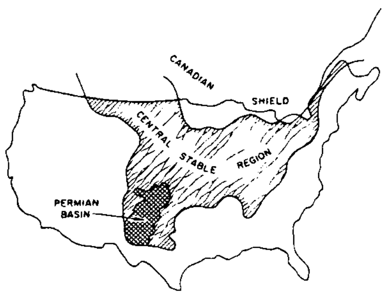

FIG. 5--Location of Permian basin in central stable region.

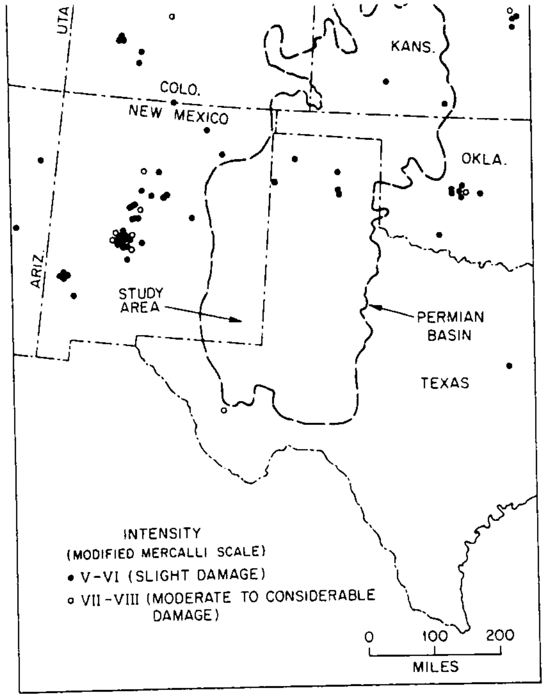

FIG. 6--Major recorded earthquakes in part of southwestern United States. Adapted from U.S. Dept. Interior Geol. Survey (1970).

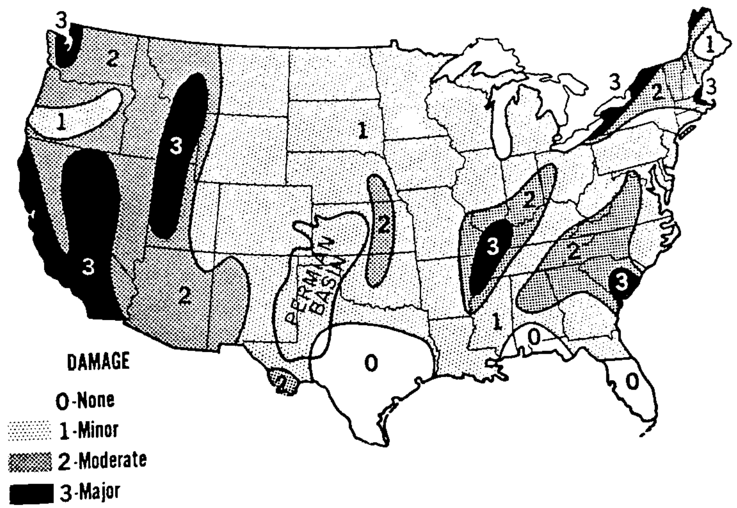

FIG. 7--Seismic risk map for conterminous United States, developed by Environmental Science Service Administration and U.S. Coast and Geodetic Survey and issued in January 1969. Subject to revision as continuing research warrants, it is an updated edition of map first published in 1948 and revised in 1951.

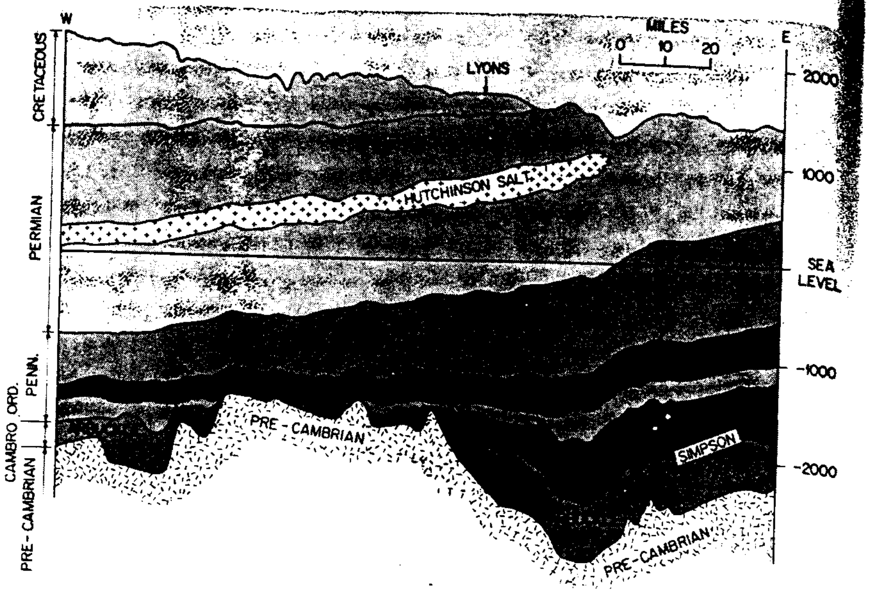

FIG. 8--Geologic cross section of part of central Kansas.

