STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES
DIVISION OF RESOURCES PLANNING

BULLETIN No. 63

SEA WATER INTRUSION IN CALIFORNIA

APPENDIX B

REPORT BY

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

ON

INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA WATER INTRUSION, WEST COAST BASIN EXPERIMENTAL PROJECT, LOS ANGELES COUNTY

As Directed by Chapter 1500, Statutes of 1951

GOODWIN J. KNIGHT
Governor

HARVEY O. BANKS
Director of Water Resources

March, 1957

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Director of Water Resources

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ADDRESS ALL COMMUNICATIONS
TO THE CHAIRMAN

P. O. BOX 1079
SACRAMENTO 5

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REDDING

A. FREW, VICE CHAIRMAN
KING CITY

GOODWIN J. KNIGHT
GOVERNOR

STATE OF CALIFORNIA
DEPARTMENT OF WATER RESOURCES

HARVEY O. BANKS
DIRECTOR

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SACRAMENTO

JOHN P. BUNKER, GUSTINE
EVERETT L. GRUBB, ELSINORE
W. P. RICH, MARYSVILLE
PHIL D. SWING, SAN DIEGO
KENNETH Q. VOLK, LOS ANGELES

March 15, 1957

HONORABLE GOODWIN J. KNIGHT, *Governor*
and Members of the Legislature of
the State of California

GENTLEMEN: I have the honor to transmit herewith Appendix B entitled, "Report by the Los Angeles County Flood Control District on Investigational Work for Prevention and Control of Sea-water Intrusion, West Coast Basin Experimental Project, Los Angeles County," January 27, 1955. This appendix, authorized by Chapter 1500, Statutes of 1951, is of several appendixes to accompany Bulletin No. 63 of the Department of Water Resources, entitled, "Sea-water Intrusion in California."

The West Coast Basin Experimental Project was constructed and operated in the Manhattan Beach-Hermosa Beach area, Los Angeles County, by the Los Angeles County Flood Control District, under supervision of the Division of Water Resources and in accordance with terms of a contract with the State Water Resources Board. The project was designed to determine the feasibility and practicability of creating a pressure ridge in confined aquifers by injection of fresh water through wells for prevention and control of sea-water intrusion.

Appendix B describes in considerable detail the results of a two and one-half year experimental study of one of the more important methods of control of fresh water through wells for prevention and control of sea-water intrusion.

Very truly yours,

HARVEY O. BANKS
Director

ORGANIZATION

STATE DEPARTMENT OF WATER RESOURCES
DIVISION OF RESOURCES PLANNING

HARVEY O. BANKS	Director of Water Resources
M. J. SHELTON	Deputy Director
WILLIAM L. BERRY	Chief, Division of Resources Planning
CARL B. MEYER	Chief, Special Activities Branch
MAX BOOKMAN	District Engineer, Southern California Office

STAFF MEMBERS ASSISTING IN EXPERIMENTS

LAURENCE B. JAMES	Chief Engineering Geologist
E. C. MARLIAVE (now resigned)	Supervising Engineering Geologist
RAYMOND C. RICHTER	Supervising Engineering Geologist
WILLARD R. SLATER	Supervising Hydraulic Engineer
PHILIP J. LORENS	Senior Engineering Geologist
DONALD L. McCANN	Assistant Engineering Geologist

RESIDENT ENGINEER AT WEST COAST BASIN EXPERIMENTAL PROJECT

JACK J. COE	Senior Hydraulic Engineer

STATE WATER BOARD

CLAIR A. HILL, *Chairman*, Redding
A. FREW, *Vice Chairman*, King City
JOHN P. BUNKER, Gustine
W. P. RICH, Marysville
EVERETT L. GRUBB, Riverside
PHIL D. SWING, San Diego
KENNETH Q. VOLK, Los Angeles

SAM R. LEEDOM, Administrative Assistant

Note: Prior to establishment of the Department of Water Resources on July 5, 1956, the following organizational positions were in effect under the Division of Water Resources and the State Water Resources Board:

DIVISION OF WATER RESOURCES

HARVEY O. BANKS *	State Engineer
L. C. JOPSON	Assstant State Engineer
HENRY HOLSINGER	Principal Attorney
T. R. MERRYWEATHER	Administrative Assistant

STATE WATER RESOURCES BOARD

CLAIR A. HILL, *Chairman*, Redding
R. V. MIEKLE, *Vice Chairman*, Turlock
A. FREW, King City
W. P. RICH, Marysville
C. A. GRIFFITH, Azuso
W. PENN ROWE, San Bernardino
PHIL D. SWING, San Diego

* A. D. Edmonstan was State Engineer until his retirement on November 1, 1955.

PART I

INTRODUCTION

Sea-water intrusion is a major problem in ground water basins bordering the California coast. The State Legislature, in directing that an investigation be made to determine plans for the prevention and control of sea-water intrusion, recognized this fact and stated that in conducting the study, consideration was to be given to the determination of criteria for control of sea-water intrusion by the creation and maintenance of a pressure ridge by the introduction of fresh water through wells into aquifers.

AUTHORIZATION

Through enactment of Chapter 1500, Statutes of 1951, the California Legislature directed that an experimental program be undertaken to determine criteria for the prevention and control of sea-water intrusion into ground water basins.

> "The sum of seven hundred fifty thousand dollars ($750,000) is hereby appropriated out of any money not otherwise appropriated in the Postwar Unemployment and Construction Fund, or, in the event that such amount of money is not available therefrom, then to the extent not so available out of any money not otherwise appropriated in the General Fund to the State Water Resources Board for investigational work and design criteria for correction or prevention of damage to underground waters of the State by sea-water intrusion in the West Coast Basin of Los Angeles County and other critical areas. The cost of such investigation and study shall include the cost of providing water, water injection wells, observation wells, water spreading grounds, pipe lines, equipment, rights of way, and other facilities necessary to introduce water into the water-bearing aquifers. The board is authorized to cooperate and contract with the Los Angeles County Flood Control District, the West Basin Municipal Water District, and any other public or private corporation or agency to the purpose of this act."

In order to carry out the intent of this legislation, the State Water Resources Board on July 6, 1951, requested that the Division of Water Resources investigate and submit recommendations as to an experimental program for the prevention and control of sea-water intrusion. These recommendations, set forth in a report entitled "Proposed Investigational Work for Control and Prevention of Sea-Water Intrusion into Ground Water Basins," dated August, 1951, advocated performance of laboratory research concerning well injection, basic parameters of sea-water intrusion, and reduction in aquifer permeability, to be carried on concurrently and coordinated with a large scale field experiment to investigate the hydraulic feasibility of creating a pressure ridge in confined aquifers. In addition, the report recommended that the State Water Resources Board and its staff further investigate construction techniques involved in actual installations of cutoff walls and the effectiveness of such walls in impeding the lateral movement of ground water. A study of the economic factors involved in prevention of sea-water intrusion was also recommended.

The State Water Resources Board, in order to implement the foregoing investigational program, executed a contract with the Los Angeles County Flood Control District on October 1, 1951, for execution of certain portions of the program. The contract, and subsequent amendments thereto, authorized the Los Angeles County Flood Control District to install and operate experimental facilities, under supervision of the Division of Water Resources, to ascertain the hydraulic feasibility of creating a pressure ridge by use of injection wells and its effectiveness in preventing sea-water intrusion, and to report thereon.

LOCATION OF WEST COAST BASIN EXPERIMENTAL PROJECT AND CONDUCT OF INVESTIGATION

The large scale experimental field project was located in the cities of Manhattan Beach and Hermosa Beach, Los Angeles County, California. The location of this site was selected after careful consideration of alternate sites in other critical areas throughout California.

Late in 1951, the Los Angeles County Flood Control District, in close cooperation with the Division of Water Resources completed plans for construction of project facilities, including injection and observation wells and feeder lines, and operation of the field test on an experimental basis. Detailed geologic exploration was also commenced at this time. By February, 1953, construction of project facilities was completed, and injection of treated Colorado River water was commenced. Throughout the field investigation, monthly progress reports were submitted to the State Water Resources Board both by the Division of Water Resources and Los Angeles County Flood Control District.

In accordance with terms of the agreement, the State retained ownership of project facilities until after completion of the prescribed work, at which time certain property was retained by the State Water Resources Board for use by the Division of Water Resources on Board work and the remaining nonexpendable property was sold to the Los Angeles County Flood Control District.

The State Water Resources Board allocated a total of $642,126.30 to Los Angeles County Flood Control District for prosecution of this experimental study. These funds, except for $9,000 reserved for completion of the District's final report, were exhausted in December, 1953. Operation since that time has been financed from local and county taxes and by local contributions.

Other phases of the investigational program recommended by the Division of Water Resources were completed by the University of California at Berkeley and Los Angeles, and United States Geological Survey, Quality of Water Branch, at Sacramento and Menlo Park. Final reports by these agencies describing the work accomplished appear as Appendixes C, D and E.

AGREEMENT FOR PERFORMANCE OF INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA-WATER INTRUSION

This Agreement, entered into as of October 1, 1951, by and between the State Water Resources Board, hereinafter referred to as the Board, and The Los Angeles County Flood Control District, hereinafter referred to as the District, witnesseth:

Whereas, by Chapter 1500, Statutes of 1951, the sum of seven hundred and fifty thousand dollars ($750,000) is appropriated to the Board for investigational work and study with the objective of formulating plans and design criteria for the correction or prevention of damage to underground water of the State by sea-water intrusion in the West Coast Basin of Los Angeles County and other critical coastal areas; and

Whereas, a report entitled "Proposed Investigational Work for Control and Prevention of Seawater Intrusion Into Ground Water Basins," dated August 1951, was prepared at the request of the Board by the State Engineer, and

Whereas, said report and recommendations contained therein were accepted by the Board and approved at a regular meeting on September 7, 1951, and

Whereas, said approved report contains a recommended program of investigations, part of which is set forth as follows:

> "2. A sum of $450,000 should be allocated from funds presently available for installation and one year's operation of a field experimental project to investigate the hydraulic feasibility of creating a pressure ridge in confined aquifers by means of injection wells and the effectiveness of such a ridge in preventing sea-water intrusion. This project should be undertaken in the vicinity of Manhattan Beach in West Coast Basin, Los Angeles County. The initial installation should comprise five injection wells and sufficient observation wells to yield the observational data necessary for complete and conclusive interpretation of the results. Capital cost for feeder and distribution pipe lines should be kept to a minimum and emphasis placed upon experimental techniques and collection of pertinent data so that the results and conclusions therefrom will be applicable on a State-wide basis.
>
> "3. Additional funds should be allocated to the Manhattan Beach field experiment if results of the first six months operation analyzed in conjunction with laboratory research studies indicate that the initial installation of five injection wells is not extensive enough to yield conclusive results."

and

> "7. The State Water Resources Board should carefully supervise the planning and execution of all experimental work in order to assure that the results obtained therefrom are interpretable and usable on a State-wide basis."

Whereas, the Board is authorized by Chapter 1500, Statutes of 1951, to cooperate and contract with the Los Angeles County Flood Control District, the West Basin Municipal Water District, and/or any other public or private corporation or agency for the purpose of making such investigations; and

Whereas, the Board desires the District to undertake certain portions of the investigational work and study under the direction and supervision of the State Engineer, acting as Engineer for the Board;

Now Therefore, in consideration of the premises and of the several promises to be performed by each as hereinafter set forth, the Board and the District do hereby mutually agree as follows:

ARTICLE I—PROJECT

Description of Work

The project will consist of providing plans, formulating procedures, and conducting investigations which will comprise but not be limited to the following:

A. Preparing plans, specifications and cost estimates for furnishing and installing:

1. Approximately 7,300 feet of pipe line from the Metropolitan Water District vault at Manhattan Beach Boulevard and Redondo Avenue in the City of Manhattan Beach westerly to the Santa Fe right of way at Manhattan Beach Boulevard, Manhattan Beach, California;
2. Pipe lines for distribution along the Santa Fe right of way approximately parallel to the coast line;
3. Five recharge wells including drilling, coring, sampling, casing, and developing;
4. Approximately 30 observation wells including drilling, coring, sampling, casing, and developing;
5. Necessary well connections, meters, valves and pressure regulators;
6. Chlorinating equipment and housing.

Advertising, receiving bids, and executing and completing a contract or contracts for the installation of that portion of the work listed in "A," which cannot be advantageously performed by District forces, and providing supervision over that portion of the work to be done under such contracts;

Furnishing labor and materials for that portion of the work listed in "A" above which is to be done by the District forces;

. Contracting with the Metropolitan Water District (or a member agency) for the purchase of Metropolitan Water District water and a temporary connection at the vault located at Manhattan Beach Boulevard and Redondo Avenue in the City of Manhattan Beach, California, for supply of water for the field experimental work outlined herein.

. Formulating plans for, and, after approval by the State Engineer, conducting investigations and experimental work to determine the feasibility and practicality of creating a pressure ridge in confined aquifers for prevention and control of sea-water intrusion as outlined herein, including collection of the necessary data to determine if possible the following factors:

1. Feasible rates of injection through wells as related to thickness, permeability and other properties of the aquifers and variation in rates of injection with pressure head built up in the well and with time;
2. Height and shape of pressure ridge that can be built up as related to thickness and permeability of aquifer and hydraulic gradient;
3. Required height of pressure ridge and amount of water necessary to inject to control intrusion of sea-water as related to properties and depth of aquifer;
4. Required spacing of injection wells to control sea water intrusion as related to thickness of aquifer, permeability and hydraulic gradient;
5. Gradient of the piezometric surface in the water-bearing deposits and its effect on the quantity of water injected and the shape of the pressure mound;
6. Rates and amounts of displacement of saline waters and/or degree of dilution of saline waters;
7. Effect of ground water extractions by pumping in the inland areas on the rate of injection necessary to control sea-water intrusion;
8. Degree of chlorination or other treatment necessary for continued injection of water at feasible rates.
9. Maintenance of wells, including procedures such as sand bailing, surging, de-aeration, and studies of formation of micro-organisms and the effect of chlorination or other methods of disinfection on their growth, and studies of base-exchange reactions and suspended solids deposition.

F. Preparation of reports on the findings as specified in Article VI herein.

Location

Manhattan Beach, Los Angeles County, California.

Amount of Allocation

An allocation of $450,000 for the initial program which may be supplemented by additional allocations for further investigation subject to the approval by the Board.

ARTICLE II—SURVEYS AND PLANS

The District will make necessary surveys and prepare the plans and specifications and cost estimates needed for the work contemplated in this Agreement. Outlines of work and procedures for conduct of the experimental work and plans and specifications and cost estimates as prepared by the District for various items of work shall be submitted by the District to the State Engineer for review and approval prior to commencement of work. No changes shall be made in the work program and plans and specifications after such approval except with the consent of the State Engineer.

ARTICLE III—RIGHTS OF WAY

The District shall obtain all necessary rights of way, licenses, permits for entry, or easements necessary for the work hereinabove described under the Project. Approval of the State Engineer will be secured prior to making any expenditures for rights of way or easements except for the incidental expense involved therein in accepting and recording same.

In accordance with terms of the agreement, the State retained ownership of project facilities until after completion of the prescribed work, at which time certain property was retained by the State Water Resources Board for use by the Division of Water Resources on Board work and the remaining nonexpendable property was sold to the Los Angeles County Flood Control District.

The State Water Resources Board allocated a total of $642,126.30 to Los Angeles County Flood Control District for prosecution of this experimental study. These funds, except for $9,000 reserved for completion of the District's final report, were exhaust December, 1953. Operation since that time has financed from local and county taxes and by contributions.

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6. Chlorinating equipment and housing.

B. Advertising, receiving bids, and executing and completing a contract or contracts for the installation of that portion of the work listed in "A," which cannot be advantageously performed by District forces, and providing supervision over that portion of the work to be done under such contracts;

C. Furnishing labor and materials for that portion of the work listed in "A" above which is to be done by the District forces;

D. Contracting with the Metropolitan Water District (or a member agency) for the purchase of Metropolitan Water District water and a temporary connection at the vault located at Manhattan Beach Boulevard and Redondo Avenue in the City of Manhattan Beach, California, for supply of water for the field experimental work outlined herein.

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The District will make necessary surveys and prepare the plans and specifications and cost estimates needed for the work contemplated in this Agreement. Outlines of work and procedures for conduct of the experimental work and plans and specifications and cost estimates as prepared by the District for various items of work shall be submitted by the District to the State Engineer for review and approval prior to commencement of work. No changes shall be made in the work program and plans and specifications after such approval except with the consent of the State Engineer.

ARTICLE III—RIGHTS OF WAY

The District shall obtain all necessary rights of way, licenses, permits for entry, or easements necessary for the work hereinabove described under the Project. Approval of the State Engineer will be secured prior to making any expenditures for rights of way or easements except for the incidental expense involved therein in accepting and recording same.

ARTICLE IV—PERFORMANCE OF WORK

The District shall do or cause to be done, under its direct supervision, the work contemplated by this Agreement, in accordance with plans and specifications, as approved by the State Engineer, and said work shall be performed to the satisfaction of the State Engineer and shall be subject to inspection at all times by the State Engineer.

No contract shall be awarded by the District until the approval of the State Engineer has been obtained. A summary of the bids received shall be forwarded promptly to the State Engineer by the District for approval of award of contract.

The District may contract to obtain the services of a consultant on water problems, particularly in relation to the chemical and bacteriological phases of the work. The expenditures for such services will not exceed fifteen hundred dollars ($1,500.00) unless approved in advance by the State Engineer. Copies of all data, reports, or memoranda prepared by the consultant shall be promptly furnished to the State Engineer.

Any District-owned equipment used for said work may be charged upon a rental basis to cover depreciation and repair, in case rental rates have been established heretofore for the District; otherwise, allowance for depreciation and repair may be charged as approved by the State Engineer.

The District shall diligently prosecute and complete said work to the best of its ability, within the limitations of the Los Angeles County Flood Control Act, and in the event the said work herein provided for is not completed within two years from and after the date of execution of this Agreement, the Board may, upon written notice, terminate this Agreement provided, however, in the event of such termination, all expenditures and expenses of the District incurred under the provisions of this Agreement, to date of such termination, shall be paid by the Board to the District. The above limitation of two years may be extended at the discretion of the Board. Written notice of such extension will be given the District three months prior to termination of this Agreement.

ARTICLE V—FUNDS

The Board will reimburse the District for all direct expenditures and expenses incurred in the performance of the said work under the provisions of this Agreement.

Salaries and expenses of administrative employees who would normally be employed by the District regardless of such work will not be allowed.

The District shall render to the Board monthly, in quadruplicate, full and complete statements of all expenditures and expenses incurred in the performance of said work under the provisions of this Agreement. The Board shall, upon approval and audit as provided by law and the regulations governing said Board, pay monthly to the District any and all amounts incurred or expended by the District as herein provided in the performance of said work.

During the progress of said work, all data and records pertaining to the work covered by this Agreement, in the possession or control of either the said District or the said Board, shall be made fully available to the other and to the State Engineer for due and proper accomplishments of the purposes and objectives hereof.

ARTICLE VI—REPORTS

Progress reports will be submitted by the District at the end of each six-month period of operation.

Within six months after the completion of the investigational work contemplated by this Agreement, or prior termination of this Agreement as provided in Article IV, the District shall submit, to the Board, a final report on the conduct of the experiments summarizing the findings and making recommendations in relation to the objectives of the project.

Within 90 days after the completion of the work contemplated by this Agreement, the District shall file with the Board a final report of expenditures on said project.

All reports, plans, specifications, estimates, statements of expenditures and expenses, and other documents required to be submitted by the District shall be in a form approved by the State Engineer.

The State Engineer shall review the reports and findings of the District and include such data in reports to the Board together with results of other investigations and studies which may be made.

ARTICLE VII—MISCELLANEOUS PROVISIONS

It is agreed, by the parties hereto, that after the completion of the work contemplated in this Agreement, the State of California will retain its ownership or interests in the installations provided hereinabove, pending decision by the Board at that time as to their proper disposition.

IN WITNESS WHEREOF, the parties hereunto have executed this agreement as of the date first herein written.

Approved as to form and procedure:

/s/ MARK C. NOSLER
Attorney for Division of Water Resources

Approved as to form and procedure: [SEAL]

County Counsel

APPROVED AS TO FORM
HAROLD W. KENNEDY
County Counsel

By /s/ ROY W. DOWDS
Assistant

STATE WATER RESOURCES BOARD
By /s/ C. A. GRIFFITH

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

By /s/ ROGER W. JESSUP
Chairman, Board of Supervisors

HAROLD J. OSTLY, County Clerk
By /s/ RAY E. LEE
Deputy

APPROVED:
/s/ JAMES S. DEAN
Director of Finance

Z.E.S. Form	F.J.M. Budget	Value	Descript.
DEPARTMENT OF FINANCE APPROVED Feb. 26, 1952			

STATE OF CALIFORNIA } ss.
COUNTY OF LOS ANGELES }

On this 11th day of December, A. D., 1951, before me HAROLD J. OSTLY, County Clerk and Clerk of the Superior Court in and for the County of Los Angeles, State of California, residing therein, duly commissioned and sworn, personally appeared ROGER W. JESSUP, known to me to be the Chairman of Board of Supervisors of the County of Los Angeles and the person who executed the within instrument on behalf of the County therein named, and acknowledged to me that such County executed the same.

[SEAL]

IN WITNESS WHEREOF, I have hereunto set my hand and affixed my official seal the day and year of this certificate first above written.

HAROLD J. OSTLY, County Clerk
By /s/ E. L. THORING
Deputy

COUNTY OF LOS ANGELES
BOARD OF SUPERVISORS
LOS ANGELES 12

RAY E. LEE
Chief Clerk of the Boord

MEMBERS OF THE BOARD
Roger W. Jessup
Choirman
Herbert C. Legg
Leonord J. Roach
John Anson Ford
Roymond V. Darby

Tuesday, December 11, 1951

The Board met in regular session. Present: Supervisors Roger W. Jessup, Chairman presiding, Herbert C. Legg, Leonard J. Roach, John Anson Ford and Raymond V. Darby; and Harold J. Ostly, Clerk, by Ray E. Lee, Deputy Clerk.

* * * * *

(Minute Book No. 38, Page 76)

In Re Flood Control: Approval of Agreement with the State Water Resources Board for Performance of Investigational Work for Prevention and Control of Sea-Water Intrusion.

On motion of Supervisor Legg, unanimously carried, it is ordered that the following agreement be approved and signed by the Chairman of this Board in behalf of the Los Angeles County Flood Control District, to wit:

Agreement entered into as of October 1, 1951, by and between the State Water Resources Board and Los Angeles County Flood Control District, providing for the undertaking by the District, at State expense, under the direction and supervision of the Board and the State Engineer, of a field experimental project to investigate the hydraulic feasibility of creating a pressure ridge in confined aquifers by means of injection wells, and the effectiveness of such a ridge in preventing sea-water intrusion in the West Coast Basin of Los Angeles County; the project to be undertaken in the vicinity of Manhattan Beach; funds to be expended on said project to be limited to $450,000, and the period of the District's work thereon to be limited to two years unless additional funds are allocated by the Board; State to pay all costs except District overhead, and to retain title to the installed facilities at the end of the test pending decision by the Water Resources Board as to their proper disposition.

(Agreement No. 573-P)

I, Harold J. Ostly, County Clerk of the County of Los Angeles, and ex officio Clerk of the Board of Supervisors of Los Angeles County Flood Control District, do hereby certify that the foregoing is a full, true and correct copy of the original Minutes of the Board of Supervisors, as entered in Minute Book No. 38, Page 76, In re Flood Control: Approval of Agreement with the State Water Resources Board for Performance of Investigational Work for Prevention and Control of Sea-Water Intrusion.

(seal)

In Witness Whereof, I have hereunto set my hand and affixed the seal of the Board of Supervisors of Los Angeles County Flood Control District, this 21st day of February, 1952.

Harold J. Ostly, County Clerk of the County of Los Angeles, State of California, and ex officio Clerk of the Board of Supervisors of Los Angeles County Flood Control District.

By /s/ Ray E. Lee
Deputy Clerk

AMENDMENT TO AGREEMENT FOR PERFORMANCE OF INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA-WATER INTRUSION

WHEREAS, the State Water Resources Board, hereinafter referred to as the Board, and the Los Angeles County Flood Control District, hereinafter referred to as the District, entered into an agreement as of October 1, 1951, for performance of investigational work for prevention and control of sea-water intrusion pursuant to Chapter 1500, Statutes of 1951; and

WHEREAS, said agreement provides that in the event the work provided for therein is not completed within two years from and after the date of execution thereof the Board may, upon written notice, terminate the agreement; and

WHEREAS, it is the desire of the Board and the District that said work be continued until December 31, 1953, with the understanding that if funds are made available from other sources the facilities of the Manhattan Beach Project may be used by the District through lease or supplemental agreement if in the opinion of the Board such operation should be continued;

NOW THEREFORE in consideration of the premises and of the several promises to be performed by each as set forth in said agreement dated October 1, 1951, said agreement is amended as follows:

1. The agreement hereby amended is extended until December 31, 1953, and may be further extended by the Board.

2. An additional allocation by the Board of $187,000 for such work is hereby made and a further allocation will be made by the Board if and when unallocated funds appropriated by Chapter 1500, Statutes of 1951, become available.

3. The District will file with the Board an interim report by December 1, 1953, on the work performed by the District and an approximate accounting of the expenditure of funds allocated to the District from the money appropriated by said Chapter 1500.

4. Further use of the facilities by the District of the Manhattan Beach Project is subject to subsequent agreement if in the opinion of the Board such operation should be continued.

Dated: November 6, 1953.

Approved as to form and procedure:

/s/ HENRY HOLSINGER
Attorney for Division of Water Resources

Approved as to form and procedure:

HAROLD W. KENNEDY
By /s/ Roy Dowds
County Counsel

(SEAL)

DEPARTMENT OF FINANCE
APPROVED
Nov. 6, 1953

STATE WATER RESOURCES BOARD

By /s/ C. A. GRIFFITH
Chairman

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

By /s/ JOHN ANSON FORD
Chairman, Board of Supervisors

HAROLD J. OSTLY, County Clerk

By /s/ Ray E. Lee
Deputy

APPROVED:

JOHN M. PEIRCE, Director of Finance
By /s/ A. Earl Washburn
Deputy Director of Finance

SECOND AMENDMENT TO AGREEMENT FOR PERFORMANCE OF INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA-WATER INTRUSION

WHEREAS, the State Water Resources Board, hereinafter referred to as the Board, and the Los Angeles County Flood Control District, hereinafter referred to as the District, entered into an agreement as of October 1, 1951, for performance of investigational work for prevention and control of sea-water intrusion pursuant to Chapter 1500, Statutes of 1951; and

WHEREAS, the Board and the District entered into an amendment to said agreement dated November 6, 1953, which provided, among other things, that the original agreement is extended until December 31, 1953; and

WHEREAS, the Board and the District entered into an agreement as of November 24, 1953, permitting the District to take possession of and operate the Manhattan Beach Experimental Project upon exhaustion of funds supplied by the Board for the operation of the project, and pursuant thereto the District took possession and commenced operation of the said project on December 1, 1953; and

WHEREAS, it is the desire of the Board and the District that the final report on conduct of the experiment referred to in Article VI of said original agreement shall include the operation of the project by the District after funds allocated therefor by the Board have been exhausted, and that $9,000 of the funds allocated for the project by the Board shall be expended on the preparation of said final report.

NOW THEREFORE, in consideration of the premises and of the several promises to be performed by each of the parties hereto as set forth in said agreement dated October 1, 1951, as amended, the parties hereto further agree that said original agreement as amended is further amended as follows:

1. The final report on the conduct of the experiments referred to in Article VI of said original agreement shall be submitted on or before December 31, 1954, and shall include the operation of the project by the District for any period prior thereto after funds allocated therefor by the Board have been exhausted.

2. Of the funds allocated by the Board for the project, $9,000 shall be reserved to be expended on the preparation of said final report.

Dated: Feb. 12, 1954

Approved as to form and procedure:

/s/ HENRY HOLSINGER
Attorney for Division of Water Resources

Approved as to form and procedure:

HAROLD W. KENNEDY
by /s/ ROY DOWDS
County Counsel

DEPARTMENT OF FINANCE
APPROVED
Feb. 17, 1954
John M. Peirce, Director
By /s/ A. EARL WASHBURN
Deputy

STATE WATER RESOURCES BOARD

By /s/ C. A. GRIFFITH
Chairman

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

By /s/ JOHN ANSON FORD
Chairman, Board of Supervisors

(SEAL)

ATTEST:
HAROLD J. OSTLY, County Clerk and ex officio Clerk of the Board of Supervisors
By /s/ RAY E. LEE

APPROVED:

Director of Finance

AGREEMENT PERMITTING LOS ANGELES COUNTY FLOOD CONTROL DISTRICT TO TAKE POSSESSION OF THE MANHATTAN BEACH EXPERIMENTAL PROJECT

This agreement, entered into as of November 24, 1953, by and between the State Water Resources Board, hereinafter referred to as the Board, and the Los Angeles County Flood Control District, hereinafter referred to as the District, witnesseth:

Whereas, the District has constructed a pipe line, wells and appurtenances and acquired certain equipment pursuant to that certain "Agreement for Performance of Investigational Work for Prevention and Control of Sea Water Intrusion" entered into by the parties hereto as of October 1, 1951, which structures and installations and equipment are the property of the Board and commonly known as the Manhattan Beach Experimental Project, hereinafter referred to as the Project, and

Whereas, the District has operated the Project pursuant to said agreement with funds supplied by the Board, and

Whereas, the funds supplied by the Board for operation of the Project will soon be exhausted, and

Whereas, it is the mutual desire of the parties hereto that operation of the Project be continued by the District with funds supplied by sources other than the Board,

Now Therefore, in consideration of the premises and of the several promises to be performed by each as hereinafter set forth, the Board and the District do hereby mutually agree as follows:

Upon the exhaustion of funds supplied by the Board for operation of the project, exclusive of funds reserved for preparation of a report, but not later than December 31, 1953, the District may take possession, custody and control of all the structures, equipment, material and supplies owned by the Board which are at that time being used or held for use for the operation of the Project.

Thereafter, the District shall operate the Project according to the same general plan and purpose as it was operated previously, however, without control or direction by the Board and without cost to the Board.

It is expressly understood and agreed that the District in so operating the Project will be acting independently of the Board and not as its agent or employee, and that the District will hold and save harmless the State of California and the Board, its members and employees, from any and all claims or actions for damages resulting from the operation of the Project by the District under this agreement.

The District shall permit representatives of the Board to freely inspect and observe the operation of the Project at all times and shall furnish the Board with copies of all records and reports kept and formulated regarding the operation of the Project as requested by representatives of the Board.

Upon the termination of this agreement or any extension thereof, the District shall return all structures, equipment, material and supplies used or held for use in the operation of the Project which belong to the Board in the same condition as when the District took possession thereof, reasonable wear and tear excepted, provided that the District shall not be responsible for supplies or material of an expendable nature which have been expended in operating the Project or for damage resulting from natural causes beyond the control of the District which reasonable care would not have prevented.

The term of this agreement shall extend until June 30, 1954, unless a prior or subsequent date is mutually agreed upon by the parties in writing.

This agreement may be terminated, and any of its provisions changed or amended by mutual consent of the parties hereto expressed in writing.

In Witness Whereof, the parties hereto have executed this agreement as of the date first herein written.

Approved as to form and procedure:

/s/ Henry Holsinger
Attorney for Division of Water Resources

Approved as to form and procedure:

Harold W. Kennedy
by /s/ Roy Dowds
County Counsel

DEPARTMENT OF FINANCE
Approved
Dec. 10, 1953
John M. Peirce, Director
by /s/ Louis J. Heinzer
Adm. Adviser

STATE WATER RESOURCES BOARD

By /s/ C. A. Griffith
Chairman

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
By /s/ John Anson Ford
Chairman, Board of Supervisors

(SEAL)

HAROLD J. OSTLY, County Clerk
By /s/ Ray E. Lee

APPROVED:

Director of Finance

EXTENSION OF AGREEMENT PERMITTING LOS ANGELES COUNTY FLOOD CONTROL DISTRICT TO TAKE POSSESSION OF THE MANHATTAN BEACH EXPERIMENTAL PROJECT

This agreement, entered into as of June 4, 1954, by and between the State Water Resources Board, hereinafter referred to as Board, and the Los Angeles County Flood Control District, hereinafter referred to as the District, witnesseth:

Whereas, that certain agreement entered into by and between the Board and the District as of November 24, 1953, entitled "Agreement Permitting Los Angeles County Flood Control District to take possession of the Manhattan Beach Experimental Project," provides: "The term of this agreement shall extend until June 30, 1954, unless a prior or subsequent date is mutually agreed upon by the parties in writing"; and

Whereas, it is mutually desired that the District continue to operate the Manhattan Beach Experimental Project pursuant to the terms of said agreement for an additional period of time;

Now, Therefore, in consideration for the foregoing, it is mutually agreed that said agreement shall be and hereby is extended until June 30, 1955,

Provided That, the Red Jacket Reda Submerga Pump, serial number 2686, one and one-half horsepower, 60-cycle, single phase 230-volt motor, serial number 1542041, and appurtenant equipment all of which is a portion of the Manhattan Beach Experimental Project equipment purchased with State funds, and is no longer needed for operation of the project, shall not be included in this extension and said equipment shall be returned to the State Water Resources Board on or before July 10, 1954.

Provided Further, that nothing herein contained shall be construed to change the date of submission of the final report by the District from December 31, 1954, as set forth in the original agreement, as amended.

In Witness Whereof, the parties hereto have executed this agreement as of the date first herein written.

Approved as to form and procedure:

/s Henry Holsinger
Attorney for Division of Water Resources

Approved as to form and procedure:

Harold W. Kennedy
County Counsel
By /s/ Roy W. Dowd
County Counsel Assistant
By ______________________________

STATE WATER RESOURCES BOARD

By /s/ C. A. Griffith
Chairman

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

(SEAL)

By /s/ John Anson Ford
Chairman, Board of Supervisors

Harold J. Ostly, County Clerk

By /s/ Ray E. Lee, Deputy

Approved:

____
Director of Finance

DEPARTMENT OF FINANCE
Approved
Aug 23 1954
John M. Peirce, Director
By /s/ Louis J. Heinzer
Administrative Adviser

ACTION BY THE LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

In compliance with the terms of the Agreement the Los Angeles County Flood Control District submitted a report entitled, "Report by the Los Angeles County Flood Control District on Investigational Work for Prevention and Control of Sea-Water Intrusion, West Coast Basin Experimental Project, Los Angeles County," January 27, 1955. The report appears, verbatim, as Part II of this appendix.

PART II

REPORT BY LOS ANGELES COUNTY FLOOD CONTROL DISTRICT ON INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA WATER INTRUSION, WEST COAST BASIN EXPERIMENTAL PROJECT, LOS ANGELES COUNTY

January 27, 1955

By H. A. van der GOOT, E. J. ZIELBAUER, A. E. BRUINGTON, A. A. INGRAM

Submitted by FINLEY B. LAVERTY, Chief, Hydraulic Division

Recommended by PAUL BAUMANN, Assistant Chief Engineer

Approved by H. E. HEDGER, Chief Engineer

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

H. E. HEDGER
CHIEF ENGINEER

MAILING ADDRESS
BOX 2418
TERMINAL ANNEX
LOS ANGELES 54, CALIFORNIA

2250 ALCAZAR STREET
LOS ANGELES
CAPITOL 2-8151

January 27, 1955

FILE NO. 573-P
SUBJECT Sea Water Intrusion Control Agreement—Final Report

STATE WATER RESOURCES BOARD
Public Works Building
Sacramento 5, California

Attention: Mr. A. D. Edmonston
Secretary

GENTLEMEN:

Transmitted herewith is the final report on the West Basin Barrier Test, in accordance with the second amendment to the agreement for "Performance of Investigational Work for the Prevention and Control of Sea Water Intrusion." This agreement was entered into as of October 1, 1951 between your Board and the Los Angeles County Flood Control District. The original agreement provided for a test period of one year after construction of facilities. Amendments to this agreement extended the period of the test to June 30, 1954, for which your Board allocated a total sum of $642,126.30.

In general, the agreement provided for formulating plans for conducting investigations and experimental work to determine the feasibility and practicability of creating a ground water pressure ridge in confined aquifers for the prevention and control of sea water intrusion. Specifically, it provided for preparing plans, specifications and cost estimates for pipe lines, distribution lines, recharge wells, observation wells, necessary appurtenances, chlorinating equipment, and housing; for labor and materials; and for contracting with the Metropolitan Water District of Southern California and the City of El Segundo for the purchase of water. This included the right to advertise, receive bids, execute and complete contracts for work which could not be advantageously performed by District forces.

The project was located in the cities of Manhattan Beach and Hermosa Beach, Los Angeles County, California. A line of recharge wells covering a reach of some 4500 feet in length was installed approximately parallel to and some 2000 feet inland from the Pacific Ocean coast line. Nine recharge wells, spaced at 500 feet along the recharge line, and thirty-six observation wells, located along, inland and seaward of the line, were originally drilled. Eighteen additional observation wells were added as the test progressed. The wells penetrated a pressurized aquifer confined by an impervious clay cap slightly below sea level. The lower limits of the aquifer were bounded by relatively impervious sediments some 110 feet below sea level. The ground waters at the test site were completely degraded by sea water intrusion. Treated Colorado River water, imported by the Metropolitan Water District, was used for injection purposes and was piped from a connection with the El Segundo branch of the West Basin Feeder located some 7300 feet inland of the recharge line.

Plans for the test project were developed in cooperation with representatives from the State Division of Water Resources. After approval by the State Engineer, geologic exploration was initiated by the drilling of observation wells along the recharge line. Full scale construction work was delayed until signed waivers of liability were executed by adjacent water producers, holding the State harmless from the possible effects of an accelerated saline encroachment. Actual construction and installation of the project facilities were completed in February of 1953. First injection was commenced at approximately that time and has continued to date. The findings of this report are based on operations until June 30, 1954. During this period the project was financed with State funds until December 1953. For the balance of the period the operation of the project was financed by local tax funds and the purchase of the water from local contributions.

It may be concluded that the subject investigational work for the prevention and control of sea water intrusion has established that for areas with similar geologic, hydrologic and topographic conditions as those found at the test site in Manhattan Beach in the West Coast Basin:

(a) Such prevention and control can be successfully realized in a confined coastal aquifer by recharge through wells.

(b) Such recharge can pressurize a confined aquifer continually through a given reach, thereby reversing any pre-existing landward gradient and preventing further sea water intrusion.

(c) Such a recharge will provide significant replenishment to the basin with an insignificant loss of fresh water seaward.

(d) Such recharge can be performed in an aquifer previously degraded by sea water intrusion and, within the limitations as established at the test site, without any deleterious effect. In fact, all evidence collected to date indicates that such a degraded aquifer can probably be reclaimed by recharge through wells.

The agreement specifically provided that certain factors be determined if possible. These factors are listed below and our findings are summarized therewith:

1. Feasible rates of injection through wells as related to thickness, permeability and other properties of the aquifers and variation in rates of injection with pressure head built up in the well and with time.

Rates of injection from 0.5 cfs to over 1.0 cfs per well were established as feasible at the test site. The rates were dependent on the type and size of recharge well, the methods used in drilling the well, the local transmissibility characteristics of the aquifer, and the injection head in the recharge well. With the wells as constructed for the test, and with aquifer characteristics as they existed at Manhattan Beach, rates of over 1.0 second foot were maintained by injection heads from 27 to 50 feet at the gravel-packed wells, while approximately the same heads were required for one half of these rates, or 0.5 second foot at the non-gravel-packed wells. The increase of injection head noted at the recharge wells following initial injection was principally a result of the build-up of the pressure ridge along the recharge line. The actual required injection head is the difference between the water surface elevation in the recharge well and the maximum elevation of the pressure mound at the face of the aquifer (the vertical undisturbed surface of the aquifer exposed to flow from the recharge well.) Within the time limits of this test it would appear that reasonably constant acceptance rates at the recharge well can be maintained with proper chlorination, as discussed in (8.) below.

2. Height and shape of pressure ridge that can be built up as related to thickness and permeability of aquifer and hydraulic gradient.

With a recharge rate of 5 second feet per mile of coast line, an aquifer depth of approximately 110 feet below sea level, an average transmissibility value of 0.18 cfs/ft., and a stabilized inland gradient of 0.0065 ft./ft., characteristics similar to those encountered at Manhattan Beach, it was possible to create a pressure mound approximately 15 to 16 feet above existing ground water levels. Initially recharge at each well establishes independent gradients radially until flow from each well merges and stabilizes an inland gradient. The quantity of this flow may then be established from the relationship $Q = Kai$ (Darcy's Law) or, as discussed in this report, $Q = TiL$, where T is the transmissibility, i is the landward piezometric gradient, and L is the length of recharge reach considered.

With a well spacing of 500 feet, the differential in pressure levels, along the recharge line, varied less than two feet from a point 20 feet from the recharge well to the point of lowest elevation midway between recharging wells. Normal to the recharge line, assuming symmetrical mound conditions immediately adjacent to the recharge well, the peak of the pressure mound 20 feet from the recharge well was only some two feet above an extension of stabilized constant gradient oceanward and landward of the recharge line. The absence of curvature in the stabilized gradients some 250 to 500 feet inland from the recharge line gave evidence that complete mergence of the individual pressure mounds had probably been established at such distances.

Although pressurization effects were observed almost instantaneously for a distance of over 1000 feet landward when injection was initiated at a single well, several days elapsed before the pressurization appeared to stabilize. Actual stabilization of the complete pressure ridge was not otained until six months after initiation of recharge. This is not representative of a minimum time required for such stabilization in that recharge operations at the individual wells were undertaken in sequence and injection over the entire recharge line was also delayed for approximately six months.

The gradients established from a recharge well are definitely a function of the total recharging flow and transmissibility of the aquifer. With properly designed gravel-packed wells and resultant higher injection rates and/or reduced well spacing, it would be possible to create much higher pressure mound levels than those obtained at the test site. The total pressure mound height at a point a given distance from a recharge well is not solely dependent on the amount of recharge at that well, but is also affected by the input from all other wells in the recharge line. However, this additional effect lessens as the distance from the wells increases.

3. Required height of pressure ridge and amount of water necessary to inject to control intrusion of sea water as related to properties and depth of aquifer.

The minimum height of a pressure ridge should be maintained sufficiently high above sea level to counteract the effect of the density difference between fresh water and salt water for the maximum depth of the aquifer below sea level. This was accomplished at the test site with a total rate of injection of about 4.5 second feet for approximately a 4000-foot reach. This rate of recharge maintained the internodal points (point of lowest elevation approximately midway between recharge wells) between recharge wells at approximately 2.7 feet above sea level. This was approximately half the amount originally estimated to be required for the formation and maintenance of the mound, based on the data then available from experiments of recharge at a single well.

4. Required spacing of injection wells to control sea water intrusion as related to thickness of aquifer, permeability and hydraulic gradient.

The spacing of the injection wells can be determined from the water requirements for a given reach to be recharged and the acceptance rate of the well, which is dependent upon the type of construction of the well and the local transmissibility characteristic of the aquifer. For example, the water requirement can be established from the transmissibility as determined from pumping tests, the initial landward gradient and an additional increment of flow (established in this test as approximately 60%) to provide for an apparent inland storage demand. (Reference is made to Chapter V, page 44, which discusses this requirement more fully.) If the water requirement were determined to be five second feet per mile and each recharge well's acceptance rate were estimated to be one second foot, then five wells would be required per mile of reach.

5. Gradient of the piezometric surface in the water-bearing deposits and its effect on the quantity of water injected and the shape of the pressure mound.

Although the initial gradient and transmissibility determine the recharging water requirement, the initial piezometric gradient has no further effect on the stabilized recharge pressure ridge except that the resultant increase in rate of flow landward distorts the symmetry of the initial recharge pressure cone. In this connection it may be noted that, as the inland gradient steepens, the inland flow increases, thereby increasing the water requirement to maintain the pressure ridge. However, since the flow towards the ocean does not depend upon the inland gradient, the proportion of the total recharging flow that will move inland to replenish ground water supplies is increased.

With conditions as they existed at Manhattan Beach and with a pressure ridge sufficiently high to offset the density difference of sea water, it can be shown that approximately 5 per cent of the total recharging flow is moving oceanward at such a slow rate that actual loss of fresh water to the ocean is not expected within a ten-year period.

6. Rates and amounts of displacement of saline waters and/or degree of dilution of saline waters.

Conductivity traverses corroborated laboratory findings by the University of California at Berkeley, that saline intrusion was taking place as an underriding wedge of salt water. These traverses also showed that the movement of the injected fresh water progressed as an overriding wedge with but slight dilution between the existing saline intruded waters and the injected fresh waters. The practical effect of this has been to minimize the landward push of the trapped saline waters. Recharging flow in general decreases the over-all salinity in the aquifer with a limited amount of dilution. The actual degree of dilution has not been accurately determined in that the location of perforations and the effect of non-homogeneity in the aquifer precluded such determinations. Within a 21-month period of recharge, (including 6 months following this report period) the movement of this overriding fresh water wedge has been traced some 2000 feet inland through waters having a minimum chloride content of some 4000 ppm, while its oceanward movement has been less than 500 feet.

7. Effect of ground water extractions, by pumping in the inland areas, on the rate of injection necessary to control sea water intrusion.

The effect of ground water extractions by the nearest pumping, some 6500 feet inland of the recharge line, had little immediate effect on recharge operations except in the manner in which such pumping may increase the general inland gradient. Hence, it may be assumed that the rate of required injection will vary in relation to such an increase. The actual valuation of this effect could not be determined within the period of this test.

8. Degree of chlorination or other treatment necessary for continued injection of water at feasible rates.

The test established that a chlorination rate of less than 10 ppm and more than 5 ppm was required at Manhattan Beach area to control the growth of slime-forming bacteria which tend to accumulate at the well perforations and in the face of the aquifer, thereby decreasing the well's acceptance rate.

Dr. Carl Wilson, District Consultant on water quality problems during this test, states:

> "Experience at Manhattan Beach seems to point strongly toward 20 ppm as a proper initial dosage to prepare the aquifer vestibule to a sufficient distance to permit injection at the desired rate. After stability has been attained and maintained for a period of, perhaps a week, then the dosage may be reduced 25 per cent. Experience leads to the belief that after stabilization has been attained, a dosage of 8 to 10 ppm will be required to maintain a constant injection rate."

9. *Maintenance of wells, including procedures such as sand bailing, surging, de-aeration, and studies of formation of microorganisms and the effect of chlorination or other methods of disinfection on their growth, and studies of base-exchange reactions and suspended solids deposition.*

The fifteen months' test period has required no redevelopment except as a result of inadequate well construction. Hence, little experience has been gained relative to the ultimate useful life of the wells and required redevelopment. However, considerable experience has been gained in relation to the construction and rehabilitation of recharge wells relative to such matters as grouting, desirability of gravel-packing, construction of cement seals, and proper operational procedures.

Particular reference is made to Appendix B of this report, which outlines the effects of base-exchange reactions. Although not apparent, during the limited period of this test, it may be expected that ultimately the injection of fresh water into the sodium-saturated sediments at the test site will increase the transmissibility to a degree, thereby increasing the total recharge requirement slightly.

Other findings pertinent to the creation of a fresh water pressure barrier to prevent and control sea water intrusion are enumerated in the Conclusions and Findings of the report.

In addition to establishing a successful method of preventing sea water intrusion as indicated by these findings and conclusions, it appears basic information has been gained which will be of value in areas threatened by sea water intrusion, not only in this State but in the various coastal areas of the United States.

Throughout the test, the District has worked closely with representatives of the Division of Water Resources, and appreciates their cooperation in carrying out the objectives assigned by the State Water Resources Board.

Yours very truly,

H. E. Hedger, Chief Engineer

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CHAPTER I
HISTORY OF THE PROJECT

A. DEVELOPMENT OF COASTAL SALINE ENCROACHMENT

Probably the most complete information available on critical sea water intrusion has been compiled for the West Coastal Basin, which is located in Los Angeles County, California. In that area heavy pumping draft and lack of adequate ground water replenishment have resulted in the increasingly rapid intrusion of ocean water which threatens the entire fresh water reserves of the basin. The possibility of serious ground water depletion in the area was predicted by M. C. Mendenhall of the United States Geological Survey in the early 1900's; subsequently, comprehensive studies were undertaken, judicial action commenced and experiments made to develop methods to prevent, or at least alleviate, this continuous destruction of one of our most valuable resources.

In 1932, J. H. Dockweiler (consultant to the Los Angeles County Flood Control District), proposed to replenish ground water reserves and create a fresh water barrier through the use of recharge wells.

In 1946, Consulting Engineer Harold Conkling was engaged by the West Basin Water Association to study the situation and make recommendations. His conclusions indicated an annual overdraft of 53,000 acre feet in 1946, and he strongly recommended the immediate import of Colorado River water, which has since been realized through the Metropolitan Water District's West Basin feeder.

In connection with its water conservation duties, the Flood Control District has been sampling ground waters in the area for a period of about 25 years. As the chemical analysis showed an alarming increase in saline concentrations along the seaward side of the basin, it was called to the attention of interested cities and local agencies. This resulted in an agreement for a cooperative ground water investigation of the critical situation in the West Basin between the United States Geological Survey's Water Resources Division, Ground Water Branch and the Los Angeles County Flood Control District in July, 1943. The District also represented the joint interests of the cities of Inglewood, Redondo Beach, Manhattan Beach, El Segundo, Hawthorne, Culver City, Gardena, Hermosa Beach, and the Palos Verdes Estates. This agreement was culminated in May, 1948, by a report entitled "Geology, Hydrology, and Chemical Character of the Ground Waters in the Torrance-Santa Monica Area, Los Angeles County, California," by J. F. Poland, A. A. Garrett, and Allen Sinnott of the United States Geological Survey. This report suggested three physical possibilities for the local control of saline water, as follows:

"(1) The construction of artificial subsurface dikes or cutoff walls;

"(2) The development, by pumping, of a water-level trough coastward from the saline front; or

"(3) The maintenance of fresh-water head above sea level at, and immediately inland from, the saline front. Only the maintenance of fresh-water head is considered to be an economic possibility. The fresh-water head required along the west coast would range from 3 to 13 feet above sea level. It could be attained only by artificial recharge through wells, trenches, or pits."

District experience indicates that in the unconfined aquifers of the area, fresh water head can be attained through the use of spreading basins or through the use of gravel-packed seepage pits.

In a report dated December 6, 1950 and entitled "Sea-Water Intrusion into Ground Water Basins Bordering the California Coast and Inland Bays," the Division of Water Resources, State of California, listed critical areas of sea water intrusion. It particularly states: ". . . The most serious intrusion to date is occurring in the overdrawn West Coast Basin in Los Angeles County and the Central Coastal Basin in Orange County." The trend of sea water intrusion can be traced by the history of chloride concentration at several West Coast Basin observation wells as depicted on Plate 2. The report also discusses possible methods of restraint of sea water intrusion as follows:

"(a) Raising of ground water levels to or above sea level by reduction and/or rearrangement of pattern of pumping draft.

"(b) Direct recharge of overdrawn aquifers to maintain ground water levels at or above sea level.

"(c) Maintenance of a fresh water ridge above sea level along the coast.

"(d) Development of a pumping trough adjacent to the coast.

"(e) Construction of artificial subsurface dikes."

It further states:

"Cost is an important factor which must be considered in the selection of proper method of control.

"The first method of control, while not necessarily the most desirable, would always be effective in the coastal ground water basins. A detailed and extensive engineering investigation would be required in order to establish salient hydrologic and geologic features necessary for the determination of a long-term basin-wide balance of draft and replenishment and program of pumping draft reduction or relocation. The other methods would require additional detailed investigation and experimentation in order to determine their feasibility as sound engineering and economic methods of restraint of sea-water intrusion. It is probable that the solution for restraint of sea-water intrusion in a particular ground water basin might utilize one or more of these latter methods in conjunction with raising of ground water levels to or above sea level by reduction or relocation of pumping draft."

B. PRIOR DISTRICT INVESTIGATIONS

In addition to its previously mentioned sampling program and cooperation with the Geologic Survey, in April, 1949, the District cooperated in a report on the "Reclamation of Water from Sewage and Industrial Wastes in Los Angeles County, California," which was published by a Board of Engineers, appointed by the Board of Supervisors of Los Angeles County, and composed of C. E. Arnold, County Engineer and Surveyor, H. E. Hedger, Chief Engineer, Los Angeles County Flood Control District, and A. M. Rawn, Chief Engineer and General Manager, Los Angeles County Sanitation Districts. Mr. Rawn acted as Chairman of the Board. This report proposed to spread reclaimed effluents from the Hyperion Plant and Los Angeles County Sanitation Districts' Plant at potential spreading sites in the sand dune areas in the vicinity of Playa del Rey and east of Redondo Beach, respectively, in order to more quickly recharge the heavily-pumped Silverado aquifer.

The District's experience with the development of ground water mounds, in connection with the operation of spreading grounds, and earlier recharge tests led it to the belief that the aforementioned fresh water ridge along the coast was the most promising solution to the problem. Consequently, in the spring of 1950, a search was undertaken for spreading sites. Surface test plots were located in the Redondo Beach and Playa del Rey areas, in which geologic reports had indicated zones where open aquifers existed. In the intermediate pressure zone in Manhattan Beach, a well recharge test was undertaken.

Experimental well recharging in one well of an abandoned Manhattan Beach well field resulted in the conclusions that:

"1. Of the possible methods for the restraint of sea water intrusion, it is believed that the creation of a fresh water ridge along the coast is the most promising solution to the problem.

* * * * *

"3. Bacterial slimes will form and clog the aquifers being recharged unless the flow is sterilized.

* * * * *

"7. It is desirable to exclude air from the recharge flow.

* * * * *

"11. The pressure elevations created by recharge are approximately proportional to the rate of recharge."

A report was prepared discussing these tests in detail, and entitled "Report on Tests for the Creation of Fresh Water Barriers to Prevent Salinity Intrusion, Performed in West Coast Basin, Los Angeles County, California," dated March 10, 1951. It recommended that bills pending at that time before the State Legislature, providing funds for the further investigation of saline intrusion, be approved and that funds be made available for a large scale test in the West Coast Basin.

C. LEGISLATIVE ACTION INITIATING BARRIER PROJECT

As a result of the recharge test at Manhattan Beach and studies of other critical areas in the State by the Division of Water Resources, and through the efforts of local interests in the West Basin area, the Legislature of the State of California provided, by Chapter 1500, Statutes of 1951, $750,000 to be used "for investigational work and study with the objective of formulating plans and design criteria for the correction or prevention of damage to underground water of the State by sea-water intrusion in the West Coast Basin of Los Angeles County and other critical areas." Planning and allocation of the funds were assigned to the State Water Resources Board.

On July 16, 1951, the Division of Water Resources was requested by the Water Resources Board to investigate and prepare a report in reference to an investigation program for the control and prevention of sea water intrusion into ground water basins bordering California coastal and inland bays. The Division, in its report dated August 31, 1951, recommended that a field experimental project be conducted in the vicinity of Manhattan Beach. (See Plate 1)

On September 7, 1951, these recommendations were approved by the Water Resources Board and an allocation of $450,000 was made for the installation and operation of a field experimental project at Manhattan Beach. Effective October 1, 1951, the Water Resources Board entered into a contract with the Los Angeles County Flood Control District for the installation and operation of the experimental recharge test for a period of one year.

Subsequently, several modifications to the contract were made concerning extensions of time and facilities. Additional funds were allocated to the project to finance the extended facilities and period of operations. Total funds allocated to the project were $642,126.30. Except for the submission of a final report, the Water Resources Board's participation in the barrier project ceased on December 31, 1953, when the last of the funds allocated to the project were exhausted.

D. SUBSEQUENT PROJECT ALLOCATIONS

Between January 1, 1954, and June 30, 1954, the project was operated by general funds of the District with funds for defraying water costs supplied by the West Basin Water Association through contributions and assessments in the West Basin area.

After July 1, 1954, Zone II of the Flood Control District provided funds, through ad valorem taxes, to purchase water for continued operation of the barrier. District general funds were again used for operating expenses.

CHAPTER II

PROJECT FACILITIES

A. PROJECT PLANNING

General planning for the project began in October, 1951 with the ratification of the contract with the State Water Resources Board. Preliminary plans had been prepared and were included in recommendations in the 1951 report on the Manhattan Beach well recharge test.

Studies were made of water supply as it concerned length of necessary pipelines and of location of recharge wells. Other considerations relative to the location of the recharge line were: the amount of recharge water needed; possible waste of recharge water to the ocean; availability of right of way for drilling recharge wells; and possible routes of an extension of the recharge line.

It was decided that the most logical location for the recharge well line would be along the Atchison, Topeka and Santa Fe Railway Company right of way, paralleling the coast along Ardmore Avenue in Manhattan Beach about ½ mile inland. The general location of the project in Los Angeles County is shown on Plate 1 and the detailed layout of facilities is indicated on Plate 3.

Following this decision, plans for the transmission pipeline proceeded rapidly. A study of utility services on two alternate routes indicated that the most favorable pipeline location for delivery of fresh water for recharge would be on Manhattan Beach Boulevard.

Many permits for access and land use, in lieu of purchasing right of way, were obtained at no cost to the project. Principal among these were those obtained from the A.T. & S.F. Railway Company for the use of its right of way for laying pipeline and drilling wells; from the City of Manhattan Beach for laying pipeline; and the cities of Manhattan Beach and Hermosa Beach for drilling observation wells. Private individuals also cooperated in providing permits for drilling observation wells on their property.

Agreements were made with the City of Manhattan Beach, California Water Service Company, General Chemical Company, and the Standard Oil Company permitting the District to make field and laboratory water quality studies of certain key water wells.

At the request of the State Water Resources Board, negotiations for waiver agreements, holding the State harmless of liability, were initiated in March 1952 with certain property owners in the vicinity of the test site whose wells were possibly subject to damage due to acceleration of saline intrusion of ground water as a result of recharge experiments. Processing of these agreements delayed drilling operations, then in progress. The last of these agreements was completed by the California Water Service Company on July 16, 1952. Agreements with Standard Oil Company of California, the City of Manhattan Beach, and the Messrs. John, Philip G., and John Shaw of Hermosa Beach had been completed previously.

B. APPURTENANT PROJECT INSTALLATIONS

Major installations for the project consisted of the laying of a pipeline and the drilling of recharge and observation wells. A combination field office and chlorination building was also constructed.

The transmission line design was prepared by the Design Division of the District. In order to minimize capital outlay, asphalt lined, thin gauge welded steel pipe was used.

To provide the water supply estimated to be needed for the test, and to allow for the contingency of a possible additional required supply, the pipe line was designed to carry 15 cfs. The contract for the pipeline installation was awarded in the fall of 1952 and completed in March, 1953.

The connection with a source of imported water for the test was made in a vault near the intersection of Redondo Avenue and Manhattan Beach Boulevard. At this point the City of El Segundo and the City of Manhattan Beach have connections with the West Basin Feeder of the Metropolitan Water District. The transmission line was connected to the City of El Segundo line in such a manner as to allow for metering and billing of the test water supply through the facilities of the City of El Segundo.

The transmission line was laid westerly along the south side of Manhattan Beach Boulevard to its intersection with the A. T. & S. F. Railway right of way and thence southerly along the railroad right of way paralleling Ardmore Avenue to the vicinity of 28th Street in the City of Hermosa Beach. (See Plate 3) Pipeline construction involved:

(a) Installation of 12,098 feet of mechanically coupled, ten and twelve gauge standard asphalt dipped welded steel pipe, valves, and fittings.
(b) Jacking of reinforced concrete pipe to house the steel pipe, valves and fittings.
(c) Construction of concrete vaults and thrust blocks.
(d) Placing approximately 250 tons of pre-mix resurfacing.

The following pipe was installed:

20 inch O.D., 10-gauge	10,000 feet
16 inch O.D., 12-gauge	1,034 feet
14 inch O.D., 12-gauge	1,064 feet

Because the National Production Authority's restrictions upon the use of steel were in effect at the time the transmission line was being planned, the Flood Control District found it necessary to purchase certain materials before the contract was let for the construction of the pipeline. Consequently, the District furnished all pipe, fittings, valves and reinforcing steel. The contractor, E. W. Cannell of Gardena, California, furnished all other materials, equipment and labor.

C. WELL INSTALLATIONS

Wells were drilled for the project for two general purposes, recharge and observation. Most of the wells were drilled by "cable tool" methods in order to obtain the best well logs possible. Drilling of the initially-planned group of wells was begun in January, 1952 and completed in January, 1953. This group included nine 12-inch wells and thirty-six 8-inch wells. Later in 1953, one 12-inch well (cable tool and rotary), replacing one of the original recharge wells, and eighteen 2-inch wells (combination rotary, driven and jetted) were added to the program. In 1954, four 4-inch wells were drilled by the "rotary" method to provide further control data for the recharge operations. The arrangement of the well pattern is shown on Plate 3. Drilling operations are pictured in Photo 1. well G-13, drilled near the ocean, is shown on Photo 4.

From the results obtained in the 1950 well recharge test and because of economic limitations, it was felt that 12 inches would be an optimum diameter for recharge wells. Of the original nine wells drilled, eight were standard 12-inch cased wells with perforations cut, after drilling was completed, in zones as indicated by drilling logs. The gravel-packed recharge well, E, of the original group was begun by drilling with cable tools in a 20-inch diameter casing and shoe down to the clay cap. A special reaming tool was used to taper the diameter of the hole in the clay cap to approximately 36 inches. The enlarged area below the 20-inch casing was filled with cement grout and the 20-inch casing was then pushed into the grout. Drilling for a 12-inch casing was begun through the cement grout utilizing a 12″x15″x3″ gravel envelope well drive shoe and continuing on down to the required well depth. The annulus between the 12-inch casing and the 18-inch drilled hole was maintained full of gravel as drilling progressed.

When it became necessary to replace recharge well I in the fall of 1953, the experience gained during recharging operations to that date was considered in deciding what type of well to drill. It had been found that a gravel-packed well was far superior in recharge acceptance rate than the other wells. Also, it had been found that the "clay cap" in the vicinity of the recharge line was easily erodible and that it was difficult to form a seal between the clay cap and the casing. To meet these difficulties, the replacement well, 1A, was drilled in somewhat the same manner as well E, with modifications as indicated in the following:

(a) A 2-inch hole was drilled well into the clay cap to obtain information on location and extent of the clay cap at that location.

(b) A 12-inch pilot hole was drilled about 5 feet into the clay cap and reamed out to a 30-inch hole throughout its entire length.

(c) The bottom 2.5 feet of the 30-inch hole was underreamed to a 44-inch diameter.

(d) A 20-inch conductor pipe was lowered into the well to the bottom of the 30-inch hole.

(e) While the hole still contained drilling mud and with the conductor pipe sealed at the top so as to be airtight and prevent grout from rising in the 20-inch conductor, the annulus between the conductor and the rotary hole was grouted through a 2-inch pipe tremie placed inside the conductor, and extending to near the bottom of the hole. Grouting was continued until grout encased the conductor to the ground surface.

(f) After grouting, a rotary rig was used to remove grout remaining inside the conductor pipe.

(g) Cable tool equipment was then used to drill and gravel-pack an 18-inch well with a 12-inch casing, utilizing a 12″ x 15″ x 3″ gravel envelope well drive shoe to the required depth into the aquifer.

Perforations in all the recharge wells, except well E, were made with a Moss Knife. The dimension of cuts ranged from $\frac{1}{8}''$ x $1\frac{5}{8}''$ to $\frac{5}{16}''$ x $1\frac{5}{8}''$. Well E was perforated with a Mills Knife, the cuts being $\frac{5}{16}''$ x $2\frac{3}{4}''$. The Mills Knife makes a vertical cut in the casing. (See Photo 2) In contrast, the Moss Knife makes a horizontal slit in the casing and through hydraulic pressure forces the casing above the slit outward into the formation forming a protective louver over the perforation in the casing. This type of perforation is valuable in a predominantly sandy formation because it inhibits the movement of sand from the formation into the well.

Following the perforation, wells were surged and bailed until sufficiently stabilized to allow satisfactory pump operation. Normal bailing and surging extended over a period of 16 to 18 hours.

Pump development was done by installing an 8-inch well pump of 1500 gpm capacity and continuing pumping and surging until discharge carried only slight traces of sand or silt immediately following three consecutive surgings. Pump development extended over a period of some 24 hours. Upon completion of pump development each recharge well was continuously pumped for periods of 6 to 12 hours to conduct transmissibility tests. Photos 3 and 4 show the development pump, surface spreading for disposal of pumped water into the dune sands, and the measurement of pumped water during transmissibility tests.

During pumping development, measurement of discharge was made by utilizing orifice plates, and during transmissibility tests a 4′ x 4′ x 16′ weir box, utilizing a V-notch weir, was installed to provide more accurate discharge data as well as a continuous automatic water stage record.

No attempt was made to determine the pumping capacity of any of the recharge wells, development procedures being limited to removing only sufficient sands and silts from the gravels of the ground water aquifer to provide an effective filter around the perforated portions of the casing.

All of the original group of observation wells were 8 inches in diameter. This size was chosen primarily due to the fact that it was desirable from the geological logging aspect to have a moderate sized cable tool drilled well. Also, wells of this size were more adequate for use in the sampling program (described hereafter) because the 4-inch submersible pump could be readily used inside the 8-inch casing, and permitted a conductivity cell to be raised past the pump in the well in order to collect data on conductivity under pumping conditions.

Perforations in all the 8-inch observation wells were cut with a Mills Knife. The common perforation was $\frac{3}{16}''$ x $1\frac{3}{4}''$, six cuts to the round, staggered on 1-foot centers, variations from this standard being determined by the size and depth of gravels encountered.

Bailing and surging for a period of 4 to 8 hours proved to be adequate development for observation and sampling. In the one instance where the development was inadequate, air-jet development utilizing a $\frac{3}{4}''$ airline with a 2-inch return pipe was successfully completed.

Extensive formation sampling was done, as drilling operations progressed, to determine the geologic and hydraulic characteristics of the underlying sediments and to select the most permeable zones for casing perforations. Twenty-five pound bulk samples were taken from the bailer, and drivecore samples were taken in selected zones with a $1\frac{1}{2}''$ diameter Dames and Moore core barrel (See Photo 2) or a 5-inch District vacuum type core barrel. The samples were forwarded to the District laboratory and the Division of Water Resources for screen analysis and permeability determinations.

Ground water samples were also taken from the bailer at approximately ten-foot intervals during drilling except in the more permeable zones, where sampling frequency was increased to four or five-foot intervals. The bailed water samples were analyzed in the field for chloride salinity, and for carbonate and bicarbonate constituents. Significant water samples were forwarded to the District laboratory for complete chemical analysis. Bacterial samples were also taken and forwarded to the District's consultant, Dr. Carl Wilson.

After recharge operations were begun, eighteen 2-inch observation wells were drilled adjacent to the recharge wells for purposes of measuring water surface above and below the clay cap, and in February, 1954, four 4-inch wells, financed with District funds, were drilled for both water surface level observation and pumping samples at the internodal points, midway between recharge wells.

Where it was not important to log the hole, the 2-inch wells were drilled by the rotary method with mud until the zone of interest was approached. These wells were then completed by driving and jetting pipe so that core samples might be obtained.

The driven and jetted wells could not be pre-perforated. Communication with the water table was established by drilling ahead of the 2-inch casing and then back-filling the open hole and the bottom 2 feet of pipe casing with pea gravel, or by drilling ahead, placing a 1-inch pipe with a screened well point within the 2-inch casing and gravel-packing around the well point. Both types proved to be satisfactory; however, both required frequent flushing to maintain reliable communication. The open bottom pipe with gravel proved the most practical, as initial installations were more economical, and also because the wells were better suited for flushing operations.

The 4-inch wells were rotary drilled using an 8-inch rotary bit and cased with a 4-inch pipe casing. Perforation was accomplished by including the 4-inch pipe casing sections of pre-perforated 4-inch pipe with 6 staggered rows of machine-cut slots, 125 mesh, 3-inch long, on 6-inch centers. These wells were provided with a gravel envelope merely by placing gravel through the annular space outside the pipe casing prior to development. Sufficient gravel was added through the drilling mud to fill the annular space up to the "clay cap" and cemented off at this point. The grout was placed through a pipe tremie.

Development was accomplished by injecting air under 85 pounds pressure to the bottom of the well through $\frac{1}{2}$-inch pipe. The maximum pumping rate during development of the 4-inch wells ranged from 60 to 90 gpm. Development period was approximately 16 hours for each well.

D. WELL HEADER ASSEMBLIES

Recharge well connections to the feeder line were made with standard 6-inch pipe and fittings in order to provide ample flow to any particular well, if needed. Included within the connection assemblies were short lengths of 6-inch pipe, elbows and flanges as required, a flexible bolted coupling, a 6-inch gate valve, and either a 6-inch meter or 6-inch flow rate controlling valve which includes a meter. Plate 4 is a section through a typical recharge well assembly and Photos 5 and 6 show the field installation of this equipment. Inasmuch as all recharge wells were adjacent to the pipeline, only minimum lengths of pipe were needed.

Flow rate controllers were installed in the assemblies of five of the recharge wells. They are designed to provide a constant flow under varying pressure conditions in the supplying line. Activation of the valve is dependent upon the flow indicated by an adjacent standard propeller-type meter. This meter can be used for totalizing, indicating and recording just as a standard meter. It was found that flow adjustment mechanism on the controller was not dependable enough for the requirements of recharging a well and, thus, that they were not useful for the project. Since it is desirable to maintain a constant flow at the injection wells, it is recommended that a pressure regulator be installed in the supply pipeline if it is subject to any more than minor variations in pressure.

A 6-inch flow conductor pipe was suspended in each recharge well to a position below the minimum water level in the well casing. For details of the suspension of this pipe, see Plate 4.

Back-pressure valves were installed in all these suspended pipes in order to keep the 6-inch pipe full of water and thereby exclude air from the recharge flow. This valve is essentially a piece of 4-inch pipe sliding up and down in a larger piece of 5-inch pipe which is slotted on the lower half, sealed on the bottom and suspended from the bottom of the 6-inch conductor pipe by means of flanges. (See Plate 4) The valve is raised and lowered in the well and actuated by means of a one-half inch galvanized pipe or cable extending from the valve to a control assembly at the surface. By raising and lowering the 4-inch pipe sleeve from a control device at the surface, the desired flow rate is obtained. Importance of the back-pressure valves, as related to recharge acceptance rate, was established by the District injection test in the City of Manhattan Beach Well No. 7, when water was permitted to plunge downward in the recharge well, thus inducing air entrainment. The water surface in the recharge well rose considerably in that test, indicating a build-up of the air-water mixture which resulted in excessive well injection head. This fact was reaffirmed in a test at well E during the barrier test, as described in Chapter V.

To collect information upon the detailed changes of the rate of flow into the recharge wells, a totalizing, indicating and recording instrument was attached to the meter of each well. In an operating barrier project it would be unnecessary to have more than a totalizing meter at each well as adequate control can be maintained through a recorder on a mainline meter.

E. FIELD OFFICE AND CHLORINATING EQUIPMENT

A wooden frame structure, to house a 20′ x 23′ field office and a 14′ x 11′ chlorination equipment room, was constructed by District force account along the Santa Fe right of way near the corner of Ardmore Street and Manhattan Beach Boulevard in Manhattan Beach. The field office included space for a small laboratory and office equipment. A roofed enclosure was provided at the north end of the structure for chlorine tank storage. (See Photo 8)

Chlorinating equipment, furnished to the Los Angeles County Flood Control District by Wallace and Tiernan Company on a rental basis, consisted of:

(a) Automatic control solution feed master chlorinator designed to meter the chlorine gas under a vacuum and automatically control the rate of feed. The chlorinator was capable of delivering from 50 to 2500 pounds of chlorine per 24 hours.

(b) Automatic chlorine evaporator, of the hot water type, to convert liquid chlorine into gas and so designed that excessively high pressures could not be developed under any condition of operation. The chlorine cylinder stood vertically in a hot water bath and was supported in such a manner as to provide free circulation of water. The evaporator was heated with immersion-type electric heating elements.

(c) Automatic chlorine shut-off valve of the pilot-operated diaphragm type using air as the pilot medium. The valve was designed to reduce the pressure of the gas, as delivered from the evaporator, to that required for the best operation of the chlorinating equipment and to prevent reliquefaction occasioned by adverse temperature conditions. The valve was also designed to automatically shut off the flow of gas, should the temperature of the evaporator bath fall below the operating range.

(d) Residual chlorine recorder which measured the residual chlorine in a continuous sampling cell and recorded on a 24-hour chart graduated from zero to 20 ppm.

(e) Air compressor complete with motor, tank and appurtenances.

(f) Chlorine solution pump complete with 20 H.P. 220/440, 3-phase, 60-cycle drip proof electric motor, magnetic starter and base plate.

(g) Pipe, valves and fittings which were rubber-lined.

The installation of the major portion of this equipment is shown in Photo 9.

CHAPTER III
GEOLOGY

A. GENERAL PROGRAM

A geologic investigation was conducted to provide subsurface geologic data in the coastal test reach seaward of Sepulveda Boulevard where no prior data was available. Resultant data constituted a basis for determination of the component materials, extent and thickness, and other pertinent characteristics of the affected major aquifer which controlled recharging, and for conclusive interpretations of the hydraulic and water quality studies. The following constitutes the pertinent phases of a detailed report on the Geology of the West Basin Barrier Test included as Appendix A.

The program involved the drilling, coring, sampling and logging of nine injection and thirty-six observation wells, located on Plate 1 of Appendix A. Some eighteen 2-inch and four 4-inch test and observation wells were added later during the test. Simplified graphic descriptions of subsurface deposits encountered during drilling may be noted on geologic sections 1-1, 4-4, 14-14, C-C, G-G, and K-K. (Plates 5, 6, and 7 of Appendix A)

In substance, drilling disclosed, at the base of the sand dunes, an extensive relatively impervious stratum called the "clay cap," which was underlain by a confined aquifer approximately 110 feet in thickness. The confined aquifer was found to be in continuity with such important prolific inland aquifers as the "200-foot sand," the "400-foot gravel," and the Silverado zones. These important inland aquifers combine or merge as they approach the coast, and are then termed the "Merged Silverado Zone." Fine grained sediments underlying the Merged Silverado constitute the lower boundary of the aquifer. A zone of sharp flexuring or faulting noted seaward of the test site may act as a partial barrier to sea water intrusion. This is corroborated by the change of piezometric gradient indicated from measurements of water levels prior to recharge at observation wells located seaward of the recharge line. (See Plate 15)

B. NATURE AND EFFECTS OF THE SAND DUNE DEPOSITS

In general, three significant horizons were noted within the sand dune deposits overlying the clay cap: a localized surficial "iron-bound" sand horizon, an intermediate horizon of relatively clean dune sands with occasional gravels in the basal portion, and a lower horizon of fine sands and silts with sandy stringers which constituted a zone of transition to the "clay cap."

Two-inch test wells adjacent to the injection wells were bottomed above the "clay cap" but within the lower horizon of the sand dunes to determine leakage from the aquifer. Subsequently, shallower test holes were drilled near the injection wells to check the levels observed in the original two-inch wells. These shallower wells confirmed the belief that recorded high water surface levels in the original test wells reflected semi-confined or partial pressure levels within the zone of transition to the "clay cap" rather than free water levels. Hence, leakage from the aquifer to sands lying above the clay cap was probably limited to that transmitted by the sandy stringers within the zone of transition to the clay cap. Inasmuch as the stringers were of minor thickness, flow through the stringers was doubtless equally limited and hence the leakage above the clay cap during the test is considered to be of minor significance.

C. EXTENT, NATURE, AND LIMITATIONS OF THE AQUIFER CAP

Logging showed, as anticipated beneath the sand dune cover in the test reach, the presence of a relatively impervious stratum called the "clay cap," averaging 20 to 30 feet in thickness. (Plate 4 of Appendix A) The upper surface of the cap varied in elevation from about 10 feet above sea level to 10 feet below sea level, (Plate 2 of Appendix A) and the lower surface from about 10 feet below sea level to 40 feet below sea level. (Plate 3 of Appendix A)

On the basis of available data, a previous geologic study in the area by another agency assumed that the relatively impervious stratum extended some distance seaward. Were this true, the underlying aquifer would have been exposed to sea water at some distance from shore. Test drilling, however, revealed the absence of the cap along the strand and at one point 800 feet inland, at test well K-9. The stripping of the "clay cap" is of significance in that it reduced the distance of travel of ocean waters and hence hastened saline encroachment at these points. The stripping may have retarded, but did not prevent pressurization of the aquifer during recharge since a continuous effective clay cap exists along the recharge line and seaward of it for some distance.

The aquifer cap was found not to be a true compact clay, being composed of vari-colored silts, sandy clays, and clays containing silty fine sand stringers. These deposits, typical of many coastal reaches of the State, obviously are not as impermeable as a true clay body and are subject to some degree of erosion if water is permitted to move at excessive velocities along or through the cap. Hence, the annular space between casings and side walls of injection wells must be properly sealed throughout the cap to prevent rupturing induced by sudden changes in injection rates, with consequent failure of repressurizing operations. The cap failures experienced at injection wells C, G and I were probably due to the creation of voids produced by overdevelopment and excessive leakage past the clay cap at the well casing. In contrast, the continuous successful operation of wells E and I-A, which were gravel-packed and grouted, would indicate the benefits of this type of well construction.

D. GENERAL PARAMETERS OF THE AQUIFER AS RELATED TO RECHARGE

The Merged Silverado Zone is the major prolific aquifer which lies beneath the clay cap but above the fine, compact blue-gray portion of the lower San Pedro formation constituting the lower boundary of the aquifer. The zone is composed of an upper brown phase, an underlying gray phase, and the basal Silverado, all in hydraulic continuity. The uppermost portion of the brown phase consists primarily of yellowish brown sands and silts, and the lower portion of gravel stringers, with occasional clay bands. The uppermost portion of the underlying gray phase consists of gravel, which grades progressively downward to very fine silty sands and clay bands. The basal Silverado is composed largely of sand and scattered gravel. The dimensions, pattern, and extent of well perforations were determined by the changing character of the aquifer sediments. The presence of fine sands and silts within the aquifer, particularly within the upper brown phase beneath the aquifer cap, requires care in well development to obviate excess removal of the supporting fine sediments.

The Merged Silverado Zone generally thickens irregularly with distance down coast (southerly) and inland from the ocean. (Plate 10 of Appendix A) This was confirmed by the original ground water gradient which also receded with distance down coast and inland in the vicinity of the test site, indicating greater transmissibility. Along geologic sections C-C, G-G and K-K, (Plates 5 and 6, Appendix A) extending in a direction transverse to the recharge line, the depth is somewhat variable, being in general deeper inland than toward the south. Although the thickness of the aquifer varies greatly both seaward and landward along the recharge line (Section 1-1, Plate 5, Appendix A), the zone averages about 110 feet and the bottom elevation averages about 130 feet below sea level. The thickness along section C-C is about 90 feet, with the average elevation at the bottom being about 130 feet below sea level. The thickness along G-G averages about 100 feet, and the bottom elevation about 140 feet below sea level. The average thickness along K-K is about 120 feet, and the elevation of the bottom is about 160 feet below sea level. The thickness of the merged aquifer rapidly increases along the coast southerly of injection well K towards Redondo Beach and generally decreases northerly of injection Well C towards El Segundo.

The necessity for stemming sea water intrusion within the Merged Silverado Zone along the coastline lies in the fact that this zone bifurcates inland into several important water-bearing members; as for example, the upper brown zone, a correlative of a productive inland aquifer designated the "200-foot sand" zone. A coincidence is evident in the marked thinning of Merged Silverado sediments at the coast (Sections C-C, G-G, K-K, Plates 5 and 6, Appendix A) and the remarkable linearity and parallelism of the topographic features of provinces I and II with the present shore line, (Plate I, Appendix A). This coincidence suggests the posibility that deposition of sediments was, at least in part, controlled by sharp flexuring or faulting parallel to the coast. Such a zone of sharp flexuring or faulting may act as a partial barrier to sea water intrusion, and is evidenced by the steep piezometric gradient, as indicated by oceanward observation well water surface levels noted prior to recharging (Plate 15) and by differentials in elevations of stratigraphic surfaces zoned by foraminiferal assemblages, as noted in Appendix A (p. 83). This pattern of thinning of aquifer sediments with approach to a fault-controlled coastline is evident along other coastal reaches of the State and is of significance in that, where applicable, geologic delineation of the zone of thinning can define the most hydraulically effective and economical route for injection wells, provided right of way acquisition is economically feasible. Effective injection within the thinnest portion of the aquifer, in continuity with affected inland wells, as at the test site, obviously not only checks further major sea water intrusion with the least possible injection rate but also permits recharging of the ground water basin with least possible waste oceanward. It is significant that such recharge can be accomplished in a local area of critically depressed ground water levels which is no longer replenished by a distant forebay area.

Contours on both the top and bottom of the aquifer indicate several large irregularities, and reveal the bottom to be somewhat more variable than the top. (Plates 8 and 9, Appendix A) These surfaces indicate channeling transverse to the present shore line. An ancient channel transverse to the present shore line, extending inland from the recharge line toward the abandoned Manhattan Beach well field, is evident in the reach between injection wells G and I. This feature, coupled with the absence of the clay cap near this point at well K-9, 1000 feet west of the recharge line, has probably permitted a more direct contact of the Merged Zone with sea water at or near the strand rather than at some distance from the shore line as presumed in earlier investigations. Saline intrusion of the original Manhattan Beach well field probably was accelerated by existence of the channel and a steepened hydraulic gradient created by an accelerated drawdown of the nearby, now abandoned, well field. (See Plate 3)

The Merged Zone is considered to be laid down largely under continental and shallow marine conditions and hence it is to be expected that the nature of the deposits would vary greatly, both areally and with depth. This is evident from an inspection of the graphic well logs presented on Plates 5, 6 and 7 of Appendix A. This variation of course had a profound effect on the permeability and thickness of the recharged aquifer. While the thickness of the Merged Silverado Zone is considerable, those portions having a high permeability were found to be considerably less in thickness. Obviously then, the aquifer was not homogeneous and, despite the fact that the deposits were in hydraulic continuity, resultant variations both in vertical and lateral permeability doubtless constituted one control of the time lapse noted following changes in injection rate to effect a pressure mergence at the pressure mound internodal points. Recharging of the non-homogeneous aquifer was thus most effectively and expeditiously accomplished by the use of gravel-packed wells such as E and I-A.

E. NATURE OF THE LOWER BOUNDARY OF THE AFFECTED AQUIFER

Beneath the Merged Silverado aquifer was an extensive thickness of dark bluish-gray, very fine sands, silts and clays, which generally became more compact with depth. These relatively tight sediments constituted the lower boundary of the aquifer inasmuch as they significantly restricted downward movement of waters from the overlying aquifer. Obviously, the finer grained sediments underlying the merged zone are not impermeable but relatively so; hence, salinity intrusion within these sediments is of relatively minor concern. Transmission of pressure effects from the Merged Silverado zone to the underlying sediments eventually occurs within those portions having at least some degree of hydraulic continuity with the Merged Zone.

F. SEISMIC EFFECTS

Occasional earth shocks were noted on observation well charts. No damage to facilities was observed, however, and noted effects on aquifer transmissibility were, as far as known, only temporary.

G. RESUME

Obviously the detailed geologic data collected was vital to the conclusive interpretations of the hydraulic and water quality studies, and to the operation of the Barrier Test. More specifically the exploration determined such vital factors as:

(a) Effectiveness of the aquifer cap, so as to provide background for such construction and maintenance as was necessary to permit continued pressurization.

(b) Physical limits and homogeneity of the aquifer in relation to the validity of the hydraulic interpretations.

(c) Extent and variation in thickness of the aquifer along the line of recharge, and continuity of such deposits with correlative inland aquifers affected by sea water intrusion.

(d) Geologic structural controls which affected the movement of ground water as related to the establishment of a fresh water barrier to sea water intrusion and as related to the intrusion via ancient coastal channels prior to recharge.

The current detailed investigation should provide a valuable supplement to the basic geologic elements outlined during prior investigations of this area by the United States Geological Survey in 1948 and subsequently by the State Division of Water Resources for the Report of Referee on the West Coast Basin. This detail covers information on a reach more seaward than the area covered by the previous studies.

In conclusion, it is significant to note that, during the exploration program, geologic conditions were encountered that were directly pertinent to the construction of the wells and are fundamental to the methods employed in the creation of the barrier. Hence, it may be predicated that the creation of a barrier along other coastal reaches of the state should be preceded by sufficient exploration to establish a sound basis for design and operation.

CHAPTER IV

CHRONOLOGY OF RECHARGE OPERATIONS

A. OPERATIONAL PLANNING

Initial plans concerning recharge procedures involved the use of five 12-inch diameter injection wells stationed at intervals of 1,000 feet along the A. T. & S. F. Railway right of way. The use of four intermediate 12-inch wells was reserved for optional injection should it later be desired to reduce the well spacing from 1,000 feet to 500 feet. The plans also provided for intial recharge to occur at the centermost well, well G to be followed in order by consecutive recharge starts at well pairs, (1) E and I, and (2) C and K.

The following table indicates the preliminary recharge schedule with particular regard to proposed well rates and time intervals between well starts:

Time from Start (in weeks)	*Proposed Well Rates (cfs)*		
	Well G	*Wells E, I*	*Wells C, K*
0	0.50	0.0	0.0
2	Incr. to 0.75	0.50	0.0
4	Incr. to 1.00	Incr. to 0.75	0.50
6	1.00	Incr. to 1.00	Incr. to 0.75
8	1.00	1.00	Incr. to 1.00

Through approximate adherence to the above indicated schedule, it was felt that lateral mergence of individual well cones would be more uniform and that gradual build-up of the fresh water mound would enable data of greater significance to be obtained.

In conjunction with injection, considerable effort was expended upon the obtaining of data regarding piezometric surface and water quality. In order to provide the former information, 10 continuous water level recorders, supplemented with 9, furnished by the District, were installed in key observation wells. Additional water level data at the remaining wells were obtained from frequent tape measurements made at regular intervals.

Basic data on the subject investigation, as indexed in Appendix F, is filed at the District's office.

Information on ground water quality was provided by a comprehensive sampling program. This consisted primarily of obtaining conductivity traverses and pumped water samples from observation wells.

B. PERIOD OF MOUND BUILD-UP, 5 WELLS

History of operations at each recharge well is shown on Plates 6 to 13. These include the water surface elevation in the recharge well, the ground water elevation in the 20-foot observation well, the injection rate and the chlorination rate. Profiles of the ground water elevation along the recharge line for several dates are drawn on Plate 14 and profiles normal to the recharge line through well G on Plate 15.

February, 1953

Recharge was first begun with a three-hour trial injection run at well G on 2/14/53. Recharge and chlorination rates during this test period were 0.50 cfs and 20 ppm respectively. Installation of the mainline Wallace & Tiernan Company chlorinator was not completed until 3/24/53 due to delays in shipment of necessary parts. Until this date, chlorination was effected through use of a small-capacity portable chlorinator located at well G. This delay postponed injection at wells other than G until 3/25/53.

Continuous recharge of 0.5 cfs was initiated at the centermost injection well G on 2/24/53. Subsequent increases in recharge rate were made at this well after intervals of operation of approximately one week each. Rate of movement of this fresh water was closely timed to the nearer observation wells. Evidence of arrival at adjacent observation well G-1-20 feet distant—was first detected some 5½ hours after injection began. In turn, the saline wave preceding the injected water was first observed at well G-2, located 250 feet landward 17 days later on 3/13/53.

March-April-May, 1953

On 3/13/53, just two days after an injection rate of 1.00 cfs was reached at well G, a surface cave-in of considerable proportions occurred adjacent to this well. This resulted in a radical lowering of acceptance rate and it became necessary to reduce inflow in order to keep the injection head below ground surface. This occurrence is discussed in detail in Chapter VII. On 3/25/53 recharge of 0.5 cfs was initiated at each of wells E and I. In order to build up a more complete pressure barrier, injection at well E was increased to 0.75 cfs on 4/7/53 and to 1.00 cfs on 4/23/53.

A definite decrease in chloride salinity was first detected at observation well E-4, located 500 feet landward of recharge well E, on 4/22/53, some 28 days after injection was initiated at the latter well. The average landward pressure gradient between wells E-1 and E-4 during this period was approximately 0.010 feet per foot. The total water injected at well E by this date was 34 A.F.

Freshening was estimated to have first begun at landward observation well I-4, located 520 feet landward of well I, on about 7/7/53, 104 days after injection was initiated at well I. Average landward pressure gradient between wells I-1 and I-4 during this period was approximately 0.0126 feet per foot, while total water injected at well I was 91 A.F.

Pressure mound build-up was extended laterally by initiating recharge at wells C and K on 4/30/53 and 4/28/53 respectively. Recharge at the former well was discontinued on 5/4/53 due to apparent excessive leakage upward past the clay cap as indicated by test holes drilled to the clay cap.

On 5/10/53, mainline chlorination was reduced from 20 to 15 ppm. No adverse effect upon well acceptance was noted. (See Plate 18)

On 5/27/53, subsidence occurred at injection well I with the same general characteristics and results as previously observed at well G. In order to keep the injection head below ground surface, the injection rate was reduced from 0.5 to 0.2 cfs, and injection at a rate of 0.2 cfs or more was continued until October 14, 1953. No further subsidence has been noted at this well.

June, 1953

Recharge at well G was again resumed on 6/9/53 at 0.2 cfs after being inoperative from 4/12/53 to 6/8/53 for purposes of rehabilitation. Subsequent increases in rate were made at this well until 6/16/53, when an injection rate of 0.75 cfs was reached and maintained.

On 6/17/53 mainline chlorination was reduced from 15 to 12 ppm. No adverse effect upon well acceptance was observed. (See Plate 18)

C. PERIOD OF MOUND BUILD-UP, 8 WELLS

In order to relieve the pressure against the clay cap at the four operating recharge wells, plans were made to utilize the existing intermediate 12-inch recharge wells (D, F, H and J). This would permit a reduced injection rate and injection head at each individual well while the total flow required to maintain an adequate pressure mound could be sustained, but with a well spacing reduced from 1000 to 500 feet.

Injection at well H was initiated on 6/20/53. The initial rate of 0.5 cfs was increased on 6/24/53 to 0.75 cfs.

July-August, 1953

With regard to movement of injected water landward from well K, decrease in chloride salinity is estimated to have first occurred at observation well K-4 on about 7/15/53, 78 days after injection was initiated at well K. Average landward pressure gradient between wells K-1 and K-4 during this period was 0.0122 feet per foot, while the volume of water injected at well K was 66 A.F.

On 7/29/53 mainline chlorination was reduced from 12 ppm to 10 ppm. No adverse effect upon well acceptance was noted. (See Plate 18)

On 7/30/53 increases in injection rate were made at wells G, H and K from 0.75, 0.75, and 0.50 cfs to 1.0, 1.0, and 0.75 cfs, respectively.

September, 1953

On 9/2/53 injection at well H was reduced from 1.0 cfs to 0.6 cfs in order to decrease the injection head. Subsequently remedial work was undertaken at this well as discussed in Chapter VII.

On 9/11/53 mainline chlorination was reduced from 10 to 5 ppm. Reference to Plate 18 shows a decrease in acceptance rate indicating that chlorination at the latter rate was insufficient to maintain preexisting acceptance rates, although at the time the method of analyzing this data was not sensitive enough to indicate that fact.

On 9/16/53 mainline chlorinator was placed on automatic control. Through an interconnection with mainline flow meter, changes in chlorine addition (pounds/24 hours) thereafter were directly proportional to any changes in the water flow in the pipeline.

On 9/21/53 recharge of 0.3 cfs was initiated at intermediate well F. Further increases in flow were made in small increments on succeeding days until 9/23/53, at which time injection of 0.5 cfs was reached and maintained.

Similarly, injection at well J was initiated at 0.3 cfs on 9/29/53, followed shortly thereafter by small increases in flow until injection of 0.5 cfs was reached on 9/30/53.

October-November, 1953

Recharge at well H was discontinued on 10/5/53 in order to effect the remedial work at this well. By the time of shutdown, the indicated surface level of water above the clay cap, as indicated by well H-T-1, had risen to +48 feet M.S.L.

Injection of 0.2 cfs was initiated on 10/6/53 at well D. Additional small increases were made until 10/7/53 when a recharge rate of 0.5 cfs was reached and maintained.

D. PERIOD OF MOUND MAINTENANCE, 8 WELLS

Injection at well I was discontinued on 10/14/53 in order to complete drilling and development of gravel-packed replacement well I-A. Injection was initiated at the latter well on 11/10/53. The initial recharge rate of 0.25 cfs was increased in small increments to 0.5 cfs. Initial acceptance was quite satisfactory. During the period from 11/12/53 to 11/23/53, the elevation of water surface in well I-A rose from +7 feet to +11 feet M.S.L. while injection approximated 0.5 cfs. (See Plate 11) The net rise in water surface was 4 feet, compared to a net rise of 55 feet at well I for a like period and rate.

Initial injection rate at all wells was the same, 0.5 cfs. At this rate, an unbalanced barrier became evident as stability was approached. Individual well flows were adjusted to more nearly balance the recharge barrier. On 11/24/53 the following adjustments in well flows were made:

Well	*Previous Injection Rate—cfs*	*New Injection Rate—cfs*
D	0.5	0.2
E	0.5	0.4
F	0.5	0.4
G	0.5	0.4
H	0.5	0.4
I-A	0.5	1.0
J	0.5	0.7

This change created a minimum pressure mound equal to or higher than +2.5 feet elevation for 1550 feet along the line of recharge and above sea level for 4100 feet. Theoretically a fresh water head of 2.7 feet above sea level is required to balance the more dense sea water, assuming an aquifer extending to 110 feet below sea level.

December, 1953

Slight changes were made during December, 1953 in order to further modify extremes in level and raise the average height of the pressure ridge. Average length of the pressure ridge maintained equal to or higher than elevation +2.5 feet M.S.L., during the period, approximated 2,450 feet and above sea level for 4,150 feet. The changes consisted of the following:

Date	*Well*	*Previous Injection Rate—cfs*	*New Injection Rate—cfs*
12/17/53	I-A	1.00	0.75
12/30/53	D	0.25	0.50

January, 1954

Recharge line pressure levels were further increased on 1/4/54 by adjustments of flow rates at the following wells:

Well	*Previous Injection Rate—cfs*	*New Injection Rate—cfs*
E	0.4	0.5
F	0.4	0.6
G	0.4	0.6
H	0.4	0.6

The average length of ridge equal to or higher than elevation +2.5 feet M.S.L. during the month approximated 3,550 feet and above sea level for 4,700 feet. Chlorination rate was temporarily increased from 5 to 15 ppm on 1/26/54 in order to observe possible effect upon acceptance rates of recharge wells. Pre-existing rate of 5 ppm was again resumed on 1/29/54. As evidenced by Plate 18, chlorination of 15 ppm for this brief period caused no immediate discernible effect.

February, 1954

On 2/25/54 mainline chlorination was reduced from 5 to 3 ppm in order to determine long-term effects of lowered chlorination rate. The trend of lowering acceptance at well J prompted a reduction in recharge from 0.63 to 0.52 cfs at this well on 2/26/54 in order to keep the injection head below ground surface.

The average length of ridge extending to an elevation equal to or above +2.5 feet M.S.L. during the month approximated 3,600 feet and above sea level for 4,800 feet.

In order to provide more complete data on the shape of the pressure ridge, additional 4-inch diameter test wells D-E, E-F, H-I, and I-J were drilled during this month.

March, 1954

Acceptance rate remained nearly constant at all injection wells except J. Due to the apparent continuation of lessening acceptance at the latter well, it was again necessary to reduce the flow at this well. The reduction from 0.52 to 0.45 cfs was made on 3/16/54. Mainline chlorination was maintained at 3.0 ppm throughout the month.

The average length of pressure ridge maintained during the month at an elevation equal to or above +2.5 feet M.S.L. approximated 3,100 feet and above sea level for 4,500 feet.

April, 1954

Mainline chlorination was reduced to 1.5 ppm on 4/1/54 and maintained at this rate during the month. At all injection wells other than I-A, consistently decreasing acceptance was observed. At wells D and J temporary chlorination in excess of 1.5 ppm was achieved through the use of a portable chlorinator located at each well site. Chlorination of approximately 16 ppm was maintained at well J from 4/1/54 to 4/7/54 and chlorination of 20 ppm from 4/14/54 to 4/21/54. Chlorination at this latter rate was maintained at well D from 4/7/54 to 4/13/54. A temporary improvement in acceptance was noted at wells D and J during these periods of relatively higher chlorination. Although no sustained benefit was noted, it did indicate that chlorination at 1.5 ppm was insufficient to maintain pre-existing acceptance rates at these wells. (See Plate 20)

Due to a lowering acceptance rate, recharge at well D was reduced from 0.50 to 0.43 cfs on 4/6/54, while that at well K was reduced from 0.69 to 0.61 cfs on 4/11/54.

Additional changes were made at the following wells on 4/21/54 in order to obtain a more uniform barrier:

Well	Previous Injection Rate – cfs	New Injection Rate – cfs
D	0.43	0.34
E	0.50	0.45
F	0.58	0.53
I-A	0.80	0.90
J	0.42	0.45

The average length of pressure ridge extending to an elevation to or greater than +2.5 feet M.S.L. during the month approximated 2,600 feet and above sea level for 3,700 feet.

May, 1954

Injection at well J was discontinued on 5/5/54 in order to carry out plans to redevelop this well as discussed in Chapter VII.

The injection rate at gravel-packed well E was raised in small increments in order to determine the maximum possible acceptance rate into this type of well. A maximum flow rate of 1.86 cfs was reached at this well on 5/12/54.

An increase of mainline chlorination was made on 5/8/54 from 1.5 to 5.0 ppm in order to effect an improvement in well acceptance. Reference to Plates 18 and 19 shows that this rate was not completely effective.

In an effort to determine what effect injecting an aerated water supply might have upon well acceptance, aeration at gravel-packed well E was initiated on 5/14/54 and continued until 5/21/54. Further discussion appears in Chapter V.

The following changes in recharge rate were made on 5/25/54 in order to offset reduction in total recharge caused by well J becoming inactive on 5/5/54:

Well	Previous Injection rate – cfs	New Injection Rate – cfs
D	0.4	0.55
H	0.65	0.75
I-A	0.85	1.10

The average length of pressure ridge maintained during the month extending to an elevation equal to or greater than +2.5 feet M.S.L. approximated 2,200 feet and above sea level for 3,400 feet.

June, 1954

Injection at well J was resumed on 6/7/54 at 0.2 cfs. Subsequent increases in flow were made at this well until 6/8/54 when an injection rate of 0.4 cfs was reached and maintained.

In order to create a pressure ridge of more uniform height, the following adjustments to individual well flows were made during the month:

Date	Well	Previous Injection Rate – cfs	New Injection Rate – cfs
6/2 /54	D	0.45	0.3
6/18/54	D	0.3	0.4
6/18/54	E	0.3	0.45
6/18/54	F	0.5	0.6
6/18/54	G	0.5	0.65
6/18/54	J	0.4	0.55

The addition of 2 ppm sodium metaphosphate glass solution to well I-A influent was discontinued on 6/15 54.

In order to make more exacting determinations of the rate of movement of injected water in the aquifer, granulated sugar was added to injection well influent as a tracer mechanism on two separate occasions. Three hundred pounds of sugar were added to well I-A on 6/17/54, within a period of 3 minutes, while a flow of approximately 0.45 cfs was being maintained. Similarly, 300 pounds of sugar were added to well G on 6/29/54 within a period of 12 minutes with an injection rate of 0.6 cfs. The results obtained in either instance were largely inconclusive because of the large amount of dilution which occurs and the resultant low concentration of sugar present in the samples.

The average length of pressure ridge maintained during the month extending to an elevation equal to or greater than +2.5 feet M.S.L. approximated 2,450 feet and above sea level for 3,700 feet.

A summary of the effect of the recharge operations can be seen on Plates 16 and 17 showing the ground water contours in the project area for February 20, 1953 and June 24, 1954, respectively, and Plate 5 which shows the build-up with time of the ground water elevations at various points in the barrier mound. Comparison of the ground water contours on the two dates clearly show the creation of a pressure mound. The points of measurement of the ground water elevations are: a well on the axis of the recharge line, 20 feet from a recharge well, well G-1; a well approximately 500 feet oceanward of the recharge line, well G-5; a well 1180 feet landward of the recharge line, well G-8; a lateral well on the axis of the recharge line to the south, well L-1; and a well 11,000 feet landward, a distance at which the landward effect of recharge is undiscernible, well MB-11. Also shown on Plate 5 are the total recharge rate and the total well production of the fields closest to the line of recharge. Total fresh water injection along the recharge line, and the resultant recharge to the basin, amounted to 3520 acre feet from the beginning of the test until June 30, 1954. Producing wells were those of the City of Manhattan Beach and the California Water Service Company located approximately 6500 feet landward of the recharge line. Discussion of the effect of inland pumping occurs in Chapter V, B.

CHAPTER V

HYDRAULIC ASPECTS OF THE RECHARGE LINE

The hydraulic properties of a line of recharge wells, the parameters involved, and the application of known data relative to the design of a line of recharge wells were the most important factors to be determined in the test. An attempt has been made in the following discussion to compare observations from this field test to theoretical derivations and model studies concerning the phenomenon of recharging fresh (or lighter) water into a pressure aquifer in the vicinity of a body of saline (or denser) water. The technical factors to be investigated, which were specified in the agreement concerning the performance of the recharge test, are discussed below.

A. THEORETICAL BACKGROUND

In order to show the relationship that a single recharge well has to its position in a line of recharge wells, it is of interest to follow the physical transition which occurs between the beginning of injection and the creation of a stable mound. The following description presents the changing conditions as if there were a time lag between occurrences. In a true pressure aquifer this is not the case, since pressure changes, theoretically, are transferred instantaneously. However, practically all pressure aquifers have a storage factor which has been variously attributed to compressibility of the aquifer materials and/or the confining strata. Actually no pressure aquifer is perfect in that leakage to a greater or lesser degree occurs through the confining strata or ground water moves to adjacent or contiguous free zones.

When injection begins at a recharge well, all the flow from the well is radial, although when the recharge cone of a single well is superimposed upon an existing ground water slope, the larger portion of the injected water flows down gradient. Radial flow emanating from the well continues along streamline trajectories until the interference effect of adjacent recharge wells begins at a point near the line of recharge about midway between the wells.

Actual mergence, that is, the intersection of the streamline boundaries of two adjacent wells, is governed by the ratio of the well recharge rate to the unit rate of flow occurring downstream from the line of recharge, which ratio determines the maximum width which the stream of injected water from any well can approach in a given aquifer. Hence, unless the spacing of the wells is smaller than this width, no intersection of streamline boundaries occurs and, therefore, no mergence takes place. Saline water would then continue to flow between the streamline boundaries of adjacent wells. Conversely, when the aforementioned ratio greatly exceeds the well spacing, more than the necessary fresh water head will be developed at the point of mergence than is required to balance the intruding sea water. This, in course of time and displacement of the saline wedge, will result in an additional waste to the ocean.

As the area of pressure due to recharge continues to expand toward stability, the pressure, at a given point, is the resultant of the effect of each well. The integrating effect of this mergence continues until a continuous mound is formed, this condition being characterized by the establishment, within a short distance from the line of recharge, of two-dimensional flow, i.e., no lateral movement of recharge water.

After establishment of the recharge mound, the most important factors involved in a line of recharge wells which is acting as a complete stable barrier to sea water intrusion are: (assuming a uniform, homogeneous, completely confined aquifer).

(a) The quantity of fresh water recharge in a given section parallel to the coast line must be sufficient to replace the previous quantity of sea water which was intruding.

(b) To stabilize an intruding sea water wedge oceanward of the recharge line, there will have to be some movement of fresh water toward the ocean.

In reference to (a) a cursory analysis of the Darcy equation of ground water flow, $Q = K\,i\,A$ (Where $Q =$ rate of flow, $K =$ permeability coefficient, $i =$ hydraulic gradient, and $A =$ cross sectional area through which ground water is moving) shows that the rate of flow through a given reach of an aquifer is proportional to the hydraulic gradient. In a large pressure aquifer, the gradient is dependent upon the location of supply source and the amount and pattern of pumping which, in general, do not change rapidly. Hence, if sea water is the source of supply, conditions can only be changed by: changing the pumping amount and pattern; and/or providing a new source of supply equivalent to the existing supply.

Factor (b) has been reduced to a quantitative equation *

* For derivation see Appendix E.

$$q = \frac{1}{4} (S_S - S_F) \frac{M}{L} T$$

where q = seaward fresh water flow per foot of ocean front

S_S = specific gravity of sea water

S_F = specific gravity of fresh water

M = thickness of aquifer, down to lowest depth which must be protected

T = aquifer transmissibility for 100% hydraulic gradient

L = length of sea water wedge, from ocean outlet to the inland toe.

The reason seaward flow occurs is, basically, because sea water is more dense than fresh water. This fact requires that a higher fresh water head be maintained in order to balance the pressure in the intruding sea water wedge. A complete barrier can be attained only if a pressure balance in the lowermost part of the aquifer is obtained. At all other higher elevations there will be an oceanward gradient, which fact is responsible for the movement of flow toward the ocean. However, the test establishes the fact that such movement is of no practical or economic significance in that the rate of movement of injected water oceanward, in relation to its movement inland, is minor and relatively slow under gradients observed.

Theoretically, the amount of recharge water required at a given location may be estimated by combining the requirements of items (a) and (b) above. The discharge rate of the intruding sea water, which must be replaced, plus the quantity necessary to waste to the ocean to stabilize the sea water wedge equals the necessary stabilized recharge rate. Since, in a completely confined aquifer there is not storage available, the rate of ground water flow depends upon the inland rate of pumping, which during the period of the test did not change appreciably. The ground water gradient depends upon the rate of flow, and therefore the gradient landward of the recharge wells should be approximately the same prior to and during recharge operations.

However, at the test site, it is found that as a result of recharge, the landward gradient was increased some 58% by recharge, indicating that the required recharge rate was greater than the prior rate of sea water intrusion. It is concluded that one or more of the following factors is responsible for this gradient increase, and the corresponding indicated increase of flow landward:

(a) Significant storage exists within the local aquifer and/or in some portion of the aquifer contiguous to that being recharged.

(b) A significant quantity of leakage is occurring upward through the confining layers inland of the recharge line.

(c) Lateral flow of the injected water is occurring to such extent that a partial barrier extends over an area much larger than is indicated, as yet, by the arrival of injected fresh water. It is feasible that a complete barrier can be formed within the reach of recharge wells and that a partial barrier is formed off the ends of the recharge line, the degree of effectiveness of the latter decreasing with lateral distance from the exterior recharge well. It may be noted that the effect of the increased gradient is significantly measurable only about 7000 feet landward. (Compare ground water elevations on Plates 16 and 17)

In order to evaluate the effect of the increased gradient, an additional ground water flow factor must be considered. To demonstrate the use of this factor and to give an example of the magnitude of the amount of water which will move toward the ocean, the conditions at the test site can be used. The total recharge water needed, per foot, Q is:

$Q = \frac{1}{4} (S_S - S_F) \frac{M}{L} T$ Oceanward flow from the recharge line

$+i\,T$ Initial ground water flow landward

$+i'\,T$ Increased ground water flow landward due to recharge,

where i = ground water gradient before recharge

and i' = increase in ground water gradient over prerecharge gradient due to recharge operations.

The ratio q/Q gives the proportion of injected water which will waste to the ocean if the recharge barrier is operated at the minimum effective elevation.

$$\frac{q}{Q} = \frac{\frac{1}{4} (.025) \frac{110}{2000}}{\frac{1}{4} (.025) \frac{110}{2000} + .0041 + .0024} = .050$$

$S_S = 1.027$	Approximate specific gravity of ocean water obtained off Manhattan Beach pier
$S_F = 1.002$	Average specific gravity of M.W.D. fresh water
M = 110 feet	Approximate value assumed for the effective aquifer
L = 2,000 feet	Distance to ocean
$i = .0041$	Average initial gradient along a normal to the recharge line through well G
$i' = .0024$	Average gradient increase along the well G normal.

Although the average elevation along the recharge line was somewhat above the minimum elevation due to the necessity of maintaining the internodal point at the minimum, this fact would not have a large effect upon the quantitative result.

The numerical values used in the above equation were estimated as follows:

This ratio applied to the average recharge rate gives the approximate amount of recharge water flowing toward the ocean:

.05 x 4.5 cfs = .22 cfs, 5% of the total recharge flow.

Since the inception of recharge, the average rate of movement oceanward has been less than 150 feet per year. At this rate, no actual waste will occur to the ocean within the next 10 years.

B. CHARACTERISTICS OF THE OBSERVED PRESSURE MOUND

In general, the pressure mound formed by the recharge wells has fulfilled the expectations of the planning and calculations for such a barrier. It was originally estimated that 1000-foot spacing of the recharge wells would be adequate at the test site. However, it was necessary to reduce the spacing to 500 feet when it was found that the acceptance rate at the 12-inch recharge wells was too low under the restricted injection head made necessary by the failure of the "clay cap" adjacent to the well casings. On the other hand, this disadvantage was balanced by the fact that a lower than estimated total recharge rate has been found adequate for the reach investigated.

Because of the difficulties of creating a seal between the recharge well casings and the clay cap, there has been a suspicion that part of the injected water leaked upward into the overlying sand dune layers. Test holes drilled to the vicinity of the top of the clay cap indicate that there may have been some leakage, but it was of such limited magnitude as to have only a minor effect upon the formation of the barrier mound.

It is necessary to define certain terminology, at this point, in order to clarify a basic concept which has been established in this test. Injection head, as used in this report, is the head required in a recharge well to cause the injected water to pass through the well perforations and the face of the aquifer. It is the difference at a given time between the water surface elevation in the recharge well casing and the maximum piezometric ground water surface near the well resulting from the existent pressure mound. The injection head is dependent upon the quantity of water being injected, the type and size of recharge well, the number of perforations and the local transmissibility characteristics of the aquifer in the immediate vicinity of the recharge well. The injection head is independent of the height or shape of the pressure mound. It is assumed that this head is influenced by the occurrence of turbulent flow through the perforations and in the aquifer immediately adjacent to the well. Normal pressure mound elevations are attained when the flow becomes laminar. Hence, the advantage of a large diameter well or a gravel-packed well to reduce the required injection head becomes obvious.