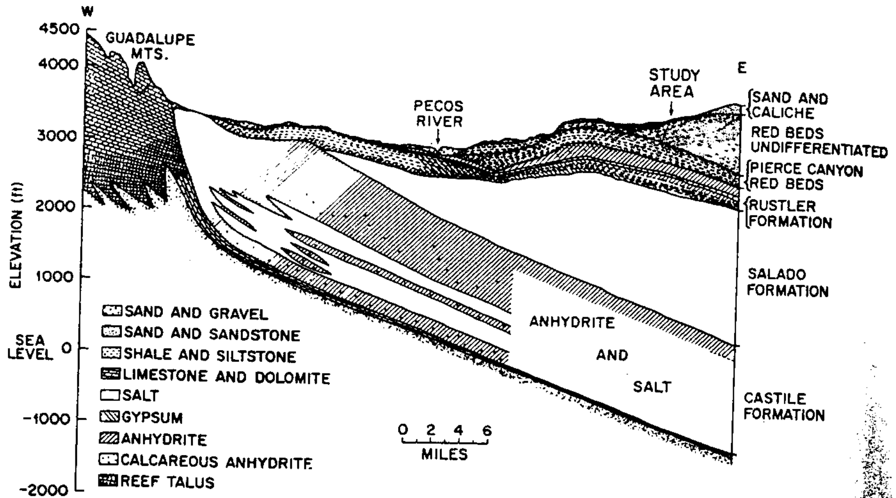

FIG. 9--Generalized geologic cross section of part of rocks in southeast New Mexcio. Adapted from Cooper (1960).

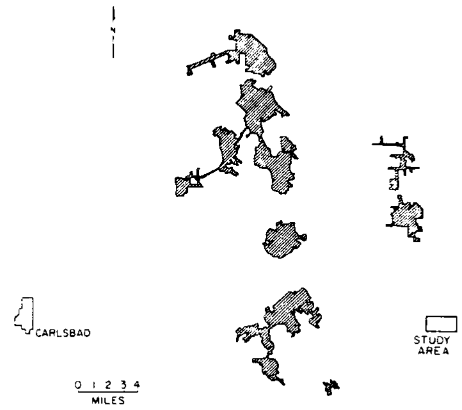

FIG. 10--Map of potash workings, southeastern New Mexico.

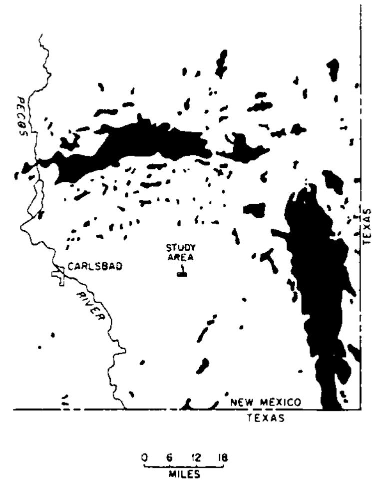

FIG. 11--Map of oil and gas fields in southeastern New Mexico.

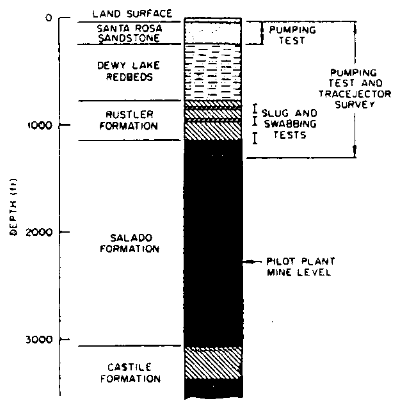

FIG. 12--Hydraulic tests in boreholes, southeastern New Mexico.

DEEP-WELL INJECTION OF DESALTING-PLANT WASTE BRINE[1]

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ABSTRACT The developing and desalting of saline waters stored in a deep artesian aquifer in northeastern Illinois, the Mount Simon Sandstone, is being considered to aid in meeting projected water deficits for the Chicago region. Because of the characteristics of the predicted Mount Simon water quality, it would be necessary to dispose of large quantities of desalting-plant waste brine. Evaluation of brine-disposal methods led to selection of disposal by injection through wells open to the lower Mount Simon aquifer. Since feedwater for desalting plants would be withdrawn from the upper Mount Simon aquifer, injection-well fields were designed to eliminate contamination of feedwater and to keep injection pressures within acceptable limits. Wells capable of injecting 1 million gal/day (mgd) were designed. Injection costs ranged from 17 cents per 1,000 gal for injection of brine from a 1-mgd reverse-osmosis plant to 35 cents per 1,000 gal for brine from a 5-mgd distillation plant. Waste-brine injection may have a harmful effect on the quality of water withdrawn from existing wells open to the aquifers above the Mount Simon.