Mound elevation is the piezometric surface (or pressure elevation) of ground water at a given point of the mound. The mound's size and shape are dependent upon the amount of water being injected in a given reach, the spacing of the recharge wells, the transmissibility of the aquifer, and the ground water gradient as it would be if unaffected by recharge.

Observations of the build-up of the recharge mound during the initial period of recharge (See Plate 15) indicated a considerable lag in the development of a stable pressure gradient oceanward of the recharge line. About two months of recharge operations were necessary before the ground water elevation oceanward of the recharge line rose above sea level and about six months of recharge were required to develop a relatively stable gradient. It is concluded that this lag was the result of a continued by-passing of flow through the internodal pressure valleys prior to the development of the entire barrier to sea level and/or the storage effect within the oceanward aquifer. Well logs indicated that the confining clay membrane oceanward of the recharge line was not continuous and would permit ground water storage.

Due to the changes necessitated in recharge rate and to the interval of time between the initiation of recharge at the individual wells, it is not possible to accurately evaluate the time required to establish a stable landward gradient. However, as recharge was commenced at a single well (G), pressurization effects were noted 1180 feet landward almost instantaneously and, under a constant recharge rate, stabilization was attained in approximately five days.

Visual inspection of the piezometric gradient indicates that the individual cones occurring around each recharge well merge into a relatively uniform barrier within 250 to 500 feet of the recharge line both landward and oceanward. This fact is indicated on Plate 15 by the relatively uniform ground water gradient between wells G-4 and M-B-4 and between wells G-5 and G-13.

After the oceanward pressure lag had been overcome, there was no difficulty involved in maintaining the desired mound elevation by maintaining a constant recharge rate into the recharge wells. However, the question as to whether a complete barrier to sea water intrusion has been formed is not as readily apparent. The most critical spot is, for obvious reasons, assumed as the point midway between the recharge wells, called hereafter the internodal point. At this location the mound elevation is the least, so that if enough fresh water head is not maintained at this location to overcome the higher density of sea water, then intrusion will continue near the bottom, although at a reduced rate. On the basis of an average elevation of the bottom of the aquifer of 110 feet, it is necessary to maintain a fresh water head of approximately 2.7 feet above sea level at the internodal point. Although geologic interpretation suggests that the bottom of the Merged Silverado may be 10 or 20 feet lower, inspection of well logs indicates relatively impervious sediments in these lower limits of the aquifer. Since the aquifer is not uniform and homogeneous and since the effective bottom of the aquifer is not known, it is of interest to know whether the sea water is completely blocked or not. An indicator of this factor is the chloride salinity at the internodal wells if they are perforated deep enough into the aquifer to indicate salinity conditions at the "bottom" of the aquifer. The gradient from the recharge wells through the internodal wells is very slight as compared to that in a landward direction from the recharge wells and, consequently, the movement of injected fresh water toward the internodal points is very slow. The chloride concentration in the internodal wells penetrating the entire depth of the aquifer have continued to diminish slowly during the test. Hence actual freshening of the internodal points may not occur until after several years of recharge. In this connection, it may be noted that if mergence of the flow from adjacent wells takes place oceanward of the recharge line, the internodal wells will eventually freshen. Conversely, if it takes place landward, a pocket of salinity or point of stagnation may occur, thereby precluding complete freshening at the internodal well. As of June 30, 1954, none of the observation wells at the internodal points had become fresh, although all of them had a decreasing trend. (To date, well G-H has decreased from 16,700 to approximately 2000 ppm chlorides.) These indicator wells are: D-E, E-F, F-G, G-H and I-J. The chloride salinity history of these internodal wells is shown on Plate 23. It should be noted that H-I penetrated only the top portion of the aquifer and may not indicate the true condition of the entire aquifer. Freshening could be hastened by increasing the injection rates and the resultant pressure levels; however, such a procedure will also increase the rates of movement oceanward.

In connection with the chloride salinity history of internodal wells F-G and G-H, the following facts should be noted. During the period from June to September 21, 1953 well G was recharged but well F was not. Thus, well F-G was not an internodal well and was largely influenced by injection at well G. From September 21 to October 29, 1953 the injection rates were unbalanced in favor of well G, which means that well F-G was still not exactly the internodal well. But after October 29, 1953, rates of recharge at wells F and G were such that well F-G was the internodal point, and the drop in chlorides which began in March, 1954 represents chloride reduction at the internodal point.

A similar analysis of the location at the internodal well applies to well G-H. Injection began at well H on June 20, 1953 and continued at a rate equal to that of well G until September 3, 1953, when it was necessary to reduce the injection rate at well H because of an impaired acceptance rate. The injection was stopped at well H on October 5, 1953. After repairs, injection was again started on October 24, 1953 at a rate equivalent to that of well G. The drop in chloride salinity at well G-H which occurred in January, 1954, probably indicated the reestablishment of this well as an internodal point indicator.

Pumping rates at the wells of the City of Manhattan Beach and the California Water Service are indicated on Plate 5. These wells, located about 7000 feet inland from the line of recharge, had no immediate effect upon the piezometric surface between the line of recharge and the pumping wells. However, pumping of ground water inland obviously contributed to the depletion of the supplies of the basin and results in an increase of the inland gradient, thereby increasing the required water supply at the recharge line. Obviously this effect is slow and cumulative unless the proximity of the pumping well results in an immediate drawdown at the recharge line.

The location of observation wells for the test was based on the experience with a single recharge well. Observation wells for the project were drilled: at a distance of 20 feet from recharge wells along the axis of the line of recharge; at the internodal points of the intially-planned 5 recharge wells; and at intervals along three separate lines normal to the line of recharge and through a recharge well. Later, when recharge was reduced to 500-foot spacing by recharging the former internodal wells, it was found necessary to drill new internodal wells.

It is concluded that adequate control for barrier recharge operations can be maintained with observation wells at internodal points, occasional observation wells near the coastline located on lines normal to the recharge line or along probable flow lines and passing through a recharge well, and occasional observation wells inland of the recharge line, preferably on a normal line or along probable flow lines and through the internodal point.

Under uniform conditions, it may be expected that the flow line (after mergence) from the wells would be normal to the recharge line. However, in the event the recharge line is not parallel to the ground water contours, the flow lines may deviate somewhat from a line normal to the recharge line. With control of the recharge rate possible by an internodal well and with the recharge rate limited by the available head in the recharge well, there is no necessity, in a purely operational line of wells, for an intermediate observation well along the axis of the line of recharge (such as the 20-foot observation wells used in this test). However, an occasional intermediate test hole to determine the maximum height of pressure mound, as compared to the required injection head within the recharge well, would be desirable to evaluate the efficiency of the recharge well.

C. RECHARGE WELL ACCEPTANCE RATE

The recharge well acceptance rate, or rate at which the well will accept flow, is a fundamental limit to determining well spacing as well as an important factor in determining the economics of building and maintaining a barrier mound. The acceptance rate of a recharge well depends upon: the type and size of well in relation to the characteristics of the aquifer; the amount of free chlorine available in the recharge water to prevent bacterial slime formation; and the degree and type of redevelopment utilized when the acceptance rate falls too low. Redevelopment is discussed later in Chapter VII.

It was found that the time plot of a factor called "unit injection head" or specific injection head for a unit injection rate is a very useful guide to analyze and control the items affecting acceptance rate. Its use, however, is limited to the analysis of items affecting each well separately and cannot be used to make comparisons among the wells. Unit injection head is the injection head in a recharge well, as defined in Section B of this chapter, divided by the injection rate at the well. To simplify the detailed analysis, the elevation of the water surface elevation in the recharge well was used in the place of the actual injection head. Although this causes a small error in the magnitude of the unit injection head, as long as there are no significant changes in injection rate along the line of recharge, the modified value of unit injection head yields a factor which can be used to depict changes in the ability of the well to accept water. These values are plotted for the recharge wells on Plates 18, 19 and 20.

Injection rates used in the test varied widely during different portions of the investigation. A maximum rate of 1.86 cfs was obtained in gravel-packed well E for a few hours. Long-time rates at gravel-packed wells E and I-A indicated the ease with which the acceptance rate can be maintained in these wells. The maximum sustained average rate at well E was 1.06 cfs with an elevation in the well of 67 feet, and at well I-A .74 cfs with 30-foot head. Average injection rates at the non-gravel-packed wells were maintained with somewhat greater relative injection heads. The following table gives the maximum average injection rate and its accompanying injection head at the test recharge wells:

Well	*Date*	*Maximum Average Injection Rate, cfs*	*Water Service Elevation in Recharge Well Ft.*	*Estimated Mound Elev., Ft.*	*Indicated Injection Head, Ft.*	*Indicated Unit Injection Head, ft./cfs*
D	10/22/53	.48	55	14	41	85
E	8/20/53	1.06	67	14	53	50
F	2/18/54	.61	48	9	39	64
G	10/22/53	1.03	69	12	57	55
H	8/20/53	1.01	68	10	58	57
I	5/14/53	.50	70	— 2	72	144
I-A	2/25/54	.74	30	10	20	27
J	11/26/53	.70	67	11	56	80
K	12/17/53	.74	54	5	49	66

In order to determine the injection head in the above table, it was necessary to estimate mound elevations by use of the composite distance-rise equation discussed in the section on transmissibility, Section D of this chapter. For estimating these mound elevations, values of "T" and "d" used were those derived in the transmissibility section by the use of the equation.

In regard to the type of well, gravel-packed wells, represented in this test by wells E and I-A, definitely had better recharging characteristics. It should be noted, however, that the aquifer encountered was primarily sand with scattered gravel layers. Under these conditions the gravel-packed well was able to directly recharge all parts of the aquifer through which it passed, in contrast to the non-gravel-packed wells. In a predominantly sandy aquifer, the advantage of increased acceptance rate and the consequent lower injection head requirement is sufficient to justify the use of gravel-packed wells for recharge purposes. Although most aquifers along the coast, where recharge mounds of this type might be needed, would most likely be similar to that encountered at the test site, it is probable that in a coarse gravel aquifer, recharge could occur through a non-gravel-packed well without excessive injection head requirements.

A comparison of the acceptance rate can be made from the data collected for gravel-packed well E and non-gravel-packed well I at which injection began at the same time and at the same rate. As indicated on Plates 7 and 11, the injection head at well E rose 5 feet, while that at well I rose 55 feet. Well I was perforated for 35 feet, while the gravel-packed well

E was perforated throughout the entire depth of the aquifer penetrated. Hence the injected water was permitted to leave the casing through a greater area of perforations at a reduced velocity, thereby reducing the required head. Further, the 20″± diameter hole for the gravel-packed well, as compared to the 12-inch hole for the non-gravel-packed wells, exposes approximately 175% more area at the face of the aquifer, thereby reducing the initial radial velocity from the well and the resulting ground water mound at the well. The injection velocity and head also depend upon the transmissibility. Reference to Plate 22 indicates that, in this case, the transmissibility is higher at the location of well I. Considering this fact, the difference between acceptance rate of the wells is even greater.

Specific studies of the optimum well size, by drilling various sizes for the recharge line, were not made because it was felt that the success of the overall program should not be jeopardized by the possibility of drilling too small a well. However, based on experience in the 1950 recharge well test and with the two types of wells drilled for the recharge line, it is felt that the best well for recharging purposes, in relation to the aquifer material at this location, is one with a relatively large gravel-packed envelope and a relatively small casing. Consideration of the economic factors and of the necessity of providing sufficient work space for well tools, sample pumping equipment, conductor pipes and valves within the casing indicates that a 24-inch gravel-packed well with an 8-inch casing may prove to be best adapted for recharging purposes.

Effect of Chlorination

It was concluded in the 1950 recharge well test that it was necessary to maintain a free chlorine residual in recharge water in order to prevent the rapid decline of acceptance rate through the formation of bacterial slimes, which apparently tend to clog casing perforations and local aquifer interstices. Thus, chlorination was considered in the earliest phases of planning for this test.

The chlorination rate required to forestall the bacterial slime formation will probably vary with different waters used. The use of the unit injection head curves to determine what the minimum necessary chlorine residual rate may be for a given water is demonstrated by the experience gained from this test. The chlorine residual was reduced in steps from 20.0 ppm to 1.5 ppm over a period of about 13 months. The experience with wells E and G, Plate 18, indicated that with the treated Colorado River water being used, a chlorine residual down to 10 ppm is capable of preventing clogging of the aquifer or perforations. This subject is discussed more fully in the subsequent chapter on Water Quality and Treatment.

Effect of Aeration

An aeration test was conducted at well E during the period May 14 to 21, 1954. Air was allowed to enter the conductor pipe through the top of the well head "T" (See Plate 4), while the back-pressure valve was open, and the water was allowed to fall freely in the conductor pipe. Although the results of this test were obscured by the necessity of changing injection rates along the recharge line occasioned by rehabilitation work at well J and also by a required change in the chlorination rate, Plate 18 indicates that the trend of the well's acceptance rate prior to and following the test showed no significant change. During aeration, an increased head in the well was apparent, which was due, at least in part, to a foaming action within the well, thereby reducing the density of the water head in the well. This action made measurement of actual head extremely difficult. The results approximately duplicate those originally observed in 1950 during the District's test at the abandoned Manhattan Beach Well No. 7. (7)*

Indications of some degree of increase in well acceptance were noted following shutdowns for well repairs. The increased acceptance may have resulted from the release of progressive air-binding and/or from the probable mild surging effected by closure and subsequent reactivation of the recharge wells.

D. TRANSMISSIBILITY

Transmissibility and gradient are the prime factors in determining the total rate of injection, over a given reach, required to create an effective fresh water barrier to sea water intrusion; while well spacing must be determined on the basis of the acceptance rate of the well in relation to the required total rate of injection.

A descriptive definition of transmissibility is:

The rate at which percolating waters pass through a unit width of a given aquifer under a unit hydraulic gradient. Transmissibility may be expressed at $T = PM$ where

$T =$ transmissibility
$P =$ permeability
$M =$ aquifer thickness.

In view of the fact that the effective depth of the aquifer is not definitely known and varies considerably over any given reach between two wells, the actual determination of permeability would be difficult and probably inconsistent. Transmissibility, being determined directly from the observed data, has therefore been used throughout in these analysis and is expressed in cfs/ft. per unit gradient.

In order to find relationships or disparities between transmissibility as it is determined by pumping drawdown and transmissibility as measured during re-

* See Bibliography (7).

charge, it was necessary to derive an equation relating the effect of transmissibility with the pressure elevation at any given location during recharge. The derivation of this equation and its application are discussed below, along with other standard methods of determining transmissibility. Pumping tests were made upon the recharge wells shortly after completion of drilling and development. Data from these tests were used to evaluate aquifer characteristics, and particularly transmissibility. Transmissibility values were also determined from the data related to recharge.

Theoretical Basis of Transmissibility Determinations

Established methods are available to determine the value of transmissibility in the field by means of data collected from pumping wells and nearby observation wells. In general, they are divided into two classes, i.e., equilibrium and non-equilibrium conditions. Equilibrium conditions are defined as those where very little change occurs over a period of time in the values of drawdown in both pumping and observation wells, with a constant rate of discharge. Non-equilibrium conditions are those where drawdown is increasing, in relation to time, at magnitudes readily measurable.

Pumping Transmissibility

The characteristics of the aquifer at the test site, the rate of pumping discharge used, and the length of pumping at the test wells precluded the use of equilibrium formulae for the pumping test data. However, non-equilibrium equations were well fitted for use of the data collected.

A simplified application of the non-equilibrium equation has been developed by Cooper & Jacob. (11) * Thus,

$$s = \frac{Q}{4\pi T} \ ln \frac{(2.25\ Tt)}{(r^2S)} \quad . \ . \ . \ . \ . \ . \ . \ . \quad I$$

where s = drawdown of the ground water surface in feet, (s = rise of ground water surface in later computations relative to recharge), Q = discharge of the well, cfs, T = transmissibility of the aquifer, cfs/ft., ln denotes logarithm to the base e, t = time elapsed since start of discharge in seconds, r = distance from the recharging well in feet, and S = coefficient of storage, dimensionless (volume of water that a unit decline of head releases from storage in a vertical prism of the aquifer of unit cross section). According to Cooper and Jacob, this approximation of the more general equation is probably sufficient if the quantity, u, is less than 0.02, where $u = \frac{r^2S}{4Tt}$. By plotting r, t, or (r^2/t) against s on a semi-logarithm paper (with s on the arithmetic scale) and finding the slope of a line best fitting the plotted points, the transmissibility may be computed in the following equations, respectively: (Δs, difference in drawdown over one logarithmic cycle, is the aforementioned slope.)

Method	*Transmissibility*	*Coefficient of Storage*
Distance—Drawdown	$T = \frac{2.303Q}{2\Delta s\pi}$	$S = \frac{2.25Tt}{r^2_o}$
Time—Drawdown	$T = \frac{2.303Q}{4\Delta s\pi}$	$S = \frac{2.25Tt_o}{r^2}$
Composite—Drawdown	$T = \frac{2.303Q}{4\Delta s\pi}$	$S = \frac{2.25T}{(r^2/t)_o}$

The coefficient of storage, as calculated from the above equations, is a measure of the degree of confinement of the aquifer in the vicinity of a tested well. The values of r_o, t_o, and $(r^2/t)_o$ are those at $s = O$ on the semi-logarithmic plot. The evaluation of the storage factor is inherently subject to a much larger error than that of transmissibility. In view of the extreme variability of the aquifer in the test area, it is felt that the use of values of S (coefficient of storage) for anything more than an indication of a trend would not be justified.

It has been shown by Theis (12) * that the time drawdown equation for transmissibility holds for recovery from pumping if the quantity of t/t' replaces t in the semi-logarithmic plotting, where t' = time since pumping ceased, and t is as before. The use of this factor permits a second, independent calculation in observation wells and, also, the evaluation of transmissibility at the pumped well itself.

Theoretically, the equations are based on an infinitely extending artesian aquifer of uniform thickness and permeability. Although the aquifer is by no means infinite, being bounded by a seacoast at an effective distance of approximately 2000 feet, analysis of observation wells in the vicinity indicate that no serious effect from the presence of the ocean is felt during the relative short pumping times used (8 hours). No measurable drawdown evidence of pumping was felt at distances from the pumping well greater than 2,000 feet. To minimize the effects of variable thickness and permeability of the aquifer, values were determined based only on the nearest observation wells, where possible.

Recharge Transmissibility

Transmissibility in the vicinity of a stabilized barrier mound which is just balanced so as to prevent saline intrusion can be determined from the Darcy equation of ground water flow, $Q = K\ a\ i$ or, as used in this report, $Q = T\ i\ L$, where L is the length of reach considered. Since, theoretically, the rate of ground water flow landward from the barrier mound

* See Bibliography (11), page 526.

* See Bibliography (12), pages 519-524.

is known, and the ground water gradient can be measured in the area landward where the flow has become linear, it is possible to compute values of transmissibility from this equation.

Along the recharge line itself it is possible to derive a theoretical equation for the shape and size of the barrier mound. Equilibrium conditions of a recharging well may be represented by the equation:

$$s = \frac{Q}{4\pi T} \ln \frac{(2d+y)^2+x^2}{y^2+x^2} \quad . \; . \; . \; . \; . \; . \; . \quad II$$

where s = rise of piezometric surface from initial conditions at a given observation well or point,

where d = distance of recharge well from the seacoast,

x = distance to observation well from the recharge well measured in a direction parallel to the coastline, and

y = distance to observation well from the recharge well measured normal to the coastline.

This equation is derived as follows:

If we assume two-dimensional flow, then the flow due to a recharge well near a straight coastline may be represented by that due to a source at the well and a sink located at the mirror image of the source beyond the coastline. Whereas a source may be represented by a well being recharged, a sink may be thought of as a pumping well. In this type of analysis the coastline, a line of equipotential, is termed a "line source."

The piezometric surface rise at any point is simply the algebraic sum of the rises due to the source and sink. In other words, a single well (point source) in a semi-infinite aquifer (bounded by the coastline or line source) is replaced by a source and sink in an infinite aquifer (13)*. The rise due to a recharge well in an infinite aquifer is:

$$s = \frac{Q}{4\pi T} \ln \frac{(2.25\,Tt)}{(r^2 S)} \quad . \; . \; . \; . \; . \; . \; . \; . \quad I$$

where s = rise at a point r feet from well.

The rise due to the source and sink is:

$$s = \frac{Q}{4\pi T}\left[\ln \frac{2.25\,Tt}{r_1^2 S} - \ln \frac{2.25\,Tt}{r_2^2 S}\right]$$

where r_1 = distance from source, and r_2 = distance from sink, therefore

$$s = \frac{Q}{4\pi T}\left[\ln \frac{2.25\,Tt}{S} + \ln \frac{1}{r_1^2} - \ln \frac{2.25\,Tt}{S} - \ln \frac{1}{r_2^2}\right]$$

$$s = \frac{Q}{4\pi T} \ln \frac{r_2^2}{r_1^2}$$

$$s = \frac{Q}{4\pi T} \ln \frac{(2d+y)^2+x^2}{y^2+x^2} \quad . \; . \; . \; . \; . \; . \; . \quad II$$

* See Bibliography (13), page 175.

By the use of Equation II, it is possible to determine a value of transmissibility measured at a given point with a single recharging well. Likewise it is possible to determine a value of transmissibility measured at a given point with multiple recharging wells as derived below:

At any point on the x-axis where $y = O$

$$s = \frac{Q}{4\pi T} \ln \frac{4d^2+x^2}{x^2} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad III$$

As the effect of recharge at any point is the sum of the individual effects of all recharging wells in the line (13)*, then at any point on the axis of the recharge wells, i.e., the x-axis:

$$s = \frac{1}{4\pi T}\, Q_1 \ln \frac{4d^2+{x_1}^2}{x_1^2} + Q_2 \ln \frac{4d^2+{x_2}^2}{x_2^2} + Q_i \ln \frac{4d^2+{x_i}^2}{x_i^2} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad IV$$

where $x_1, x_2, \ldots x_i$ = distance to recharge wells from point of measurement of s, and $Q_1, Q_2, \ldots Q_i$ = rate of injection at recharge wells. The use of this equation has been called, in this report, the composite distance-rise method.

After equilibrium has been reached, the values of s measured at an observation well and Q at the recharge wells can be used with the constants d and x to obtain T for each point of measurement of s. The values thus obtained from the field data were not consistent with values obtained by other methods and, therefore, it is indicated that the equation does not fit conditions at this test site. The transmissibility values obtained by use of the equation are a little more than *one half* of those computed by the other methods. The fact that this equation does not yield values consistent with those determined by other methods is probably due to the assumption of a perfect pressure aquifer and because it is based upon the reaction of stabilized pressures at a considerable distance from a "line source." Hydrologic data indicates, and the geology shows, that portions of the aquifer oceanward of the line of recharge are not under pressure. In a more completely confined aquifer this equation may prove more adaptable.

However, since the values of T and also d, indicated by the solution of Equation IV, were consistent with each other, they were considered to be constants which include the effect of the nonpressure characteristics of the oceanward aquifer. These constants were used to estimate recharge well mound heights in Chapter V, B. It is felt that the value of mound height, derived in this manner, is a reasonable approximation to an otherwise difficult value to estimate.

* See Bibliography (13), page 509.

Determination of Field Transmissibility

The accuracy of using values of transmissibility determined in field tests as averages representing the entire aquifer is limited by the effects of the natural variations which exist in the aquifer materials. Localized areas of high or low transmissibility in the vicinity of the pumping well and, to some extent, near the observation wells can have a large influence upon the value derived. Hence, individual values can vary considerably from those which might represent the average aquifer. As discussed below, several methods were used to compute transmissibility and the values thus derived are summarized in a table at the end of this section. To graphically illustrate the variation in transmissibility, these values for the various wells are plotted on a profile along the recharge line on Plate 22.

During Pumping

Determinations of field transmissibility from pumping tests were obtained from data collected in the following manner: A pump was installed at the well to be tested with its discharge line connected to a weir box to provide accurate measurements of the pumping rate. (See Photo 3) In each pumped well, an airline (calibrated by measurements with a steel tape before and after pumping) measured the pumping drawdown. Taped and airline measurements of water surface elevation, after pumping ceased, continued until recovery was virtually completed—usually within a 12-hour period.

Observation wells were equipped with recorders which gave a continuous record of drawdown and recovery during the pumping tests. In general, observations at wells beyond 500 feet from the pumped well were not depictive of applicable results.

It was found that the distribution of observation wells about the recharge wells, together with the limited pumping time, was such as to prohibit an accurate evaluation of transmissibility by the distance-drawdown method. Results by the other two nonequilibrium methods were relatively consistent, and are felt to be representative of the test area. A typical computation by the time-drawdown method is presented on Plate 21.

The value of transmissibility at well E for the pumped-well time-recovery method appears to be in error but is included on Plate 22 to provide continuity between wells D and F. Data for this test was too incomplete for proper evaluation. The value of transmissibility at well I, using either pumped-well time-recovery or initial recharge indicates that a local area of low transmissibility exists near the well. Determinations based on the 20-foot observation well are more consistent with the determinations for the balance of the recharge area.

During Recharge

When recharge is initiated at a specific well, a nonequilibrium condition, comparable to that of a pumping well, occurs for a limited period—usually less than a period of two days. Provided no other disturbing elements exist or occur during the period, another set of transmissibility data may be obtained. It was found that results were comparable in magnitude to those determined by pumping tests.

In order to compute transmissibility values during stabilized recharge conditions from the Darcy equation, it is necessary to make assumptions as to the length of reach of the recharge line which is unaffected by the lateral gateway at the ends, and the rate of ground water flow which is applicable to this reach. The assumption made herein was that the portion of the mound from the centerline of well E to the centerline of well J was relatively unaffected by end conditions. The validity of this assumption can be studied by reference to Plate 14, showing the profile of the barrier mound. The rate of ground water flow applicable to the above-chosen reach was then assumed to be the sum of the injection rate of the wells between wells E and J and one half of the rate at the latter two wells. However, since all the flow from the mound is not landward, it is necessary to modify this quantity somewhat.

It would be possible to determine the average gradient from the recharge mound to the ocean and correct it for the head differential due to the difference in density between intruding water and injected water. However, it is more advantageous, in the presence of a mound stabilized at the minimum barrier elevation, and probably sufficiently accurate, to use the theoretical equation concerning waste to the ocean discussed above. Thus, about 95% of the water injected between wells E and J was assumed to be moving landward. The landward, linear gradients used to determine transmissibility values by this method were those indicated on the ground water profile normal to the line of recharge through well G (Plate 15).

Spacing of Recharge Wells

As discussed in Section A of this chapter, having determined the rate of intrusion of sea water throughout a given reach of coastline and estimated the acceptance rate of the recharge wells as related to the transmissibility of the aquifer, the spacing of such wells can be determined merely by dividing the total required flow by the estimated acceptance rate. Practically, the above criteria might be applied to the design of a recharge line as described below.

Assuming that the geology of the general area has been thoroughly investigated, the conditions of overdraft in the basins having been established and

SUMMARY OF TRANSMISSIBILITY DETERMINATIONS ALONG THE RECHARGE LINE

Pumped or Recharge Wells	Date	Q cfs	Time of Pumping Hrs.-Min.	Draw down in Pumped Well s Ft.	Specefic yield Q/s x10-2	Observation Well	Dist. from Pumped Well Ft.	Transmissibility, T—cfs/ft.						Storage Coeff. S 10-4 Units
								Pumping tests			Recharge			
								1	2	3	4	5	6	
								Time Draw-down at 20′ Wells	Time Re-covery at 20′ Wells	Time Re-covery at Pumped Wells	Time Rise Initial Well Mound	Stabilized Barrier Mound 12/3/53	Stabilized Barrier Mound 6/24/54	
C	12/15/52 4/30/53	.29 .28 .17	1-34 2-44	57.0	.48	C-1 C-1	27 27	—	.09	.10	.09	Average value based upon the recharge rate over a reach of 2,500 feet (wells E to J)	Average value based upon the recharge rate over a reach of 2,500 feet (wells E to J)	7.0
D	12/21/52 10/ 6/53	.43 .85 .4	5-12 5-13	30.5 76.5	1.4 1.1	— C-1	485			.09	.14			
E	12/ 5/52 3/25/53	.23 .48 1.02 2.05 .50	2-30 3-0 5-18 0-30	12.0	8.5	E-1 E-1 E-1	20 20 20	.16	.10	.06	.13			.3
F	2/ 7/53 9/21/53	.75 0.3	6-32	21.3	3.5	— F-G	255			.18	.12			
G	3/29/52 6/ 9/53 6/10/53	1.31 .20 .54	9-24	86.5	1.5	G-1 G-1 G-1	20 20 20	.24	.24	.22	.17 .17			62.0
H	2/12/53 6/20/53	1.21 .49	7-15	16.0	7.6	— G-H	245			.22	.18			
I	2/ 4/53 3/25/53	1.20 .51	7-12	20	6.0	I-1 I-1	20	.25	.20	.06	.04			
J	1/24/53	1.2	5-27	59.5	2.0	—			.10					
K	1/12/53 4/28/53	.58 .29	7-0	36.9	1.6	K-1 K-1	11	.10	.06	.05	.11			.19
	Numerical Average							.19	.14	.12	.12	0.17	0.18	

It may be noted in Appendix B that laboratory determinations of permeability range from less than 50 percent down to less than 1 percent of the values noted in the table above (transmissibility of 0.18 cfs/ft. equals 141 ft./day assuming an aquifer thickness of 110 feet). No obvious explanation is presently apparent; however, the possibility exists that such a difference may result because of the high degree of stratification of the aquifer materials in their natural state as compared to the disturbed condition of the aquifer materials as tested in the laboratory.

having found suitable right of way, the procedure would be to:

(a) Based on the best available geologic information, divide the coastline to be protected into reaches of estimated uniform transmissibility.

(b) Drill a test hole near the center of each reach throughout the entire depth of aquifer materials to definitely confirm the geology and establish the depth of recharge well required. This test hole later could be used either as an adjacent observation well or a pilot well for the recharge well.

(c) Based on the established lithology and estimated permissible injection head in relation to the transmissibility of the aquifer, a recharge well would be designed and constructed at or near this location.

(d) A pumping test would be performed on the recharge well after the completion of drilling to establish the aquifer's transmissibility. Such a pumping test should be made with moderate rates of pumping to prevent over-development of the well.

(e) The well drilling and pumping test will provide a check on the initial assumed conditions. Observing the principles as outlined in this report, a more accurate calculation could then be made to determine the required spacing.

(f) With the spacing thus determined, the adjacent internodal observation wells could be drilled providing a further check on the geology. This would be followed by the drilling of the adjacent recharge wells at each side and the above check of geology and transmissibility duplicated. Such a procedure could then be continued until all recharge wells were completed.

This procedure would, of course, also apply to an extension of any existing recharge line. Obviously, since the many variable factors can be determined only with approximate accuracy, a corresponding safety factor must be considered in determining the actual spacing and the estimated acceptance rate of the recharge wells.

The change in spacing ultimately adopted at the test site, i.e., from 1000 to 500 feet, was due to the limited acceptance rate of the nongravel-packed wells which, at higher rates, induced excessive injection pressures on the clay cap. This reduction in spacing permitted maintenance of the barrier at an average injection rate of approximately 0.50 cfs per well. An inspection of Plates 7, 9 and 11 shows that injection rates from 0.75 cfs to 1.0 cfs could easily be maintained at the gravel-packed wells. (Following the remedial work done at well G, it could be considered as a quasi-gravel-packed well.) This data would indicate that an injection rate of 0.75 cfs at 6 gravel-packed wells with a spacing of 750 feet would have maintained an equivalent barrier at the test site. A higher acceptance rate would, of course, permit even greater spacing.

E. MOVEMENT OF INJECTED WATER

Variation in chloride salinity constituted the basic indicator for tracing ground water movement landward and seaward as well as along the line of injection wells. The variations were detected by chemical determinations made of ground water samples collected by pumping, by use of a "thief sampler," and by conductivity traverses made in observation wells. The chloride concentration history of the key project observation wells are shown on Plates 23 to 28. These plates show comparisons of the chloride history of the internodal wells along the recharge line; of the line of wells normal to the recharge line through well G; of the wells lying approximately 500 feet landward of the recharge line; of the wells lying approximately 1000 feet landward of the recharge line; of the wells laterally north and south of the recharge line; and of a group of wells used as special indicators of the chloride concentration on the perimeter of the immediate test area.

Comparison of the isochlor maps for February, 1953, and June, 1954, Plates 33 and 34, show the size, shape and mergence of the fresh water bulbs resulting from injection. Of particular interest is the fact that the isochlors indicate that the movement of water is influenced by a southerly component of the ground water gradient which has caused the flow to veer southerly from the expected route of travel normal to the recharge line. This deviation to the south places most of the landward project observation wells in the zones of mergence rather than in the main areas of fresh water movement.

Sampling Techniques and Field Procedure

Previous District experience with water well sampling has shown that samples taken from a nonpumped well, particularly from one not pumped for a considerable period of time are not necessarily representative of the quality of the ground water body. The desirability of obtaining pumped samples without the disadvantage of disturbing ground water flow with a large capacity pump led to the use of a small diameter, 10 gpm submersible pump for sampling purposes. Most of the project's ground water samples were taken with this type of pump.

The principal ground water sampling equipment utilized consisted of: (1) A 1½ HP submersible pump about 4 inches in diameter and 5 feet long, pumping about 10 gpm; (2) Pickup truck for transporting the pump and miscellaneous equipment, and upon which was mounted a winch and cable reel for lowering the pump into the wells; (3) Generator, powered by a gasoline engine, mounted on a trailer and pulled by the pickup truck. (See Photo 6)

To provide comparative data, each sampled well was initially pumped until the concentration of chlorides stabilized. Stabilization time varied from 1 hour to 17 hours. Numerous analyses of the pumped effluent were made during the initial pumping to determine the point of stabilization. Thereafter, upon pumping a given well, the stabilization period previously determined was used and checked by spot analyses.

Generally, the pump was placed between two sets of perforations to facilitate interpretation of conductivity data (described below). The small capacity of the pump produced such a small amount of mixing in the well that very little change in well water salinity was noted below the bottom set of perforations. When salinity declined in the aquifer due to recharging operations, it was noted that the water below the bottom perforations remained highly saline.

In conjunction with the water sampling program, numerous "thief" samples were taken. A "thief" sampler is a device by which water samples may be taken at any desired depth within a well. The samplers used consisted of a small "thieving" head unit which could be fitted to sample containers of various sizes. The "thieving" head unit used is designed so that when a brass shim serving as a sealing diaphragm is pierced by a plunger, the attached container is filled. The sampler is lowered into a well by a $\frac{3}{32}$-inch stainless steel cable utilizing a small reel. When at the desired depth, the sample is taken by releasing a cylindrical weight, drilled and fitted around the suspension cable. The weight runs down the cable and activates the plunger which breaks the brass sealing shim and allows the container to fill. As the hole punched through the shim is small in relation to the capacity of the container, little mixing of consequence occurs during the removal of the container from the well. Examples of two types of samplers used in the test are shown on Photo 7.

Partial chemical analyses for chloride, carbonate and bicarbonate were made at the field laboratory, and complete analyses for significant constituents of well samples were made at the District's testing laboratory. Detailed results are available in this District's files. (See Appendix F)

Electrical conductivity equipment was used, in conjunction with the sampling pump, to obtain information on chloride concentrations at various horizons of the merged aquifer—those corresponding with various sets of well perforations. In many cases the interface between fresh and salt water could be identified by means of this technique.

Conductivity equipment consisted of a conductivity cell, a reel of conductor cable, and an alternating current Wheatstone Bridge. Power was supplied either by the pump generator at 110 volts AC or by a 6 volt DC storage battery converted to 110 volt AC with a vibrator converter. The conductivity cell consisted of a pair of electrodes rigidly mounted, protected by an insulated shield, but exposed to the water when submerged. The resistance between the electrodes was measured by means of the Wheatstone Bridge. In order to convert resistance to conductivity and to occasionally check the conductivity cell constant, a one-tenth normal solution of potassium chloride was used as a standard.

The effect of temperature variations upon conductivity values was significant. In pumping tests, the temperature of the pumped effluent was measured with a thermometer. In non-pumping tests a thermistor was lowered into the well. Resistance of this instrument measured on a Wheatstone Bridge, when converted, indicated the water temperature.

A set of conductivity traverses was generally taken in conjunction with the pumping of well water samples as follows:

(a) Static Traverse

The conductivity cell was lowered to a position below the lowest well perforation before pumping in order to avoid the necessity of lowering the cell after the pump was in position which had proved to be difficult. Readings were taken at predetermined depths (usually at 5 or 10-foot intervals) as the cell was lowered. Without removing the cell, the pump was then lowered into the well and the well pumped until the chloride concentration was stabilized.

(b) Stabilized Traverse

With the pump running and stabilization reached, the conductivity cell was then withdrawn from the well with conductivity readings being taken during the process.

Additional static traverses, made in a manner similar to that described above, were often made at non-pumped observation wells to determine static trends.

Effects of Fresh Water Injection

The progressive effects of recharging on the ground water body were reflected in the salinity time histories of observed project wells. The initial effect of continued injection was the creation of an inland-moving temporary saline wave. The sustained effects of continued injection were: (1) Formation of an interface between waters of different salinities and development of an overriding fresh water wedge; and (2) progressive mergence of the individual fresh water bulbs into a continuous fresh water front.

The temporary saline wave must not be confused with the continual increase in chloride salinity resulting from sea water intrusion which was occurring prior to the commencement of recharge and which continued subsequently as the trapped saline waters continued to move landward under the influence of existing ground water gradients, even though recharge had cut off and replaced the source of supply. The magnitude of the temporary wave, as reflected by chloride concentrations, is indicated on Plate 24 and by the curves for observation wells G-2, G-4, and G-8. The wave at G-2, 250 feet inland, increased the chloride concentration about 3.7% during March 1953; at G-4, 500 feet inland, it increased the concentration by 1.9% during May and June 1953; and at G-8, 1,180 feet inland, the wave was not distinguishable. In general, the area affected by the temporary saline wave is approximately the same as the radius of influence noted during pumping drawdown tests. An indication of a saline wave was detected within specified horizons in the aquifer at each well by conductivity traverses, but the magnitude was masked by interference of waters of different quality in the well.

It may be noted that injection commenced in an aquifer zone which had been degraded by sea water intrusion to a chloride concentration approximately equal to that of sea water. Originally fears were evidenced that this procedure might produce and push a saline wave of similar concentration inland to the present fresh water-producing areas. It may be concluded, from the test results, that this wave diminishes rapidly with distance from the recharge line and will become insignificant within a relatively short distance of travel.

Salinity time histories of wells along the "G" line showed the progressive effect of recharging. Well "G," being at approximate center of the recharge line was one of the most significant wells in relation to the line of recharge wells. The arrival of fresh water at the landward wells (even-numbered wells) was noted by the decrease in chlorides. The conductivity traverses indicated that fresh water appeared to arrive as an overriding wedge, slowly displacing the underlying more dense saline water.

The arrival of fresh water and/or the temporary saline wave at a given location can be estimated by the use of the equation:

$$t = \frac{d^2}{c\Delta s}$$

where t = time of travel over the distance, d, i.e., from one given point to the next, c = a constant to be derived from data on known times of travel between given points (which value depends upon permeability and porosity), Δs = the difference of the piezometric surface elevation over the distance d. (If the gradient changes with time because of recharge rate changes or any other reason, the values of s must be the average value.)

By further breakdown of the above equation concerning time of travel of the fresh water front estimates of the porosity of the landward aquifer can be made. Thus, substituting the factors permeability and, inversely, porosity for the constant c and converting permeability to transmissibility, the following relationship evolves:

$$p = \frac{T\Delta st}{Md^2}$$

where p = effective porosity.

The following table lists the actual velocity of travel of the injected water d/t, the average gradient under which this flow occurred, and the calculated values of effective porosity, based on a transmissibility of .18 cfs/ft. (discussed in the previous section) and data observed at the project observation wells.

Reach	*Average gradient*	*Actual velocity*	*Effective porosity*
G–G-2	0.0166	9.8 ft./day	24.0%
D–C-4	.0130	6.9	25.8
I–I-4	.0117	5.0	28.6
K–K-4	.0125	6.0	29.6
Average		6.9 ft./day	27.0%

Based on the above average value of effective porosity, the influence of a stabilized landward gradient of 0.0065 feet/foot and an aquifer thickness of 110 feet, a rate of advance of injected fresh water of 1250 feet per year can be expected. It may be noted that the more rapid rate of average travel, indicated in the above table, was under the influence of steeper gradients immediately adjacent to the recharge well and is not representative of velocities under the influence of the stabilized inland gradients.

The interface between waters of different salinity within the merged aquifer was readily identified at a given well by means of stabilized conductivity traverses, provided the interface fell within the limits of well perforations. The tests at observation wells over a period of time showed the progressive arrival of fresh or saline water. Traverses collected graphically illustrated that:

(a) Historically, a zone of mixed waters had developed as a result of alternate periods of landward flow of sea water and oceanward flow of fresh water which, in early years, probably occurred seasonally and resulted in increased diffusion or mixing between the two bodies of water.

(b) With the gradients that existed prior to and during recharge, undiluted sea water was intruding as a wedge beneath the mixed waters.

(c) During injection, the imported less dense fresh water moves as an overriding wedge into the underlying intruded saline or native mixed ground water bodies.

Creation of an overriding fresh water wedge within the merged aquifer was noted as injection operations progressed. The progressive freshening of the upper part of the lower perforations of well G-4, located 500 feet landward of recharge well G (Plate 29) indicates arrival at that horizon within the aquifer of the overriding fresh water wedge created by recharge operations.

The potential saline underriding which will occur when recharge is halted is indicated by an analysis of data (Plate 30) of well I, the original recharge well which was shut down on October 14, 1953; observation well I-1, 20 feet distant from well I; and well I-A which replaced well I. Total freshening, equivalent in chloride salinity to recharge water of well I-1, as compared to the native saline character of ground water prior to March 6, 1953, is evident by inspection of the well traverse of May 1, 1953, approximately one month following initiation of injection at well I. A subsequent underriding effect of saline water due to a shutdown of well I for a period of approximately 27 days during construction of well I-A, is indicated by the well traverse of well I-1 on November 30, 1953.

The location of the interfaces between native waters, intruding sea water and injected fresh water is shown, somewhat ideally, on Plates 31 and 32. Many more observation wells would be necessary to permit accurate depiction of the effects of variable transmissibility on the movement of the injected fresh water body.

The overriding characteristic of the injected fresh water minimizes the danger of pushing a concentrated saline wedge inland when injection is undertaken in a zone of intrusion where saline water is already present. Observations during the test indicate that, in general, injection in a saline area dilutes and minimizes the effect of the intruded saline wedge as the injected fresh water overrides the native, denser water. Thus the test has established the feasibility of reclaiming an aquifer which sea water has already polluted.

CHAPTER VI

WATER QUALITY AND MAINTENANCE OF AQUIFER TRANSMISSIBILITY

The following discussion constitutes a resume of pertinent phases of a detailed report (Appendix B) on water quality and various chemical and physical factors affecting the permeability of the aquifer.

A. CHEMICAL CHARACTER OF GROUND WATER PRIOR TO AND DURING THE RECHARGING TEST

A comprehensive ground water sampling program at the test site, which included complete analyses of chemical constituents as well as trace elements, substantiated the source of pollution in the recharge area to be sea water rather than connate water.

Historically, as of 1903-04, in the reach from Playa del Rey to Palos Verdes Hills, natural fresh ground water of the Merged Silverado Zone was escaping to the ocean. With the extensive drawdown of water levels to and below sea level, however, particularly between 1931 and 1946, sea water invaded this entire reach. The intrusion was most critical in the vicinity of Manhattan Beach.

During the period of reversal of flow of fresh ground water seaward and sea water landward, which probably occurred seasonally for several years, waters near the coast were affected by increase in chloride concentration. Changes in the quality of ground waters also resulted from a process known as cation exchange.

Cation exchange occurs when ground water is in contact with certain minerals present in clay and other sediments, notably montmorillonitic and zeolitic. This phenomenon involves the exchange of loosely bound sodium in the soil minerals with calcium and to a lesser degree, magnesium in the water. This exchange, which is similar to the softening action of zeolitic water softeners, occurs when the ground water has a normal low concentration of dissolved salts. However, when the ground water is sea water, or a mixture of native good quality water and sea water, the exchange will be reversed, and sodium now more highly concentrated in the ground water will exchange with the calcium (and magnesium) loosely bound to the minerals in the sediments. The exchange just described is analogous to the process utilized for renewal of water softeners with rock salt. This process depends on the concentration of sodium salts present in the water. Within the Merged Silverado aquifer at the test site, the highest concentration of sodium salts is limited to that present in undiluted sea water.

Hence, prior to recharging, the chloride concentration and the quality of native ground water in the vicinity of the test site had already been altered as the result of the admixture of native ground waters and sea water and the effect of cation exchange. The chloride concentration of native ground water, determined from analyses as the project wells were drilled, ranged from essentially sea water near the ocean to slightly polluted water near Sepulveda Boulevard. Although conductivity traverses indicated an increase of chloride concentration with depth, concentrations as discussed herein are based on pumped samples, which represent the average chloride concentration of waters at any specific observation well. The range is indicated by the isochlors as delineated on Plate 33 and also by isochlor ratios delineated on Figure 3 of Appendix B. The increase of cation exchange activity is reflected in the altered quality of the ground water, particularly the increased proportion of calcium present. Five general zones of original water quality were differentiated on the basis of calcium content, delineated on Plate 3 of Appendix B.

Subsequent changes in concentration and quality of ground water brought about by recharging with fresh water is illustrated by well G-2. This well, prior to injection of fresh water, represented a mixture of 85% sea water and 15% original native water. On June 4, 1954, the chloride concentration in this well had dropped to 94 ppm—almost equivalent to the 88 ppm chloride concentration of the recharge water. The sea water obviously had been completely displaced, since the water in G-2 was practically undiluted recharge water.

In moving from injection well G to well G-2 (see Geologic Section G-G), the ground water also experienced cation exchange. This is evident from a comparison of the analyses of recharge water and ground water from well G-2. A typical analysis of recharge water has shown a sodium content of 190 ppm and a calcium plus magnesium content of 46 ppm. On June 4, 1954 water from well G-2 showed a sodium content of 242 ppm and a calcium plus magnesium content of 8.3 ppm. This increase of approximately 27 per cent in sodium shows an appreciable softening of the ground water and the influence of cation exchange. This process of cation exchange is typical of what is occurring along other portions of the recharge line (see Geologic Sections C-C and K-K) as injected fresh water moves inland and seaward.

From the above, it is apparent that in areas where sea water is intruding a fresh water basin or where fresh water is being injected into the intruded saline waters, cation exchange can be expected. This is significant, in that it indicates that a recharge source of inferior waters may be used as replenishment without any deleterious effect on the native ground waters, so long as the sodium-saturated aquifer sediments will act to remove calcium and magnesium.

B. EFFECT OF RECHARGE WATER ON AQUIFER PERMEABILITY

As sodium sediments are more impermeable than calcium sediments, it would therefore be expected that sediments in contact with saline water would, on conversion to a sodium sediment, become more impermeable and that the reverse would occur when the saline water was displaced by a recharge water containing a normal amount of calcium. However, this is not an immediate effect, and actually requires an interval of time to occur because

(1) The permeability of the aquifer sediments does not depend on the amount of exchangeable sodium present in the sediment when the water in contact with the sediments has a high concentration of dissolved salts. This is true whether sodium or calcium sediments are involved. The salts present in solution, when sufficiently high, will cause the minute (colloidal) clay particles present in the sediment to collect into distinct flocs. This flocculating effect of dissolved salts will render the sediment more permeable, regardless of whether calcium or sodium sediments are involved.

(2) The flocculating effect of dissolved salts on sediment colloids is decreased with a reduction in concentration; hence, as the concentration of dissolved salts in the aquifer is reduced, the sediments become less permeable.

(3) The flocculation effect is eventually completely removed and the point of least permeability is reached. Sodium sediments, however, still exist at the recharge line, as evidenced by active cation exchange (indicated by chemical analyses of ground waters).

(4) With continued cation exchange, a progressive increase in permeability then occurs as sodium is removed from the sediments and replaced by calcium from the injected water. Maximum permeability is reached when cation exchange activity decreases to that point where the sodium cations in the recharge water are in equilibrium with the calcium and magnesium cations in the sediments. The conversion to calcium sediments is a slow process. To date analyses indicate that exchange from the fresh injected water is occurring to the greatest degree within the aquifer sediments at the recharge line; to a somewhat lesser degree immediately inland where the sediments are much less saturated wtih sodium; and to the least degree seaward of the recharge line. It would appear that the permeability of sediments within the aquifer along the recharge line is reaching a minimum, prior to the stage when an increase in permeability occurs, as calcium slowly replaces sodium in the sediments. It is obvious that during the period of time when reduction in permeability in the merged aquifer is occurring, a reduction would be expected in the required input of water necessary to maintain the barrier mound. Some indication of this has been evident during recharge operations; however, conclusive proof has not yet been obtained due to the limitations imposed by time and variations of recharge input. Further, it may be expected that an increase in recharge input will be necessary at some future date when cation exchange is stabilized.

C. GEOCHEMICAL EFFECTS OF RECHARGE WATERS ON THE "CLAY CAP" OVERLYING THE MERGED SILVERADO AQUIFER SEDIMENTS

Pressurizing the Merged Silverado Zone depends on the existence of a relatively impermeable stratum generally called the "clay cap" which is 17 to 40 feet thick along the recharge line and which caps the Merged Silverado aquifer. (See Isopach of Clay, Plate 4 of Appendix A) This cap, composed of clays, silts and sandy silts, prevents upward movement of the recharge water and forces it to spread laterally within the Merged aquifer. During the test, the cap failed adjacent to two injection wells, and an investigation was made to determine possible causes. (See Appendix C) The causes were believed to be one or more of the following:

(1) Chemical causes, due to the nature of the recharge water used.

(a) A change in the chemical constituents affecting the plasticity of the "clay cap," resulting in flow of the clay by failure of the plastic state.

(b) An increase in the erodibility of the "clay cap" due to chemical dispersion, so that the dispersed particles are carried through voids in the surrounding granular materials.

(2) Physical causes, arising from the type of drilling operations used, and the methods of development of the wells.

(a) Erosion of the "clay cap" by lateral flow from the well.

(b) Erosion of the "clay cap" by vertical percolation.

(c) Physical collapse of the "clay cap" into adjacent large voids due to gravity alone.

(d) Erosion of the "clay cap" by flow through voids adjacent to the casing.

(e) Collapse of the clay cap due to sudden pressure changes associated with well operation.

Chemical effects and erosion by physical causes (a), (b) and (c) above were found to be relatively unimportant. The physical causes investigated indicated that erosion of the "clay cap" by flow through voids adjacent to the well casing, and collapse of the cap due to sudden pressure which is associated with well operation, to be most serious. Hence, future wells should be gravel-packed to eliminate large voids adjacent to the casing, and properly grouted to seal the "clay cap" to the well casing.

D. EFFECT OF MICROBIOLOGICAL GROWTHS AND CHLORINATION ON AQUIFER PERMEABILITY

The effect of microbiological growths natively present in the aquifer and in the recharge water is of great concern. Dr. Carl Wilson, the District's Consultant, states: (See Appendix D)

"In connection with recharge operations at Manhattan Beach, it was assumed that the steadily diminishing rate at which water could be introduced into an aquifer was principally due to the growth of microorganisms in the sands and gravels of the aquifer. It is a well established fact that virile bacteria, and some other micro-organisms, are always present in soils and sands and gravels, where they persist indefinitely in small members in static equilibrium with the environment. Should conditions within the environment become more favorable, these dormant organisms would burst into activity. The introduction of an imported surface water, like the Colorado River water used to recharge the underground basin, which is relatively high in organic matter, would provide a powerful stimulant to the growth and multiplication of the native organisms at the same time that it brought with it a foreign flora to complicate the situation."

The types of bacteria found within the aquifer and the effects of chlorination on such bacteria are discussed in detail in Appendix D.

Since chlorine is added to the recharge water to prevent bacterial slimes from accumulating at the well perforations, a criterion for controlling its use is necessary. The criteria used at Manhattan Beach were: (1) adding sufficient chlorine to the recharge water to obtain a minimum residual at the adjacent "20-foot" well; (2) adding sufficient chlorine to obtain a minimum bacterial count in the "20-foot" well; and (3) adding sufficient chlorine to maintain maximum acceptance of the recharge water.

The chlorine dosage was gradually reduced from an initial 20 ppm dosage to 1.5 ppm, according to the following schedule:

Inclusive Dates	*Chlorine Dosage, ppm*
2/24/53 to 5/10/53	20
5/10/53 to 6/17/53	15
6/17/53 to 7/29/53	12
7/29/53 to 9/11/53	10
9/11/53 to 2/26/54	5
2/26/54 to 4/1/54	3
4/1/54 to 5/8/54	1.5
5/8/54 to 6/30/54	5

The lowest concentration that would satisfy the first two criteria was 1.5 ppm. However, the required injection head (at constant recharge rate) started to increase at this low dosage as shown on Plate 19. The dosage was then increased to 5 ppm, which still resulted in an appreciable continued increase in the required recharge well head. A low bacterial count and a detectable residual chlorine in the "20-foot" well are therefore not reliable criteria for controlling chlorine dosage. The third criterion, that of adding sufficient chlorine to permit maximum acceptance, would appear to be the logical one to use. The one difficulty inherent in this method is that even an optimum dosage may permit an insiduous bacterial or organic growth to collect in the perforations without being reflected in an immediate reduction in acceptance rate. By the time reduction is noticed, a sufficient quantity of slime may have accumulated that would be difficult to remove by increasing the dosage in small increments. However, by giving the recharge well a "shock" treatment of 20 ppm or more, it may be possible to remove most of the accumulated slime.

Dr. Carl Wilson, District Consultant, states that—"The best criterion of chlorine sufficiency is undoubtedly found in a constant acceptance rate, but in this connection, he who fixes the chlorine dosage must remember that physical conditions within the aquifer will always limit the rate at which water may be injected into it, and when this limit is reached, no increase in chlorine dosage can further augment the acceptance rate." (The constant acceptance rate, as far as it is affected by growths, can be accomplished only by heavy chlorine dosage.) "Experience at Manhattan Beach seems to point strongly toward twenty parts per million as a proper initial dosage to prepare the aquifer vestibule to a sufficient distance to permit injection at the desired rate. . . . After stability has been attained and maintained for a period of perhaps a week, then the dosage may be reduced twenty-five per cent. Experience leads to the belief that after stabilization has been obtained, a dosage of eight to ten parts per million will be required to maintain a constant injection rate."

To determine the potential corrosive properties of Colorado River water when heavily chlorinated (12 ppm) prior to injection into the ground, Dr. Wilson analyzed a series of water samples. The least benign of the waters had a pH of 7.45 and a free carbon dioxide content of 10 ppm which, in his opinion, was unlikely to be actively corrosive. Hence, chlorination at a dosage of 12 ppm appeared to impose no special hazard of corrosion.