INTRODUCTION

The Illinois State Water Survey is engaged in a comprehensive water-supply study of the metropolitan Chicago region (northeastern Illinois; Fig. 1), an area of about 4,000 sq mi with a population of about 7 million. In 1970, about a fourth of the population was dependent on groundwater as a source of supply. Because the Supreme Court decree of

[1]Manuscript received, May 29, 1973.

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1966 limited the maximum quantity of water diverted from Lake Michigan to 3,200 cu ft/second, there will continue to be an increased dependence on groundwater as a source of supply. According to Schicht and Moench (1971), some areas dependent on groundwater are already withdrawing water in excess of natural recharge. By as early as 1990, small parts of the area may have demands for water greater than the amount available from mining.

One of the alternatives to conventional groundwater development being investigated is the developing and desalting of large quantities of brackish water stored in a very deep sandstone aquifer in the Mount Simon Sandstone, which underlies the region. This paper summarizes a recently completed study (Illinois State Water Survey, 1973) describing the feasibility of this alternative. The study was funded in part by the Office of Saline Water. Hittman Associates, Columbia, Maryland, under subcontract to the Water Survey, carried out the feasibility study of the technical and economic aspects related to desalting.

AQUIFER DESCRIPTION

The stratigraphy and descriptions of the aquifers in the Chicago region are shown in Figure 2. The Mount Simon Sandstone lies at the base of the Cambrian deposits. In the Chicago region, the Mount Simon Sandstone and the lower, loosely consolidated sandstone unit of the Eau Claire Sandstone are hydrologically connected and considered as one unit, the Mount Simon aquifer (Suter et al., 1959). It is separated by a confining layer, the upper part of the Eau Claire Sandstone, from the Cambrian-Ordovician aquifer, a major source of potable groundwater in the Chicago region. The Mount Simon aquifer, which is present 1,500-1,900 ft below ground surface, consists of fine- to coarse-grained sandstone. The thickness of the aquifer (Fig. 3) is estimated to range from less than 1,200 ft in the northwest part of the area to more than 2,800 ft in the southwest part.

Many water-supply wells penetrate the upper 200-300 ft of the aquifer, which generally contains potable water in the northern two thirds of the region. It was estimated in 1971 that, of the 151 mgd pumped from deep wells, 26 mgd was contributed by the Mount Simon. Deteriorating mineral quality in many of these wells has been reported. Below 200-300 ft of penetration into the aquifer, the amount of total dissolved minerals increases gradually with depth to almost 90,000 mg/l at the bottom of the aquifer.

The recharge area of the Mount Simon aquifer is in southeastern Wisconsin, where the Mount Simon is the lowermost unit of the "sandstone aquifer," a series of sandstones and dolomite beds extending from the Mount Simon upward through the St. Peter Sandstone. This series behaves hydrologically as one unit, and recharge takes place as precipitation percolates down through the glacial drift and overlying bedrock units.

Groundwater in the Mount Simon in the Chicago region occurs under artesian conditions, because the upper part of the Eau Claire Sandstone acts as a confining layer and head differences exist between the Mount Simon and shallower aquifers. Heads in the Mount Simon were reported in 1960 to be more than 50 ft higher than those in overlying aquifers in some areas of the region.

AQUIFER MODELING

The yields of wells, the mineral quality of water withdrawn from wells, production-well field design, and injection-well field design are dependent primarily on hydraulic properties of the aquifer and water quality - depth relations. The Mount Simon aquifer was modeled from existing data so that the hydraulic properties and the quality-depth profile could be determined for any given well-field location.

Although many wells in the Chicago region are completed in the upper part of the Mount Simon aquifer, well-test data from wells open exclusively to the Mount Simon are scarce. To offset this, data from core analyses and well tests associated with underground gas-storage projects outside the study area were collected and correlated with depth variables to describe a model of the hydraulic character of the Mount Simon. On the basis of the application of this model to the geologic situation in the study area, the average transmissivity and coefficient of storage were estimated to be 10,000 gal/day/ft and 10^{-4}, respectively. The ratio of vertical to horizontal permeability is generally 0.5.

The mineral quality of water in the Mount Simon aquifer is described only briefly in the literature. The scarcity of mineral data exists because the aquifer is either too deeply buried or too highly mineralized for extensive development to have taken place economically. There are only two wells in the Chicago region for which analyses of water samples exclusively of Mount Simon water are available. One well, in Cook County, penetrated only 281 ft into the Mount Simon. The other well, in DuPage County, penetrated the entire thickness of the Mount Simon. Water samples were collected at several points. Table 1 shows the results of the analyses for selected water properties and chemical constituents. Sixty-one additional analyses were available from outside the area, primarily for samples

collected during gas-storage-reservoir test borings.

In order to define a water-quality depth profile at any proposed well-field location, a relationship between the mineral constituents and depth-related variables such as bottomhole elevation and depth was established. A water-quality depth profile for the Mount Simon aquifer at Aurora is shown in Table 2.

The flow in the Mount Simon model was simulated by digital computer analysis (modified from techniques developed by Prickett and Lonnquist, 1971) to enable water-quality predictions for desalting-plant design and production-well and injection-well field designs to be made. Analyses indicate that the mineral concentration of water that would be withdrawn from the Mount Simon wells would increase rapidly at first and, later, at gradually slower rates, similar to drawdowns within a pumped well. The ultimate mineral concentration of the water withdrawn will depend not only on the well location but also on its depth of penetration. In a multiple-well system, the ultimate concentration would also depend on spacing and total pumpage. Table 3 shows the total-dissolved-mineral predictions for a 5-mgd well-field pumping at a constant rate for 30 years.

WASTE-BRINE DISPOSAL

Five different desalting processes were considered by Hittman Associates: ion exchange (IE), electrodialysis (ED), reverse osmosis (RO), distillation, and freezing. Desalting plants capable of producing 500 mg/l product water that were considered as feasible were (1) 1-mgd RO and freezing plants and (2) 5-mgd distillation plants. The desalting plants were designed to produce desalted water at a constant rate. Since the quality of feedwater deteriorates with time, it is necessary that the feedwater rate increase with time. For example, providing feedwater for a 5-mgd desalting plant from a well-field with wells that penetrate 50 percent of the aquifer requires feedwater at a rate of 9.3 mgd initially and 19.3 mgd after 30 years. Thus the rate at which waste brine is produced would increase from 4.3 mgd initially to 14.3 mgd after 30 years.

Deep-well injection was considered to be the most suitable method for disposing of the waste brine from the desalting plants. Injection wells were designed on the basis of criteria of the Illinois Environmental Protection Agency. For a maximum injection rate of 1 mgd per well, each well would include 16-in. outer casing to a point below the depth at which Mount Simon water quality reaches 10,000 mg/l of dissolved minerals and a 12-in. long string to the top of the injection zone in the lower part of the Mount Simon (Fig. 4). An 8-in. injection string would be run inside the 12-in. long string; the annulus between the two strings would be filled with oil or fresh water. A seal would be placed at the bottom of the annulus, and a monitoring device would be used at the surface to detect changes in the annulus fluid pressure. A 10-in. borehole would continue below the long string to the bottom of the Mount Simon and would constitute the injection zone. The length of the injection zone must accomodate the following constraints: (1) flow velocity into the borehole well must not exceed 2 ft/min; (2) injection pressure must not exceed 0.65 psi per foot of depth at the injection zone; and (3) the mineral concentration of the injected brine must not exceed that of the native water in the injection zone. The required length of the injection zone was estimated to be 850 ft.

Waste-disposal well-fields were designed to accommodate the increasing waste-brine rates, to eliminate possible contamination of pumped feedwater, and to keep injection pressures within acceptable limits. An injection-well scheme for maximum injection of 15 mgd is shown in Figure 5. Pressure buildups computed for this scheme would reach a maximum of 1,020 ft above original static water levels. Although the injection and production wells were spaced in such a way that the quality of the feedwater would not be affected by waste-brine injection during the 30-year project life, water pumped from existing wells open to both the Mount Simon and overlying aquifers might deteriorate in quality. The predicted degree of deterioration should be investigated before pilot desalting plants are considered.

COST SUMMARY

Injection costs were from 13 to 19 percent of the total cost of producing water with a concentration of 500 mg/l TDM. Injection costs ranged from 17 cents per 1,000 gal for injection of brine from a 1-mgd reverse-osmosis plant to 35 cents per 1,000 gal for brine from a 5-mgd distillation plant. Total costs range from $1.33 per 1,000 gal for 1-mgd reverse-osmosis plants to $1.85 per 1,000 gal for the 5-mgd distillation plant. The costs are much higher than other alternatives; however, when this method is used in conjunction with the available fresh groundwater, costs become more competitive. The cost to one area in the region was estimated to be 74 cents per 1,000 gal for a 15-mgd combined supply, in which 11 mgd was available from conventional groundwater sources and 4 mgd was supplied from a desalting plant.