E. ADDITIONAL EFFECTS ON AQUIFER PERMEABILITY

1. The Effect of Iron in Recharge Water

Although chemical analyses indicated an iron pickup from the water transmission line, it was too small to adversely affect the recharge water. However, there was a sufficiently large amount of iron in the recharge observation wells to constitute a problem. The formation at the perforations of a gelatinous iron hydroxide which is encouraged by the presence of oxidizing substances, chlorine and oxygen, would tend to reduce the acceptance of the recharge water. Examination of water samples revealed presence of iron oxide floc embedded in bacterial slime. Iron present in water in insoluble forms, usually as hydroxides but often as sulphides, is found in most alluvial fills. Where iron is taken into solution, it occurs as ferrous bicarbonate due to the carbon dioxide released by bacterial activity. When such water comes into contact with the air, the iron is oxidized and is finally precipitated as red-brown gelatinous ferric hydroxide. The effect of hydroxides of iron is not only to reduce aquifer permeability but also to cause accumulations of corrosion products in back-pressure valves, as well as to affect the operation of other equipment.

2. The Effect of Calcium Carbonate (Sludge) and Suspended Solids

The presence of suspended solids and rust particles have been of no apparent consequence during the test to date. However, on one occasion during the test, a temporary Metropolitan Water District plant operational difficulty released an appreciable amount of sludge into the Metropolitan Water District feeder line which proved to be primarily calcium carbonate. Although this sludge had a temporary effect on the operation of the chlorinator, appearance of such sludge would not be normally expected.

3. Effect of Dissolved Oxygen

Although analyses of ground water taken during development tests indicated a dissolved oxygen content of from 0.6 ppm to 2.6 ppm, it is believed that a leaking pump column made these results questionable and that the actual dissolved oxygen in ground water was well below 1.0 ppm. Little dissolved oxygen released by sulphate reduction would be anticipated, largely because of the rapid utilization of such small increments of the gas by bacteria present within the aquifer.

The injection of recharge water with a high dissolved oxygen content would not only significantly contribute to the growth of slime-forming organisms, but also probably tend to affect the aquifer by air binding.

The average temperature of the imported Colorado River water was found to be about 18°C which would permit a maximum dissolved oxygen content of 9.6 ppm. The concentration at times at Manhattan Beach exceeded the maximum solubility of oxygen and the excess was made apparent by the appearance of trapped air bubbles. Further tests are required to conclusively establish the effect of air binding.

CHAPTER VII

MAINTENANCE AND OPERATIONAL PROBLEMS

A. WELL G—REHABILITATION

The most critical maintenance problems, causes, effects and remedial measures undertaken were all related to the subsidence that occurred at well G early during the test. Subsequent well rehabilitation work, preventative measures and operational procedures adopted were directly connected to this experience. The success of these measures and procedures in bringing the test to an ultimate successful conclusion substantiate the validity of the analysis relating to the causes of this failure. It is therefore felt that a chronological account of this occurrence and the subsequent repair provide a valuable background to the problems that might occur in connection with recharge. Specifically, this failure established criteria for the: (1) design, (2) development, (3) rehabilitation, (4) operation, and (5) redevelopment of recharge wells.

Recharge at the test site was initiated at well G on February 14, 1953. The recharge rate was increased in stages and by March 11 the input was one second-foot. During this period the constant rate flow controller valve gave considerable difficulty and constantly required adjustment. This resulted in frequent rapid increases and decreases of input rate. On March 13 during the period of such an adjustment and with several people present, a subsidence occurred on the south side of the well. The subsidence occurred almost instantaneously. The resulting cavern was roughly circular in area, and exposed a hole at the surface some 10 feet in diameter. This area increased below the surface to some 15 or 16 feet in diameter and some 25 to 30 feet in depth, exposing about 15 feet of the well casing. The inflow to the well was immediately reduced to approximately 0.4 second-feet and the cavern filled with native material.

During the original drilling of the well, two substantial horizons of water-bearing gravels were encountered in the aquifer and the well was perforated in these two zones. At the request of the State Engineer's office, the lower zone was perforated first, and then developed by bailing, surging and pumping, and subsequently pump-tested for transmissibility. Following this, the upper gravel horizon was perforated and likewise bailed, surged and pumped, and a second pumping transmissibility test performed. During the second stage of development, materials removed from the well definitely showed that the intervening sediments between the two gravel horizons had broken down and that the upper yellow-colored gravels had combined or merged with the lower blue-gray-colored gravels. Hence it may be assumed that the well had been overdeveloped and large voids had probably formed below the clay cap.

Exploration was next undertaken in an attempt to determine if the failure had occurred in the clay cap. A 2-inch test hole was jetted and driven to a depth of 145 feet in the area of subsidence, about two feet south from the original well. No evidence of a clay cap was encountered. A similar test hole was drilled about three feet north of the recharge well and definitely indicated that the clay cap in this area was still in place and apparently had not been disturbed. The well was grouted through this hole near the lower portion of the clay cap, employing a pressure grout pump and a grout mix of 27 sacks of cement and three parts of water. Maximum pressure employed was about 50 pounds per square inch at the ground surface. To prevent the grout mix from sealing the casing perforations, injection at the recharge well was continued during grouting operations. Following the grouting, the 2-inch pipe casing at the test hole was flushed clean and pulled clear of the upper level of the clay cap to serve as a test well to measure water levels in the overlying sand dune materials. Upon testing recharge at well G, water levels in the test well indicated that this grouting was not effective. The recharge well was then dismantled and filled with sand above all perforations to prevent grout from moving through the perforations into the well, and the area was again grouted through the northerly 2-inch test hole by stages (i.e., raising the pipe at intervals during grout placement), using 34 sacks of cement with a 3 : 1 water/cement mix and a maximum of 50 pounds pressure. The recharge well was then cleared of sand. A third test hole was drilled to check water levels above the clay cap. The recharge well was again tested by injection, and water levels above the clay cap immediately responded, indicating the ineffectiveness of the second grouting procedure. The recharge well was then filled with sand to a depth of 105 feet below ground surface, the approximate elevation of the bottom clay cap; and grout was placed in stages by using the recharge well itself as the conductor and by cutting two sets of additional perforations within the clay zone. Grout was pumped at 50 pounds pressure, utilizing 270 and 41 sacks of cement, respectively. Clearing of well casing was again commenced; however, running sand and mud prevented complete cleanout. The well was again filled to approximately 110 feet with sand and regrouted above the previous grouting zone with a 428-sack grout mix. On commencing cleanout, running sand and mud indicated that the grouting had not completely filled the grouting perforations and the well was again grouted with an additional 49-sack mix.

During the operations noted above, additional subsidence had occurred at the location of the original cave-in, particularly during periods of cleanout operations at the recharge well. After the final grouting mentioned above, and commencement of bailing operations, this subsidence continued. In order to stabilize this movement, it was decided to gravel-pack the well through two 6-inch conductor pipes which were placed in rotary-drilled holes on either side of the recharge well. These two 6-inch casings were placed to a depth of 106 feet and both conductor pipes were thoroughly grouted with a 52-sack mix and a 35-sack mix, respectively. After setting, the grout was removed with a rotary drill and the hole continued on into the aquifer. Subsequent bailing operations at well G and attempted gravel-packing through these conductor pipes proved unsuccessful. During the bailing operations, the original subsidence area continued to move. This indicated that the well might be gravel-packed through the area of subsidence; hence additional perforations were added so that the well was completely perforated within the aquifer from 121 feet to 212 feet below ground surface and bailing was continued as gravel was added to the subsidence area. A total of 76 tons of gravel was added in this manner. As this gravel continued to move, decomposed granite was added in an attempt to form a seal over the gravel. Bailing continued until the decomposed granite had moved approximately to the vicinity of the clay cap. That this gravel actually moved into the aquifer and successfully gravel-packed the well was indicated during bailing by the pick-up of small particles of imported gravel. Injection was reinitiated at well G on June 9 and has continued to date with no indications of further subsidence or failure in well acceptance rate.

B. WELL C ABANDONMENT

Following the experience at well G, test holes were installed immediately adjacent to the remaining recharge wells in order to indicate water levels in the sand dune materials overlying the clay cap. Recharge at well C resulted in immediate response in its test hole indicating excessive leakage through the clay cap. The well log obtained during drilling at well C indicated a relatively thin clay cap some 8 feet in thickness. During development, the pump discharge showed that excessive quantities of fine sands and silts were being removed from the aquifer.

In view of the above and based on the experience at well G, it was not considered advisable to continue recharge at this location. Therefore, operations were suspended and the well was retained as an observation well.

C. WELL I, FAILURE AND ABANDONMENT

Late in May, 1953 a subsidence similar to that experienced at well G occurred at well I. However, due to the rehabilitation costs encountered at well G and the uncertainty of the permanency and effectiveness of such rehabilitation, no such measures were deemed advisable until the work at well G could be properly evaluated. The subsidence area was back filled with gravel and injection was continued at a reduced rate. In accordance with a decision from the State Engineer's Office, a new gravel-packed well (1-A) was planned and installed near this site, as described previously in Chapter II. During the interim period, injection at well I was continued until October 14, 1953.

D. WELLS D, F, H, AND J—GROUTING

In order to prevent excessive leakage through the clay cap adjacent to the well casings and possible subsequent subsidence, remedial grouting work was undertaken. Prior to the initiation of recharge at these intermediate recharge wells, the wells were all filled with sand to approximately the bottom of the clay cap and stage grouted through added perforations in the casing. Wells D, F and J were grouted with high pressure grouting equipment under independent contract between the State Department of Public Works, Division of Water Resources, and BJ Service, Inc. (Well H was grouted by District forces). Grouting at these four wells indicated that voids of significant capacity were present, in that a considerable quantity of grout was used at each of the wells. As an example, over 400 sacks of cement were used in grouting well H. The existence of such voids around the casing was also indicated by the occurence of casing misalignment at each well during grouting procedures. Casing misalignment was severe at wells D and H and required swaging to realign the casing before well tools could be employed to clean out the wells. At both wells D and H swaging operations apparently ruptured the casings. These ruptures were immediately apparent at well D, which was repaired by filling the casing with grout through the damaged area and drilling a smaller 8-inch hole through the grout, thereby effectively cement lining the casing. The rupture at well H was not apparent until later when the acceptance rate of the well declined and soundings showed that sand had run into the well covering a portion of the aquifer perforations. During bailing cleanout operations, the rupture became obvious. The repair of this rupture was accomplished by grouting an 8-inch steel liner within the 12-inch casing.

All four wells are still operating successfully to date; however, after seven months of recharge, the acceptance rate at well J decreased radically and soundings indicated that the well had commenced to run sand. On commencing bailing operations, it became obvious that the upper aquifer perforations were running mud and sand, thereby indicating that the original grouting had not been effective. This well was rehabilitated by grouting off the upper 26 feet of aquifer perforations and cementing in an 8-inch steel liner to cover the upper perforated area. During these operations approximately 125 sacks of cement grout were forced into the formation. Subsequent recharge operations indicate that this stabilization procedure was successful.

E. LEAKAGE THROUGH THE CLAY CAP TO THE OVERLYING SAND DUNE MATERIALS

Although evidence of some leakage at the well casings, through the clay cap, has been noted at the adjacent test holes which were drilled to check such leakage, no subsequent well failures have occurred. Such leakage is considered insignificant in that, although relatively high levels have occurred in some of the test holes, additional adjacent test borings have failed to disclose any free water levels in the sand dune materials. As previously mentioned in the chapter on Geology, the test holes probably tap semi-pressure stringers near the top of the clay cap and do not reflect free water levels in the sand dune material.

F. CONCLUSIONS RELATIVE TO MAINTENANCE PROBLEMS

(1) From the above, it is apparent that a well drilled into coastal deposits, consisting of fine to coarse sands with limited stringers of gravel, should be a gravel-packed well in order that the materials removed from the aquifer during well development procedures can be replaced by gravel and prevent the formation of excessive voids and subsequent serious well subsidence problems. Further, it is obvious that in a pressure aquifer where the confining clay layer is perforated by the well casing, an area of weakness may develop and result in excessive leakage; hence a properly constructed cement seal in this zone will always be desirable.

(2) It is of particular interest to note that in relation to the grouting operations mentioned, the major grout movement appeared to be lateral and upward, while there was very little tendency for the grout to move downward. This was substantiated by the absence of any evidence of grout within the perforations below the grouting zones and particularly by the rotary drilled holes at well G which passed through previously grouted areas.

(3) It is also obvious that recharge operations will place the clay cap under additional pressure, particularly at the zone of weakness along the well casing; therefore, care should be taken during operations to avoid rapid pressure changes resulting from rapid increases or decreases in injection rate. Recharge should be commenced and stopped with small increments of change in the flow, allowing sufficient time for pressures to stabilize between such changes. In this connection, it is recommended that a water supply to the recharge line be controlled with a pressure regulator and that changes at individual wells be made with manually controlled valves.

(4) As a result of the above-mentioned experiences, operation procedures were adopted approximately as follows: Well injection was started by increasing flows slowly until the 6-inch conductor became filled. After reaching this point, at which the well was under complete control, continued increases were made at 2-hour intervals in increments of 0.10 cfs or less. Adjustments to well Q's requiring reductions of flows were made in decrements of 0.05 cfs at 4-hour intervals, the same procedure being followed in complete well shut-downs.

(5) Redevolpment of recharge wells as distinguished from rehabilitation required by failure of the clay cap, was not necessary until after the period of this test report. Although not discussed in the foregoing, following the test period, well K was successfully redeveloped by moderate surging and bailing. A study of the unit acceptance rate for the well during this later period shows a marked improvement in acceptance rate following the redevolpment. Due to the aquifer conditions encountered at the test site, it is not expected that pumping will be used as a redevelopment procedure because it removes excess quantities of sand which would probably be conducive to additional failure of the clay cap. However, in a gravel-packed well, pumping could probably be safely used so long as gravel was added to the gravel envelope to replace the removed aquifer sediments.

CHAPTER VIII

ANALYSIS OF PROJECT COSTS

In accordance with aforementioned legislative authorization, the State Water Resources Board was authorized to cooperate and contract with other agencies in order to make this investigation. The Board contracted with the Los Angeles County Flood Control District for the purpose of conducting the field experimental project, including preliminary investigation, planning, construction of facilities, and operation. Originally $450,000.00 was allocated for this work. Before the completion of the investigation, supplemental sums of $187,000.00 and $5,126.30 were allocated, bringing the total State funds to $642,126.30. A sum of $9,000.00 of the above total was reserved for payment to the District upon completion of a final report.

From a detailed analysis of total District expenditures reimbursable by the State (October 1, 1951 to December 31, 1953), a cost breakdown was made. The expense items were segregated as follows:

A. Capital Costs	$334,135.72
B. Operation and Maintenance Costs	98,833.53
C. Engineering, Investigation and Testing	109,705.33
D. Supervisory Costs	84,201.00
E. Miscellaneous Costs	14,860.52
Total	$641,736.10 *

* Final accounting is not yet complete.

A detailed cost analysis follows:

A. CAPITAL COSTS

The following breakdown of capital cost expenditures includes all related costs such as labor, materials and supplies, rented equipment, District equipment, private auto mileage, utilities, services, photos and blueprints, contract expenses, damages, and applicable District overheads.

COSTS

1. Preliminary Study, Planning and Design		$5,262.67
2. Right of Way Acquisition, Right of Way Engineering		1,829.58
3. Pipeline [1]		
a. Design and Survey	$17,747.87	
b. Installation	107,695.90	
		$125,443.77
4. Well Costs		
a. 9—12" Recharge Wells [2]	$44,030.91	
b. 36— 8" Observation Wells [3]	98,620.50	
c. 14— 2" Test Holes [4]	6,563.50	
		$149,214.91
5. Transmissibility Pump Tests (9 wells)		4,455.17
6. Recharge Well Appurtenances, Materials & Installation		20,521.78
7. Measuring Equipment, Meters, Recorders and Instrument Shelters		12,349.75
8. Field Office and Chlorinator Housing Costs		
a. Design	$1,121.96	
b. Construction	10,269.78	
		$11,391.74
9. Design and Installation of Chlorine Equipment, Storage Facilities		3,666.35
Total Capital Costs		$334,135.72

[1] Pipeline consists of 10,000 feet of 10 ga., 20" O.D.; 1,034 feet of 12 ga., 16" O.D.; 1,064 feet of 12 ga., 14" O.D. welded steel pipe and appurtenances.
[2] Includes drilling and development of 8-cable tool standard-cased wells and 1 cable tool, gravel-packed well.
[3] Includes drilling and development. All wells installed by cable tool drill rigs with standard casing.
[4] All wells driven and jetted into place by district personnel.

The capital costs represent over 50% of the total expenditures. It may be noted that right of way acquisition costs were a minimum due to the cooperation of local agencies and property owners. Pipeline, design and survey costs were relatively high; however, it must be borne in mind that survey costs were significantly increased because of the extensive underground exploration required to locate existing city water mains, sewer and gas lines.

B. OPERATION AND MAINTENANCE COSTS

This breakdown includes those costs for the period February 1953 through December 31, 1953, related to labor, materials and supplies, rented equipment, District equipment, private auto mileage, utilities, photos and blueprints, services, right of way access and applicable District overheads. Such costs are also included in C and E below.

1. Recharge Water	$32,967.80
2. Chlorine	2,853.91
3. Recharge Line Operation	36,589.58
4. Recharge Well Rehabilitation *	20,734.17
5. Routine Project Facility Maintenance	5,688.07
Total operation and maintenance costs	$98,833.53

* Does not include B/J Service, Inc., charge of $3,426.59 for grouting wells D, F, and J. This was billed directly to State.

It is believed that the operation and maintenance costs are in proportion to the magnitude of the project and may be used as a basis for estimating operation and maintenance costs involved in constructing a similar barrier project. The use of these costs on such a basis are discussed under Section G below.

The water used for this project was *treated* Colorado River water imported by the Metropolitan Water District in that no other supply was available. Additional water not included in the above costs was purchased for the test by the West Basin Water Association. All water was obtained at a cost of $20.00 per acre foot.

C. ENGINEERING, INVESTIGATIONAL AND TESTING COSTS

Engineering investigation and testing costs for the test project were, of course, much more detailed, comprehensive and costly than similar engineering work required for operating a project; this also is discussed in more detail in Section G below. The items of work included in this category are as follows:

1. Well Sampling Operation (Including "waiver wells")	$33,540.16
2. Preliminary and Operational Test Data: Collection, Compilation, Interpretation, and Computations	40,596.05
3. Geological Analysis	13,280.76
4. Water Analysis and Soils Testing	20,788.36
5. Water Consultant	1,500.00
Total	$109,705.33

Attention should be called to the fact that the geologic analysis costs noted above do not include field work in connection with logging and coring of test holes. These costs are included in capital outlay for well construction.

D. SUPERVISORY COSTS

1. Immediate Supervision	$33,604.18
2. Divisional Supervision	50,596.82
	$84,201.00

E. MISCELLANEOUS COSTS

1. Well I-A, Drilling and Development *Partial	$1,870.02
2. Final District Report to State (reserved)	9,000.00
3. District Accounting	3,990.50
Total Miscellaneous Costs	$14,860.52

* Does not include contractual cost of drilling and developing gravel-packed replacement well I-A. This contract cost of $7,156.15 was expended independently by the State.

F. UNIT COSTS

1. Pipeline
   a. 10,000 feet of 10 ga., 20" welded steel pipe; 1034 feet of 12 ga., 16" pipe and 1064 feet of 12 ga., 14" pipe—Total $125,443.77
   b. Average Unit Cost
      $125,443.77 ÷ 12,098 feet = $10.37 per foot.
2. 12" Recharge Well—Standard Casing
   a. Eight wells, 2,134 lin. ft. of casing—Total $37,021.70
   b. Average Unit Cost
      $37,021.70 ÷ 2,134 = $17.35 per lineal foot.
   c. Average Cost per Well of 266' average depth
      $37,021.70 ÷ 8 = $4,627.71
3. 12" Gravel-packed recharge well (E)
   a. 240' depth of casing—Total Cost $7,009.20
   b. Unit Cost
      $7,009.20 ÷ 240 = $29.20 per lineal foot.
4. 12" Gravel-packed recharge well (I-A)
   a. 258' depth of casing—Total Cost * $9,026.17
   b. Unit Cost
      $9,026.17 ÷ 258 = $34.99 per lineal foot.

* Includes state contractual cost of drilling and developing well I-A.

5. 8" Observation Well—2-ply casing
   a. 33 wells, 10,803 lin. ft. of casing—Total Cost $94,447.90
   b. Average Unit Cost
      $94,447.90 ÷ 10,803 = $8.74 per lineal foot.
   c. Average cost per well of 328' average depth
      $94,447.90 ÷ 33 = $2,862.06
6. 2" Test holes, driven by District
   a. 14 wells, 1,497 lin. ft. of casing—Total Cost $6,563.50
   b. Average Unit Cost
      $6,563.50 ÷ 1,497 = $4.38 per lineal foot.
   c. Average cost per well of 107' average depth
      $6,563.50 ÷ 14 = $468.82
7. Recharge Well Appurtenances
   a. Nine Wells (incl. well C)—Total cost $20,521.78
   b. Average cost per well
      $20,521.78 ÷ 9 = $2,280.20
8. Recharge Line Operation
   (excluding water costs, well sampling and engineering)
   a. Period March through December, 1953.
   b. Total cost $36,589.58
   c. Average cost per month
      $36,589.58 ÷ 10 = $3,658.96

The unit well costs are of particular interest to this analysis and included all costs related to logging and coring, and the collection of samples in the field. It also includes one field geologist's time in relation to such duties. The costs of the two gravel-packed wells were $29.20 per lineal foot and $35 per lineal foot, as compared to a non-gravel-packed well at a cost of $17.35 per lineal foot. However, in view of the increase in acceptance rate, the minimizing of maintenance difficulties and the overall efficiency of the well, a well designed gravel-packed well is considered economically more advantageous. The increase in acceptance rate will permit increased spacing distance, thereby reducing the required number of wells. The costs of remedial work where a well fails due to the collapse of the clay cap, as was experienced at well G, will probably in itself more than offset the differential costs.

Experience with the observation wells at the test site would indicate that an 8-inch diameter observation well would be the minimum acceptable size if pumped samples and conductivity traverses are desired. This would also be the minimum size in the event a continuous recorder was to be installed to record ground water levels, in that a smaller well could not be used to contain adequate size floats, counterweights and clock weights. A 2-inch test hole to observe water surface elevation only is not recommended in aquifers containing considerable fine sand and silt in that the perforations of such a small well are subject to clogging and the reliability of water levels measured therein are therefore subject to question. The 2-inch wells at the test site required frequent flushing and other rehabilitation measures. A 4-inch rotary-drilled hole is believed to be desirable and most economical for observation wells where water surface measurements only are required. Such wells were drilled as internodal wells at four locations and have, to date, given no difficulty.

G. TEST COSTS AS RELATED TO OPERATING PROJECT COSTS

In order to adapt the test cost experience to a routine operational barrier project, analyses of the pertinent test costs were made, as outlined hereafter. Total capital outlay and annual operation and maintenance costs per mile of recharge line were $186,000 and $32,000, respectively.

Capital Costs

Experienced preliminary planning and design costs are considered to be approximately applicable.

Right of way costs were not considered in that this factor would vary widely, in accordance with local conditions.

Geologic investigation and analysis for the test were considerably more detailed than would be required for normal operation. It is estimated that less than one half of that expended for the test project would be sufficient for one mile of reach.

Pipeline costs indicated in the following table assume a distribution line only and that water would be available at the distribution line. Although a permanent distribution pipeline for an operating project would be more substantial than that used for this test, it is felt that the cost per lineal foot of approximately $10.50 is applicable. The pipeline sizes, anchorage and difficult underground problems, increased the cost of the test project line to a point where its unit cost is commensurate with a more permanent type of installation.

Under the assumption of aquifer characteristics similar to those experienced at the test site, and for reasons discussed in Chapter V, Section D, it is felt that seven recharge wells and twelve observation wells would suffice for each mile of reach. This would permit a recharge well spacing of 750 feet, observation wells at the internodal points, and one line of observation wells normal to each mile of recharge line, assuming two observation wells oceanward and three landward. A unit cost of $30.00 per lineal foot for gravel-packed wells and $9.00 per lineal foot for observation wells were used.

The experienced costs of transmissibility pump tests for this project are considered too favorable in that bids for this work from other well contractors were approximately double those experienced at Manhattan Beach, and it is doubtful that a similar contract with a driller could be repeated. Hence, a value of $750.00 per well has been assumed.

Test experience indicated that a simpler well connection can be designed and that flow controllers are not desirable. Therefore a unit cost for well appurtenances of $1,000.00 each could probably be realized.

It is also estimated, in contrast to the test, that less than one half of the measuring and recording equipment would suffice, thereby reducing this cost to approximately $5,000.00 per mile.

Field office, chlorinating housing, and chlorinating equipment would also probably suffice for an entire five-mile reach, and was proportioned accordingly.

Based on the above, the estimated capital cost *per mile* for an operational recharge line is indicated as follows:

CAPITAL COSTS

1. Preliminary Planning and Design	$5,000
2. Geologic Investigation	6,000
3. Distribution Pipeline 5280 feet x $10.50/ft.	56,000
4. Recharge Wells (gravel-packed, 250′ deep) 7 x $7,500 each	52,500
5. Observation Wells (8″ cased, 250′ deep) 12 x $2,250 each	27,000
6. Transmissiblity Pumping Tests 7 Wells x $750 each	5,200
7. Recharge Well Appurtenances 7 Wells x $1,000	7,000
8. Measuring Equipment	5,000
9. Field Office and Chlorinator Housing	2,000
10. Chlorinating Equipment	3,000
Total	$168,700
10% Contingency	16,900
Total Capital Cost	$185,600
Say,	$186,000 per mile

Annual Operating and Maintenance Costs

The operations personnel required for the short reach of this test would be more than sufficient for the operation of approximately five miles of recharge line, in that the collection and analysis of the detailed data collected for the test would not be required in the practical operation of a recharge barrier project. An operating crew of some five trained operators to provide for three shifts a day, seven days a week, would probably suffice. In addition, one Hydrographer would be required for such a reach to measure observation wells and maintain recording equipment. The salaries for such a group, including an engineer for immediate supervision of operations and engineering control computations, and including general supervision costs totals some $45,000 a year, or $9,000 per mile of recharge line. Including power, telephone and other related costs, the operational costs, as indicated below, have been estimated at $10,000 annually per mile of recharge line.

Routine project maintenance costs as experienced at the test site could probably be reduced slightly on a larger scale and have been estimated accordingly.

It is estimated that recharge well development for a gravel-packed well would be required about one each two years, at a cost of $1,000 for each operation.

Water costs have not been considered in that such costs would vary widely with the availability of supplies and must be evaluated in relation to the benefit of such a supply as a source of replenishment to the basin.

Control data required for operations could be minimized as compared with those experienced during the test, as indicated in the following cost analysis:

ANNUAL OPERATING COSTS

Operation and Maintenance	
1. Supervision and Operations Personnel	$10,000
2. Routine Project Maintenance and Miscellaneous Materials	4,000
3. Recharge Well Redevelopment 3½ wells x $1,000 per well	3,500
4. Chlorine (5 cfs, 8 ppm)	4,500
Engineering Control Data	
1. Well Sampling Operations	5,000
2. Water Analysis and Related Laboratory Work	2,000
Total	$29,000
10% Contingency	2,900
Total Annual Operating Cost	$31,900
	Say, $32,000 per mile

Economic Justification

Application of the costs itemized above to a proposed sea water barrier, and the economic justification of such expenditures, would require a careful analysis of each item to fit conditions existing in the area considered. If it were assumed that the aquifer conditions along the entire coast were similar to those encountered at the test site, the application of these costs and the economic justification thereof in relation to a complete barrier in the West Coast Basin might be analyzed, as indicated in the following.

Such a barrier would ultimately insure the annual safe pumping yield from the basin. This yield has been established, at present as approximately 30,000 acre feet per year (West Coast Basin Reference by State Engineer, February 1952). It is obvious that if some protection is not provided, the entire resources of the basin will eventually be destroyed.

As indicated below, the costs of water have not been considered, in that the protection of 30,000 acre feet of annual production alone would, under the assumptions made above, approximately justify the capital outlay and annual operating costs.

Annual Costs vs. Annual Benefits

Annual Operating Cost $32,000 x 11 miles of reach	$352,000
Annual Costs of Capital Recovery $14,900 * x 11 miles of reach	164,000
Total Cost	$516,000
Annual value of 30,000 A.F. at $21/A.F. less pumping costs ($4.50) †	$495,000

* An analysis of the capital outlay items, assuming 10-year life for recharge wells and appurtenances and measuring equipment; 20-year life for the balance of the replaceable items; and with interest at 4 percent per annum, indicates an annual capital recovery cost of $14,900.

† Based on an average pumping lift of 150 feet at a cost of $0.03/A.F./ft. of lift.

In addition to the real value of 30,000 acre feet, a value of considerable proportions must be considered in the loss of perhaps some million acre feet of basin storage capacity. The present import facilities of the Metropolitan Water District have insufficient capacity to provide for either peak daily demand or peak seasonal demand; hence surface storage would have to be provided in the event ground water supplies were destroyed. Although sufficient data are not at hand to accurately evaluate this required storage, it is obvious that an economic analysis of the costs involved in developing such surface storage considered in relation to the costs of a pressure barrier would result in a definite economic benefit in favor of the barrier. The costs for such storage have been estimated by the West Basin Water Association to range from $4,500 to $18,000 an acre foot.

The costs of a water supply could be considered on the basis of its value as recharge to the basin, thereby permitting a greater safe pumping yield from the basin. Hence, assuming a very conservative value of $15 per acre foot of ground water, any cost of water less than this would provide a definite additional economic return. For example, it has been estimated that some 52,000 acre feet per year is presently intruding as ocean water. In order to control this intrusion, it might be estimated that some 75,000 acre feet a year will be required. If such a water supply could be purchased at $10 an acre foot, then an economic benefit to the basin of $5 per acre foot, or a total of $375,000 per year, could be realized.

Considering these costs in relation to the extension of the existing recharge line for the entire eleven miles of affected coast line in the West Basin, it is presently indicated that due to the limited thickness of the aquifer to the north, the cost would be considerably lower than those presented for an average reach. While to the south where the aquifer becomes deep, quite permeable and unconfined, it is anticipated that the costs would be considerably higher. Throughout the southerly reach, recharge through seepage pits and/or surface basins would become a consideration. This would dictate a need for further geologic exploration and a study of the applicability of the experiences to date to the problem of creating an effective barrier in the unconfined aquifer reaches near Redondo Beach.

CHAPTER IX

FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

It may be concluded that the subject investigational work for the prevention and control of sea water intrusion has established that for areas with similar geologic, hydrologic and topographic conditions as those found at the test site in Manhattan Beach in the West Coast Basin:

(a) Such prevention and control can be successfully realized in a confined coastal aquifer by recharge through wells.

(b) Such recharge can pressurize a confined aquifer continually through a given reach, thereby reversing any pre-existing landward gradient and preventing further sea water intrusion.

(c) Such recharge will provide significant replenishment to the inland ground water basin with only an inconsequential loss of fresh water oceanward in relation to the total quantity of injection.

(d) Such recharge can be performed in an aquifer previously degraded by sea water intrusion and, within the physical and hydrologic limitations as established at the test site, will not cause any consequential deleterious effect on inland pumped supplies. In fact, all evidence collected to date indicates that the degraded portion of the aquifer can be reclaimed by recharge through wells.

The project was located in the cities of Manhattan and Hermosa Beaches, Los Angeles County, California. A line of recharge wells covering a reach of some 4500 feet in length was installed approximately parallel to and some 2000 feet inland from the Pacific Ocean coast line. Nine recharge wells, spaced at 500 feet along the recharge line, and thirty-six observation wells, located along, inland and seaward of the line, were originally drilled. Eighteen additional observation wells were added as the test progressed. The wells penetrated a pressurized aquifer confined at the top by an impervious clay cap slightly below sea level and bounded at the lower limits by relatively impervious sediments some 110 feet below sea level. The ground waters at the test site were completely degraded by sea water intrusion while decreasing effects of intrusion extended approximately 5000 feet inland. Treated Colorado River water, imported by the Metropolitan Water District, was used for injection purposes and was piped from a connection with the El Segundo branch of the West Basin Feeder located some 7300 feet inland of the recharge line.

Prior to recharge, sea water was flowing landward under the influence of an inland ground water gradient. As injection was initiated at the first well, a pressure cone formed, centered at the well. The pressure effect extended almost instantaneously in all directions and within a few hours was noted 1000 feet distant from the well. As injection was initiated at adjacent wells, the pressure cones overlapped, forming a pressure ridge along the recharge line with alternate peaks and valleys. As stability along the entire line was reached, the cones merged into a stable, constant landward and oceanward gradient at a distance of some 250 to 500 feet from the recharge line. At this distance the individual pressure cones lost their identity and the shape of the mound was essentially uniform. This ridge reversed the pre-existing landward gradient between the recharge line and the ocean, and prevented further sea water intrusion.

Initially, fresh water moved radially from the recharge well and finally merged with flow from the adjacent wells. After moving some 250 to 500 feet from the recharge line, all radial movement was transformed into essentially landward and oceanward flow. As the gradient oceanward was relatively flat, movement oceanward was extremely slow, while the steeper landward gradient maintained relatively higher ground water velocities landward. As the fresh water advanced, it overrode and displaced the pre-existing saline ground water. Although the pressurizing effect of injection was relatively rapid and further sea water intrusion was thereby prevented, the movement and actual mergence of the injected fresh water was relatively slow.

Detail findings, conclusions, and recommendations are enumerated as follows:

Project Facilities

1. Experience with the well types used during the test indicated that a gravel-packed recharge well with a minimum diameter for the gravel envelope of 20 inches is desirable in aquifers similar to those found at Manhattan Beach, where fine to medium sands may be encountered with limited gravel stringers. In aquifers composed of such non-homogeneous materials, all critical portions should be penetrated to insure an effective and rapid pressurization.

2. In addition to gravel-packing, the recharge well casing should be of sufficient diameter to permit the entry of standard sized well tools. In this connection the 12-inch casings used at the test project proved adequate. In the event no rehabilitation work was anticipated, a somewhat smaller casing could be used.
3. Test holes through the unconfined surface sands and bottomed near the top of the "clay cap" immediately adjacent to the recharge wells, gave evidence of some leakage from the confined portions of the aquifer. This leakage apparently occurs along the well casing through the confining clays as a result of the casing having perforated the clay and having left a zone of weakness along the casing. The test established that the construction of a proper cement seal is required through this zone of weakness.
4. Observation well diameters should be in relation to their anticipated use. If water stage recorders are to be installed and/or pumped water samples with conductivity traverses are to be obtained, then an 8-inch casing is a minium required diameter. If water level elevation measurements only are to be obtained, a 4-inch observation well is recommended as a minimum size.
5. All observation well perforations should be carefully located and cut in relation to the position and extent of gravel stringers encountered in the aquifer. The need for this care is obviated in a gravel-packed well, which should be perforated to permit a maximum discharge with a minimum velocity in the perforations.
6. Development by pumping and/or sand bailing should be limited in order to avoid the creation of voids in the aquifer by the removal of excess quantities of sand. Such limitation is not as important in a gravel-packed well in that the removed sand will be replaced by gravel.
7. Operations during the test showed the need for accurate control of the recharge flow. Hence, if the supply is subject to pressure variations, the installation of a pressure regulator on the main distribution line would be recommended. A manual control valve with a reliable flow meter at each well would probably provide the simplest and most economical flow control mechanism. Each well would require independent control of recharge in relation to its acceptance rate and the transmissibility characteristics of the local aquifer. In addition, a well conductor pipe from the surface of the ground to the ground water level should be provided, with means of preventing aeration.
8. Although the chlorine equipment used at the test was provided with automatic chlorine feed control and a chlorine residual recorder to facilitate adjustment to the frequent changes made in both recharge and chlorination rate, in a practical operation, after initial stabilization, both recharge and chlorination rates would be expected to remain fairly constant. Hence, manual feed control and occasional field tests to check chlorine residual would suffice to operate a stabilized pressure barrier.

Geology

9. The aquifer at the recharge site is extensive, confined, heterogeneous, and pressurized (the latter imperfectly). It is probably similar to some aquifers occurring along other seaward margins of coastal ground water basins of California.
   At Manhattan Beach the geologic sections definitely showed correlation between the Merged aquifer at the coast with important inland water-producing aquifers.
10. The confining "clay cap" has been eroded near the sea coast some 2000 feet oceanward of the recharge line, which may have provided a limit to pressurizing and resulted in a delay in the stabilization of the seaward gradient by permitting storage in the unconfined portion of the aquifer.
11. A zone of sharp flexuring or faulting appears to exist oceanward of the recharge line. Its effect upon the test results is probably limited to a localized reduction in transmissibility. This zone has probably somewhat restricted sea water intrusion in the past, and caused a reduction of normal oceanward flow during initiation of recharge.
12. All wells drilled in connection with recharge should be carefully logged and analyzed in relation to the general geology of the area in order to: (a) determine the structural effectiveness of the confining cap of the confined aquifer; (b) determine the physical limits and homogeneity of the aquifer in relation to the construction of recharge wells and to its hydraulic parameters; (c) identify the correlative formations of the important inland water-bearing aquifers; and (d) detect special conditions within the aquifer which may accelerate, limit, or prevent ground water flow in a given area or direction.

Hydraulic Aspects of the Recharge Line

13. The total required recharge rate per foot in a long line of recharge wells is the sum of: (a) oceanward flow from the recharge line; (b) initial ground water flow landward, i.e., prior to

recharge; and (c) increased ground water flow landward due to recharge. Whereas items (a) and (b) depend on measurable ground water elevations, aquifer constants and the physical dimensions of the aquifer, item (c) evidently depends upon a quantity unmeasurable before recharge begins. At the test site the increased gradient and therefore the increased recharge rate required was about 60% greater than the rate of sea water intrusion under the existing landward gradient prior to recharge. This additional requirement may be due to an inland storage demand or to leakage through imperfectly confined portions of the aquifer.

14. The total head measured in a recharge well consists of two parts—the mound elevation and the injection head. Mound elevation is that elevation of the ground water piezometric surface which is created adjacent to the recharge well due to recharge operations. Injection head is that head required in the well to overcome the energy loss occurring when the injected water passes through the casing perforations and enters the aquifer face. (The aquifer face may be defined as the vertical undisturbed surface of the aquifer exposed to flow from the recharge well.)

15. The mound elevation depends upon the amount of water being injected in a given reach, the spacing of the recharge wells, the transmissibility of the aquifer, the ground water gradient as it would have been if unaffected by recharge, and the degree to which this later gradient may be increased by recharge.

16. Injection head is dependent upon the quantity of water being injected, the type and size of recharge well, the number of perforations, and the local transmissibility characteristics of the aquifer face immediately adjacent to the recharge well.

17. At the test site at Manhattan Beach it was found that the average transmissibility determined by pumping tests prior to recharge was about 0.165 cfs/ft. under a unit gradient; that during recharge it was about 0.18 cfs/ft. per unit gradient; that the average landward gradient before recharge commenced was about 0.0041 feet/foot; that the apparent porosity landward of the recharge line averaged 27%; and that based on a stabilized landward gradient of 0.0065 feet/foot during recharge, the rate of landward movement of injected fresh water was about 1250 feet per year.

18. Maximum acceptance rate tests at well E indicated rates up to 1.86 cfs. Although the maximum acceptance rate of well I-A was not determined, it indicated even more efficient acceptance characteristics at the lower flows experienced. Rates of 0.5 to 0.75 cfs were maintained for long periods at the non-gravel-packed wells. Hence, with the aquifer characteristics as experienced at the test site and a well designed and constructed gravel-packed recharge well, it may be expected that a well acceptance rate of 1.5 cfs could be maintained.

19. The spacing of recharge wells can be determined by estimating the unit recharge requirement and the expected acceptance rate of the wells. As a result of the findings of this test, it is believed that the installation of recharge wells should be based on a sequence of well installations. As each well in an assumed uniform transmissibility reach is completed, a pumping test should be made to determine any error in assumption of the value of transmissibility. If the transmissibility has changed to any extent, the adequacy of the original estimated spacing should be reevaluated and, if necessary, a new spacing adopted.

20. Test experience indicates that the number of observation wells required to control the operation of a barrier mound is limited to internodal wells and occasional sets of wells (perhaps one set each mile) landward and oceanward of the recharge line. An occasional observation well adjacent to a recharge well is recommended to measure maximum pressure against the confining strata and to determine the actual injection head in relation to the well's efficiency in acceptance.

21. Pumping at nearby well fields, unless the cone of depression approaches the line of recharge, does not affect the required rate of recharge except as it contributes to the depletion of the supplies of the basin and results in an increase of the landward gradient in the basin, thereby creating a continuously increasing recharge rate to maintain the sea water barrier.

Pumping at nearby producing well fields, where the resulting cone of depression is sufficiently remote (6500 feet at the test site) no immediate effect is apparent at the recharge line. However, as such pumping contributes to the general depletion of the ground water supplies and increases the landward gradient in the basin, the required recharge rate necessary to maintain the barrier will increase. In the event pumping was sufficiently close and the cone of depression approached the recharge line, it would be expected that the influence would be immediate and require a corresponding increase in recharge rates.

22. Results of conductivity traverses taken in the project observation wells corroborated previous studies performed by the University of Cali-

fornia at Berkeley which showed that sea water intrudes as an underriding wedge; however, the traverses indicated that historically some mixing had taken place and that prior to recharge the existing waters at the recharge line and for over 4000 feet inland, were saline to a varying degree. The traverses also showed that injected fresh water moved across the more dense native water as an overriding wedge. The traverses further showed that there was a minimum of mixing between the injected fresh and partially degraded native water.

23. In connection with the initiation of fresh water injection into an aquifer degraded by sea water intrusion, a minor "saline wave" is produced which moves landward from the recharge line. As it moves into areas of the aquifer which are not as highly degraded, the magnitude of its characteristically higher saline content is rapidly dissipated. At the test site, the saline wave was indistinquishable at well G-8, a distance of 1180 feet inland of the recharge line.

Water Quality and Maintenance of Aquifer Transmissibility

24. If fresh water recharge is initiated into an aquifer containing sea water, a cation exchange reaction between the sodium-saturated aquifer sediments and the calcium and magnesium ions of the injected water will occur. Thus, if an inferior water (inferior due to hardness) is used as a source for recharge supply, a softening effect may be expected so long as the sodium concentrations will act to replace calcium and magnesium ions in the recharge water, thereby minimizing any possible degradation of the native ground waters.

25. Ultimately, it may be expected that the injection of fresh water will increase the aquifer's transmissibility slightly through cation exchange. Although the indication of this effect has not occurred during the limited period of the test, ultimately a corresponding slight increase in recharge rate may be required.

26. Tests established the presence of many "slime-forming bacteria" in the aquifer. Chlorination of the recharge water is necessary to inhibit the growth of such slime-producing organisms which, if uncontrolled, will rapidly reduce the acceptance rate of the well. The necessary chlorination rate may vary with individual recharge waters and aquifers. At the test site, it was found that initially a chlorine dosage of 20 ppm was sufficient to prepare the aquifer for continued injection, and that the minimum necessary sustained chlorine dosage was more than 5 ppm, but less than 10 ppm.

27. The effect of suspended solids was not evaluated in that the recharge waters were almost devoid thereof. Other chemical effects, such as deterioration of well casings and facilities due to rust, were also not evaluted due to the relatively limited period of operations. The effect of entrained air in relation to air-binding and stimulation to the growth of slime-forming organisms also requires further research to evaluate. Reference is made to Chapter VI, E for a brief discussion of these factors.

Maintenance and Operational Problems

28. The test demonstrated that overdevelopment of a recharge well in coastal aquifer materials could be serious in that voids could be created within the aquifer and that during recharge operations a structural failure of the confining clays adjacent to the casing could occur with a subsequent major subsidence of the overlying materials. Test experience indicates that the danger of subsidence is minimized in a gravel-packed well.

29. The cement seal at the clay cap, mentioned in Conclusion No. 3, will probably prevent or at least minimize leakage along the well casing and subsequent structural failure of the clay cap adjacent to the well casing. Although the danger of subsidence in a gravel-packed well is more remote than in a non-gravel-packed well, the proper sealing of the clay cap at the well casing to prevent leakage is considered essential regardless of the type of well installed.

30. Experience with the project wells showed that rehabilitation of a well, where failure of the clay cap and later subsidence has occurred, can be difficult and costly with no assurance of success.

31. Rapid changes in recharge rate subject the confining clay cap to excessive pressures and/or erosion adjacent to the well casing where a zone of weakness may permit excessive leakage. Hence, it is recommended that all changes in injection rates be made in small increments at a sufficient time interval to permit local equalization of pressures created by injection.

32. Although the necessity of redevelopment as distinguished from rehabilitation did not occur during the test period, following the test, well K was successfully redeveloped by moderate surging and bailing. Due to the preponderance of fine to medium sand in the aquifer, redevelopment by pumping is not recommended in a non-gravel-packed well.

Costs

33. The test's costs cited in Chapter VIII are considered to be typical of those required for a similar research project that might be considered for an area where it is desired to evaluate detailed controls and conditions relative to recharge for pressurizing and/or replenishing a ground water aquifer.

Omitting right of way and water expenditures, the costs have been analyzed in relation to those required for developing, constructing and operating a practical barrier operation (i.e., omitting the collection of the detailed control test data required in the research project). *This analysis was based on a minimum reach of five miles, assuming aquifer characteristics and ground water overdraft conditions similar to those at Manhattan Beach.* On this basis, the capital outlay has been estimated to be about $186,000 per mile and the annual operation about $32,000 per mile. Considerations of right of way expenditures and water costs were omitted in that these would vary widely with local conditions, e.g., right of way costs for the test project were negligible; water costs were $20 per acre foot.

In ground water basins where significant overdraft exists and imported supplies are limited to rates of delivery less than peak consumer demands, the benefits to be realized from a pressure mound barrier to sea water instrusion are: (a) the protection of the existing safe yield of potable water from the basin; (b) the value of replenishment provided through recharge; and (c) the prevention of sea water intrusion and the resultant preservation of fresh water storage. Such benefits can economically justify the expenditures entailed in constructing and operating the barrier facilities. The latter benefit must be evaluated by considering the costs of surface storage which would be necessary if pumping from the aquifer to meet peak demand were not available as well as the intangible value in relation to a safe emergency water supply.

ACKNOWLEDGMENTS

State Water Resources Board and Division of Water Resources

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State Supervisory Personnel

A. D. Edmonston____State Engineer
Secretary, State Water Resources Board
Harvey O. Banks____Assistant State Engineer
Max Bookman_______Principal Hydraulic Engineer
Jack J. Coe_________Associate Hydraulic Engineer
Elmer C. Marliave___Supervising Engineering Geologist

State Geological Staff

Raymond C. Richter_Senior Engineering Geologist
Lawrence B. James__Senior Engineering Geologist
Donald L. McCann__Assistant Engineering Geologist
Philip J. Lorens_____Assistant Engineering Geologist

United States Geological Survey

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Joseph F. Poland____District Geologist
Arthur A. Garrett___Engineer

District Engineering and Operations Personnel

In addition to the authors of the report and report appendices, the following District personnel contributed to the final successful completion of the test and development of this report through their field and office work on the investigation.

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Castro, Rose M.
Connell, Garland Jr.
Davis, Everett
Dysart, Ben W.
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Spencer, Edwin N.
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Volpe, Thomas J.
Watkins, Philip S.
Wood, Donald
Wood, Verne

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Other Public and Civic Agencies

The coordinated efforts and splendid cooperation of the following public agencies were vital to initiation, progress and continued operation of the barrier test:

West Basin Water Association
Metropolitan Water District of Southern California
City of Manhattan Beach
City of Hermosa Beach
City of El Segundo

Other Cooperative Organizations

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Peck and Sons, well drillers
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California Water Service Company
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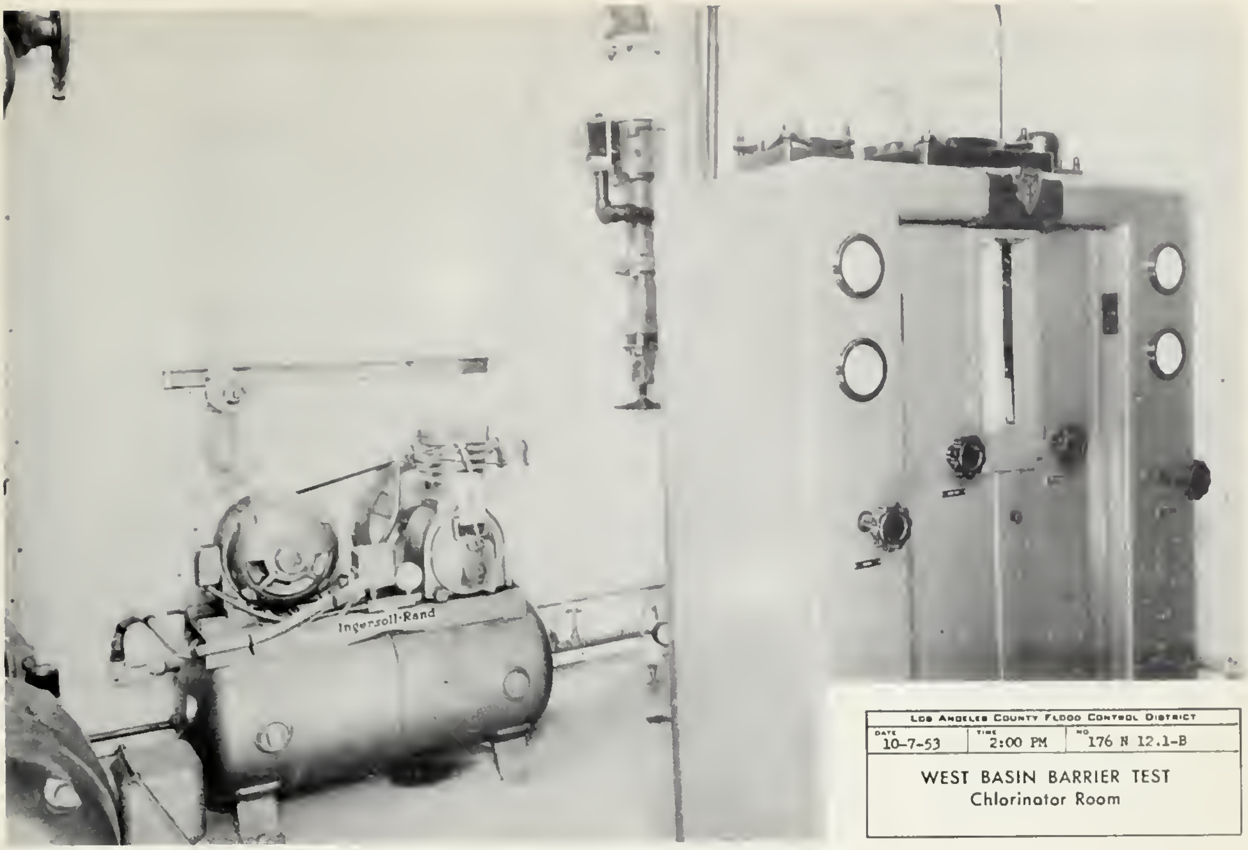
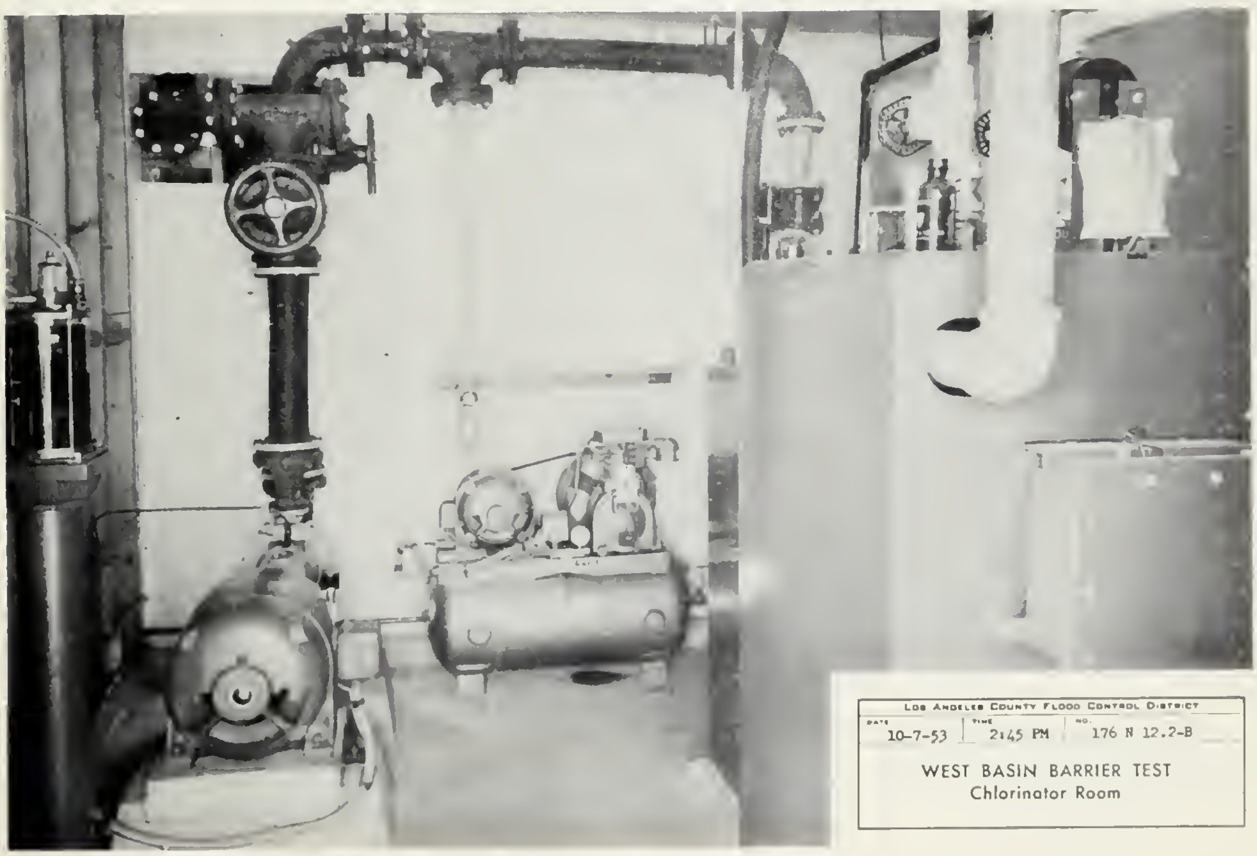


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

DATE	TIME	NO.
10-7-53	2:00 PM	176 N 12.1-B

WEST BASIN BARRIER TEST
Chlorinator Room



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

DATE	TIME	NO.
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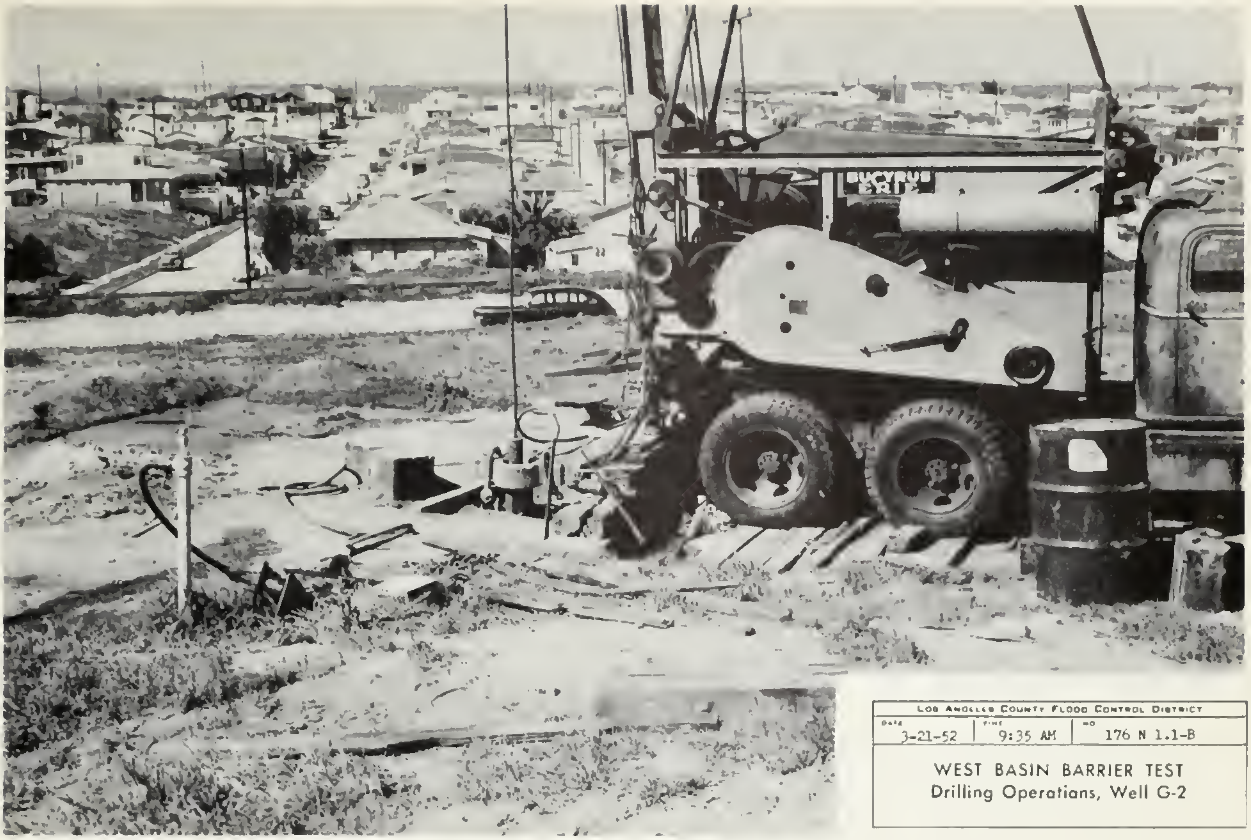
Chlorinator Room



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

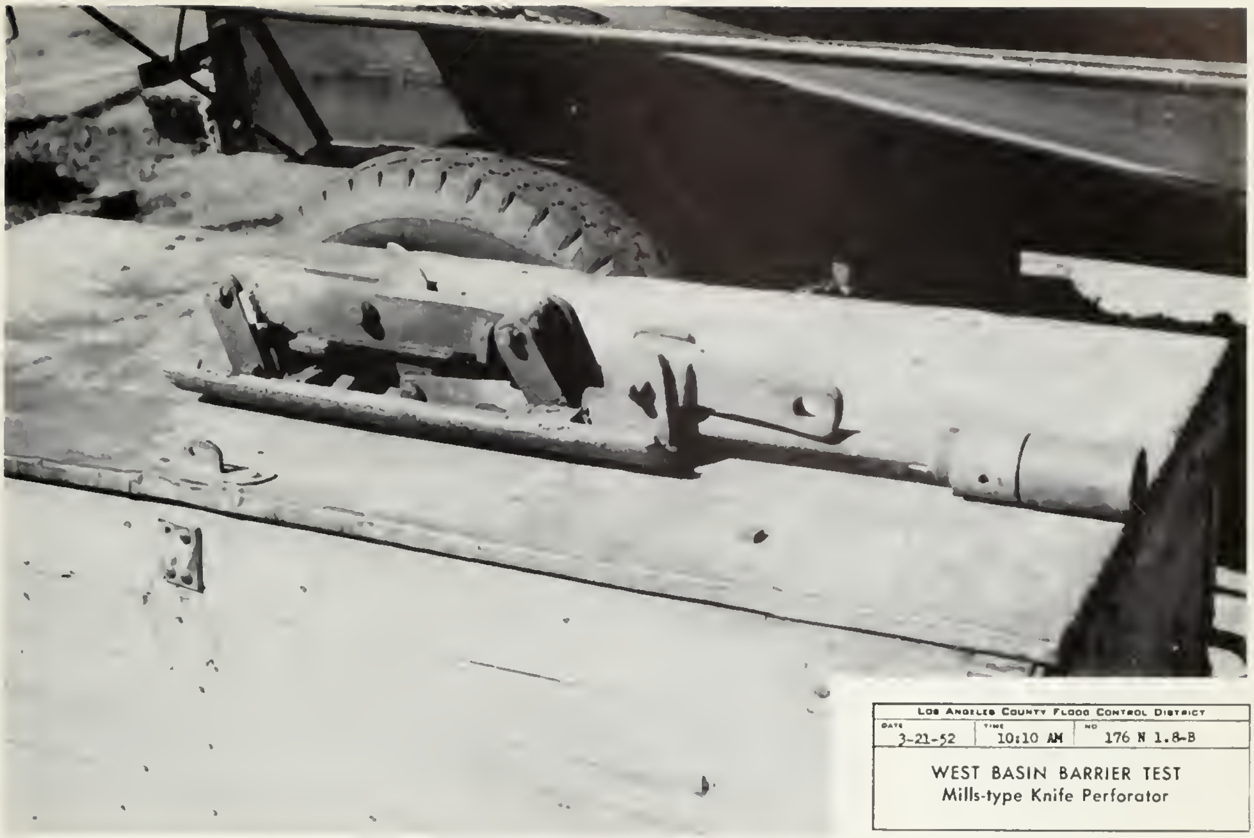
DATE	TIME	NO
3-21-52	9:35 AM	176 N 1.1-B

WEST BASIN BARRIER TEST
Drilling Operations, Well G-2

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

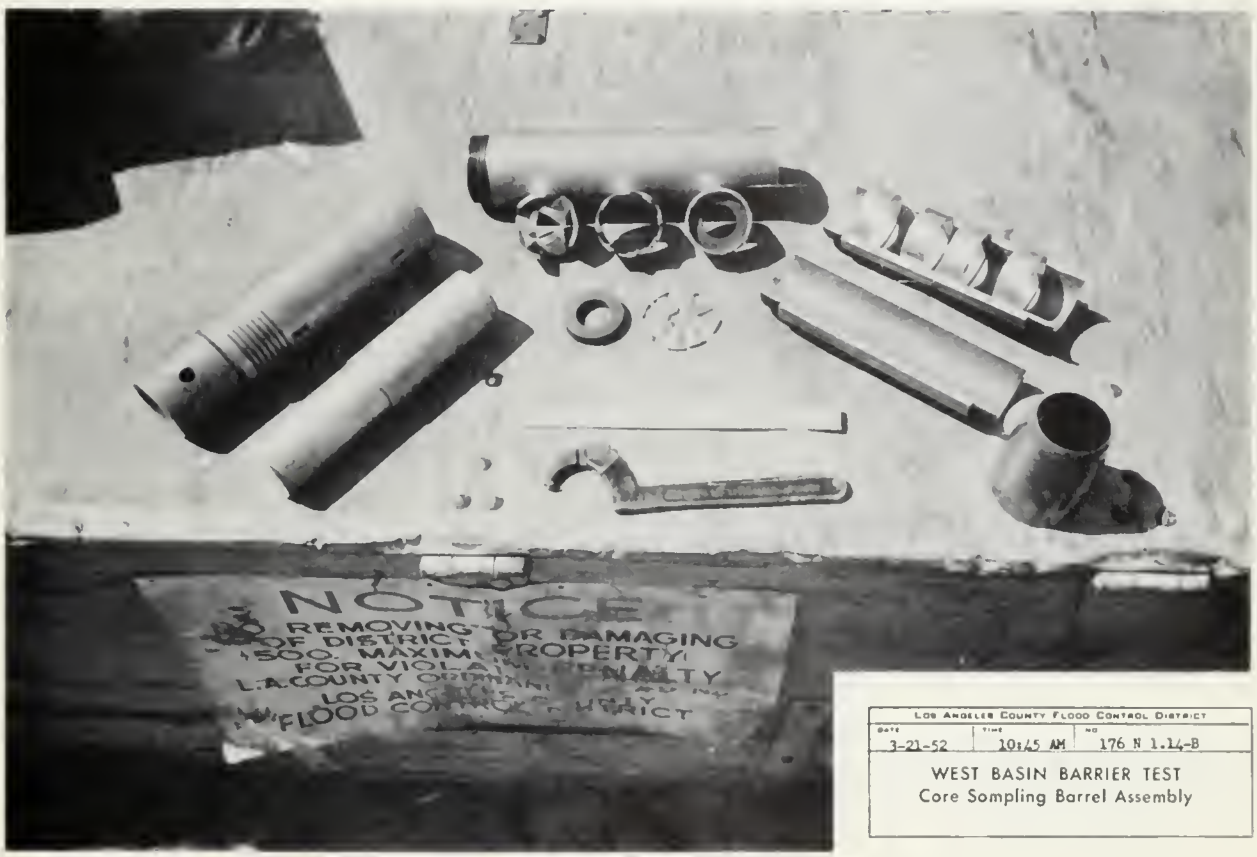
DATE	TIME	NO
3-21-52	10:00 AM	176 N 1.6-B

WEST BASIN BARRIER TEST
View of Bailer, Well G-4



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT		
DATE 3-21-52	TIME 10:10 AM	NO 176 N 1.8-B

WEST BASIN BARRIER TEST
Mills-type Knife Perforator



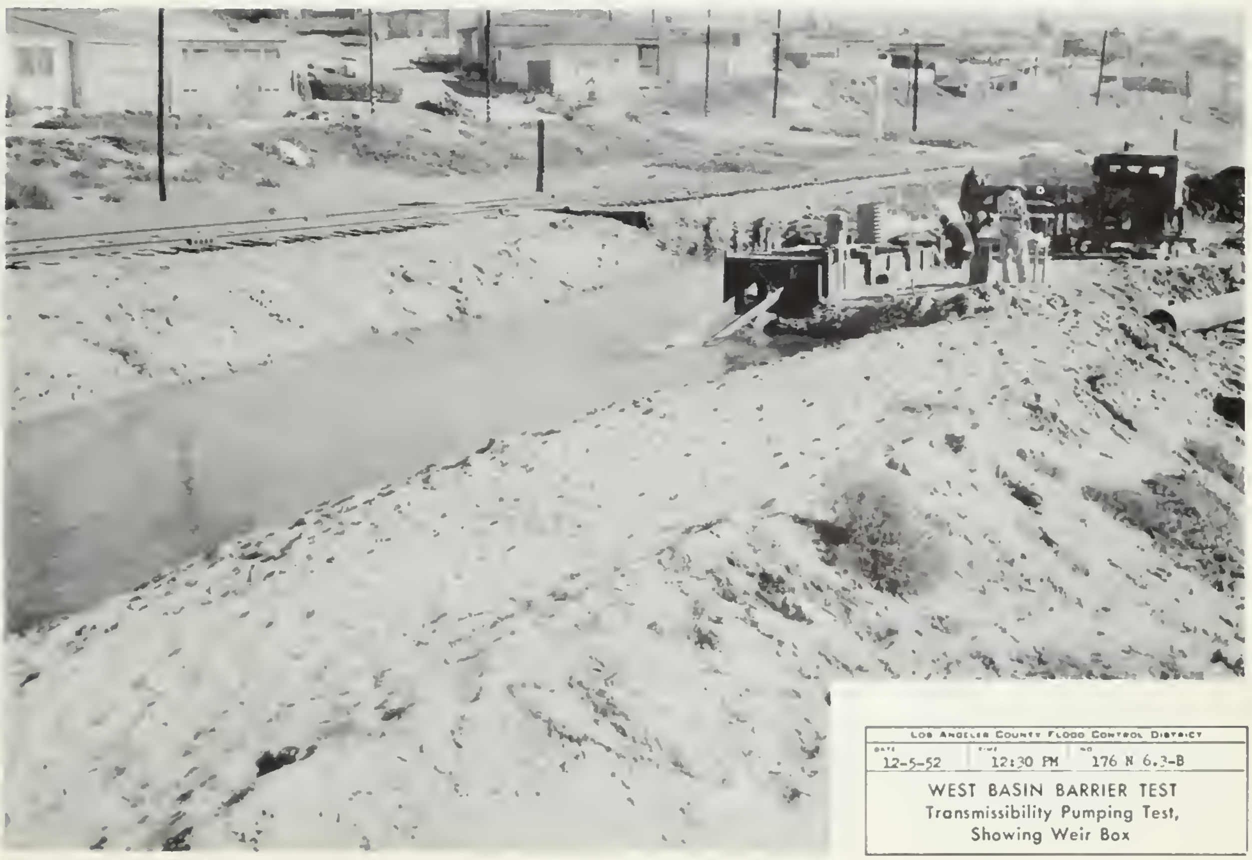
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT		
DATE 3-21-52	TIME 10:45 AM	NO 176 N 1.14-B

WEST BASIN BARRIER TEST
Core Sampling Barrel Assembly

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

DATE	TIME	NO
10-23-53	10:15 AM	176 N 14.1-B

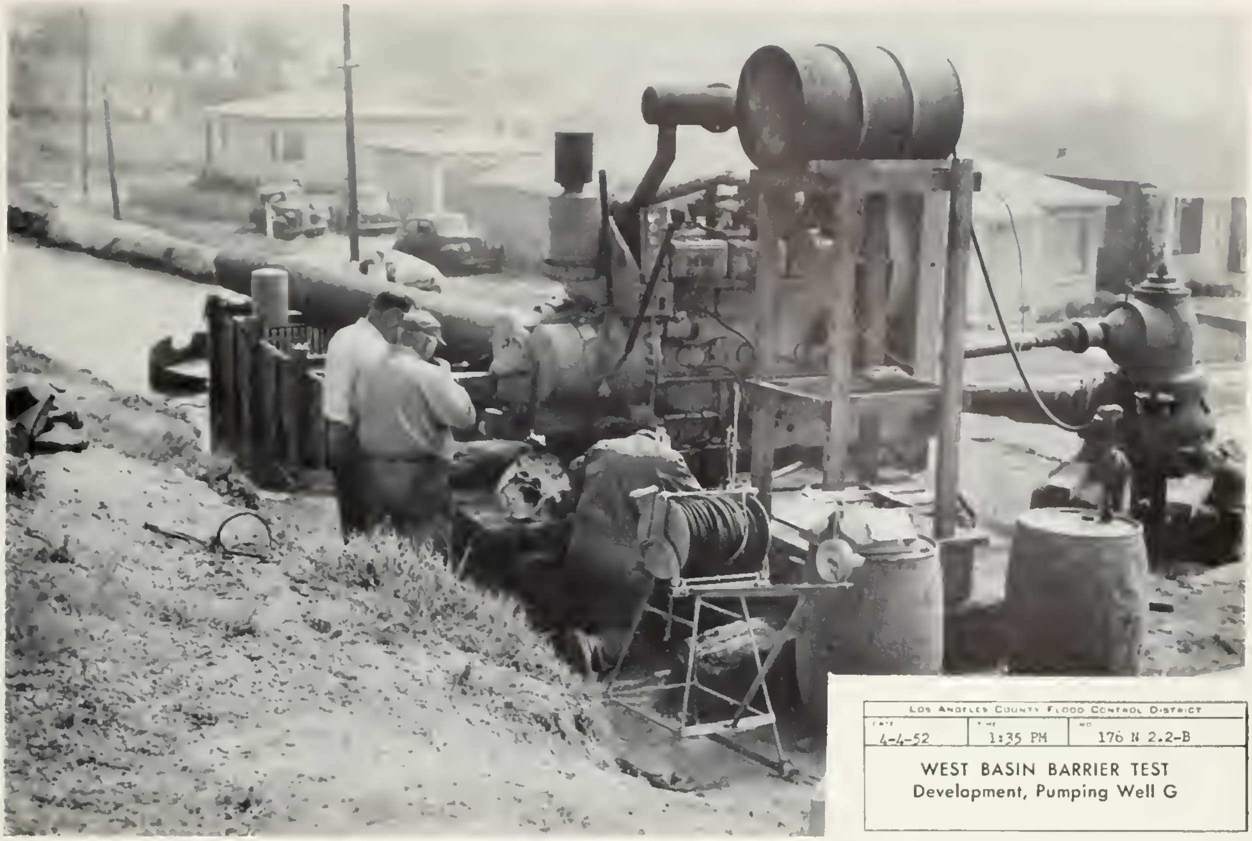
WEST BASIN BARRIER TEST
Well I-A, Well Development
Spreading Areas With 30"
Seepage Holes



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

DATE	TIME	NO
12-5-52	12:30 PM	176 N 6.3-B

WEST BASIN BARRIER TEST
Transmissibility Pumping Test,
Showing Weir Box



Los Angeles County Flood Control District

4-4-52 | 1:35 PM | 176 N 2.2-B

WEST BASIN BARRIER TEST
Development, Pumping Well G

Los Angeles County Flood Control District

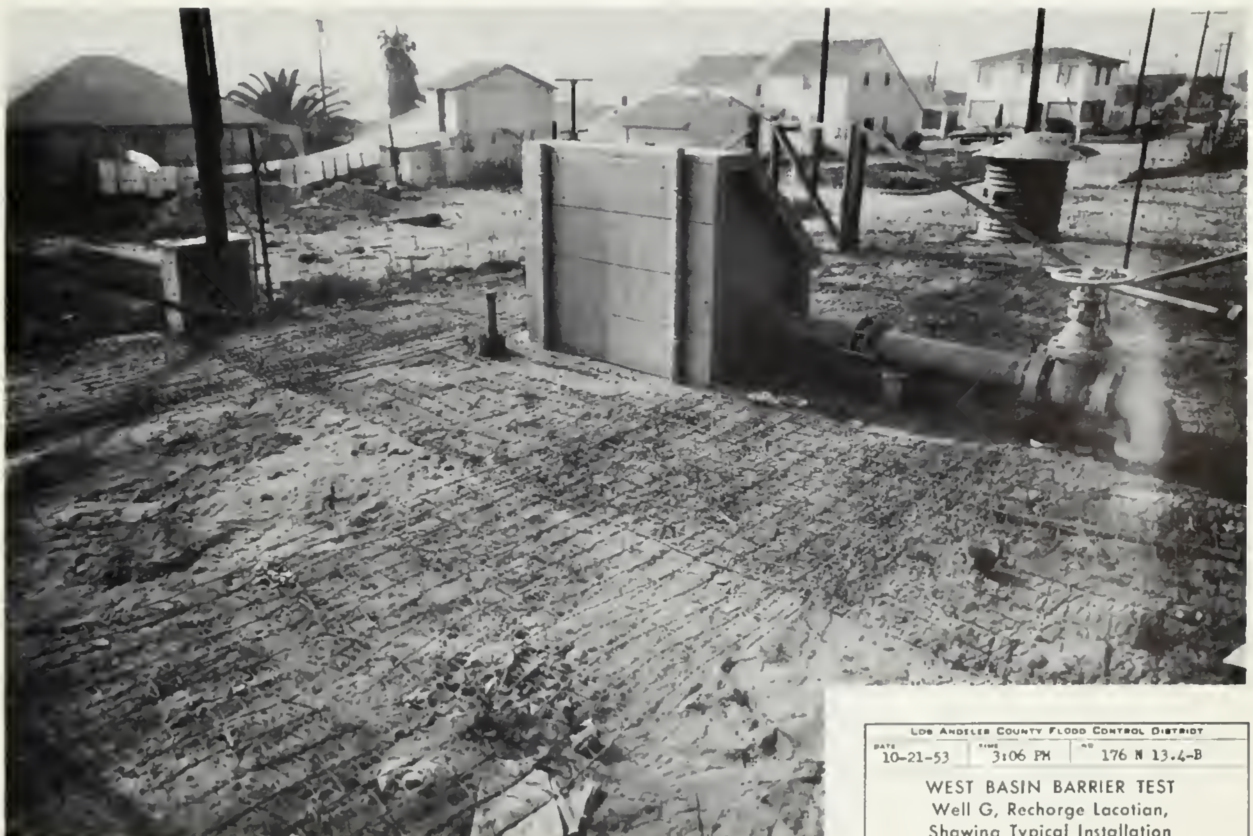
3-21-52 | 11:00 AM | 176 N 1.15-B

WEST BASIN BARRIER TEST
Observation Well G-13

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

DATE	TIME	NO
2-24-53	10:40 AM	176 N 8.1-B

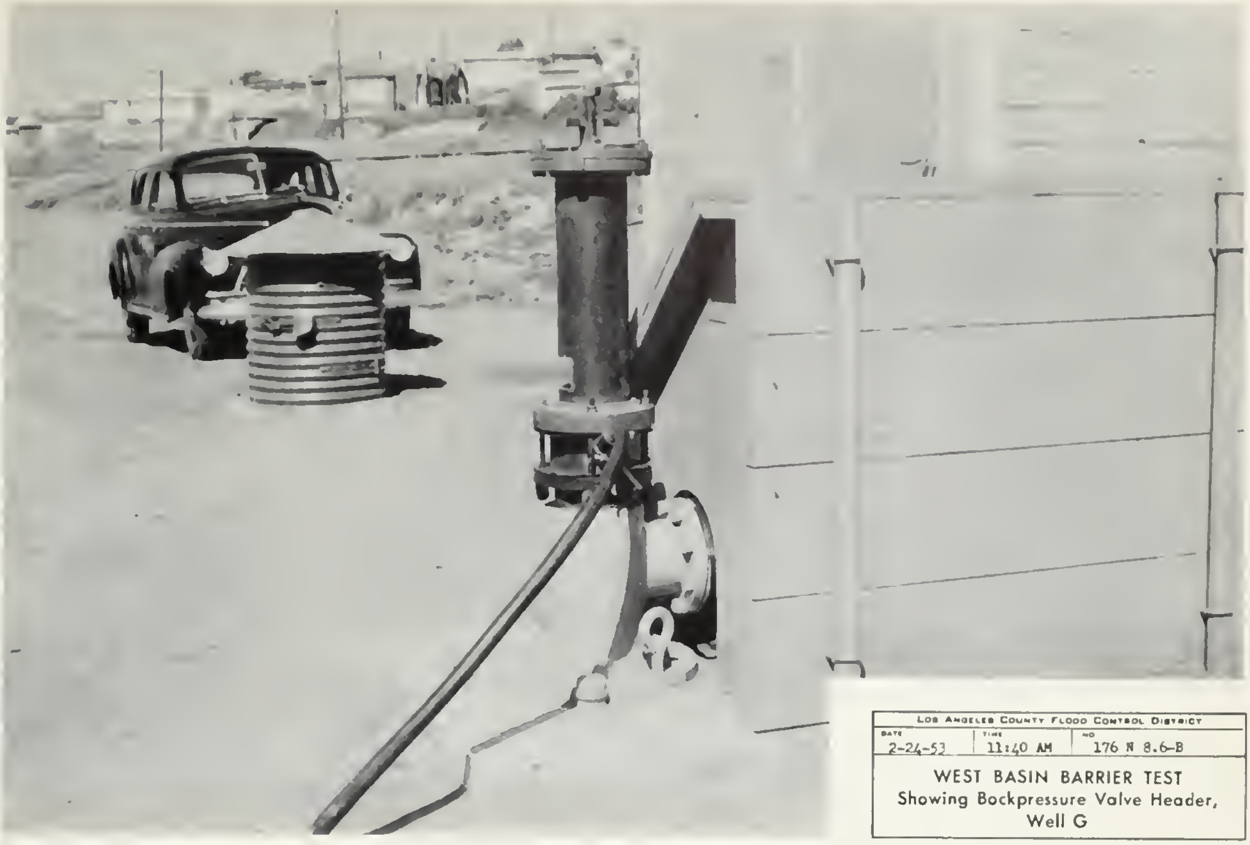
WEST BASIN BARRIER TEST
Well C, Showing Meter, Metra Valve and Recarder



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

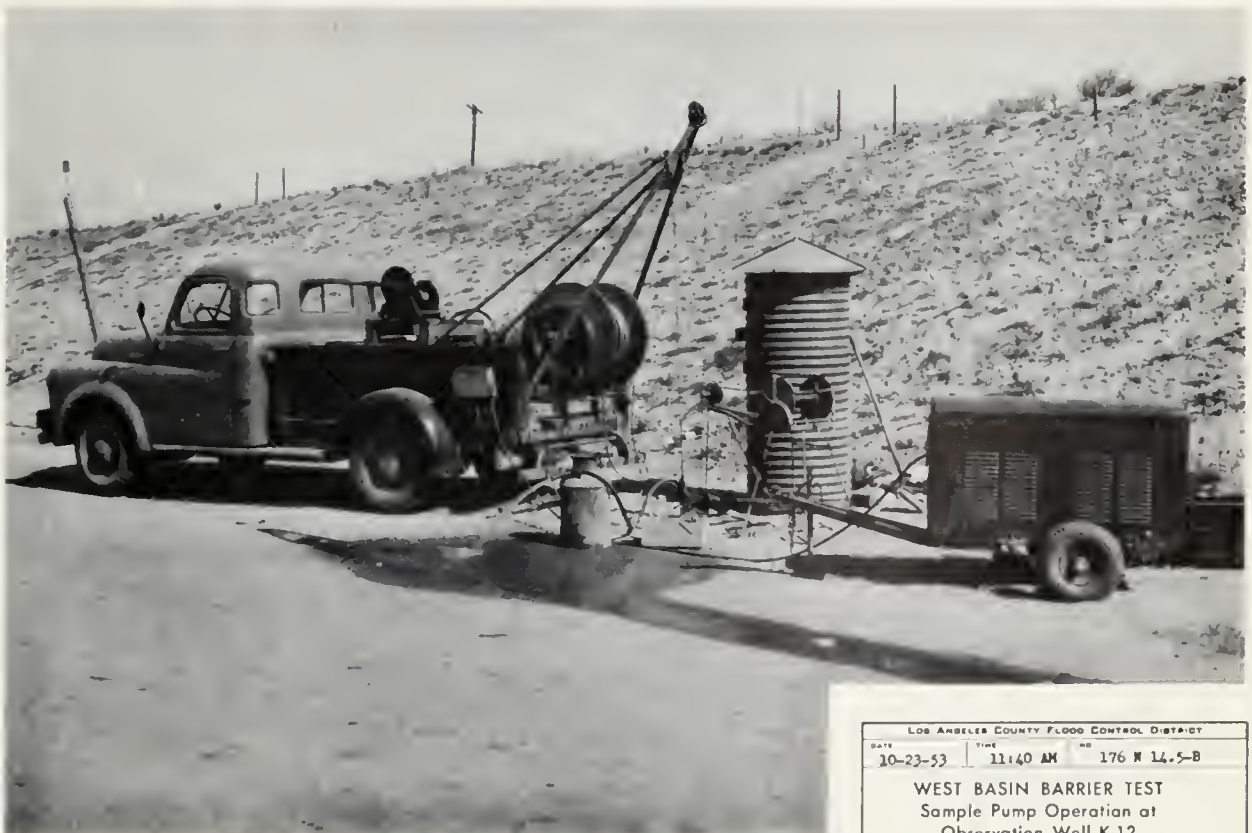
DATE	TIME	NO
10-21-53	3:06 PM	176 N 13.4-B

WEST BASIN BARRIER TEST
Well G, Rechorge Lacotian, Shawing Typical Installation



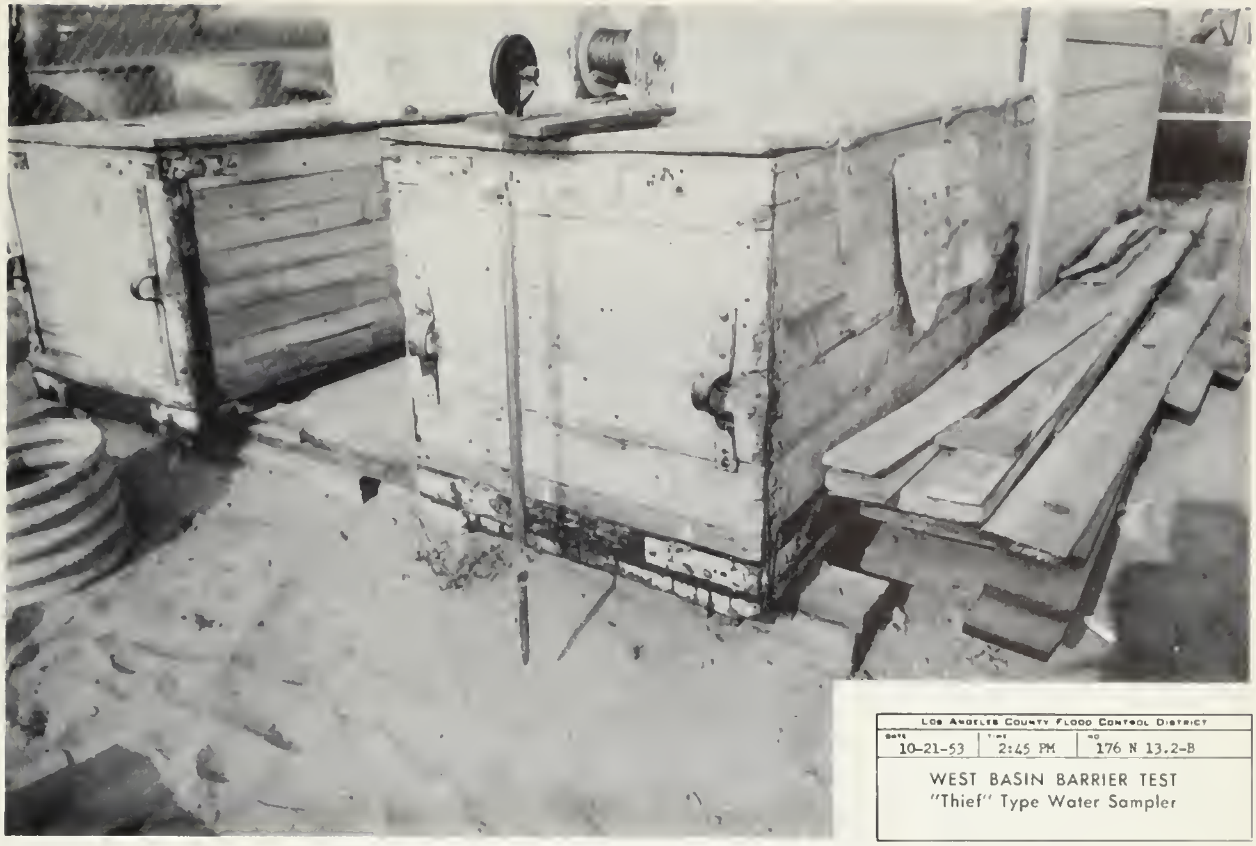
Los Angeles County Flood Control District		
DATE 2-24-53	TIME 11:40 AM	NO 176 N 8.6-B

WEST BASIN BARRIER TEST
Showing Bockpressure Valve Header, Well G



Los Angeles County Flood Control District		
DATE 10-23-53	TIME 11:40 AM	NO 176 N 14.5-B

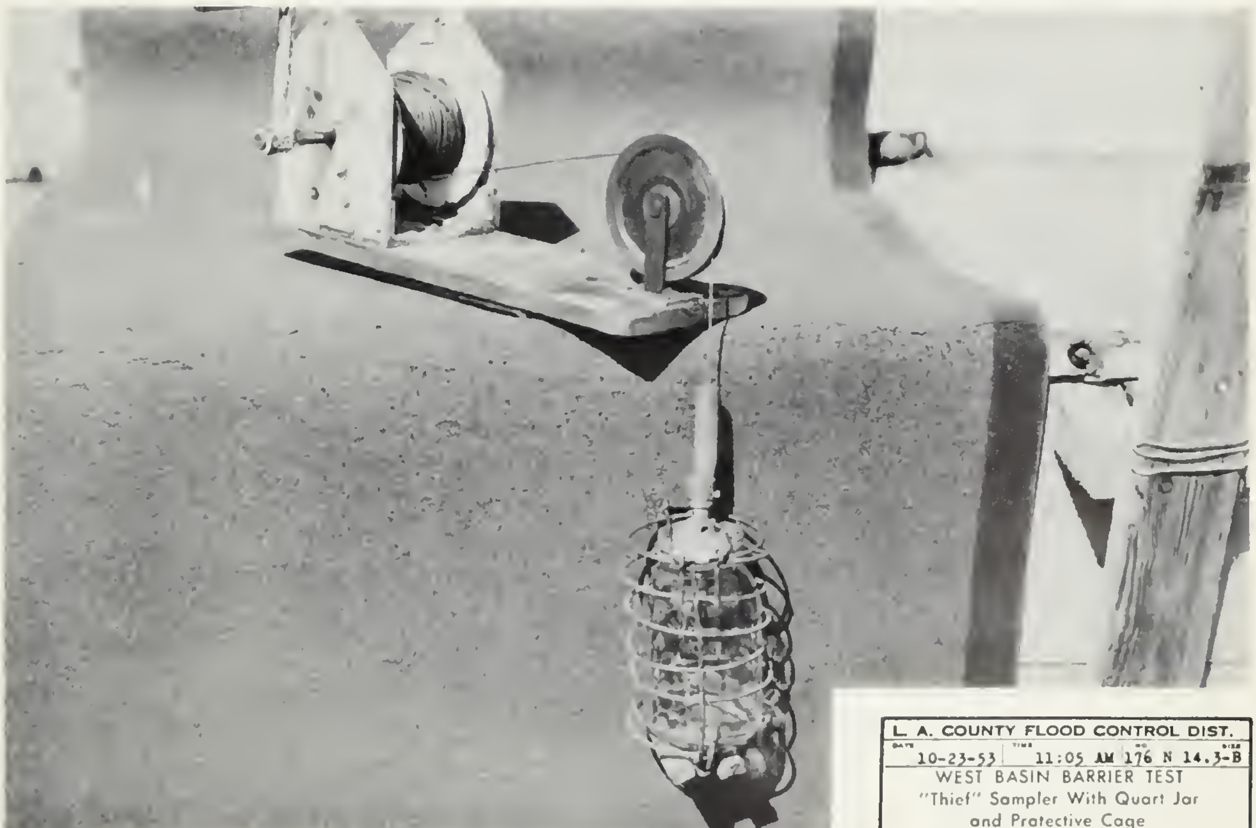
WEST BASIN BARRIER TEST
Sample Pump Operation at Observation Well K-12



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

DATE	TIME	NO
10-21-53	2:45 PM	176 N 13.2-B

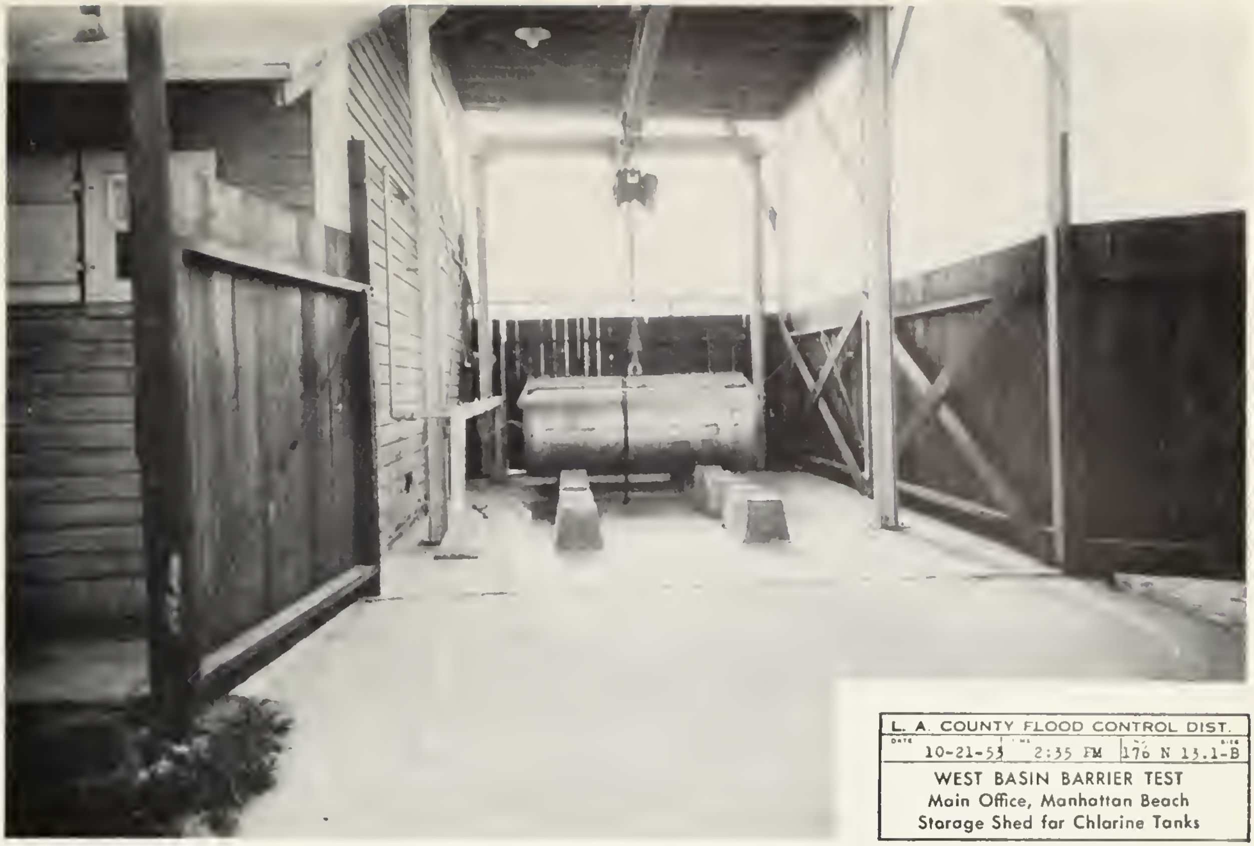
WEST BASIN BARRIER TEST
"Thief" Type Water Sampler



L. A. COUNTY FLOOD CONTROL DIST.

DATE	TIME	NO	SIZE
10-23-53	11:05 AM	176 N	14.3-B

WEST BASIN BARRIER TEST
"Thief" Sampler With Quart Jar
and Protective Cage



L. A. COUNTY FLOOD CONTROL DIST.
DATE 10-21-53 | TIME 2:35 PM | NO. 176 N 13.1-B

WEST BASIN BARRIER TEST
Main Office, Manhattan Beach
Storage Shed for Chlorine Tanks

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
DATE 8-28-53 | TIME 1:20 PM | NO 176 N 9.1-B

WEST BASIN BARRIER TEST
New Office Building, 11th and Ardmore Sts., Manhattan Beach

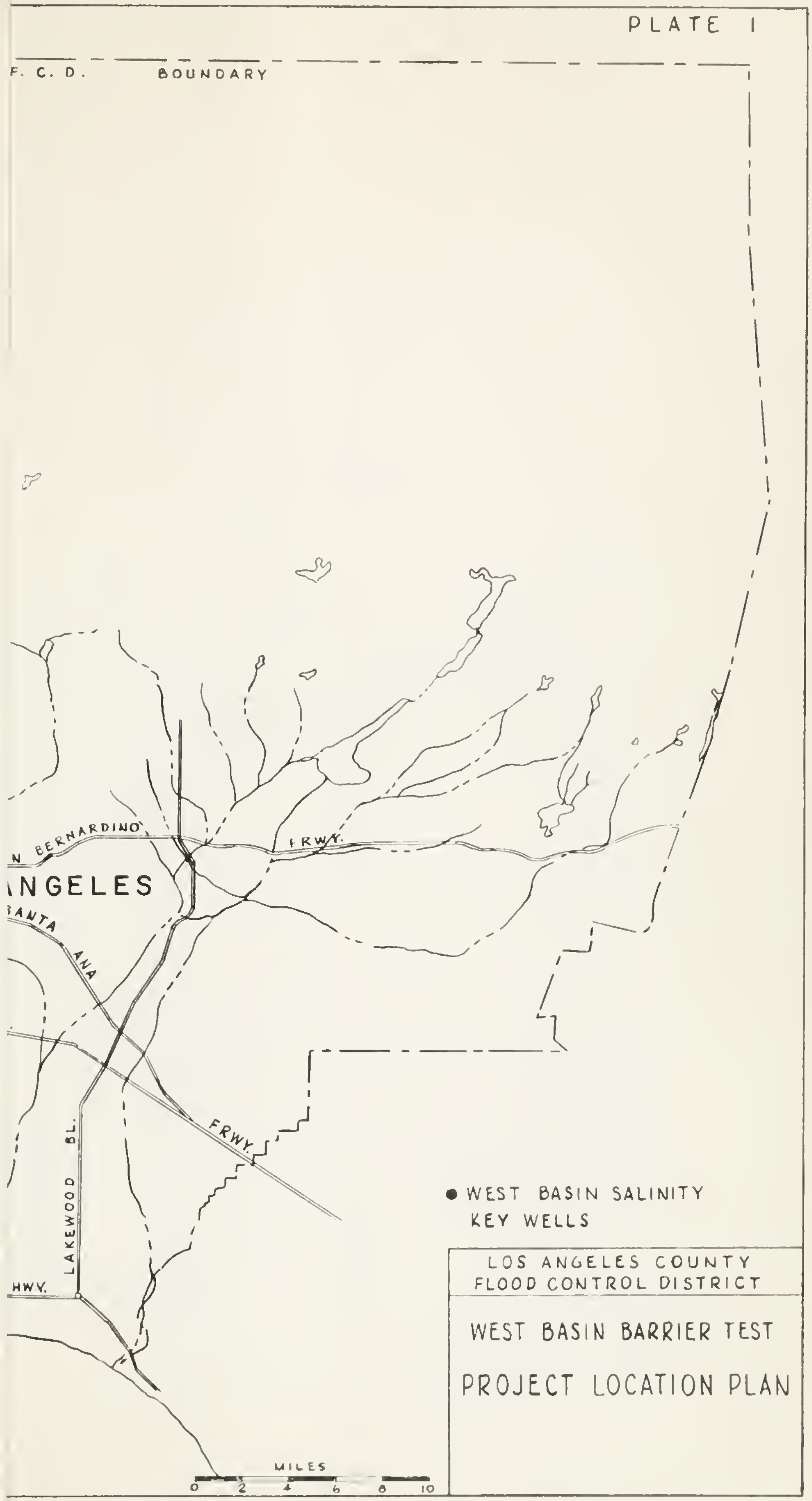
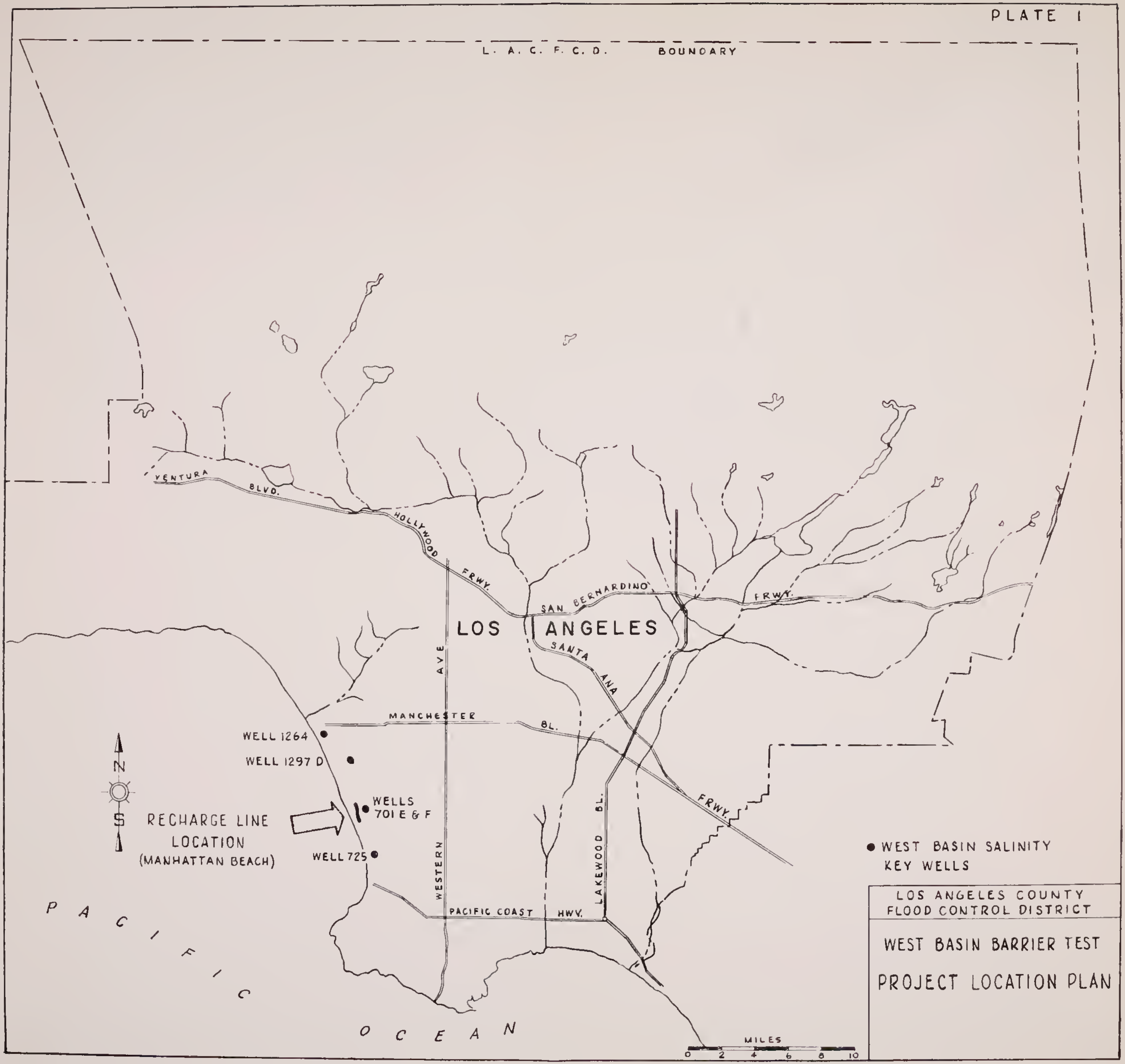
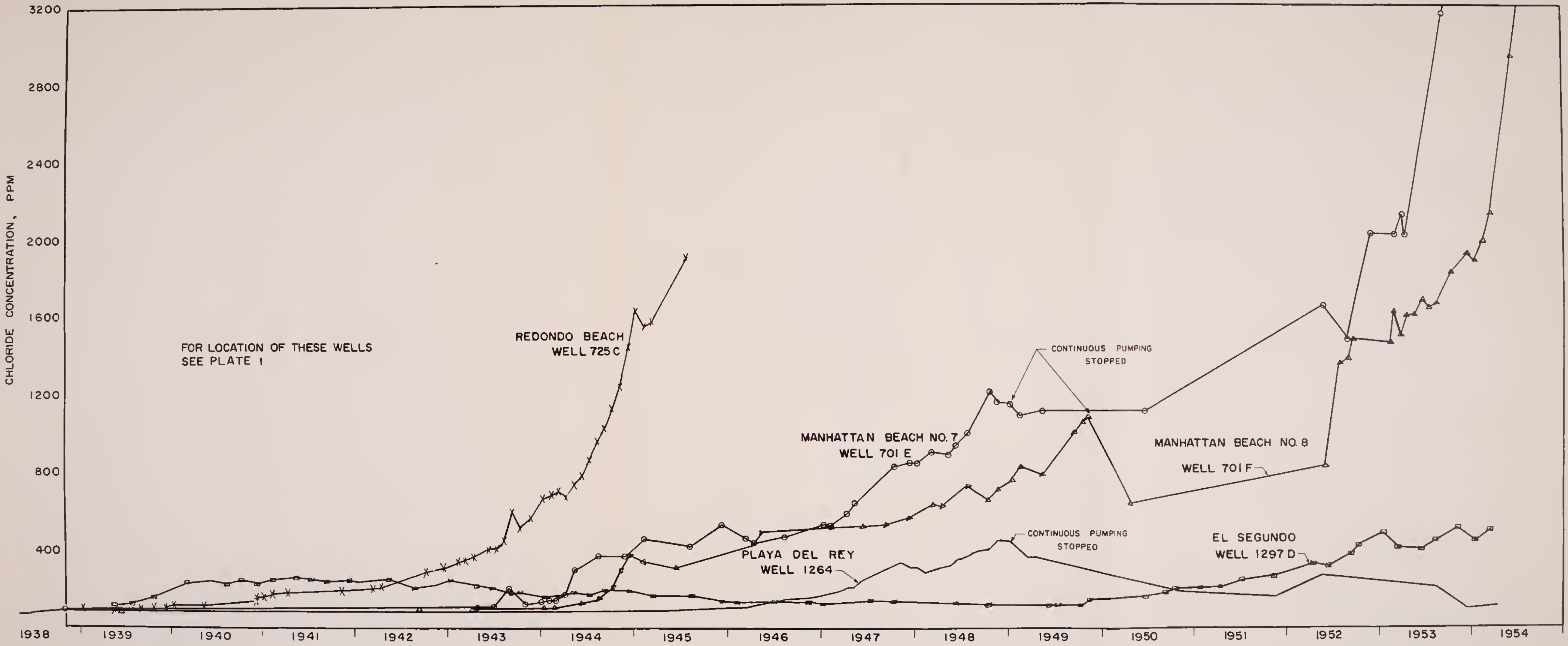


PLATE I
F. C. D. BOUNDARY
N BERNARDINO
FRWY.
NGELES
SANTA
ANA
FRWY.
LAKEWOOD BL.
HWY.
WEST BASIN SALINITY
KEY WELLS
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
PROJECT LOCATION PLAN
MILES
0 2 4 6 8 10

PLATE I

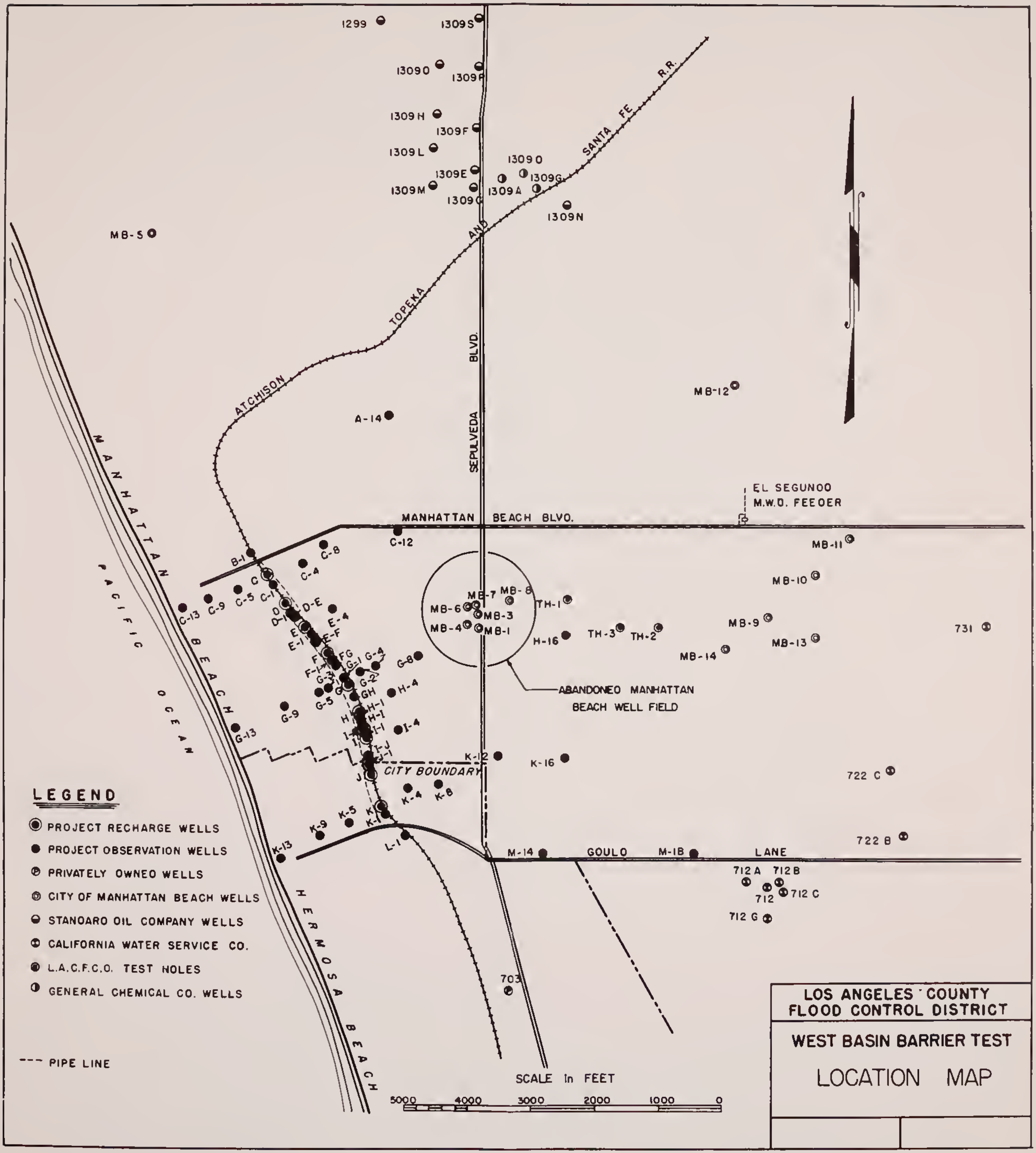

L. A. C. F. C. D. BOUNDARY
VENTURA BLVD.
HOLLYWOOD FRWY.
SAN BERNARDINO FRWY.
LOS ANGELES
SANTA ANA
FRWY.
MANCHESTER BL.
WESTERN AVE
LAKEWOOD BL.
PACIFIC COAST HWY.
WELL 1264
WELL 1297 D
WELLS 701 E & F
WELL 725
N
S
RECHARGE LINE LOCATION (MANHATTAN BEACH)
PACIFIC OCEAN
● WEST BASIN SALINITY KEY WELLS
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
PROJECT LOCATION PLAN
MILES
0 2 4 6 8 10



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CHLORIDE CONCENTRATION OF TYPICAL WELLS POLLUTED
BY SALINE INTRUSION — WEST COAST BASIN
FROM 1938 TO 1954