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Aquifer Penetration* (ft)	TDM	Cl	SO_4	Alkalinity ($CaCO_3$)	Hardness ($CaCO_3$)
450	620	88	39	284	202
738	4,076	2,150	224	204	1,370
1238	33,070	19,500	770	196	13,200
2038	79,330	47,500	1177	124	29,200
2138	80,100	48,000	1154	88	29,800
2190	81,780	50,500	1133	84	28,900

*Analyses are for point samples collected at these depths

Table 2. Estimated Water-Quality Depth Profile at Aurora (after Illinois State Water Survey, 1973) (Chemical constituents in milligrams per liter)

Aquifer Penetration* (ft)	T°F	TDM	Cl	Alkalinity ($CaCO_3$)	Hardness ($CaCO_3$)
65	60	500	100	280	345
195	63	1,000	500	275	450
325	64	1,500	650	269	600
455	65	3,500	2,000	250	900
585	67	6,200	3,500	235	1,200
715	68	11,500	6,500	215	2,500
845	71	19,500	11,500	205	4,200
975	73	25,500	14,700	195	7,200
1105	74	30,500	17,700	187	9,800
1235	76	35,500	21,000	175	12,000
1365	78	39,500	23,000	170	14,200
1495	80	46,500	27,500	160	17,500
1625	82	51,500	30,500	147	19,800
1755	84	58,000	34,500	135	22,700
1885	85	62,000	36,700	130	24,600
2015	87	65,500	39,000	125	26,700
2145	89	73,000	43,000	108	29,700
2275	91	77,000	45,200	100	32,700
2405	93	83,000	48,700	90	33,700
2535	95	88,000	52,000	80	36,700

*Water properties and mineral concentrations are averages for 130 foot intervals, midpoints of which are given in column 1.

Table 3. Predicted TDM Concentrations for a Constant Pumping 5 MGD Well Field (from Illinois State Water Survey, 1973) (Wells penetrate 31 percent of the aquifer)

Years of Pumping	TDM (mg/l)
0	9,000
1	15,400
2	19,200
3	22,000
4	24,000
5	25,400
10	30,000
15	32,800
20	34,400
25	35,600
30	35,900

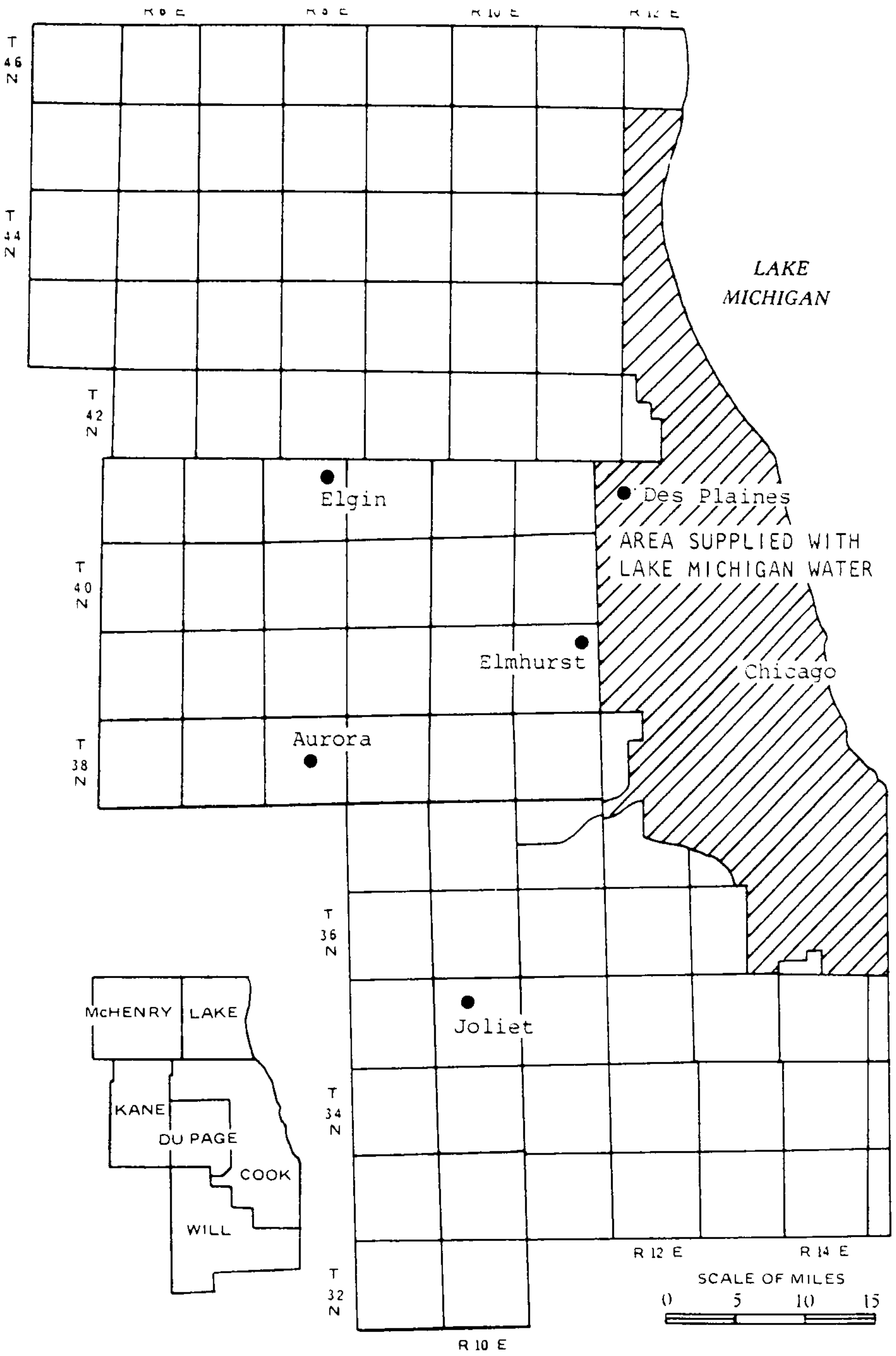


FIG. 1--Location of study area.

SYSTEM	SERIES	GROUP OR FORMATION	GRAPHIC LOG	THICKNESS (FEET)	DESCRIPTION	AQUIFERS
QUATERNARY	PLEISTOCENE			0 - 400+	Unconsolidated ice - and water-laid deposits, pebbly clay (till), silt, sand and gravel, generally discontinuous and interbedded; alluvial silts and sands commonly present along streams	Glacial drift aquifers
PENNSYLVANIAN				0 - 175	Shale; sandstones, fine grained; limestone; coal; clay	Shallow bedrock aquifers
MISSISSIPPIAN DEVONIAN						
SILURIAN	NIAGARAN			0 - 400+	Dolomite, very pure to very silty, cherty; shale partings; thin shales and argillaceous beds frequently present in lower parts of Silurian dolomite.	
	ALEXANDRIAN			0 - 165	Upper and middle units - shale, light gray to green, plastic to brittle, some dolomite, silty; dolomite, mostly silty, argillaceous; minor limestone Lower unit - shale, dark gray, black, brown, plastic to brittle; some dolomite in upper part; silty, argillaceous.	
ORDOVICIAN	CINCINNATIAN	Maquoketa		0 - 250+		
	CHAMPLAINIAN	Galena Platteville		150 - 350+	Dolomite, cherty; sandy at base; limestone; shale partings.	Cambrian-Ordovician aquifer
		Glenwood St. Peter		75 - 650	Sandstone, fine to coarse grained; shale at top; locally cherty red shale at base.	
	CANADIAN	Prairie du Chien		0 - 340	Dolomite, sandy, cherty, interbedded with sandstone.	
CAMBRIAN	CROIXAN	Eminence Potosi		0 - 225	Dolomite, white, fine grained sandy at base; drusy quartz.	
		Franconia		45 - 175	Sandstone, dolomite, and shale, glauconitic, green to red, micaceous.	
		Ironton Galesville		103 - 275	Sandstone, fine to medium grained, well sorted, upper part dolomitic	
		Eau Claire		235 - 450	Shale and siltstone, dolomitic, glauconitic; sandstone, dolomitic, glauconitic; dolomite, sandy	
		Mt. Simon		2000±	Sandstone, coarse grained, white, red in lower half; lenses of shale and siltstone, red, micaceous.	Mt. Simon aquifer
PRECAMBRIAN					Not penetrated by wells in Chicago area. Nearby wells encounter red or gray granite or similar rocks.	

FIG. 2--Geologic column and aquifer description of northeastern Illinois. *After Hughes et al. (1966).*