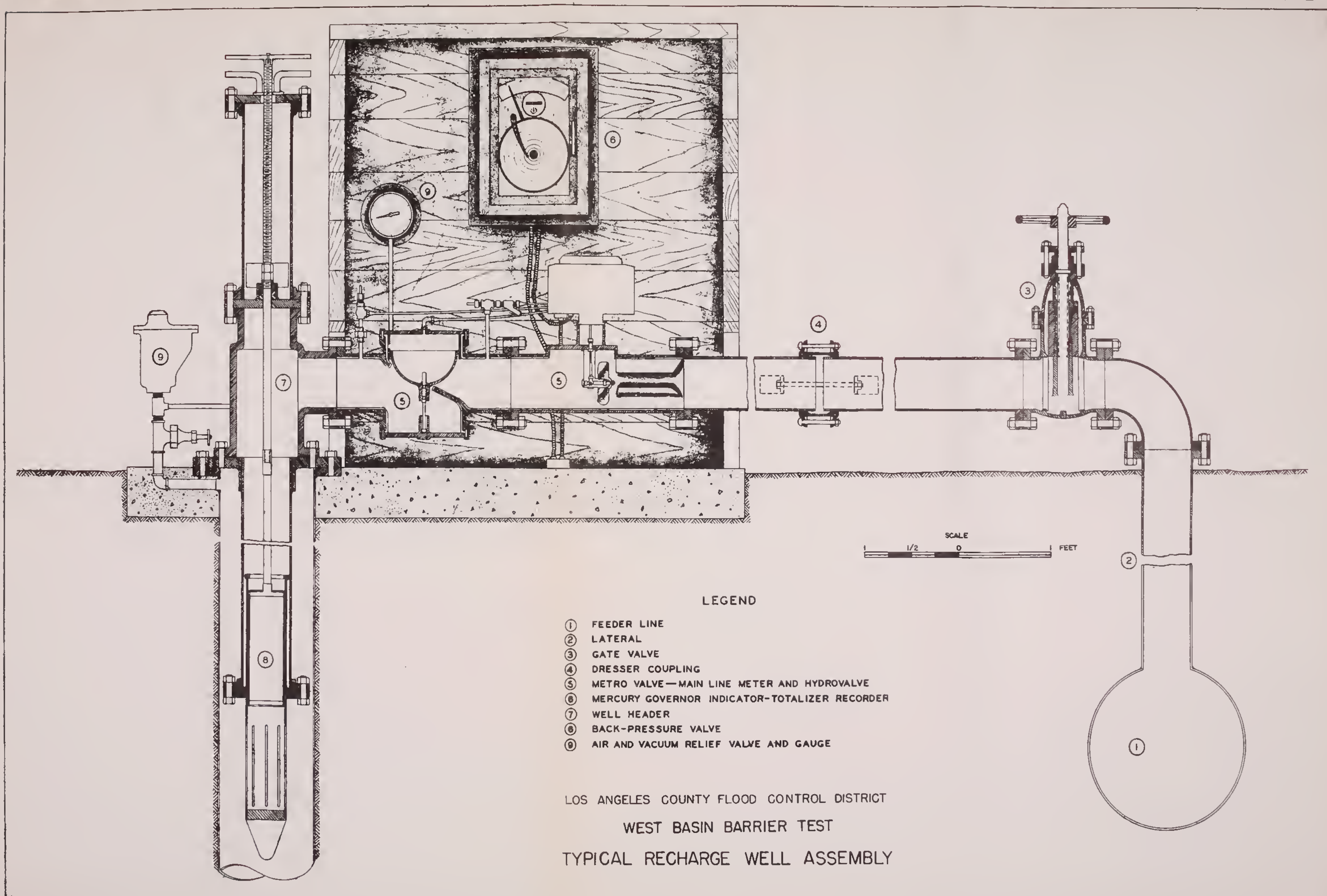


LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

LOCATION MAP

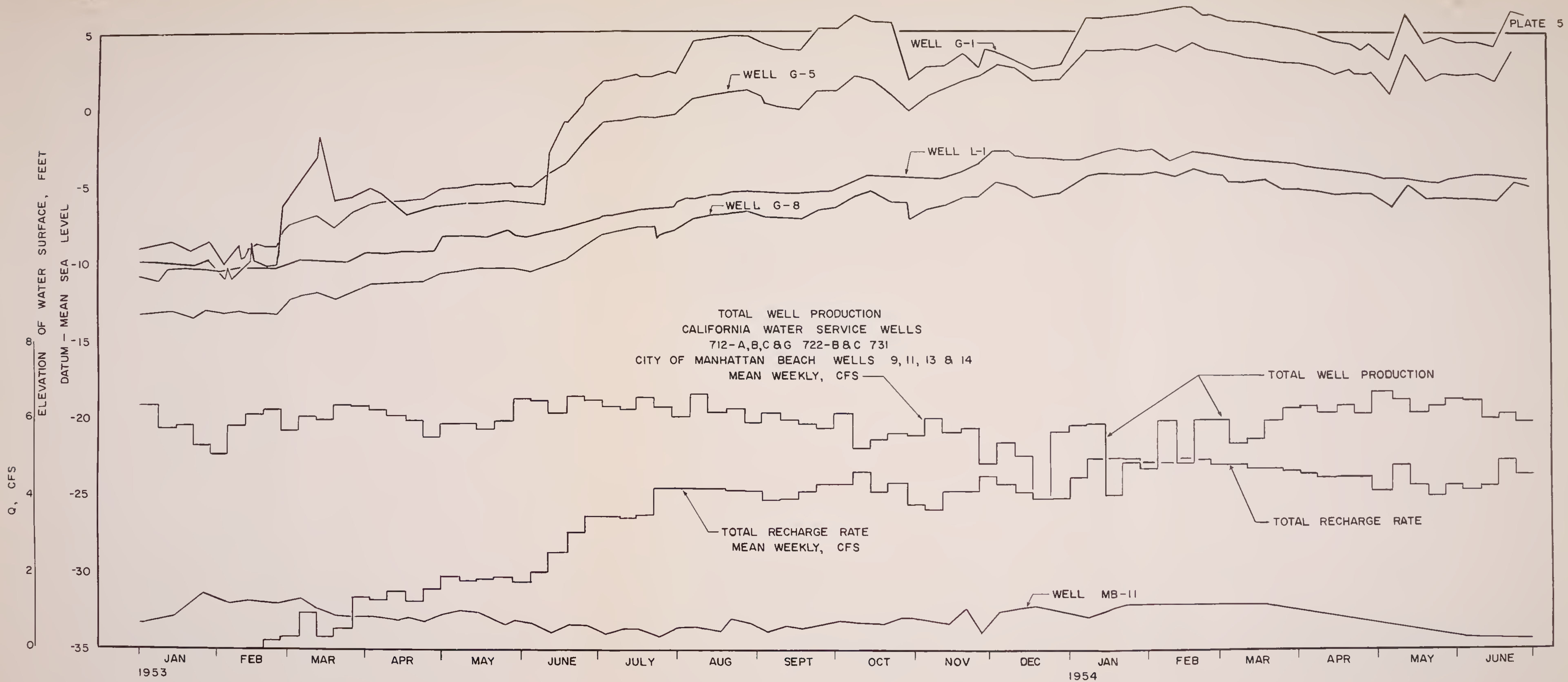
PLATE 4



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

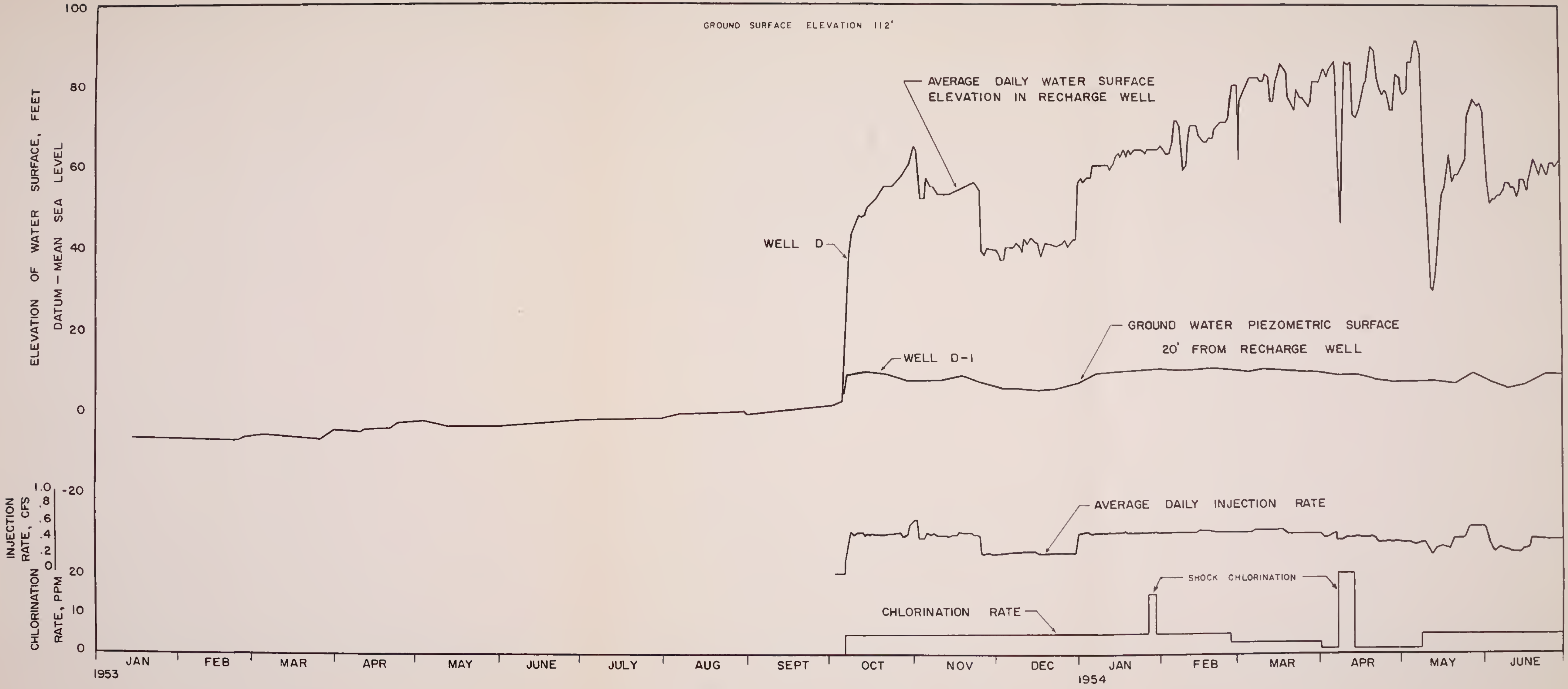
TYPICAL RECHARGE WELL ASSEMBLY



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

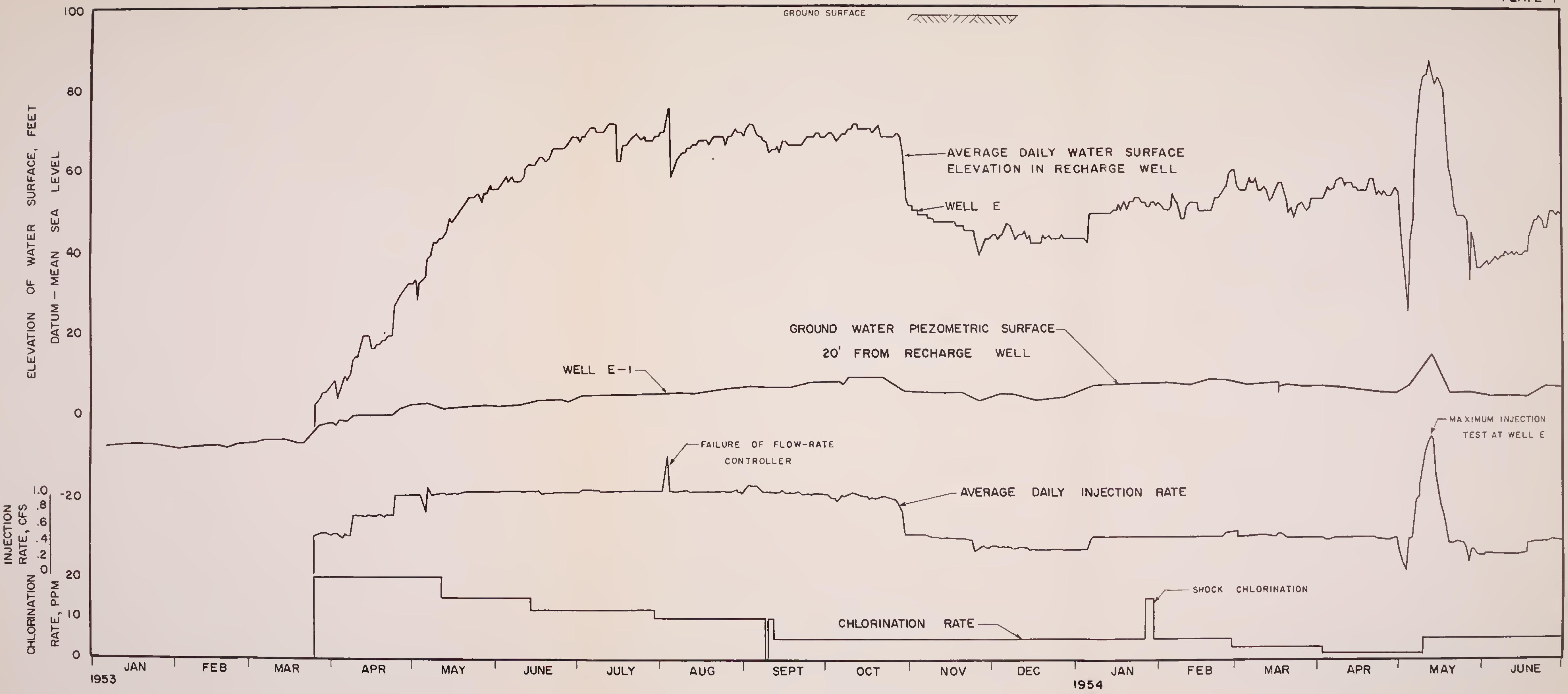
HYDROGRAPHS OF PROJECT WELLS,
RATE OF PUMPING IN IMMEDIATE VICINITY OF PROJECT, AND
RATE OF PROJECT RECHARGE



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

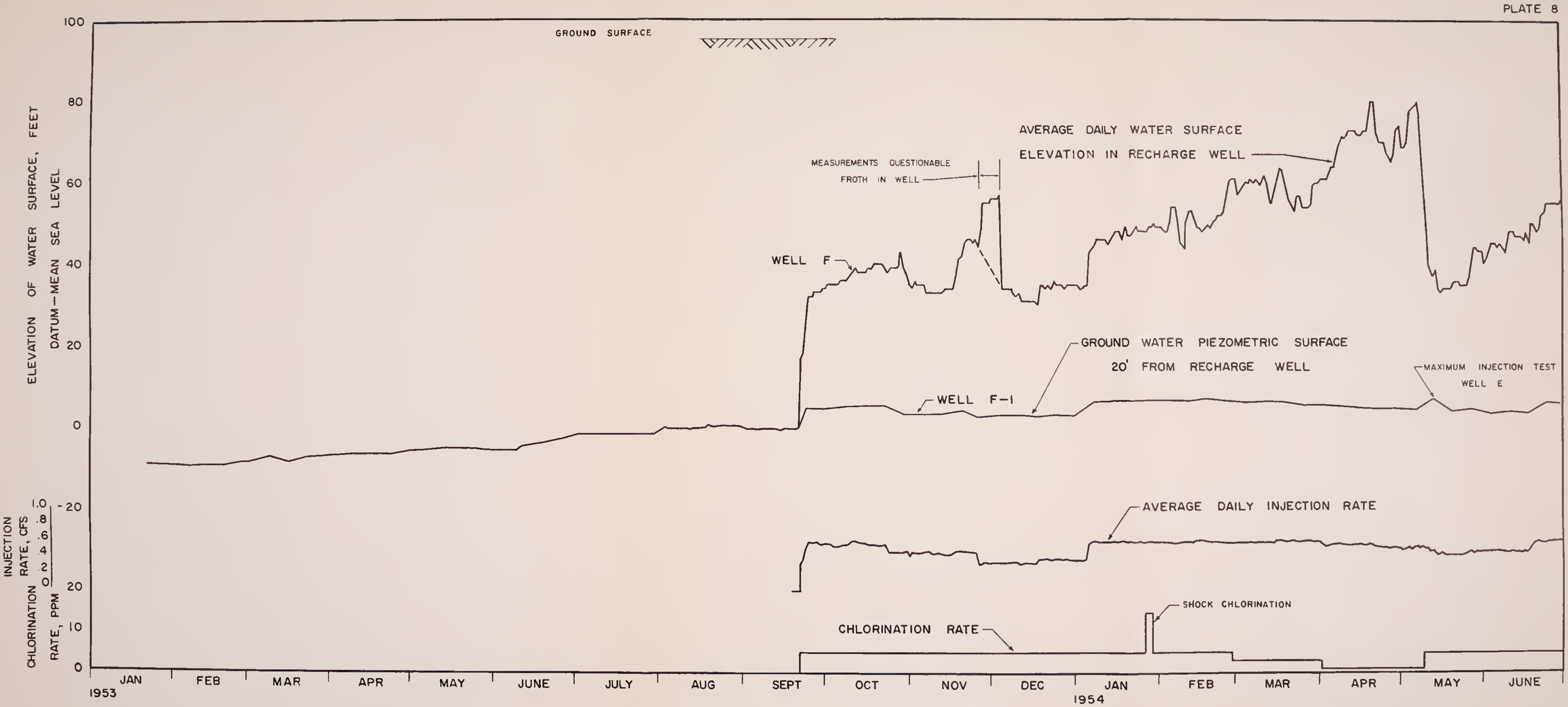
HISTORY OF OPERATIONS AT RECHARGE WELL D



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

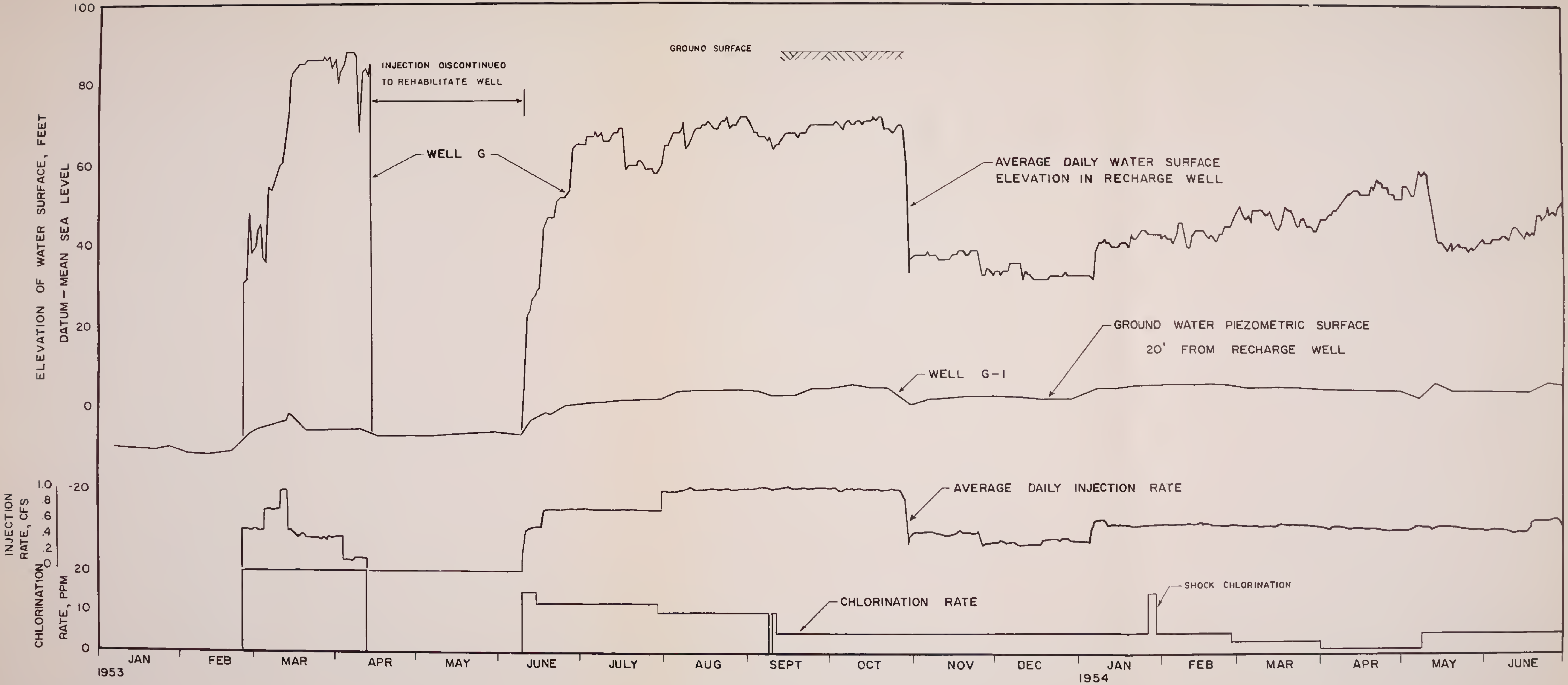
HISTORY OF OPERATIONS AT RECHARGE WELL E



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

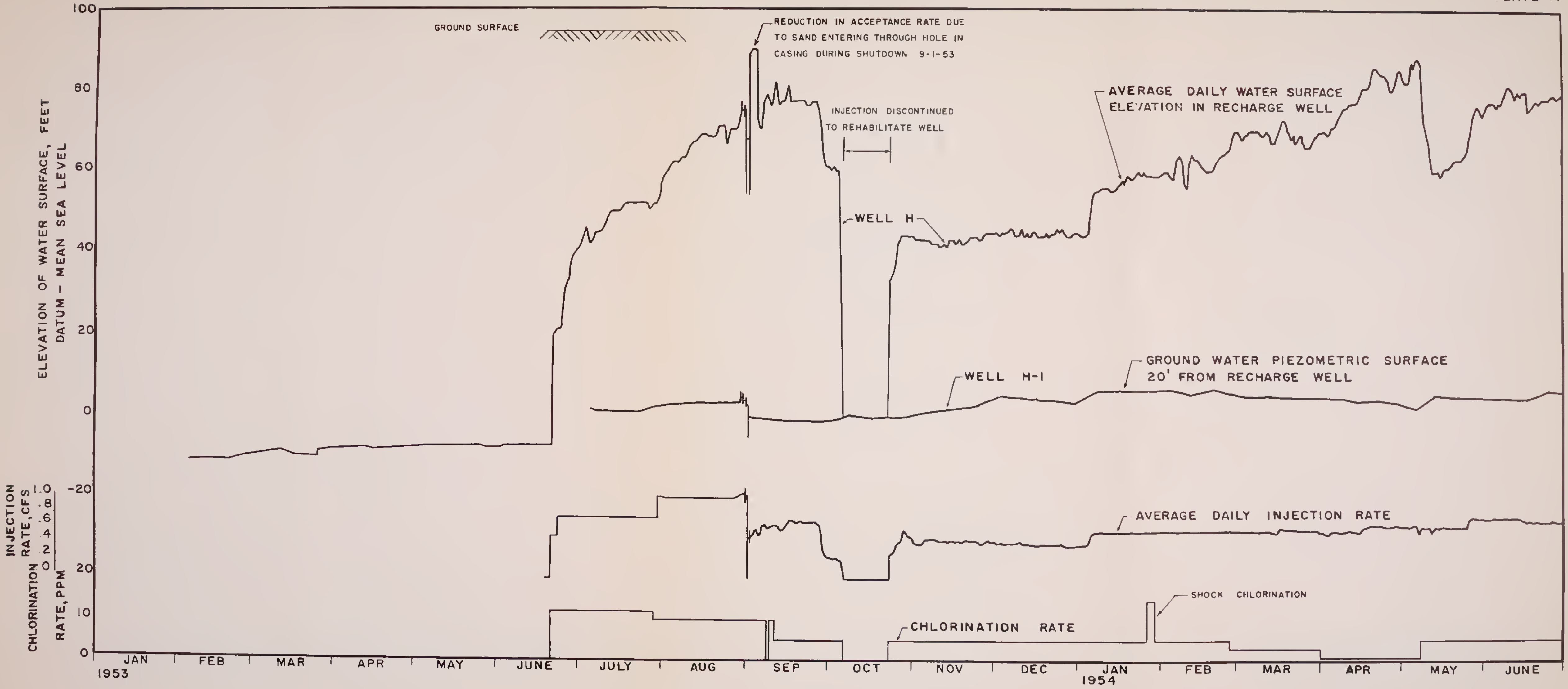
HISTORY OF OPERATIONS AT RECHARGE WELL F



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

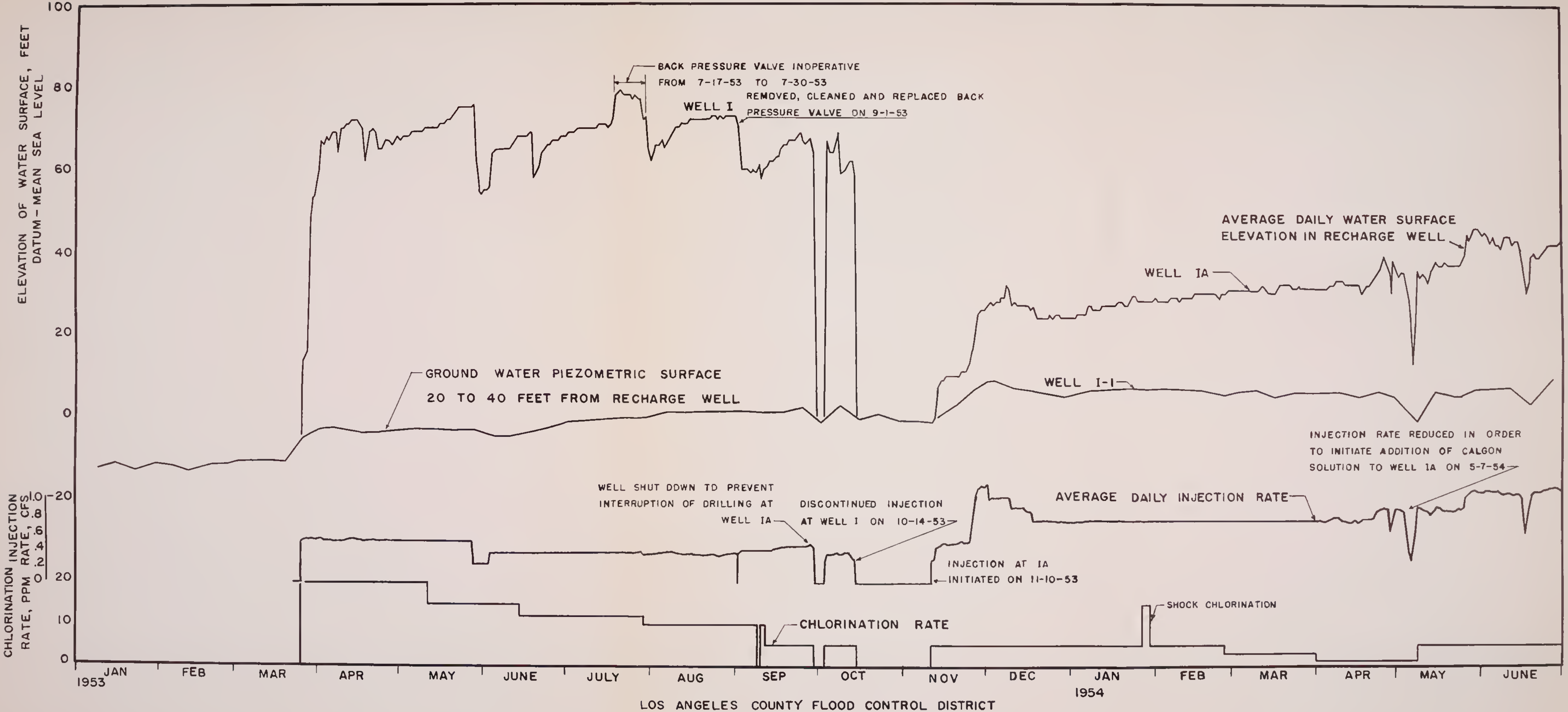
HISTORY OF OPERATIONS AT RECHARGE WELL G



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

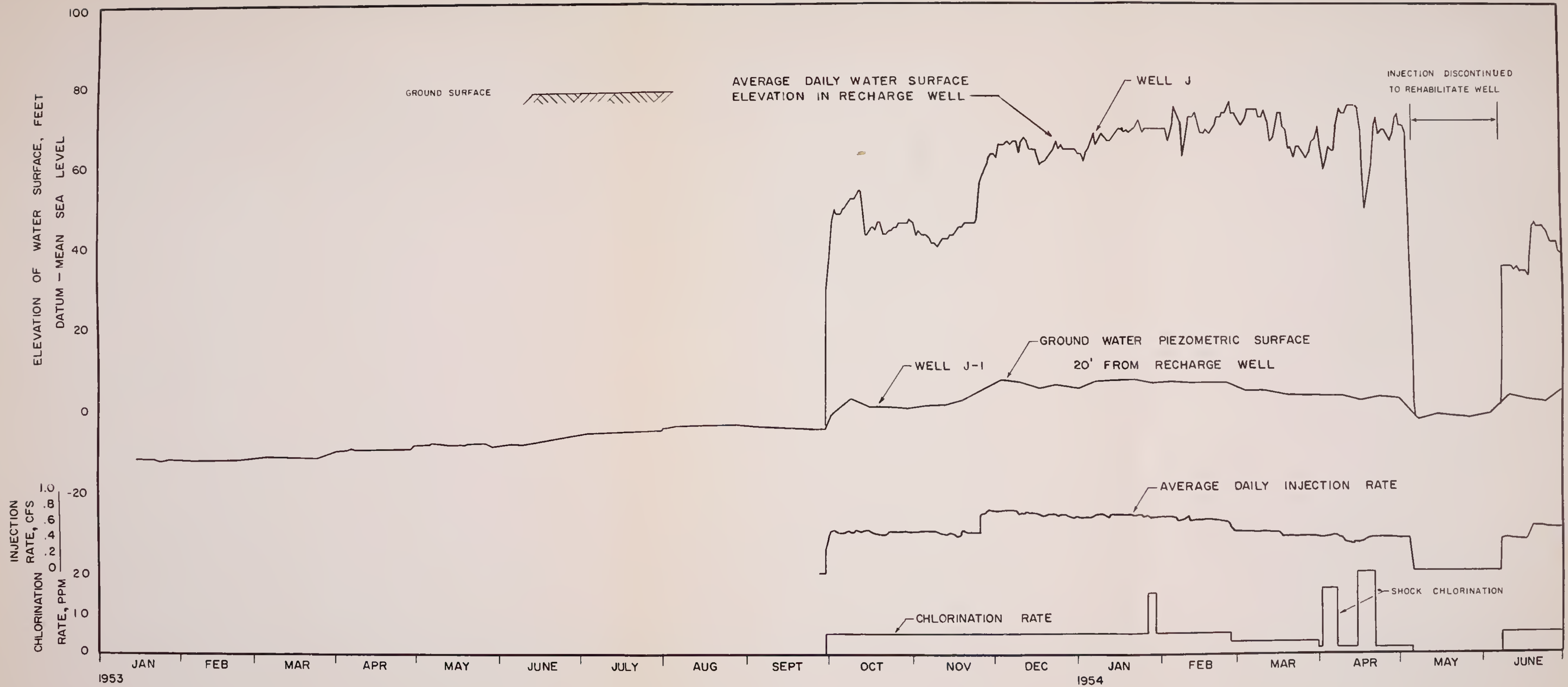
HISTORY OF OPERATIONS AT WELL H



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

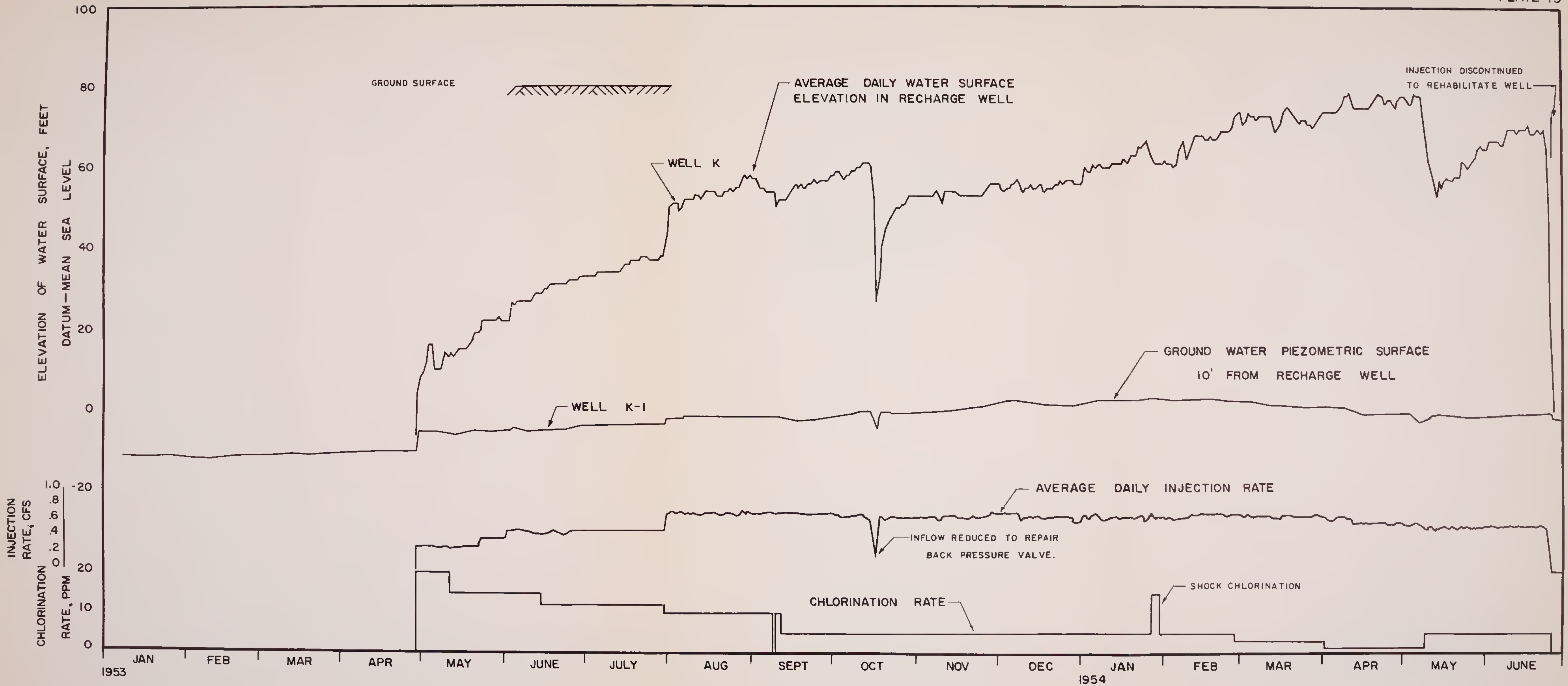
HISTORY OF OPERATIONS AT RECHARGE WELLS I & IA



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

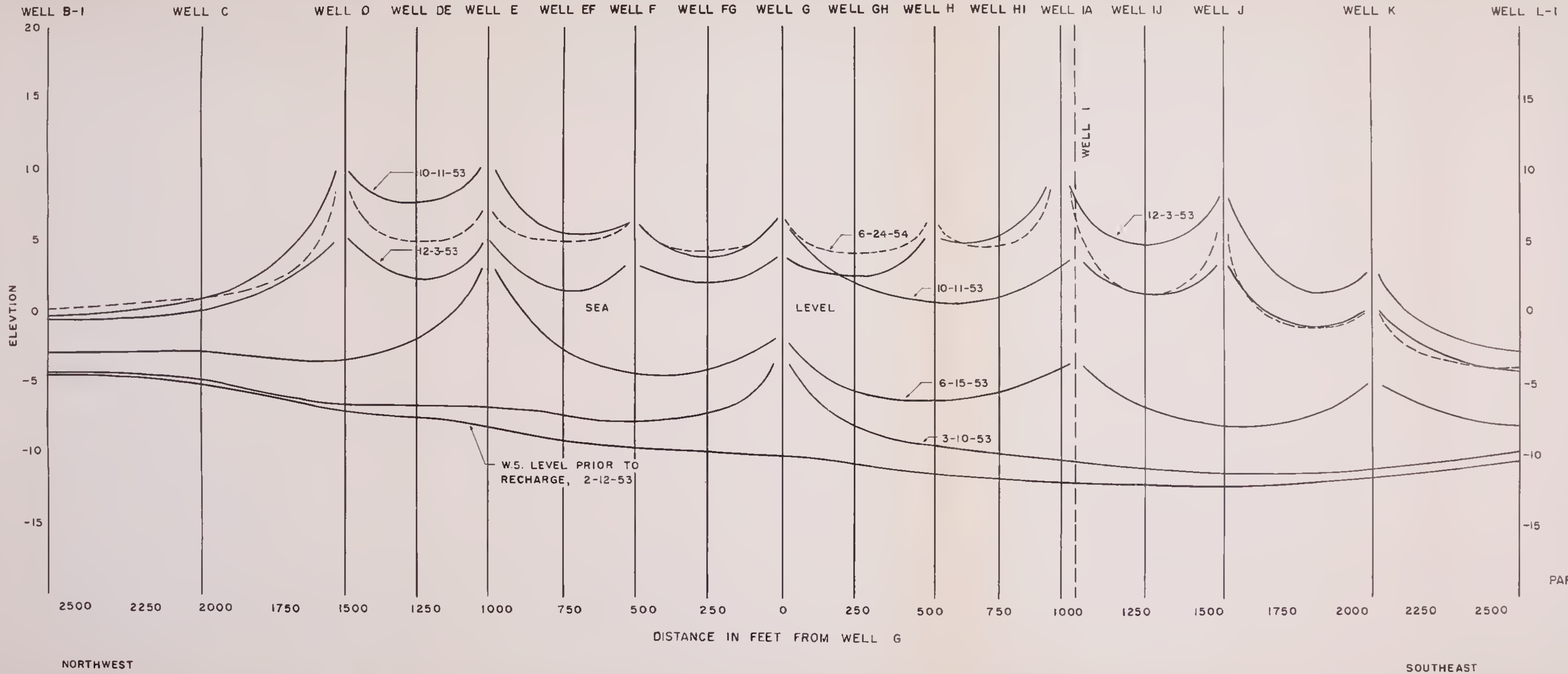
HISTORY OF OPERATIONS AT RECHARGE WELL J



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

HISTORY OF OPERATIONS AT RECHARGE WELL K



INJECTION RATES, C.F.S.										
DATE	WELLS									
	C	D	E	F	G	H	IA	J	K	TOTAL
2-12-53	0	0	0	0	0	0	0*	0	0	0
3-10-53	0	0	0	0	0.75	0	0*	0	0	0.75
6-15-53	0	0	1.05	0	0.54	0	0.36*	0	0.48	2.43
10-11-53	0	0.50	1.04	0.63	1.05	0	0.37*	0.51	0.74	4.84
12-3-53	0	0.24	0.39	0.37	0.39	0.47	1.03	0.78	0.74	4.41
6-24-54	0	0.41	0.45	0.59	0.64	0.70	1.13	0.55	0.30	4.77

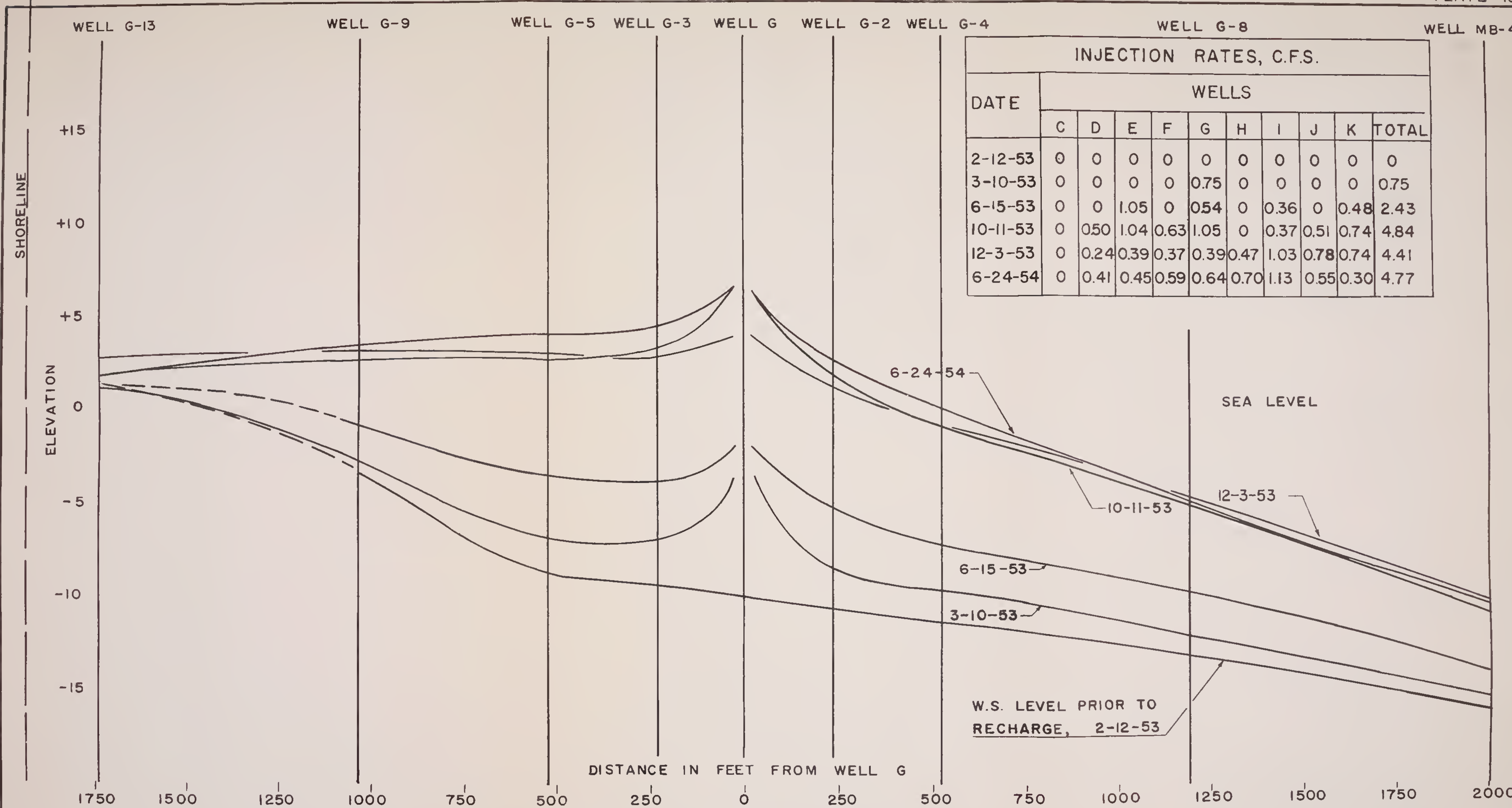
* WELL I

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

GROUND WATER PROFILES

PARALLEL TO COAST THROUGH LINE OF RECHARGE



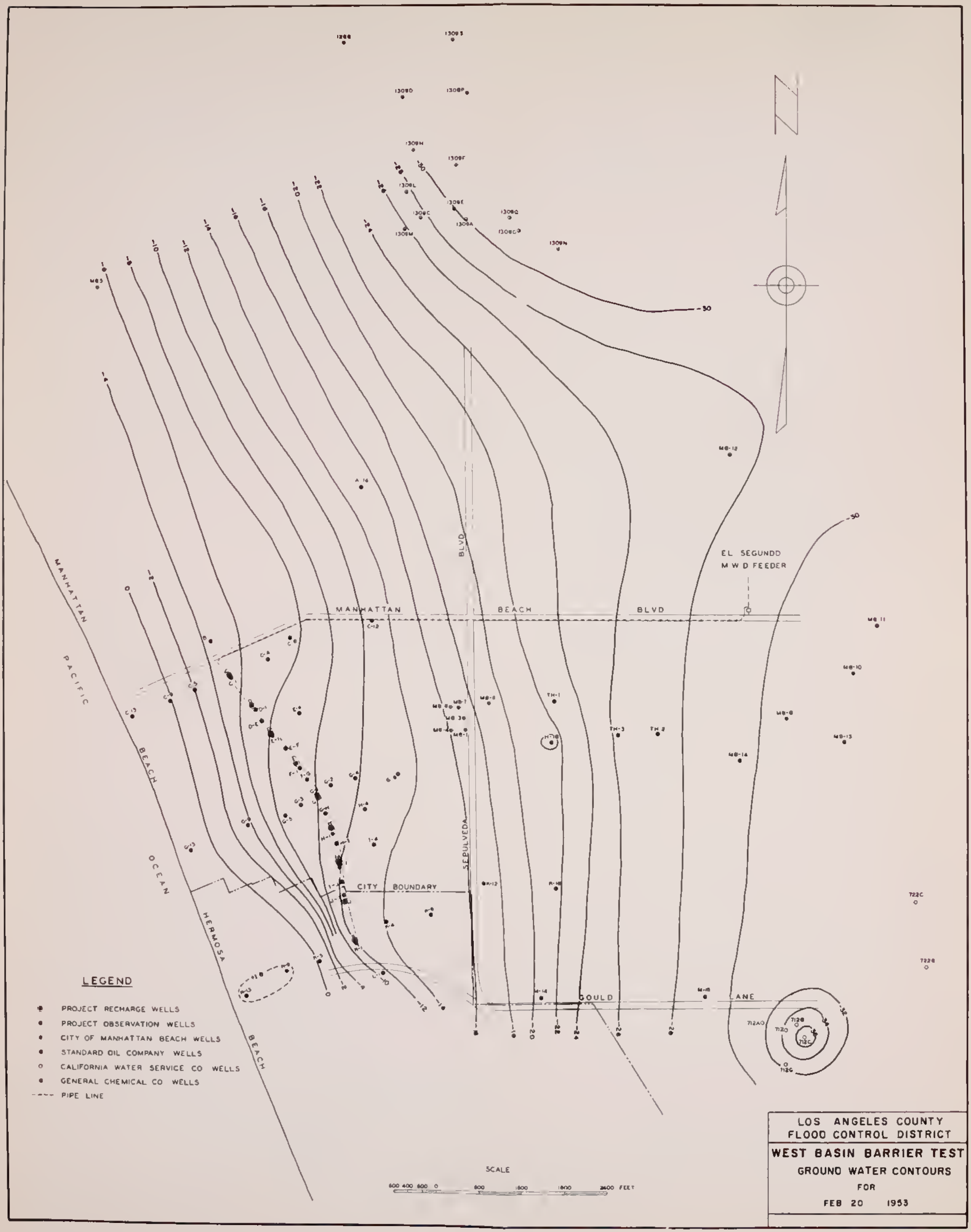
INJECTION RATES, C.F.S.										
DATE	WELLS									
	C	D	E	F	G	H	I	J	K	TOTAL
2-12-53	0	0	0	0	0	0	0	0	0	0
3-10-53	0	0	0	0	0.75	0	0	0	0	0.75
6-15-53	0	0	1.05	0	0.54	0	0.36	0	0.48	2.43
10-11-53	0	0.50	1.04	0.63	1.05	0	0.37	0.51	0.74	4.84
12-3-53	0	0.24	0.39	0.37	0.39	0.47	1.03	0.78	0.74	4.41
6-24-54	0	0.41	0.45	0.59	0.64	0.70	1.13	0.55	0.30	4.77

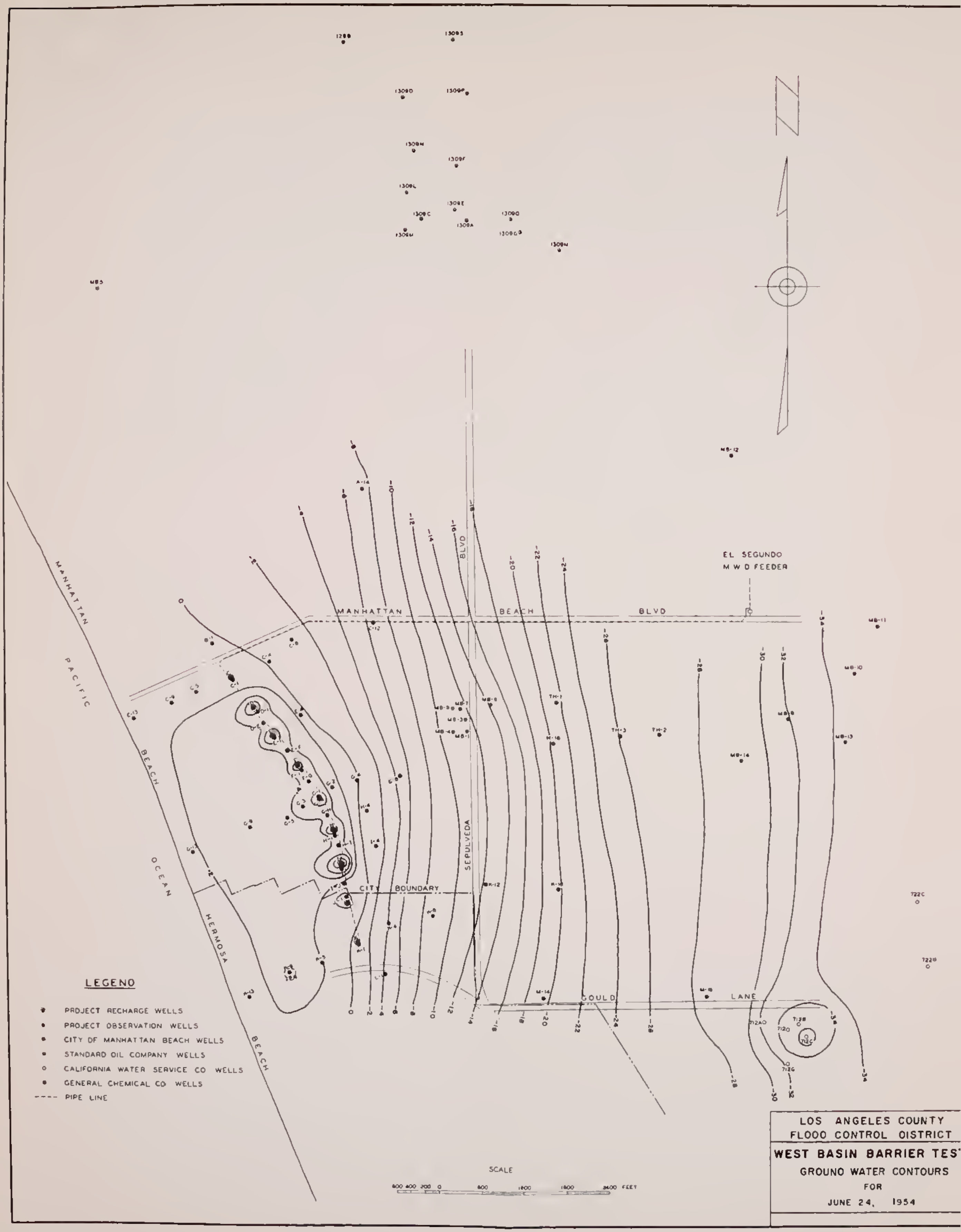
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

GROUND WATER PROFILES

NORMAL TO LINE OF RECHARGE THROUGH WELL G

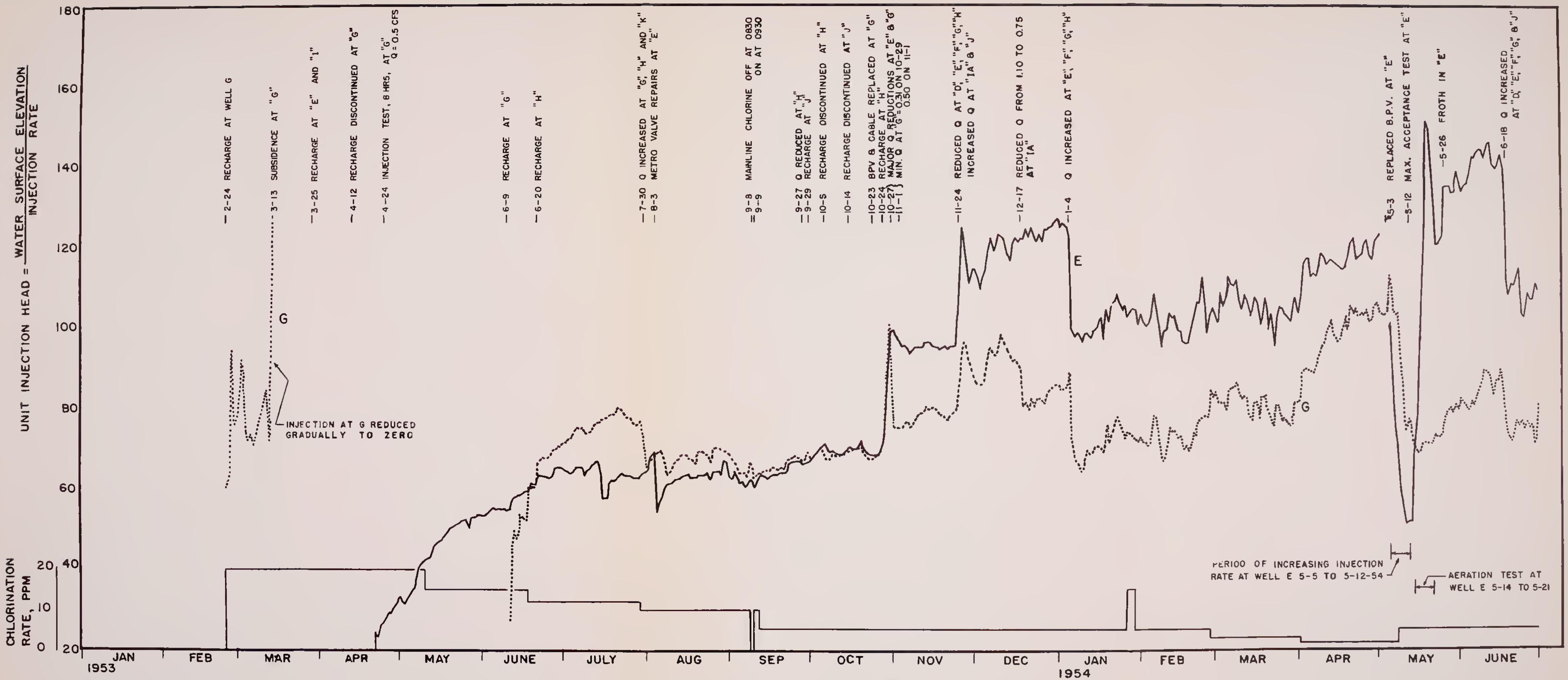

LEGEND
PROJECT RECHARGE WELLS
PROJECT OBSERVATION WELLS
CITY OF MANHATTAN BEACH WELLS
STANDARD OIL COMPANY WELLS
CALIFORNIA WATER SERVICE CO WELLS
GENERAL CHEMICAL CO WELLS
PIPE LINE
MANHATTAN BEACH BLVD
SEPULVEDA BLVD
GOULD LANE
CITY BOUNDARY
EL SEGUNDO
M W D FEEDER
PACIFIC OCEAN
MANHATTAN BEACH
HERMOSA BEACH
SCALE
FEET
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
GROUND WATER CONTOURS
FOR
FEB 20 1953



LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

GROUND WATER CONTOURS
FOR
JUNE 24, 1954

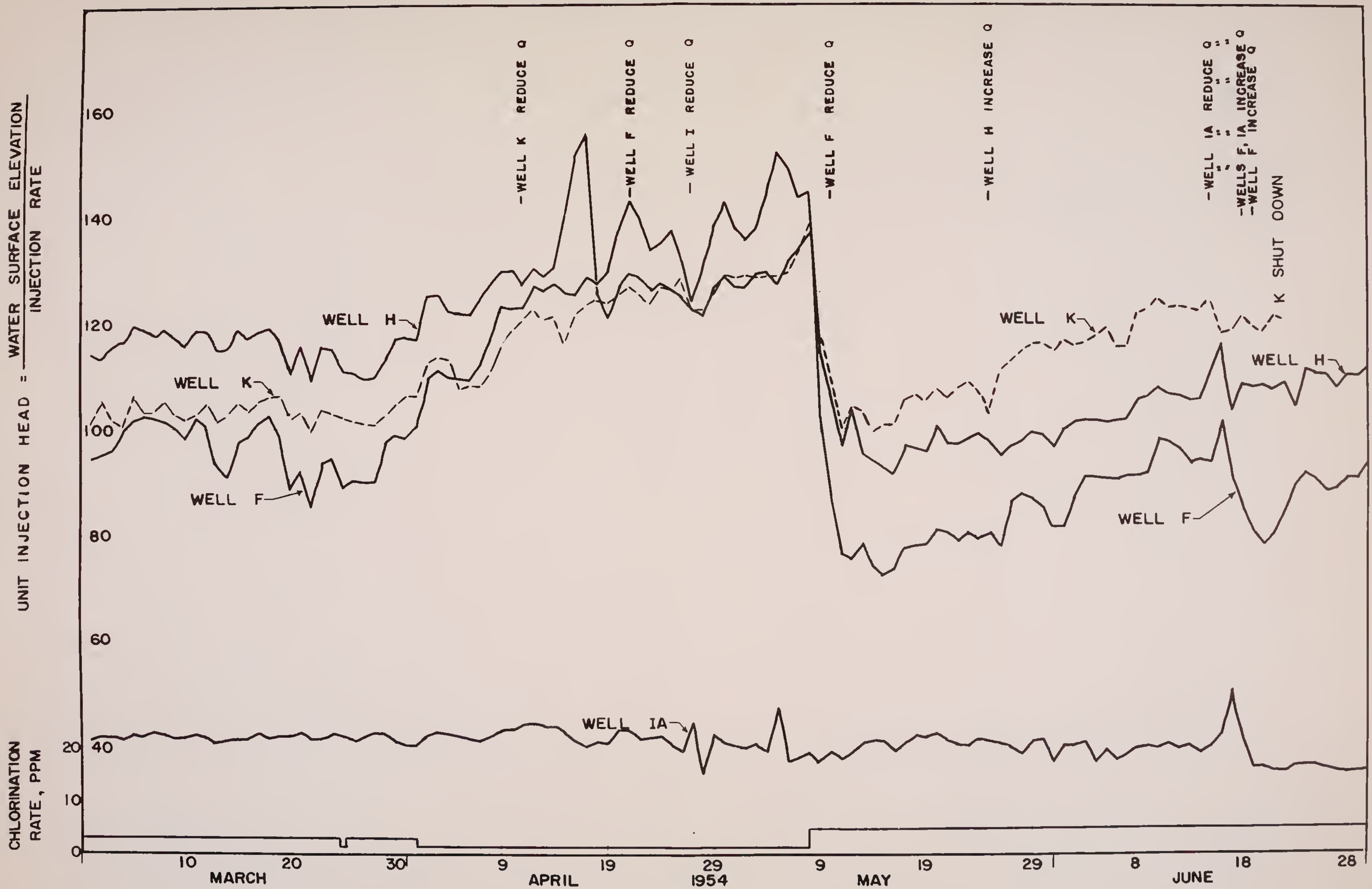


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

UNIT RECHARGE WELL INJECTION HEAD FOR WELLS "E" & "G"

PLATE 19

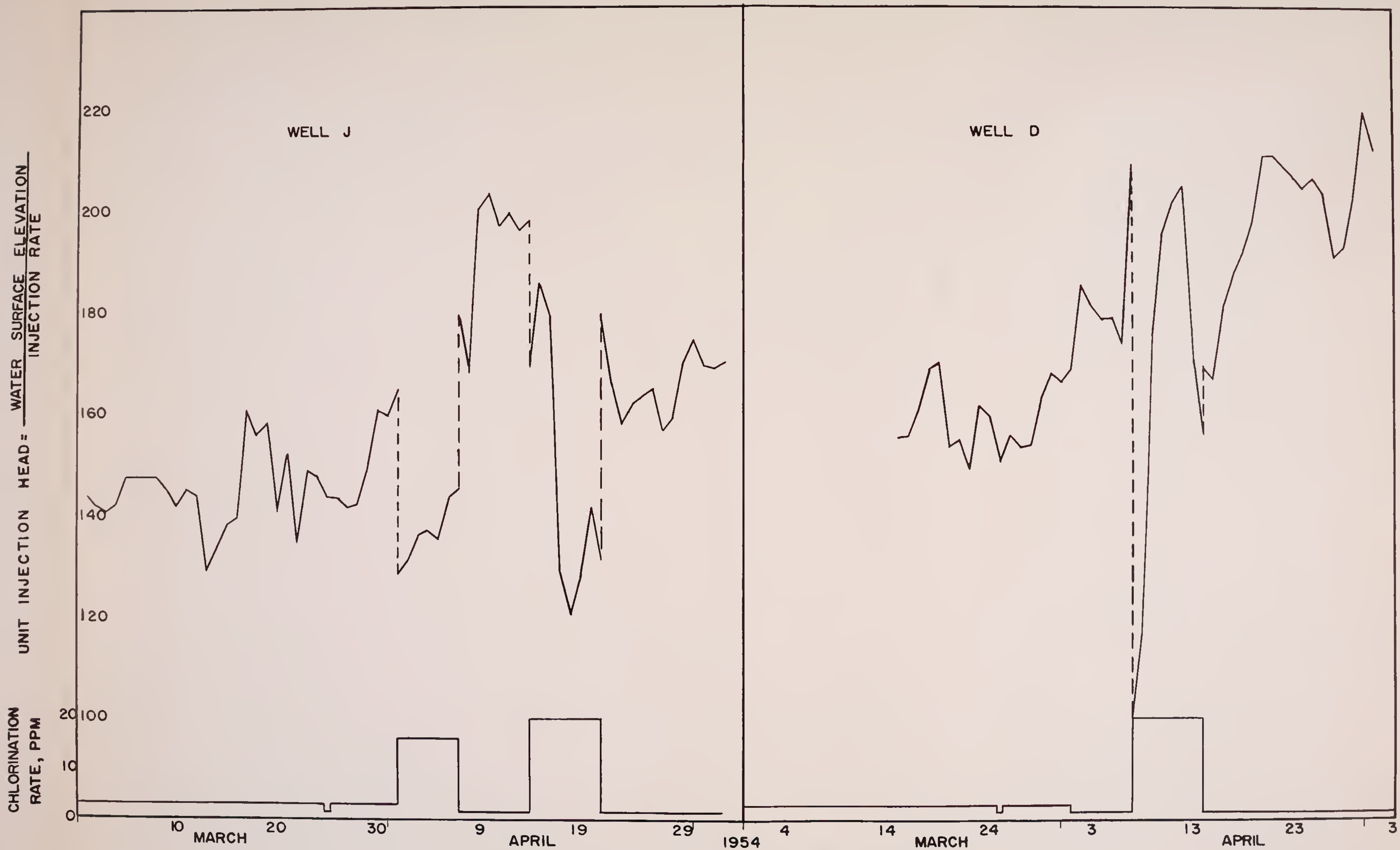


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

UNIT RECHARGE WELL INJECTION HEAD FOR

WELLS F, H, IA, AND K

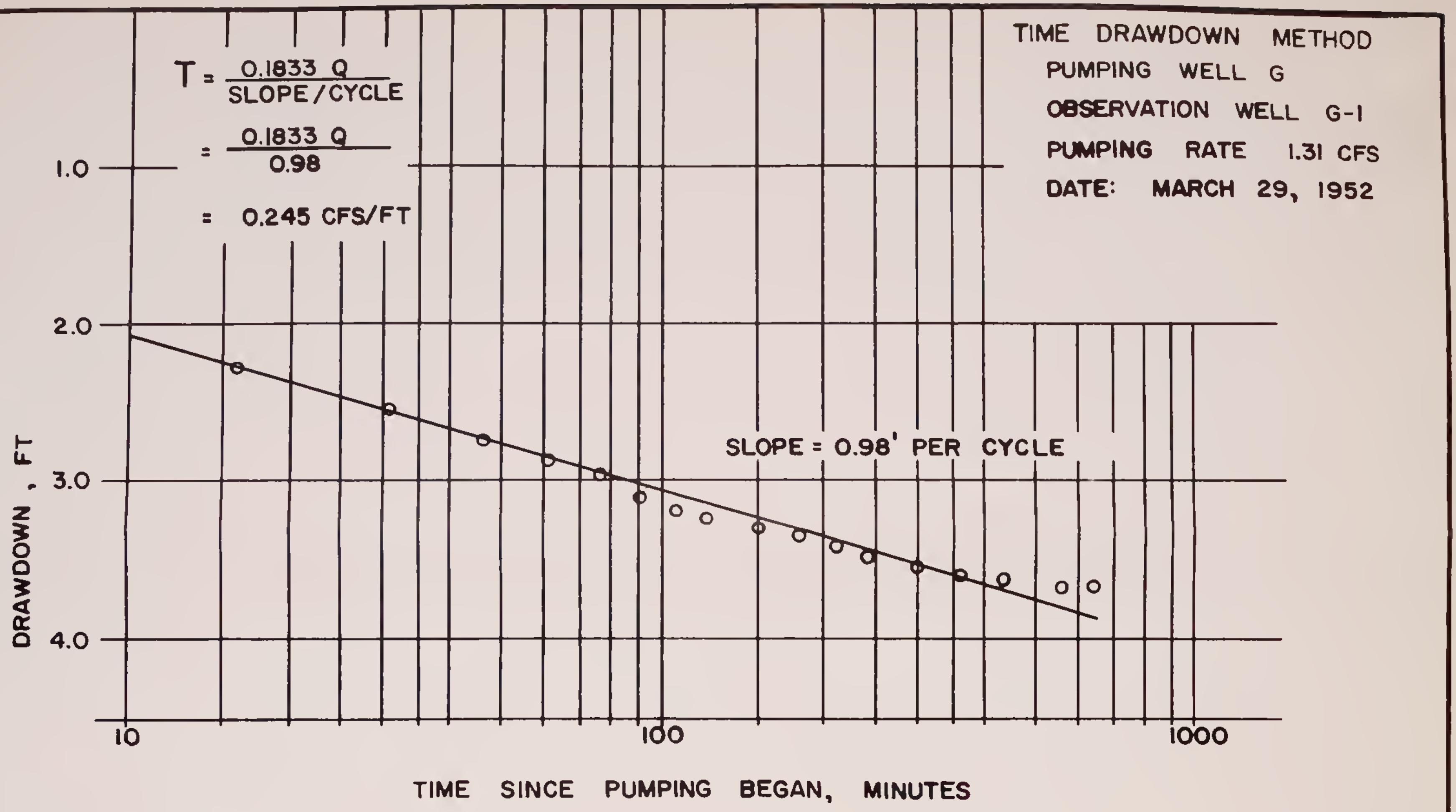


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

UNIT RECHARGE WELL INJECTION HEAD FOR WELLS "J" AND "D"

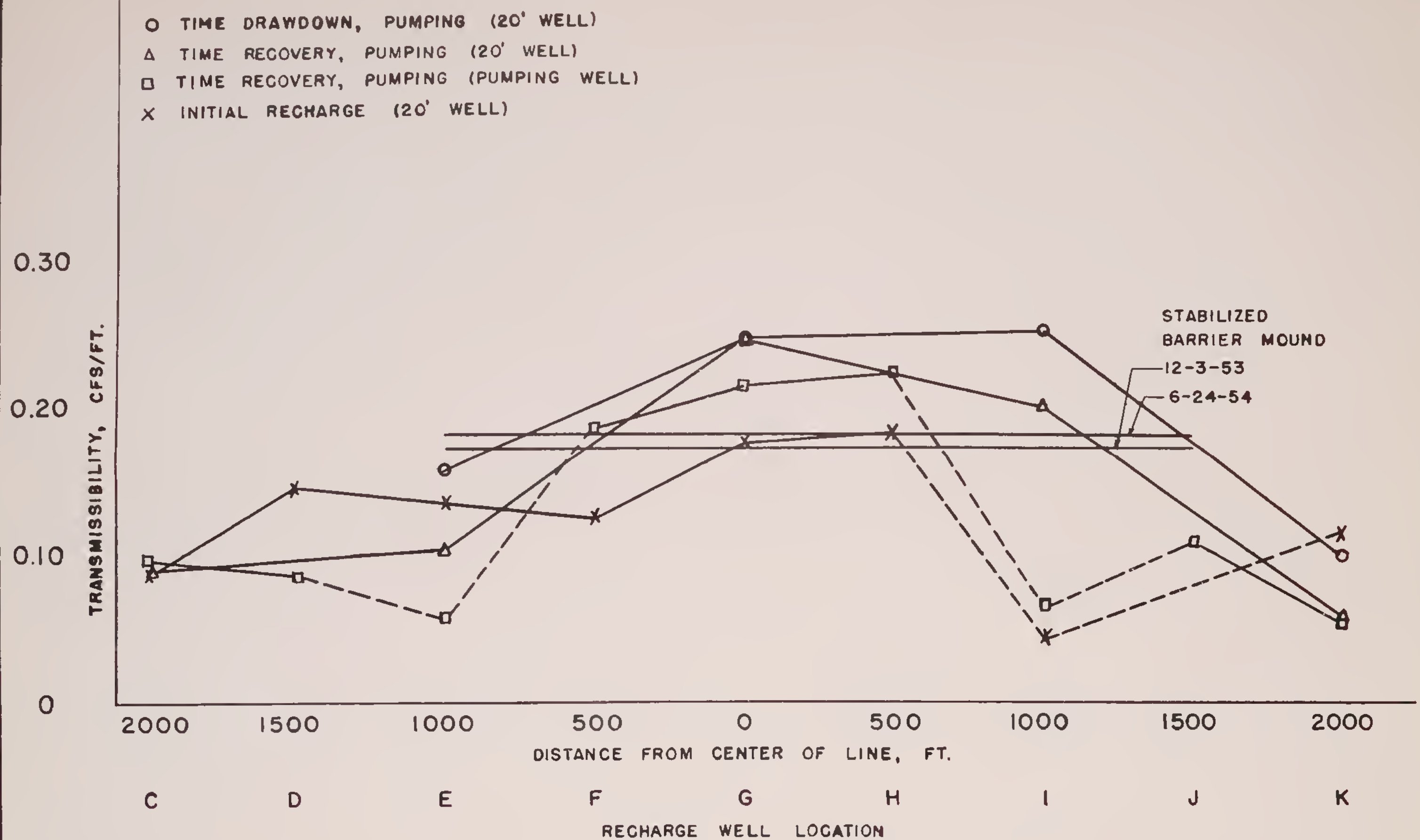
EFFECT OF SHOCK CHLORINATION



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

TYPICAL TRANSMISSIBILITY PUMPING TEST



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

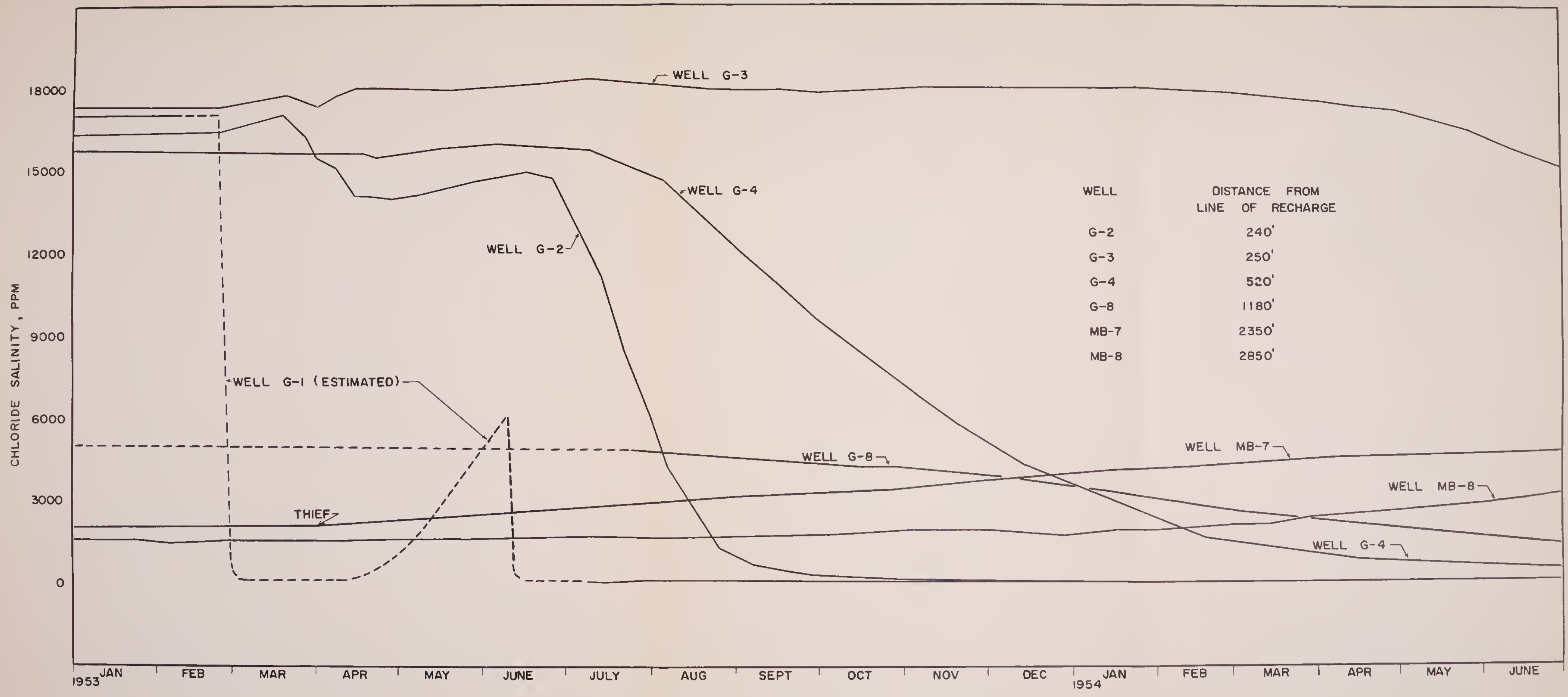
TRANSMISSIBILITY VALUES ALONG THE RECHARGE LINE

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CHLORIDE CONCENTRATION HISTORY OF PROJECT OBSERVATION WELLS

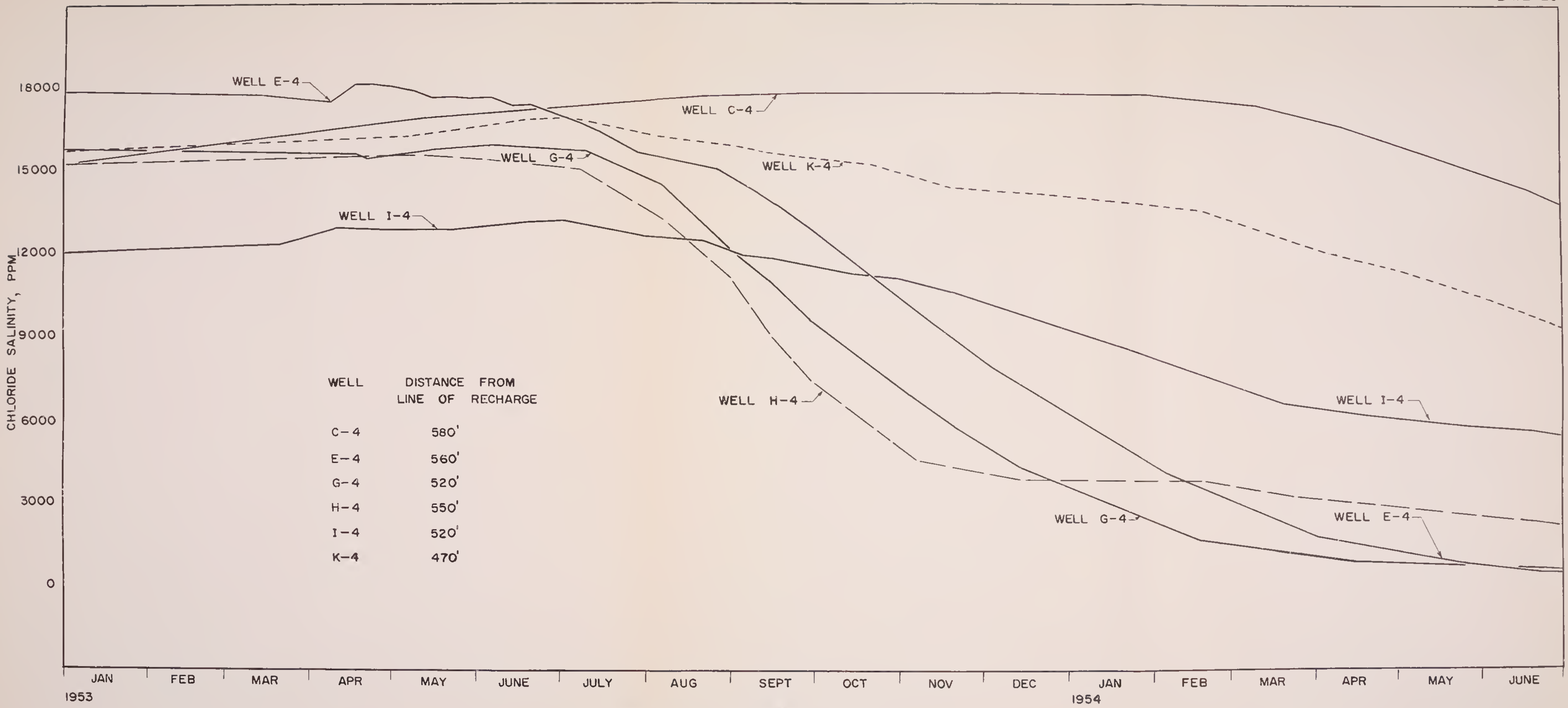
INTERNODAL WELLS OF LINE OF RECHARGE



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

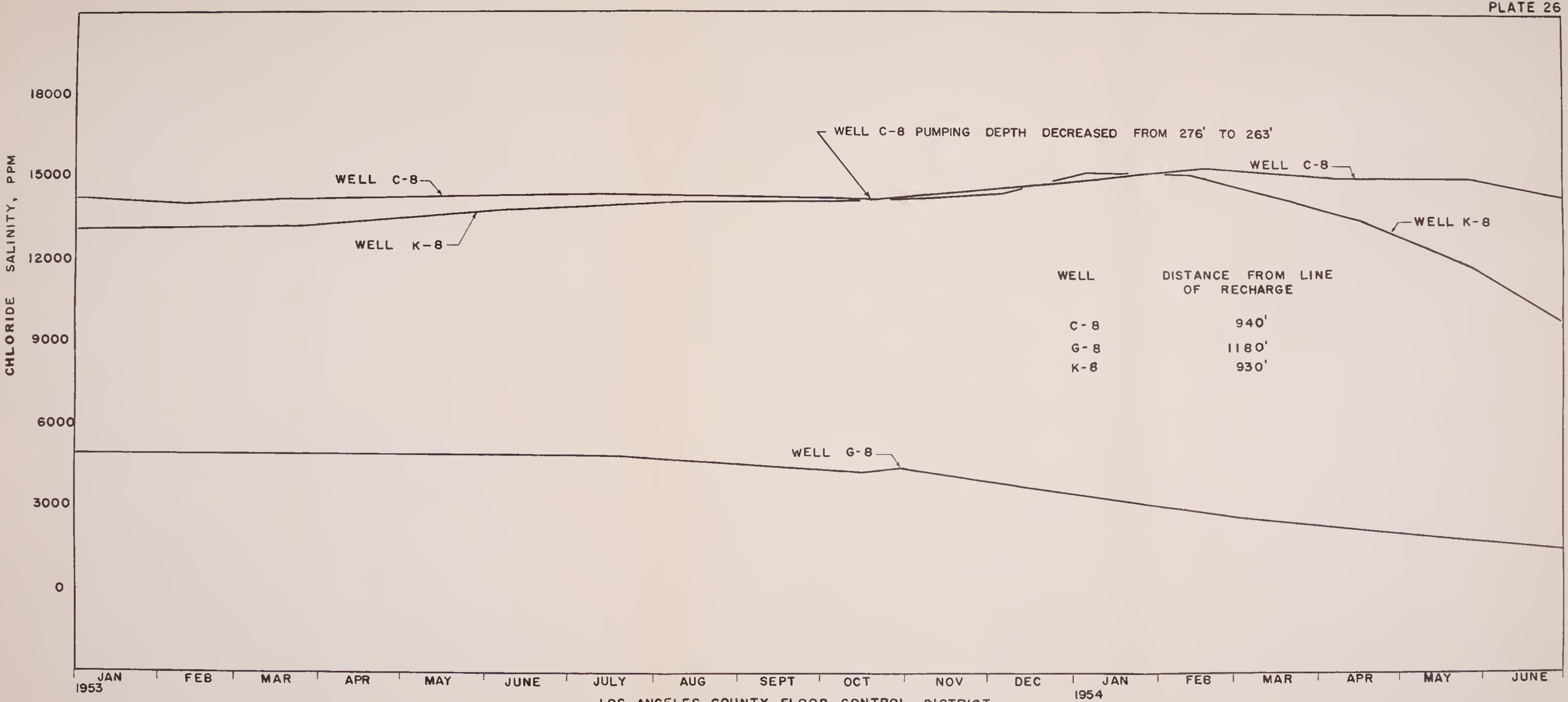
CHLORIDE CONCENTRATION HISTORY OF PROJECT OBSERVATION WELLS NORMAL TO CENTER OF LINE OF RECHARGE



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CHLORIDE CONCENTRATION HISTORY OF PROJECT OBSERVATION WELLS
APPROXIMATELY 500 FEET LANDWARD OF LINE OF RECHARGE

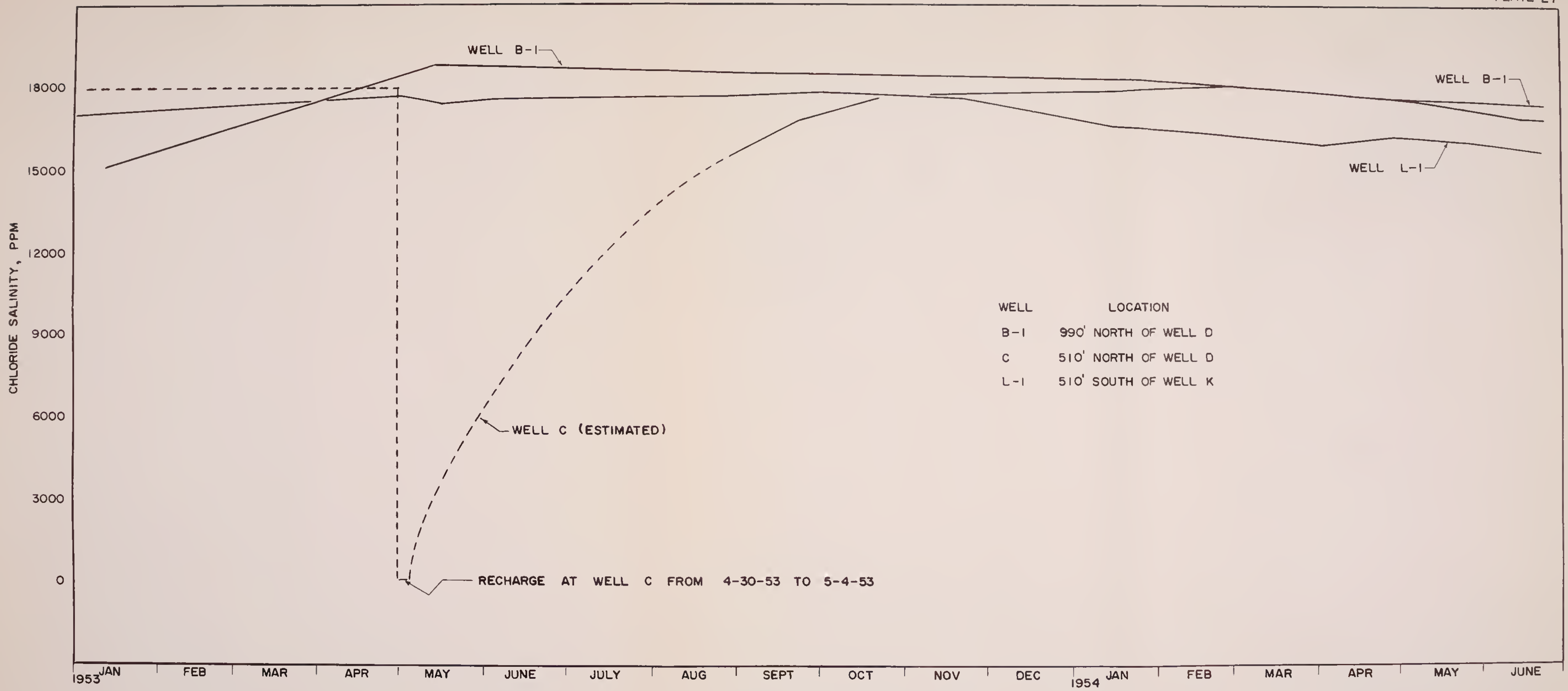


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CHLORIDE CONCENTRATION HISTORY OF PROJECT OBSERVATION WELLS

APPROXIMATELY 1000 FEET LANDWARD OF LINE OF RECHARGE

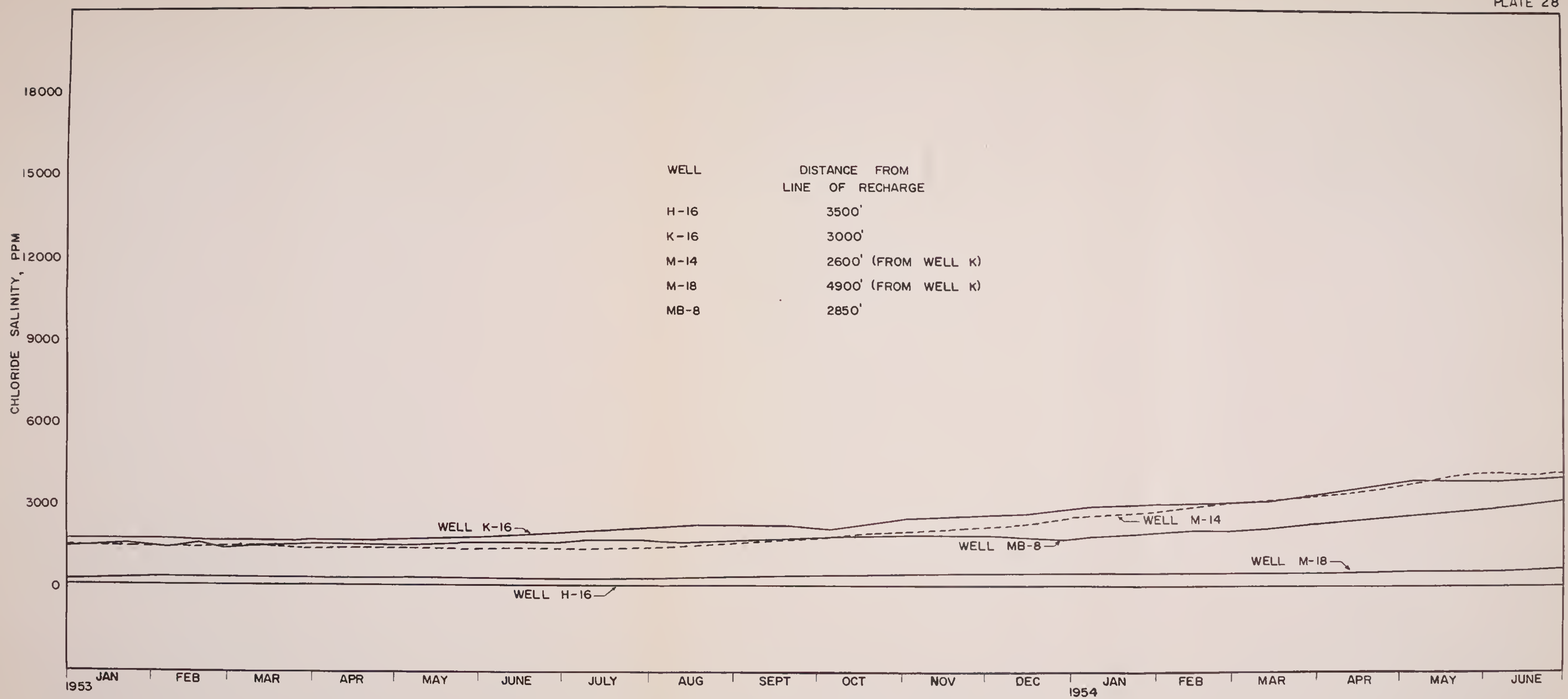


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CHLORIDE CONCENTRATION HISTORY OF PROJECT OBSERVATION WELLS

LATERAL WELLS NORTH AND SOUTH OF RECHARGE LINE

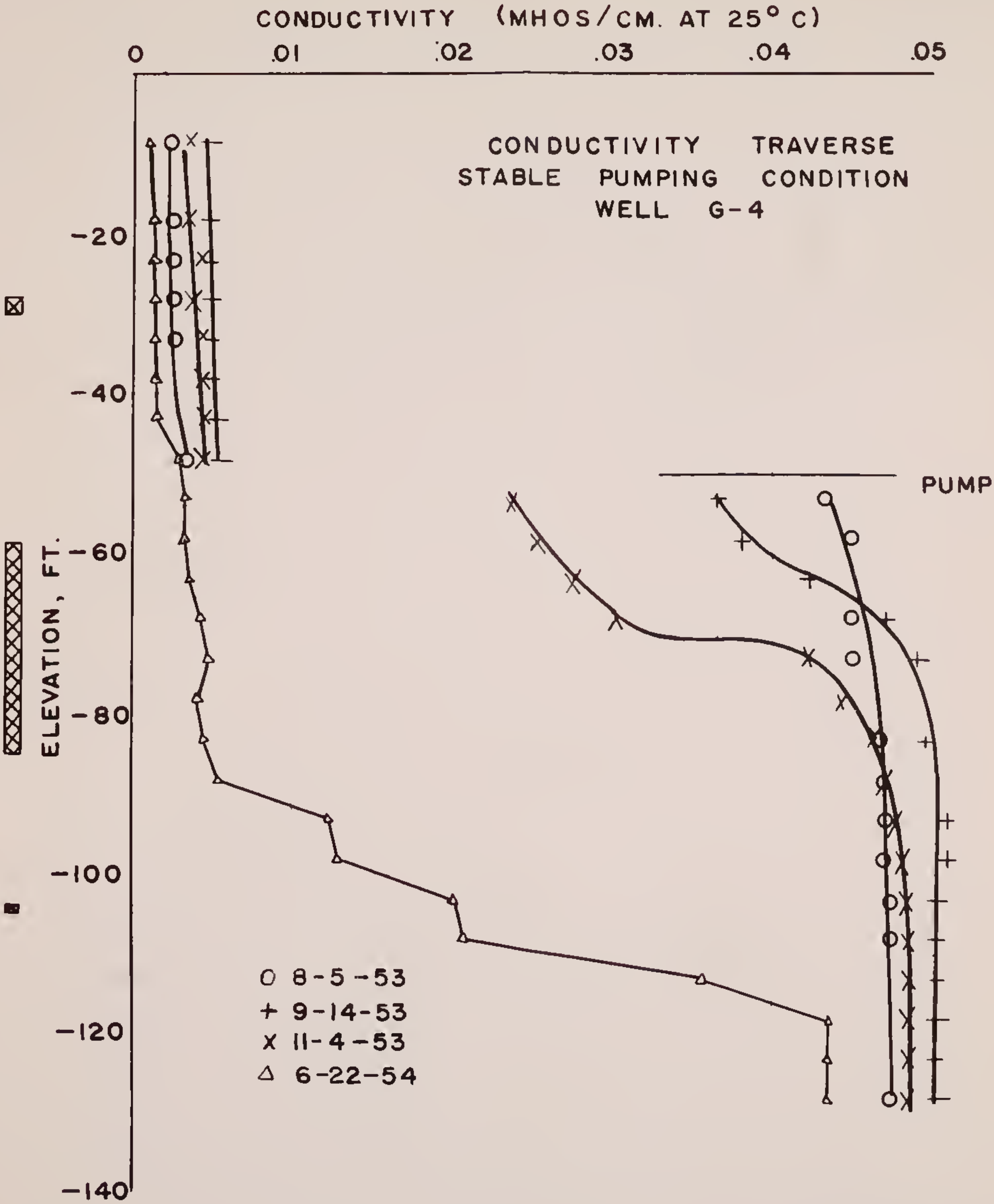


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CHLORIDE CONCENTRATION HISTORY OF PROJECT OBSERVATION WELLS

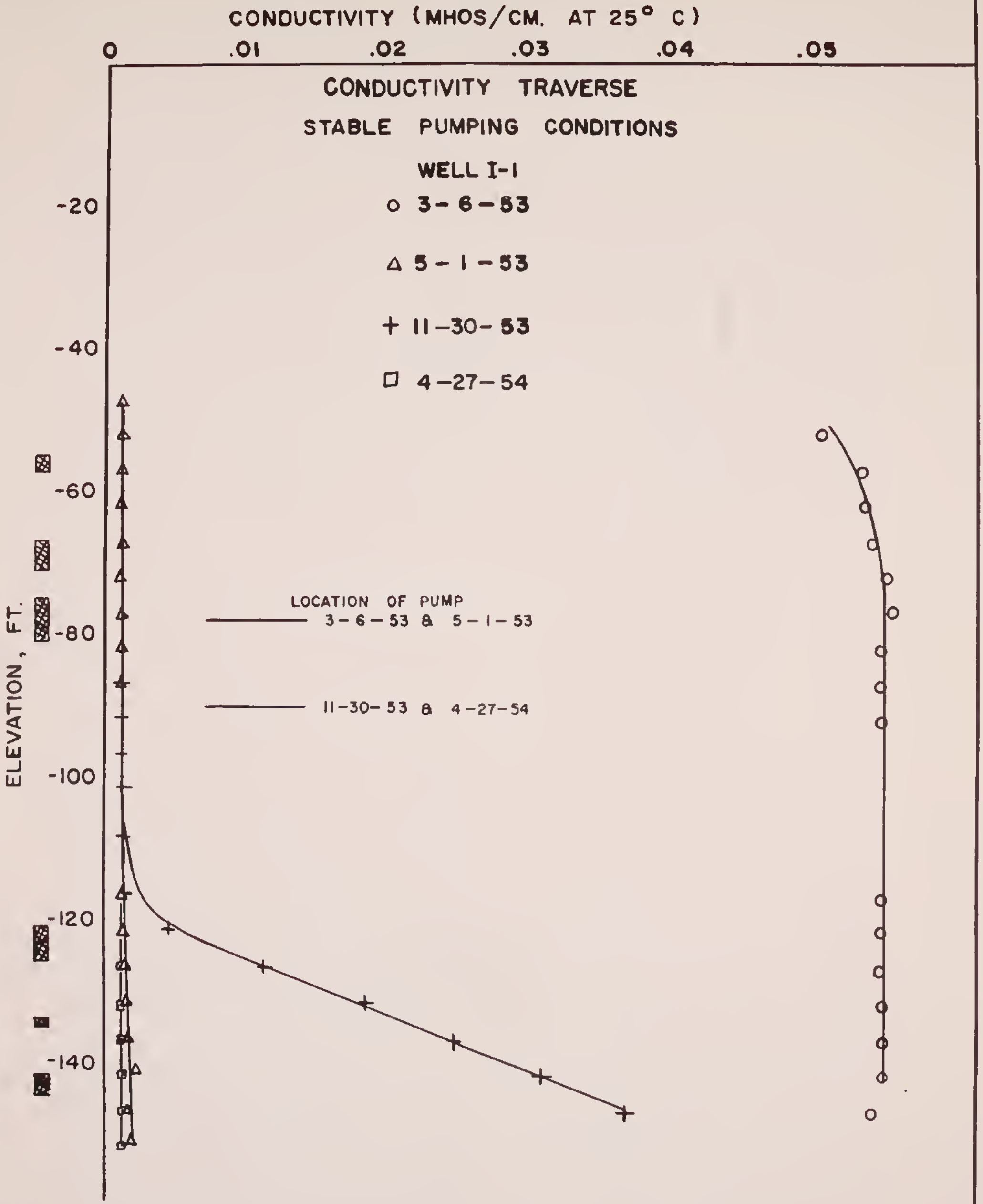
APPROXIMATELY 3500 TO 5500 FEET LANDWARD OF LINE OF RECHARGE



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

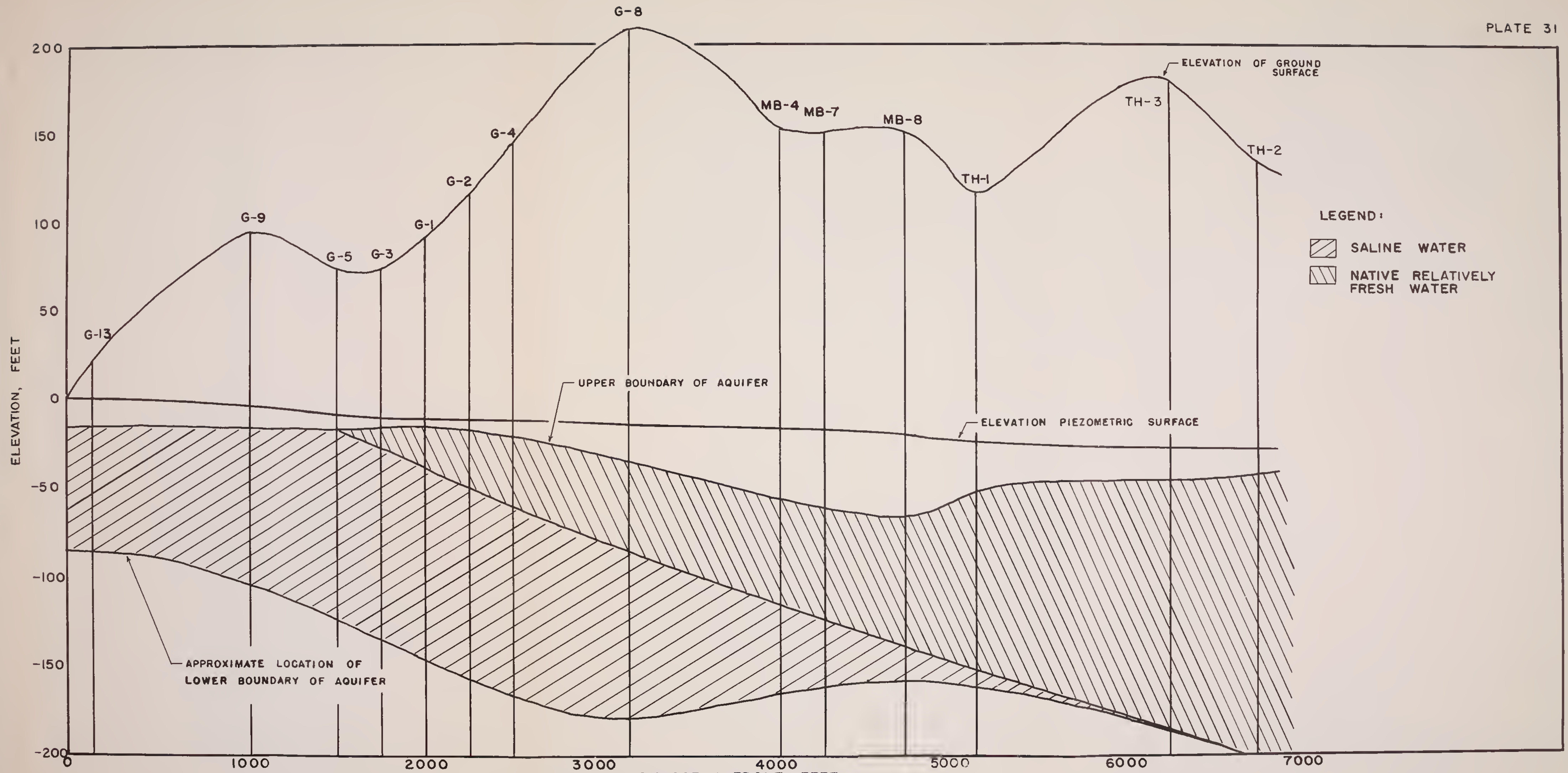
OVERRIDING EFFECT OF FRESH WATER WEDGE



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

UNDERRIDING EFFECT OF SALINE WATER

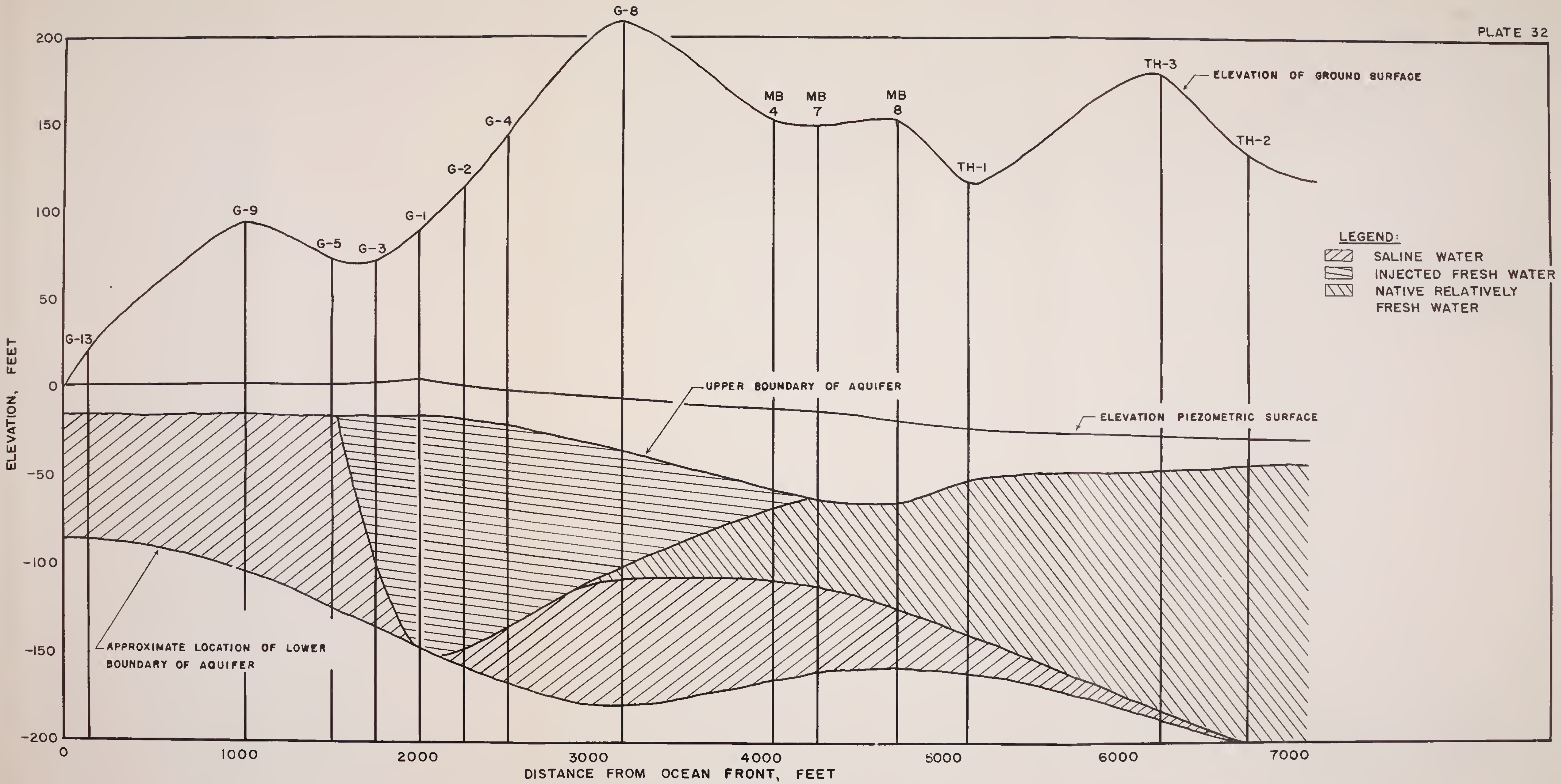


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

IDEALIZED SECTION SHOWING EXTENT OF SALINE INTRUSION PRIOR TO RECHARGE

JANUARY, 1953

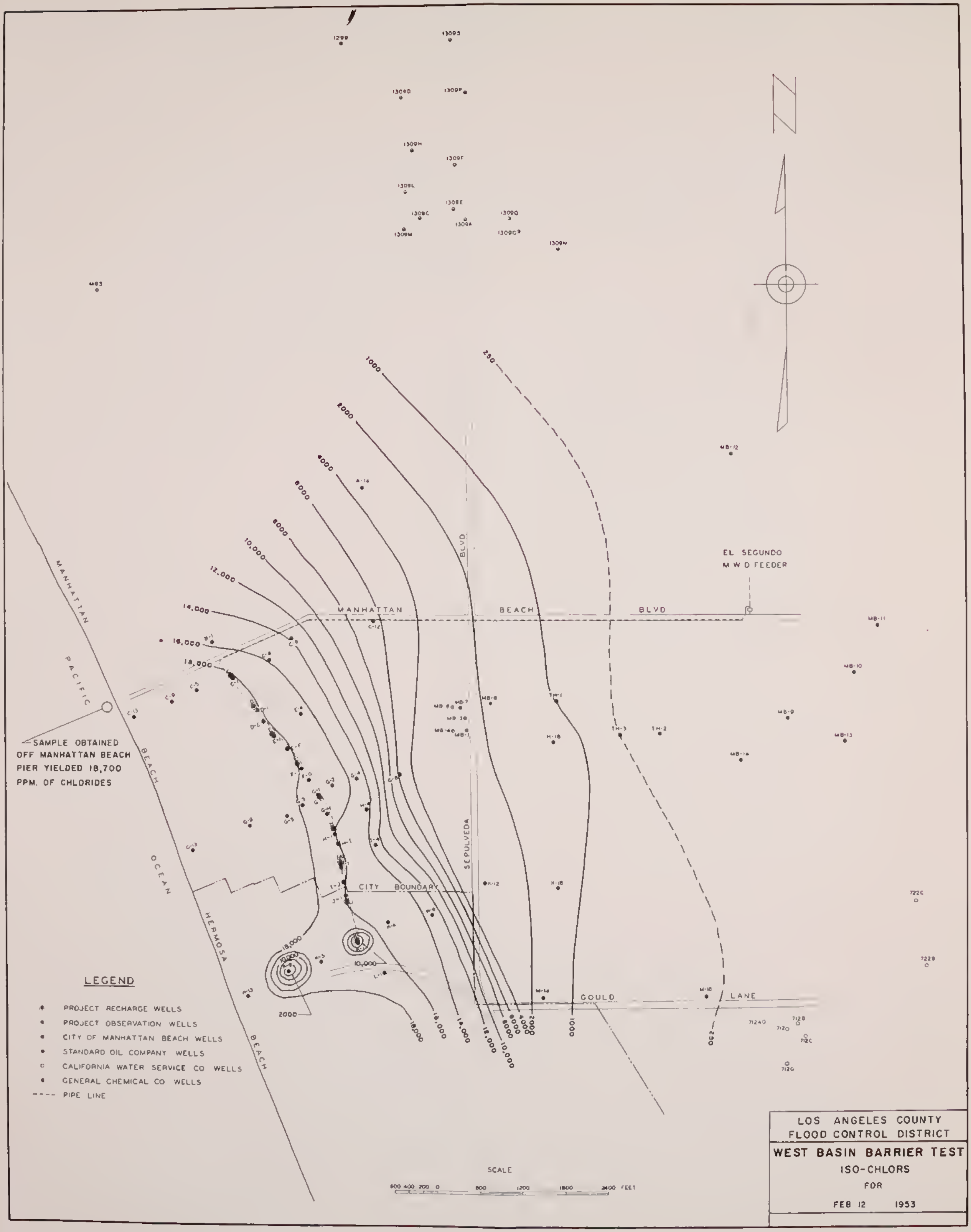


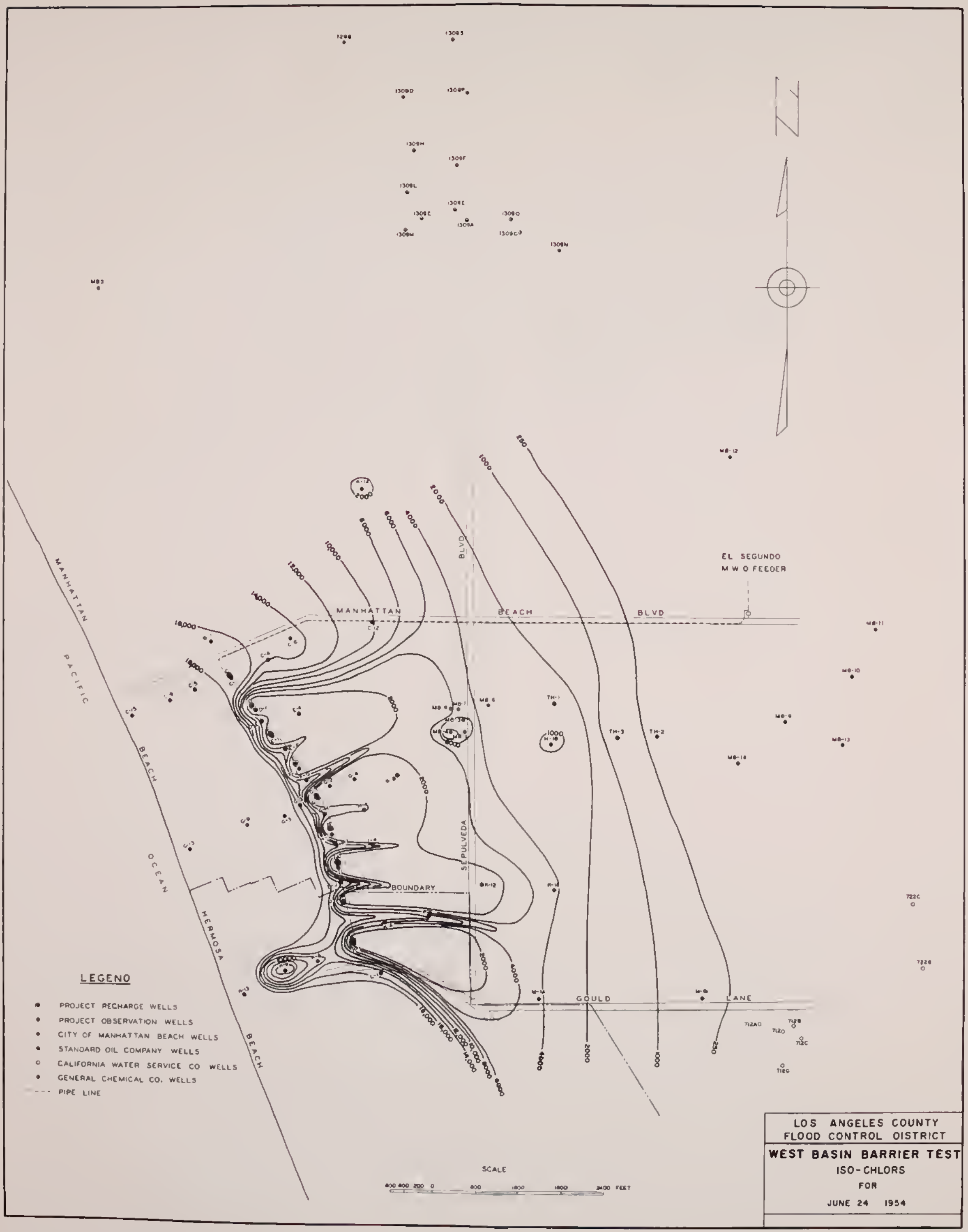
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

IDEALIZED SECTION SHOWING DEVELOPMENT OF THE INJECTED FRESH WATER BODY

JUNE, 1954


MANHATTAN BEACH BLVD
SEPULVEDA BLVD
GOULD LANE
CITY BOUNDARY
EL SEGUNDO
M W D FEEDER
PACIFIC OCEAN
MANHATTAN BEACH
HERMOSA BEACH
SAMPLE OBTAINED OFF MANHATTAN BEACH PIER YIELDED 18,700 PPM. OF CHLORIDES
LEGEND
PROJECT RECHARGE WELLS
PROJECT OBSERVATION WELLS
CITY OF MANHATTAN BEACH WELLS
STANDARD OIL COMPANY WELLS
CALIFORNIA WATER SERVICE CO WELLS
GENERAL CHEMICAL CO WELLS
PIPE LINE
SCALE
FEET
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
ISO-CHLORS
FOR
FEB 12 1953



LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

ISO-CHLORS

FOR

JUNE 24 1954

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

APPENDIX A

GEOLOGIC STUDIES RELATIVE TO INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA WATER INTRUSION

By EDWARD J. ZIELBAUER and RICHARD S. DAVIS

5—52568

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PLATES

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TABLE

PHOTO

GEOLOGIC STUDIES RELATIVE TO INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA WATER INTRUSION

ABSTRACT

Geologic studies were undertaken to determine the nature, extent, thickness, and other pertinent characteristics of the affected Merged Silverado aquifer recharged at the West Basin Barrier test site at Manhattan Beach and Hermosa Beach, California.