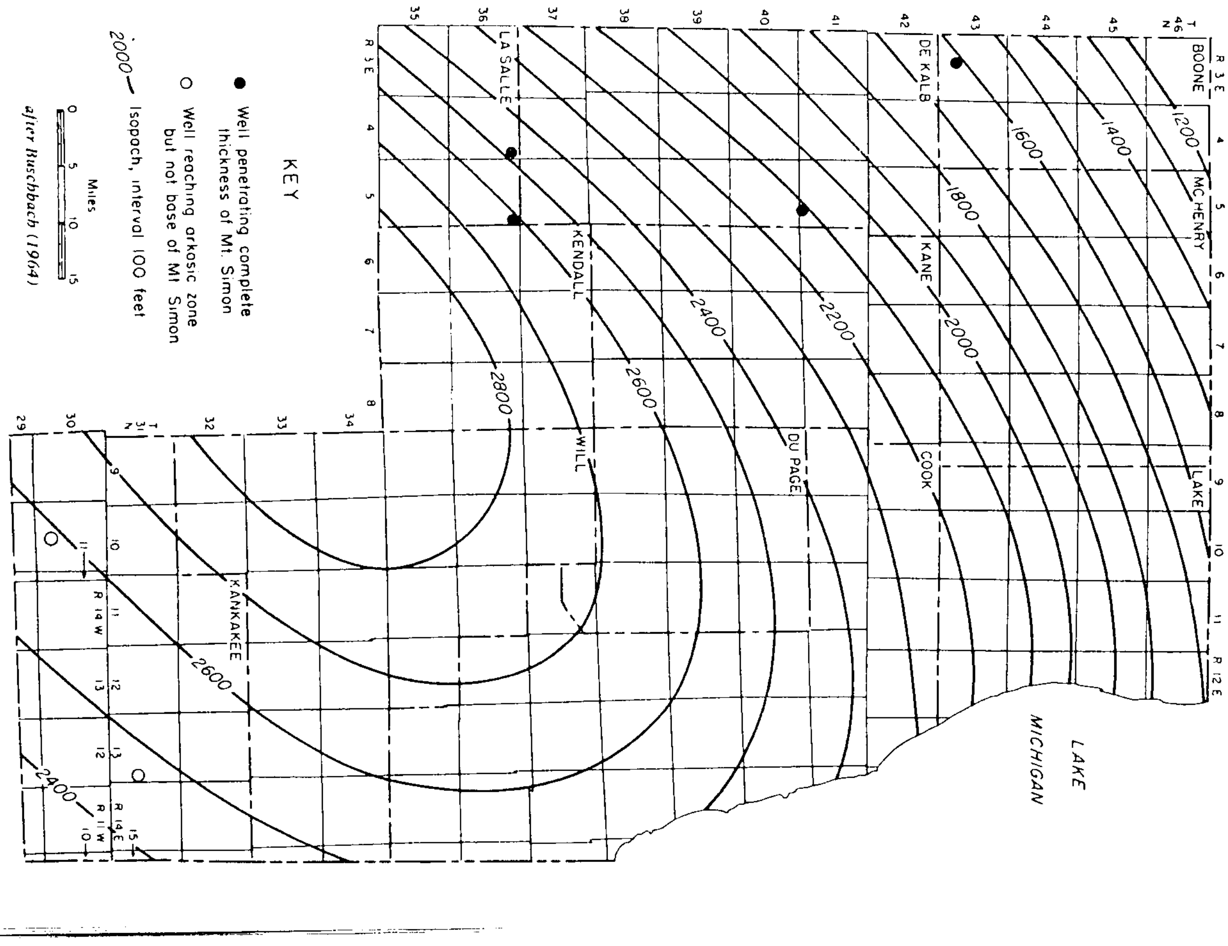
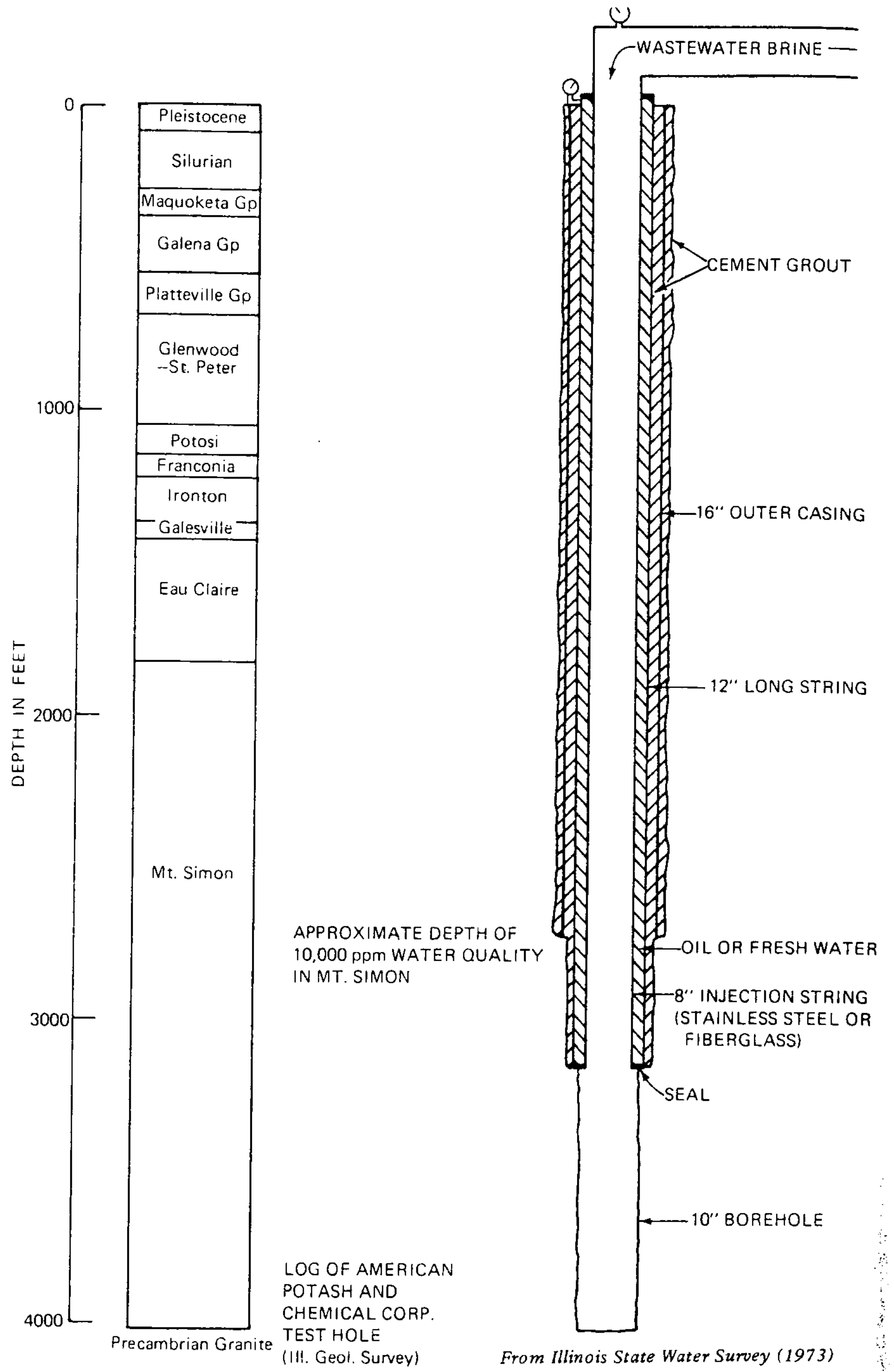
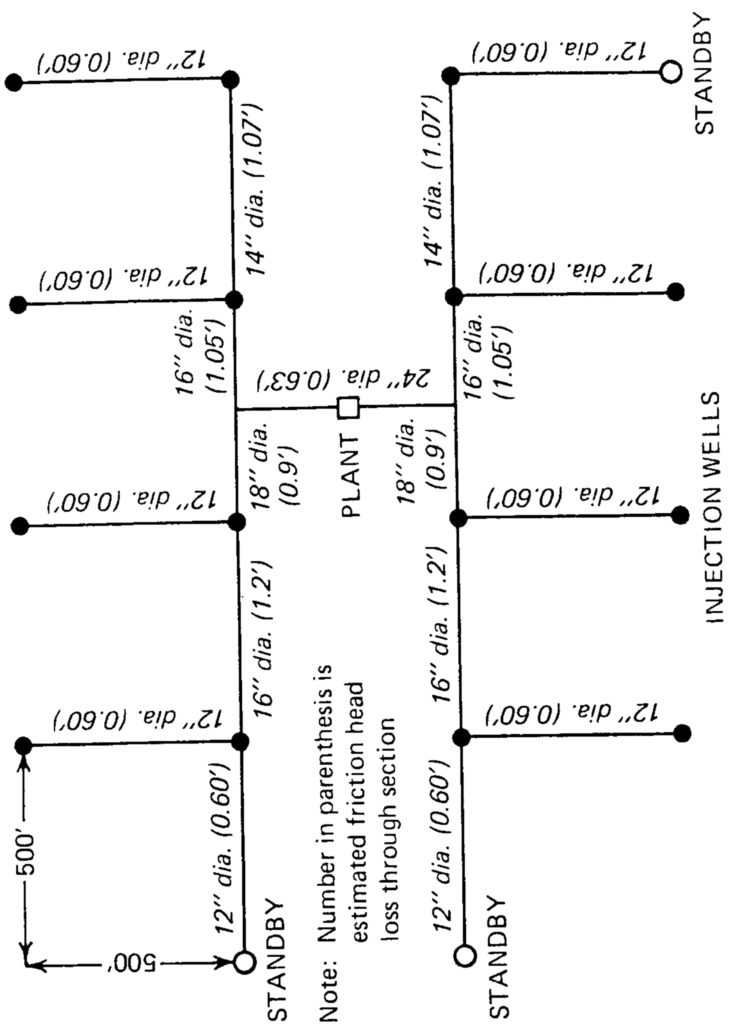


FIG. 3--Thickness of Mount Simon Sandstone.



From Illinois State Water Survey (1973)

FIG. 4--Injection-well construction features.



From Illinois State Water Survey (1973)

FIG. 5--Injection well layout for maximum injection of 15 million gal/day.