Drilling disclosed at the base of dune sands and coastal deposits, a continuous, relatively impervious body of sediments averaging 20 to 30 feet in thickness. This body extended over the entire project area except for a discontinuous band adjacent to the strand, where it appears to have been removed by along-shore current action and transverse channeling.

The clay horizon was underlain by the Merged Silverado Zone, the major prolific aquifer which bifurcates inland into several important water-bearing members. This zone, averaging about 110 feet in thickness along the recharge line, was found to be typically divided into two phases—an upper brown phase and a lower gray phase. The upper brown phase consisted primarily of continental and littoral deposits, portions of which were correlative with the "200-foot sand" zone, an important inland aquifer. The lower gray phase appears to have been deposited in a shallow marine environment.

Beneath the Merged Silverado aquifer was an extensive thickness of fine-grained sediments which generally became more compact with depth, and constituted the lower boundary of the aquifer.

It was found that recharging of the non-homogeneous aquifer was most effectively and expeditiously accomplished by the use of gravel-packed wells. Stripping of the capping sediments may have retarded but did not prevent pressurization of the aquifer by recharging. Preservation of the cap sediments to permit continued and effective pressurization required both grouting throughout the cap and care to prevent overdevelopment of injection wells. Stripping of the cap sediments along the strand and ancient channeling extending inland transverse to the present shore line, coupled with a steepened hydraulic gradient, probably accelerated sea water intrusion at the nearby, now abandoned, City of Manhattan Beach well field. The coincidence in the marked thinning of Merged Silverado sediments at the coast, the remarkable linearity and parallelism of topographic features with the present shore line, and differentials in elevations of stratigraphic surfaces zoned by foraminiferal assemblages, suggests the possibility that deposition of sediments was controlled to some degree by sharp flexuring or faulting parallel to the coast. This flexuring appears to have established a partial barrier to sea water intrusion, as evidenced by the steep piezometric gradient indicated by oceanward observation water surface levels noted prior to recharging.

PHYSIOGRAPHY

The project area lies within the limits of three of four minor physiographic provinces, I, II, III, and IV, outlined by Metzner [1] and delineated on plate (*1*). The project recharge line lies within province II. In brief:

Province I, extends from the Pacific Ocean inland to the first valley paralleling the coast, a distance of from 1,500 to 2,000 feet. Rather steep, recent sand dune escarpments lie along the major portions of both boundaries.

Province II, next inland, includes a series of sand dune ridges and valleys paralleling the coast line. At Manhattan Beach the province swings sharply coastward and then southerly flattens into a terrace. The remarkable parallelism and linearity of the topographic features of provinces I and II, and of the shore line, suggests the possibility of fault control.

Province III, further inland, includes a much wider band of sand dune hills and depressions having a general trend nearly perpendicular to that of province II and the coast line.

Province IV, extends from the eastern boundary of province III to the Newport-Beverly Hills Uplift (Inglewood Fault Zone) and includes the main portion of the West Coast ground water basin.

SOILS

Soils within the area investigated are, as mapped by the U. S. Department of Agriculture Bureau of Soils,[2] of three principal types: (1) Coastal beach and dune sand, (2) Oakley fine sand, and (3) Ramona sandy loam.

Coastal beach and dune sands occur directly along the shore and are composed of pervious yellow-brown to buff, fine to very coarse sand, quite clean and poorly consolidated.

The Oakley fine sand series is brown, with variations to grayish brown and light brown to buff, and with the lower part of the section frequently lighter

[1] See Bibliography.
[2] See Bibliography.

in color. The soils generally contain considerable fine material, giving them a somewhat loamy appearance, and often contain irregular zones of clay-binding material which makes them relatively impervious.

The Ramona series, presumably derived from altered old unconsolidated water-laid deposits, is brown in color with slightly reddish brown to grayish brown variations. It is underlain by heavier, more compact, brown, reddish brown, light brown or red soils. These sandy loams exist only in the most landward extension of the area and there seems to be some reason to believe that locally the series may be dune material rather than water-laid and should actually, despite fineness and compactness, be classified as the Oakley wind-blown sand.

DESCRIPTION OF MAJOR DEPOSITS AND CHARACTERISTICS AFFECTING THE RECHARGE TEST

Dune Sand and Coastal Deposits (Recent)

The main portion of the surface of the area is occupied by the El Segundo Sand Hills. These hills are largely dune sand and are composed of light yellowish brown to dark reddish brown, fine to coarse sand. (Plate 1). The sands show some iron staining and clay binding, and are locally silty, compacted and relatively impervious. The sorting is fair and the shape ranges from angular to subangular. These coarse sands, primarily of granitic origin, contain an appreciable amount of magnetite, pyroxenes, amphiboles, and micas with lesser amounts of zircon, topaz, garnet, epidote, etc. Near shore, the older dune sands are directly overlain by active dune material which is loose and highly permeable.

In general, three significant horizons were noted within the sand dune deposits above the "clay cap" —the relatively impervious stratum confining the underlying major aquifer: a localized surficial "ironbound" sand horizon, an intermediate horizon of relatively clean dune sands with occasional gravels in the basal portion, and a lower horizon of fine sands and silts with sandy stringers which constituted a zone of transition to the "clay cap."

Two-inch test wells adjacent to the injection wells were bottomed within the lower horizon of the sand dunes but above the "clay cap," to determine leakage from the aquifer. High water surface levels were observed in the test wells. Subsequently, shallower test holes were drilled near the injection wells to check the levels observed in the original two-inch wells. These shallower wells confirmed the belief that recorded high water surface levels in the original test wells reflected semiconfined or partial pressure levels within the zone of transition to the "clay cap" rather than free water levels. Hence, leakage from the aquifer to sands lying above the clay cap was probably limited to that transmitted by the sandy stringers within the zone of transition to the "clay cap." Inasmuch as the stringers were of minor thickness, flow through the stringers was doubtless equally limited, and hence the leakage above the "clay cap" during the test is considered to be of minor significance.

The sediments along the beach and underlying the dune sand are recent marine coastal deposits composed of sands and gravels, with included cobbles from three to eight inches in diameter. The two types of deposits are normally not differentiated.

Palos Verdes Formation (Uppermost Pleistocene)

Deposits constituting the Uppermost Pleistocene Palos Verdes formation are marine sands and gravels underlying the dune sands. They are very similar to the coastal deposits, except that they occur somewhat farther inland and include calcitic fragments not found in the coastal deposits. Diagnostic marine megafossils are often noted. In portions of the area the formation is absent, probably due to stripping by marine currents and/or fluviatile activity.

To the east of the El Segundo Sand Hills and parallel to the coast, a very small area of Upper Pleistocene sediments is exposed within the area investigated.

"Clay Cap" (Upper Pleistocene)

Directly underneath the dune sand and coastal deposits and lying at or near sea level is a continuous horizon of relatively impervious deposits referred to, for simplicity in this investigation, as the "clay cap." This cap was not a true compact clay, but composed of varying colors of silts, silty fine sand stringers, sandy clays, and clays normally yellowish brown but occasionally gray, green, or mottled. In cases where sediments comprising the cap are gray in color, the upper brown member of the Merged Silverado Zone may be non-existent. In certain cases in the easterly portion of the area, thin discontinuous sand and gravel lenses are included as a part of this cap.

These deposits, typical of many coastal reaches of the State, obviously are not as impermeable as a true clay body and are subject to some degree of erosion if water is permitted to move at excessive velocities along or through the cap. Hence, the annular space between casings and side walls of injection wells must be properly sealed throughout the cap to prevent rupturing induced by sudden changes in injection rates, with consequent failure of repressurizing operations. The cap failures experienced at injection wells C, G, and I were probably due to the creation of voids produced by over-development and excessive leakage past the clay cap at the well casing. In contrast, the continuous successful operation of wells E and I-A, which were gravel-packed and grouted, would indicate the benefits of this type of well construction.

Test well data within the subject area revealed the upper surface of the "clay cap" to be unusually flat and to vary from 10 feet above to 10 feet below sea level. (See plate 2). Existing well data indicated larger irregularities inland due, at least in part, to differences in logging by various drillers. The relative flatness of the "clay cap" is indicative of probable planation and deposition of an older, more irregular surface.

Contours on the bottom of the "clay cap" appear to reflect, to a degree, transverse drainage channels. (See plate 3.) Thickness of the "clay cap" along the recharge line, as delineated by isopachs on plate 4, averages from 20 to 30 feet and varies from zero at the strand to 48 feet on the inland side of the recharge line. This is also shown on geologic sections, plates 5, 6, and 7.

The presence of this "clay cap" over the entire area definitely identified the aquifer as a pressure or confined aquifer. This was of considerable importance in the establishment of a pressure mound or ridge along the recharge line. On the basis of data available prior to the test, an investigation by another agency indicated the aquifer as being exposed to saline water at some distance seaward of the coast. Wells drilled along the strand, however, showed the absence of clays as well as at one point some 800 feet inland from the strand, at test well K-9. This apparently resulted from stripping and channeling by geologically recent erosional activity.

Local stripping of the "clay cap" along the shore margin is of significance in that it reduced the distance of travel of ocean waters and hence hastened saline encroachment at these points. The stripping may have retarded but did not prevent pressurization of the aquifer during recharge, since a continuous effective "clay cap" exists along the recharge line and seaward of it for some distance.

"200-foot Sand" Correlative (Upper Pleistocene)

Beneath the "clay cap" and basinward of a line about one mile east of the coast is a zone of interbedded sands and clays overlying the San Pedro formation. These sediments are normally barren of megafossils and, on the basis of lithologic studies to date, are distinguished only in a general way from the San Pedro formation. This zone appears to be, at least in part, correlative with the "200-foot sand," an important inland aquifer mapped by the United States Geological Survey in this general area in 1948.[3] Its existence is inferred due to the absence of fossils, which usually occur in the lower formation, and the presence of a substantially thick separating clay member.

Where this clay member exists (see logs of wells M-14, H-16, and K-16), it often separates the upper brown phase of the Merged Silverado Zone from the lower gray phase, both of which are described below. This appears to indicate, as logged by the State Division of Water Resources, that the upper brown phase constitutes the seaward correlative of the "200-foot sand" zone, also previously mapped by that agency.[4] Continuity with the "200-foot sand" zone is significant in that barrier operations not only check sea water intrusion into the zone, but also permit its recharging with fresh water.

Merged Silverado Zone (Upper and Lower Pleistocene)

The term "Merged Silverado Zone," proposed in a prior report on the West Coast Basin,[4] was expanded from the name "Silverado Zone" used by the U.S.G.S. in 1948.[3] This zone, in the subject area, spans both the Upper and Lower Pleistocene and can be divided lithologically into several components: an upper brown phase, a lower gray phase, and the Silverado—a generally very permeable portion of the San Pedro formation. Within the subject area the zone varies in thickness from 29 feet at Manhattan Beach City well No. 4 (F.C. 701-C) to nearly 155 feet at Project Well No. G-8; however, it thickens appreciably basinward.

UPPER BROWN PHASE

Although possibly distinct from the underlying Silverado, the upper brown phase is generally in hydraulic continuity with, and hence is included as a portion of, the Merged Zone. Although this phase is quite extensive, local gaps are noted. The upper portion of the phase normally consists of yellowish brown sands and silts, and the lower portion of fine sands and gravel stringers with occasional clay bands.

Sand fractions are angular to subangular while the gravel in the lower portion contains fragments that are somewhat more rounded. It is arkosic, having a feldspar to quartz ratio of about 3 to 2. The heavy mineral content varies from 1% to about 15% and consists largely of magnetite and ilmenite, pyroxenes, amphiboles, micas, epidote, topaz, garnet, and minor amounts of other minerals. Study of rock fragments (See table 1) indicates only minor differences between this phase and an underlying lower gray phase.

The upper brown phase is much more prominent in the seaward portion of the area and does not appear to exist in certain of the more inland wells. (See logs of wells K-16 and M-18).

LOWER GRAY PHASE

A gradation is noted from the upper brown phase to the underlying lower gray phase. The upper portion of the lower gray phase normally is rather greenish and becomes progressively gray and then bluish-gray with depth. The gravel horizon constituting the basal portion of the upper brown phase seems to

[3] See Bibliography.

[3, 4] See Bibliography.

merge with the upper portion of the lower gray phase, which contains the gravel horizon of that phase. The lower gray phase becomes finer and more compact with depth. Lithologically, there is very little apparent difference between the two zones despite the great difference in color. This color difference seems primarily to be the result of the weathering of iron-rich biotite. In the upper phase, this produces an iron staining which imparts a distinct yellowish brown color. In the lower phase weathering is not noticeable, and the black coloration of the biotite lends a gray appearance to the sediment. A more detailed petrographic analysis may establish diagnostic criteria differentiating the upper and lower phases.

Below the gray gravel horizon, as previously stated, the sediments become progressively finer and darker in color. They consist of fine to very fine silty sands and clay bands. Occasionally lenses of medium to fine sand are noted.

Examination reveals the feldspar to quartz relation to be about 2 to 1 and the heavy mineral content, which is composed largely of biotite, to vary from about 2% to about 8%.

The larger fragments are largely granitoid type rocks, some metamorphics, such as quartzite and gneiss, some basaltic or dark fine-grained volcanics, pegmatites or vein type quartz, and a minor amount of sedimentary rocks.

SILVERADO PHASE

That portion of the San Pedro formation in the West Coast Basin which is most permeable and composed largely of sand and gravel is know as the Silverado water-bearing zone. This zone is the most important and most prolific aquifer in the basin. In the subject area, it consists largely of sand with scattered gravel and is combined or merged with the brown and gray phases previously described. In some instances, it is gray in its entirety.

Another aquifer of Lower Pleistocene age, the "400-foot gravel" zone, which is important in the basin interior, does not exist recognizably in the investigation area.

Lower Member of the San Pedro Formation (Lower Pleistocene)

In this area the lower member of the San Pedro Formation is composed of bluish gray compact very fine sands, silts and clays. The relatively tight sediments are important aquicludes which retain waters in the overlying permeable members of the Merged Silverado Zone and constitute the lower boundary of the aquifer. Recognition of this member is based on the presence of megafossils of a cold water type and, where they occur, foraminifera. Obviously, the finer grained sediments underlying the merged zone are not impermeable but relatively so; hence, salinity intrusion within these sediments is of relatively minor concern. Transmission of pressure effects from the Merged Silverado Zone to the underlying sediments eventually occurs within those portions having at least some degree of hydraulic continuity with the Merged Zone.

GENERAL PARAMETERS OF THE AQUIFER AS RELATED TO RECHARGE

Geologic sections showing the extent, thickness, and character of the zones described may be noted on plates 5, 6, and 7. The Merged Silverado Zone, for the purpose of this report, has been chosen as that material below the "clay cap" and above the fine, compact, blue-gray portion of the lower San Pedro formation except where the correlative of the "200-foot sand" zone is distinguishable. In some cases where minor clays or clayey or silty strata are interbedded with the coarser material, particularly near the bottom, they have been included in this zone.

Although the thickness of the Merged Silverado varies greatly, both seaward and landward, along the recharge line, the zone averages about 110 feet in thickness and the bottom elevation averages about 130 feet below sea level. Along geologic section lines C-C, G-G, and K-K extending in a direction transverse to the recharge line, the depth is somewhat variable, being in general deeper inland and toward the south. The thickness along section C-C is about 90 feet with the average elevation of the bottom being about 130 feet below sea level. The thickness along G-G averages about 100 feet and the bottom elevation is about 140 feet below sea level. The average thickness along K-K is about 120 feet and the elevation of the bottom is about 160 feet below sea level.

Extending eastward from the ocean, the Merged Silverado Zone thickens irregularly. Contours drawn on the top (Plate 8) and bottom (Plate 9) of the aquifer indicate several large irregularities in the vicinity of the test site and reveal the bottom to be somewhat more variable than the top. These surfaces are significant in that they indicate channeling transverse to the present shore line. There appears to be, in addition, a rather strong suggestion of parallelism with the present shore line which may be a reflection of deposition controlled to a degree by sharp flexuring or faulting. Such a zone of sharp flexuring or faulting further evidenced by a discontinuity of stratigraphic surfaces, may act as a partial barrier to sea water intrusion, as indicated by observation well water surface levels noted prior to recharging. This pattern of thinning of aquifer sediments with approach to a fault-controlled coastline is evident along other coastal reaches of the State and is of significance in that, where applicable, geologic delineation of the zone of thinning can define the most hydraulically effective and economical route for injection wells, provided right of way acquisition is economically feasible. Isopachs on the Merged Silverado, (see plate 10) based in part on old water well logs, indicate differences in localized areas of special interest:

(1) An abrupt thinning, at the abandoned City of Manhattan Beach well field at 8th Street and Sepulveda Boulevard, with a minimum thickness of about 29 feet at Manhattan Beach well No. 4 (F.C. 701-C)

(2) An appreciable thickening to a maximum of 290 feet in the area occupied by California Water Service Company wells.

The Merged zone is considered to be laid down largely under continental and shallow marine conditions and, hence, it is to be expected that the nature of the deposits vary greatly both areally and with depth. This variation, of course, had a profound effect upon the permeability and the thickness of the aquifer recharged during project operations. While the thickness of the Merged Silverado Zone was appreciable, those portions having a high permeability were found to be considerably less in thickness. A list of thicknesses of the more permeable phases existing in wells along the recharge line follows:

Well	*Thickness of More Permeable Phase*	*Thickness of Most Permeable Phase*
B-1	43 feet	17 feet
C	18 "	0 "
C-1	20 "	12 "
D	28 "	7 "
E	24 "	11 "
E-1	8 "	4 "
F	38 "	29 "
F-G	47 "	32 "
G	44 "	25 "
G-1	42 "	23 "
G-H	42 "	31 "
H	60 "	31 "
I	60 "	17 "
I-1	67 "	17 "
J	32 "	20 "
K	3 "	0 "
K-1	28 "	0 "
L-1	19 "	4 "

NOTE: More permeable phase—Fine to medium sand and coarser.
Most permeable phase—Fine to very coarse sand and coarser without included silt.

Despite the fact that the deposits were in hydraulic continuity, resultant variations both in vertical and lateral permeability in the non-homogenous aquifer doubtless constituted one control of the time lapse noted following changes in injection rate to effect a pressure mergence at the pressure mound internodal points. Recharging of the non-homogeneous aquifer was thus most effectively and expeditiously accomplished by the use of gravel-packed wells such as E and I-A.

Test well drilling delineated a channel transverse to the present shore line between test wells G and I, extending from the proposed recharge inland line to the City of Manhattan Beach well fields. This feature, coupled with the absence of the "clay cap" along shore, doubtless permitted more direct contact of the Merged zone with sea water at or near the strand. Saline intrusion of the original Manhattan Beach well field probably was accelerated by the channeling and a steepened hydraulic gradient created by the accentuated drawdown of the nearby now abandoned well field.

PALEONTOLOGY

Determination of stratigraphic ranges, as usual, was on the basis of foraminiferal and megafossil assemblages.

Dr. M. L. Natland, who made the microfaunal determinations states:*

"The foraminiferal assemblages found in shallow test holes drilled in the Hermosa Beach area by the Los Angeles County Flood Control District were typical of those found in general in most of the uppermost beds of the Los Angeles Basin. In most wells there was a layer of reddish colored sand about 100 feet thick. These sands grade downward into carbonaceous silty greenish-gray sand and silts with lagoonal or littoral zone foraminiferal fauna marked respectively by *Rotalia becarrii* and *Elphidium spinatum*.

"Below the carbonaceous zone occurs the Silverado zone which is generally composed of sand and gravel barren of fauna. Abruptly below this coarse clastic zone, a bed of silty micaceous sediments appears with a typical Timms Point fauna marked by *Cassidulina limbata*. This latter assemblage correlates with Woodring's Lower Pleistocene which includes the Lomita Marl (Upper Pliocene of Arnold and Smith). It also belongs in the newly proposed Lower Hallian stage. In the work of this report this lowermost zone, for convenience, has been called Zone A."

A significant differential in elevations of the stratigraphic surfaces of Zone A was noted seaward of the recharge line along transverse sections C-C, G-G, and K-K (Plate 1).

In general, sediments deposited in waters during the Pleistocene were typically cold in the lower Pleistocene and typically warm in the Upper Pleistocene. The lower San Pedro formation in this area is, at least in part, correlative with the Timms Point member of the formation on the basis of cold water forms and faunal similarity to assemblages from the Timms Point type locality. A single foraminiferal species, *Cassidulina limbata*, as noted above, is one index to this member, as is a megafossil species, *Pecten (Patinopecten) caurinus*.

* Personal communication by Dr. M. L. Natland of Richfield Oil Corporation.

As defined by Woodring [5] and Poland [3] in previous investigations, the deposits of San Pedro fine sands, fine grained silts, and clays, which extend some depth below the base of the most permeable phase of the San Pedro formation and which contains Timms Point fauna, are considered to constitute a basal portion of the San Pedro formation and hence delineate the lower boundary of the Merged Silverado Zone.

Foraminiferal data indicated the probable order of deposition of Merged Silverado Zone sediments of the area to be as follows:

1. Marine sediments deposited at depths in excess of 100 feet. Fine sands, silts, and clays containing fauna correlative with the Timms Point member of the San Pedro formation, underlying the generally most permeable portion of the formation in West Coast Basin—the Silverado Zone.
2. Upper San Pedro sediments of shallow marine environment. (Neritic) These sediments were possibly deposited at a depth less than 100 feet.
3. Deposits containing abundant carbonized plant remains.
4. Sediments indicative of deposition in a lagoon environment. Heavy plant growth and consequent eventual production of carbonic acid may well have created an environment unfavorable to the preservation of foraminifera.
5. Continental deposits. These deposits are composed of rust brown quartzose and feldspathic sands with included minor phases of micaceous sands. They were probably redeposited from Lower Pleistocene sediments apparently derived from the Palos Verdes Hills, and from granitic sources to the north.

[3, 5] See Bibliography.

ACKNOWLEDGMENTS

Special acknowledgment is due Dr. M. L. Natland, District Geologist for the Richfield Oil Company, for examination and interpretation of foraminiferal assemblages. Dr. W. P. Popenoe and Mr. Takeo Susuki, of the University of California at Los Angeles. cooperated in identification of megafossil assemblages. Acknowledgment is also due the Department of Geology of the University of Southern California for permitting use of laboratory facilities for mineralogical study.

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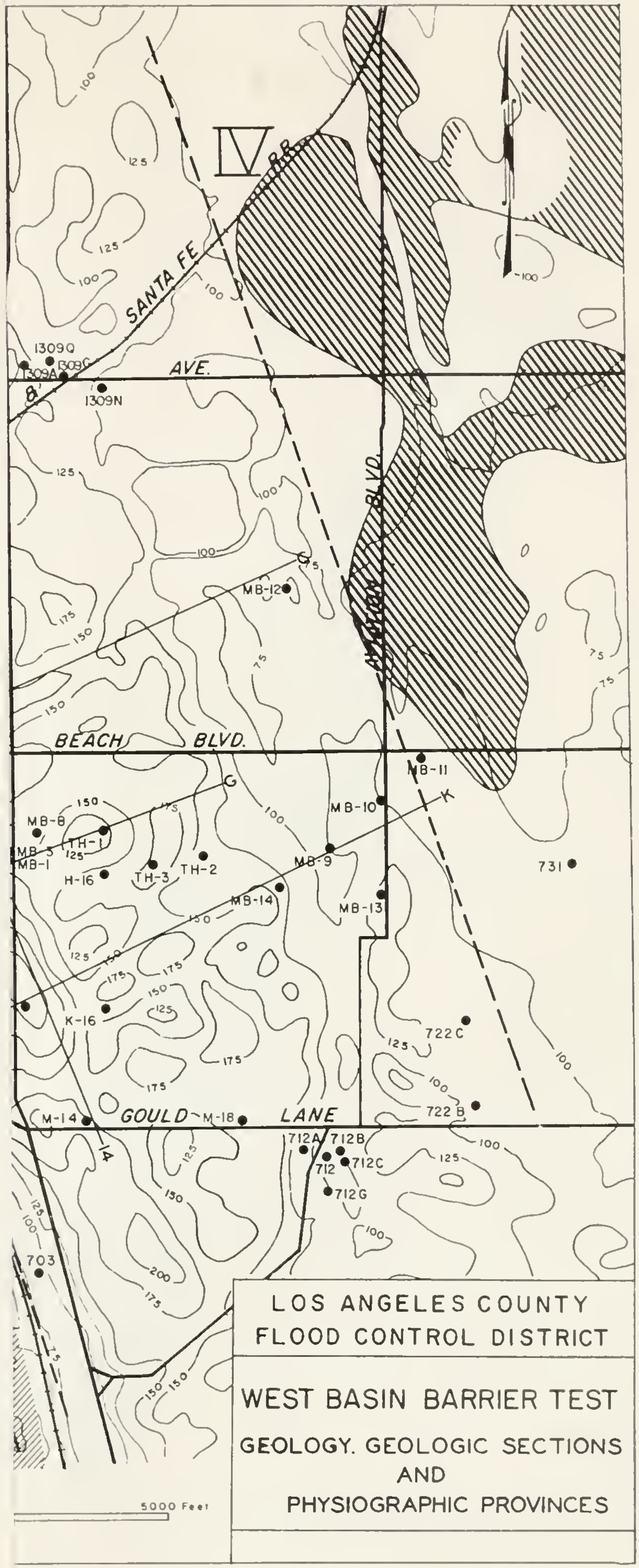
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SANTA FE
AVE.
BEACH BLVD.
GOULD LANE
BLVD.
1309Q
1309A
1309C
1309N
B'
G
G
K
MB-12
MB-11
MB-10
MB-9
MB-13
MB-14
MB-8
MB-3
MB-1
TH-1
TH-3
TH-2
H-16
K-16
731
722C
722B
M-14
M-18
712A
712B
712
712C
712G
703
14
5000 Feet
LOS ANGELES COUNTY
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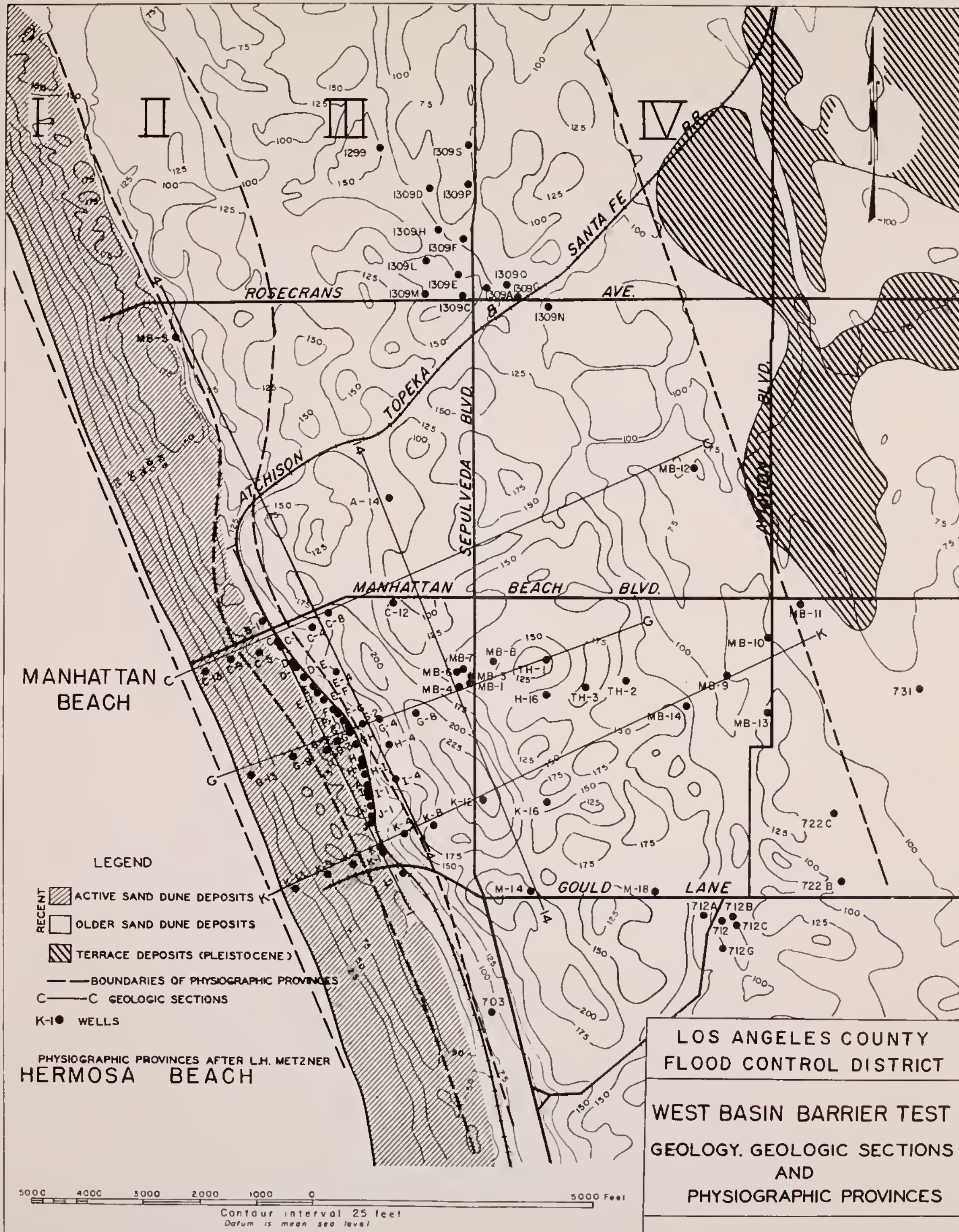
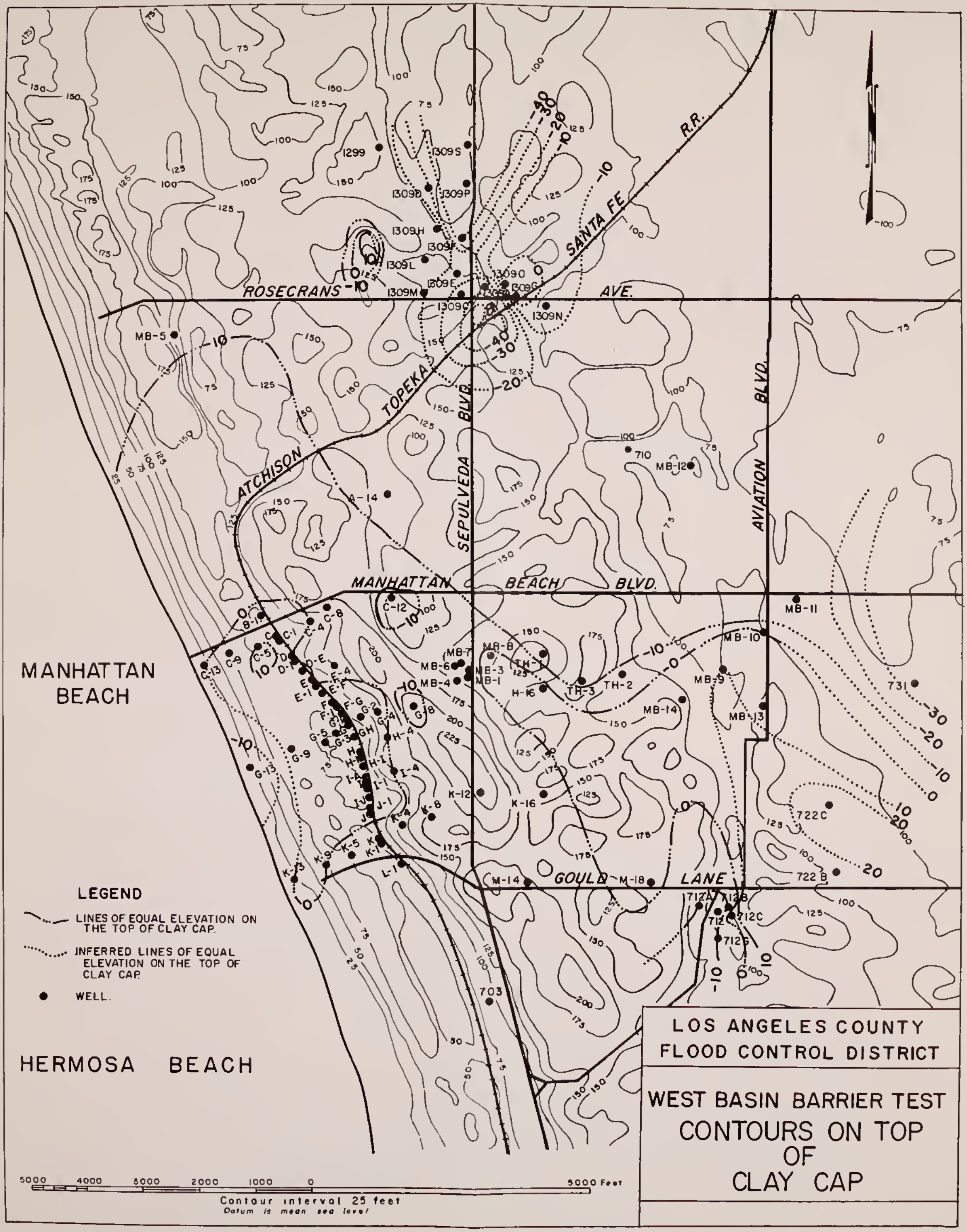
GEOLOGY. GEOLOGIC SECTIONS
AND
PHYSIOGRAPHIC PROVINCES



LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

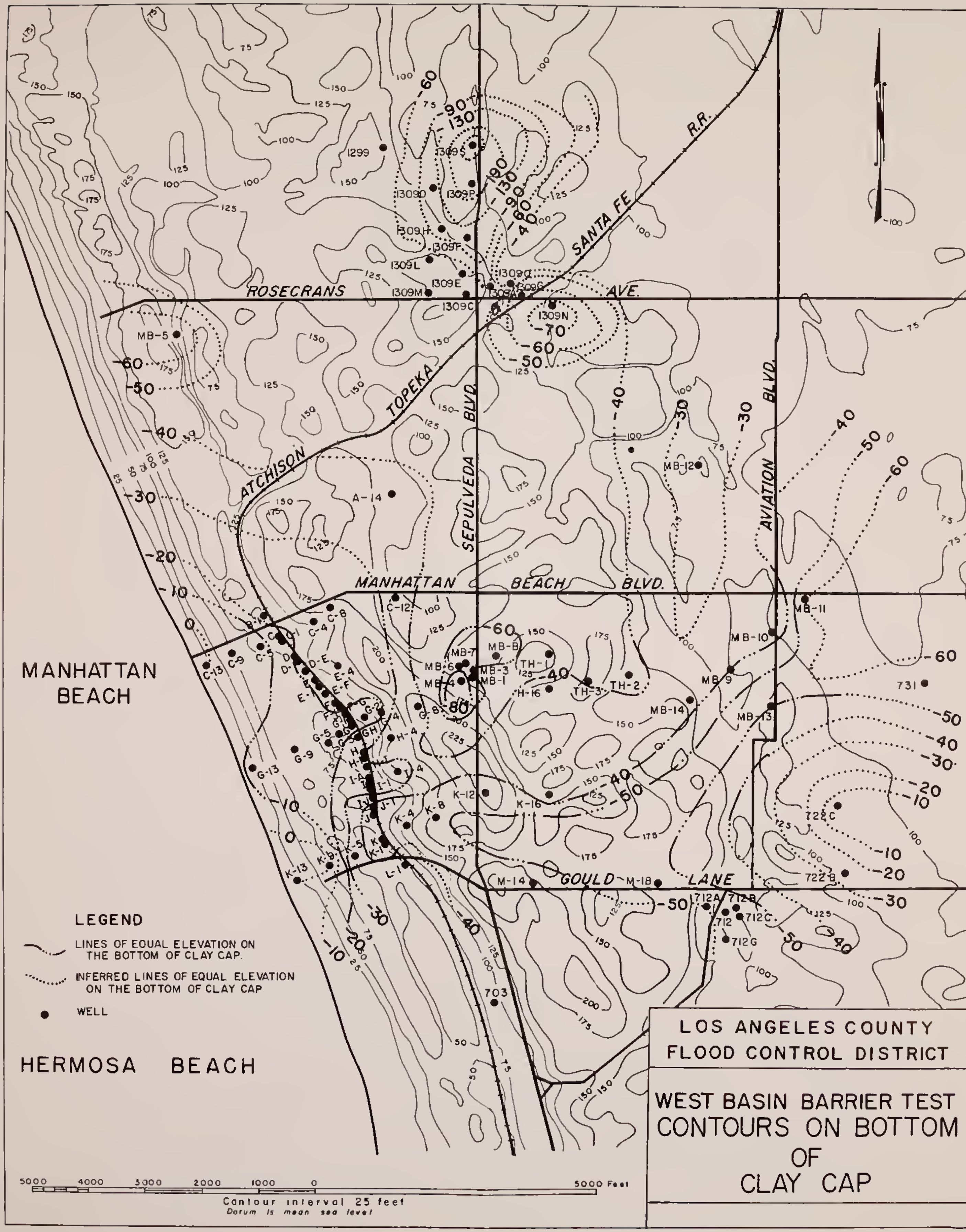
GEOLOGY, GEOLOGIC SECTIONS AND PHYSIOGRAPHIC PROVINCES

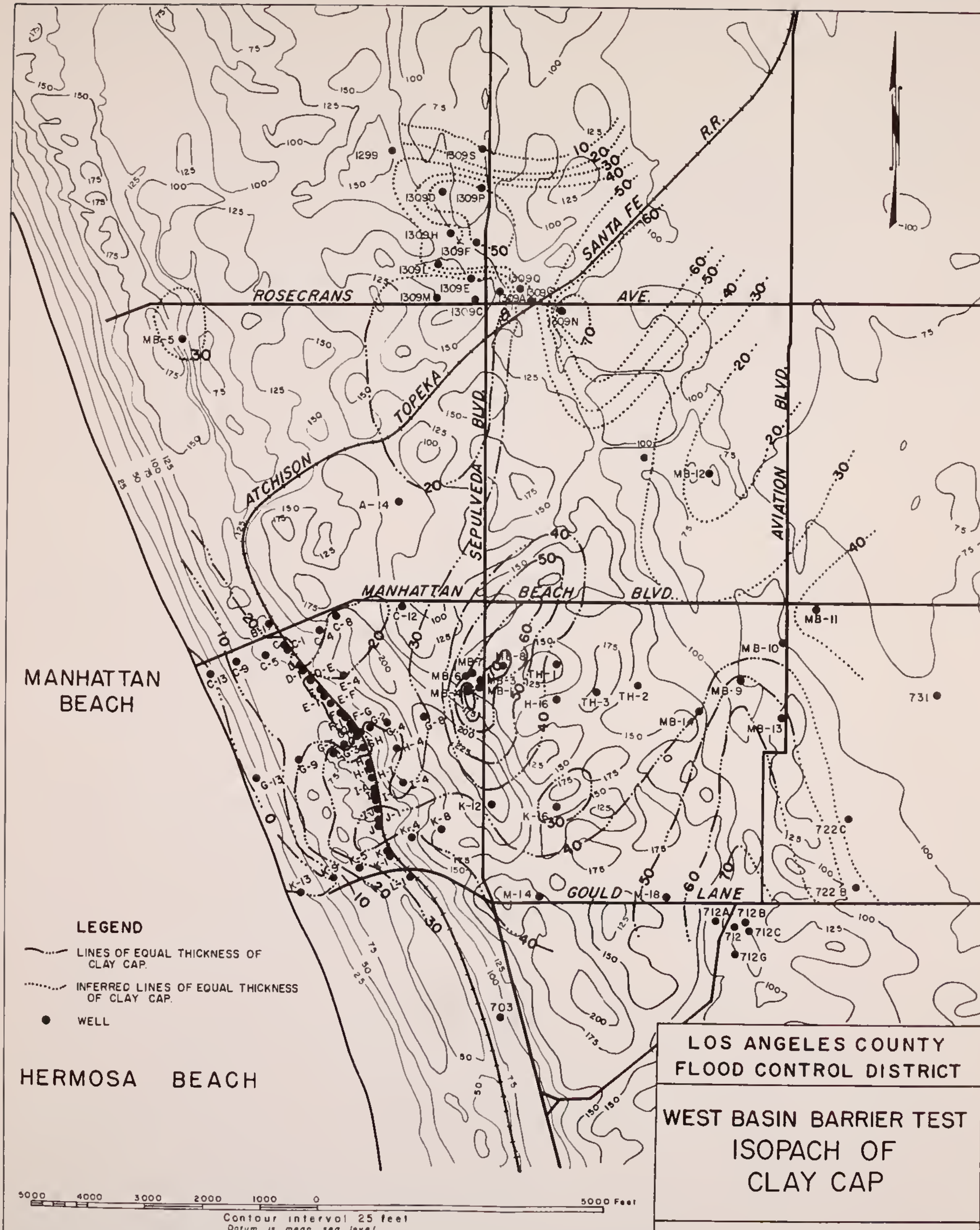


LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

CONTOURS ON TOP OF CLAY CAP

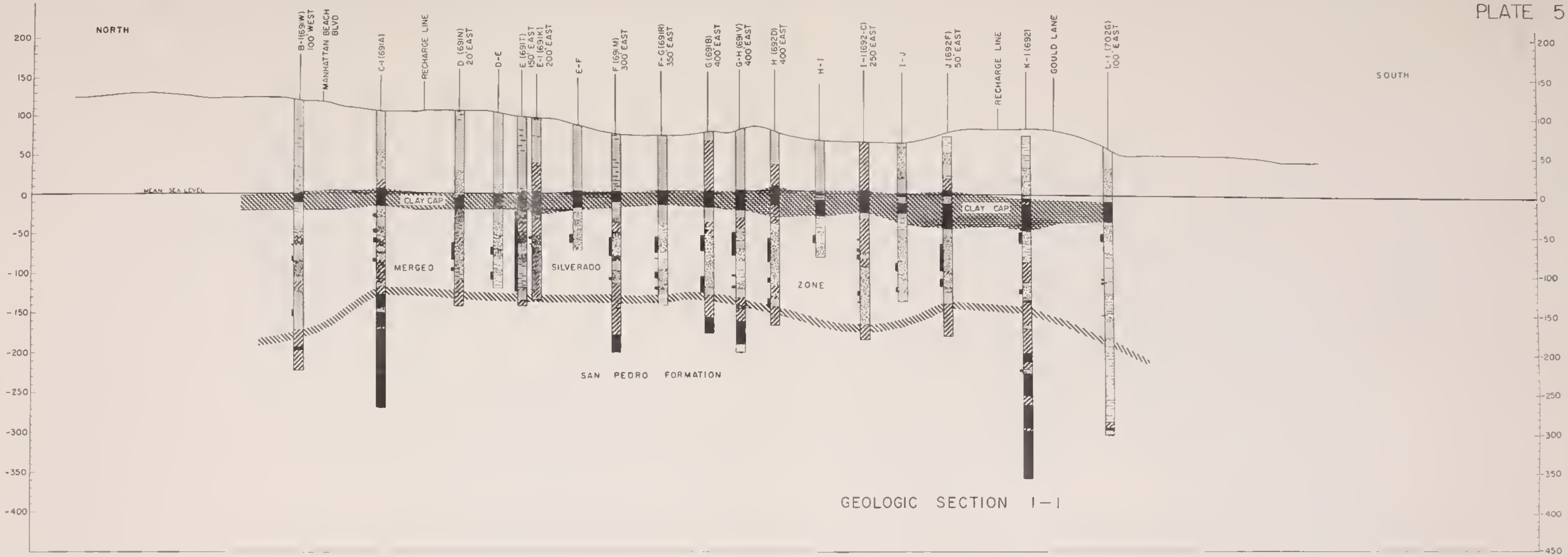

LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
CONTOURS ON BOTTOM
OF
CLAY CAP
LEGEND
LINES OF EQUAL ELEVATION ON THE BOTTOM OF CLAY CAP.
INFERRED LINES OF EQUAL ELEVATION ON THE BOTTOM OF CLAY CAP
WELL
MANHATTAN BEACH
HERMOSA BEACH
ROSECRANS AVE.
MANHATTAN BEACH BLVD.
SEPULVEDA BLVD.
AVIATION BLVD.
GOULD LANE
ATCHISON TOPEKA SANTA FE R.R.
5000 4000 3000 2000 1000 0 5000 Feet
Contour interval 25 feet
Datum is mean sea level



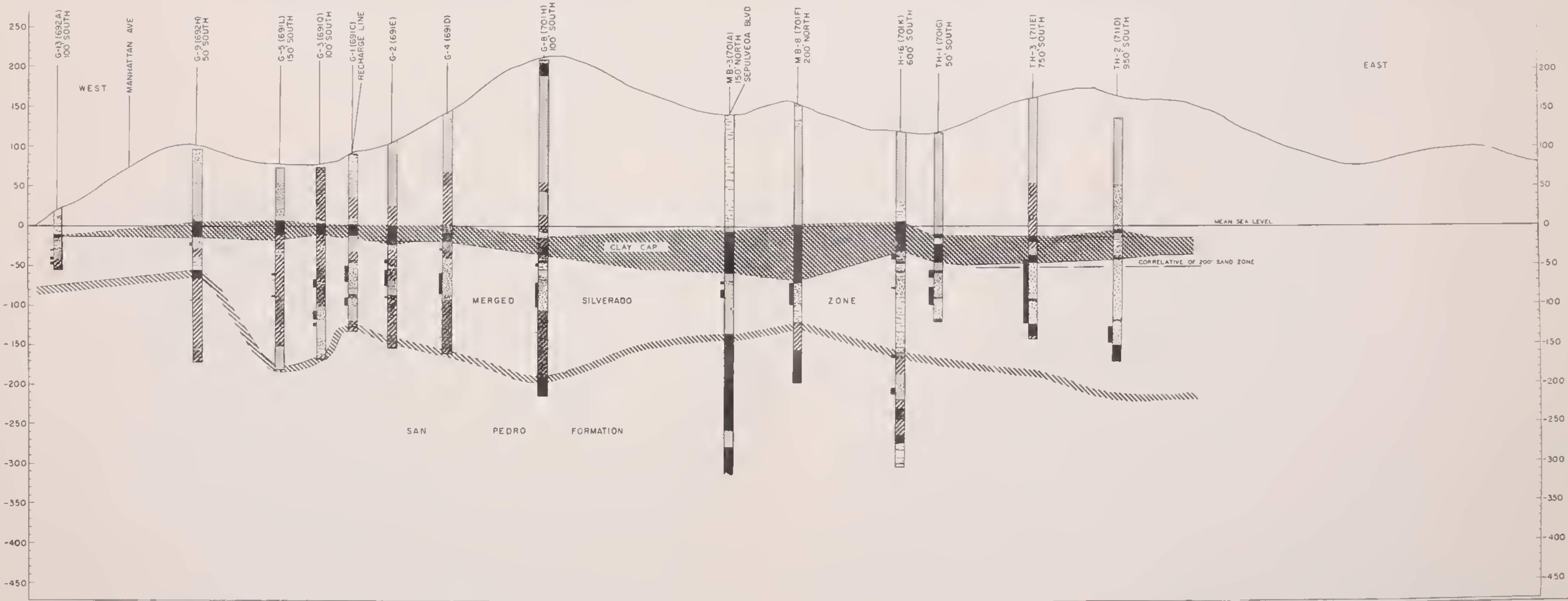
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
ISOPACH OF CLAY CAP

PLATE 5



GEOLOGIC SECTION I—I



GEOLOGIC SECTION G-G

LEGEND

Casing Perforation
Gravel
Gravel & Sand
Sand
Fine To Very Fine Silty Sand
Silt or Clay
Silt or Clay Phases
Relatively Impervious Strata
Base of Merged Silverado Zone

HORIZONTAL SCALE
200 0 400 800
IN FEET
ELEVATIONS IN FEET
USGS DATUM
LOCATION OF SECTIONS
SEE PLATE I

LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
GEOLOGIC SECTIONS
I—I AND G-G

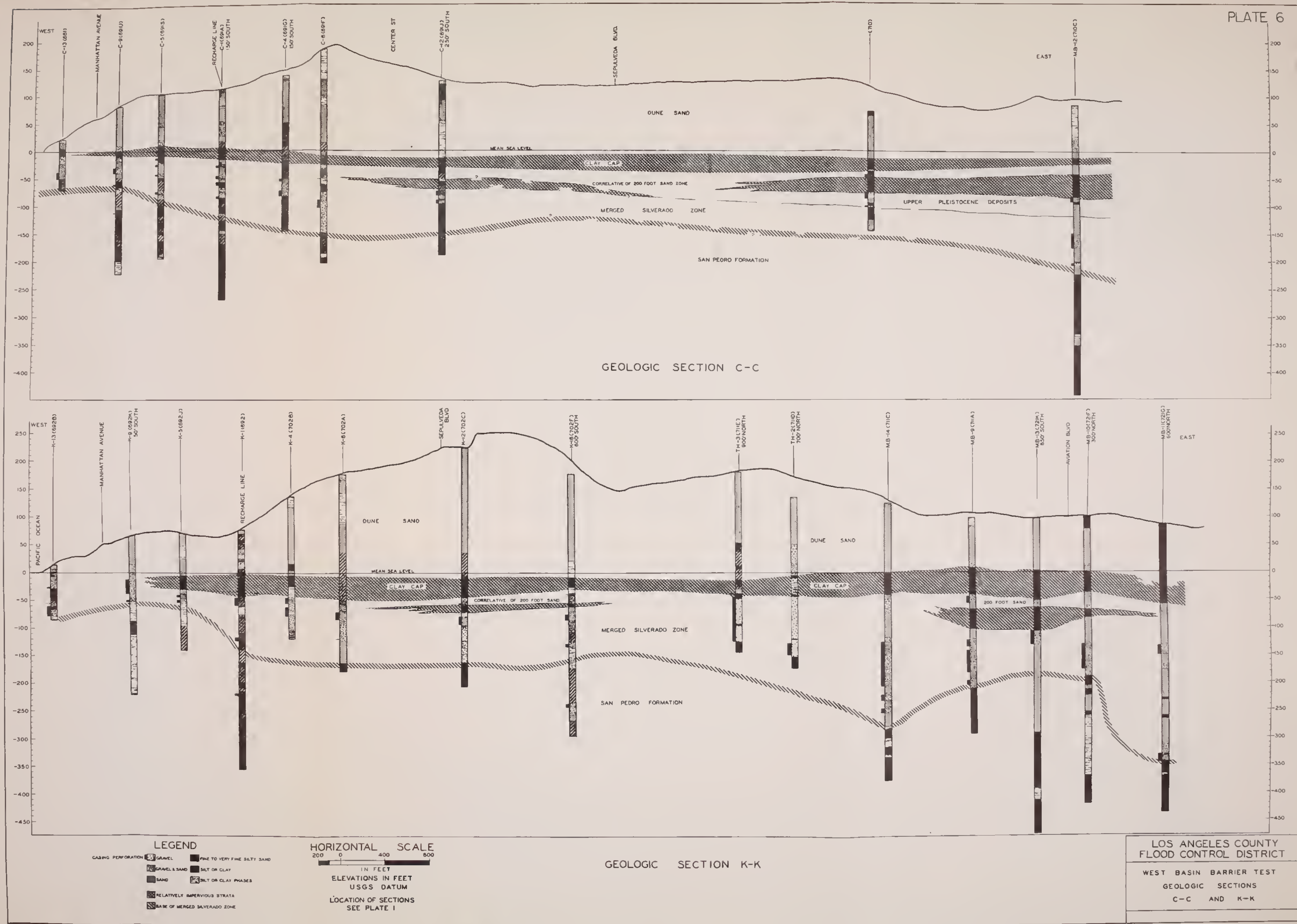
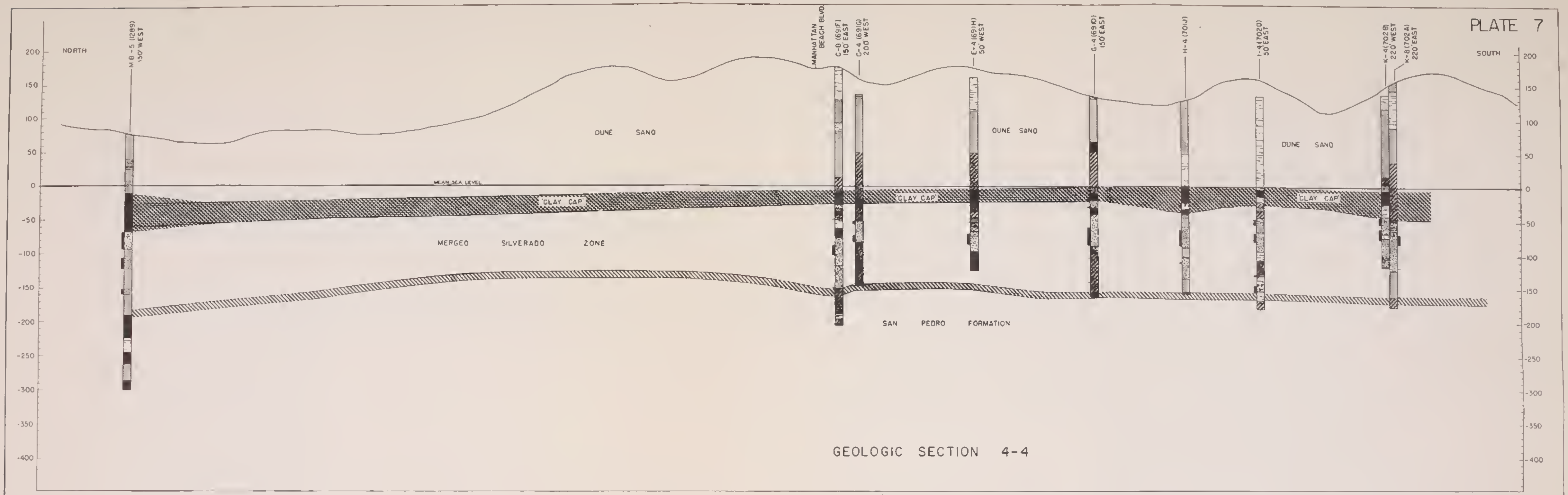
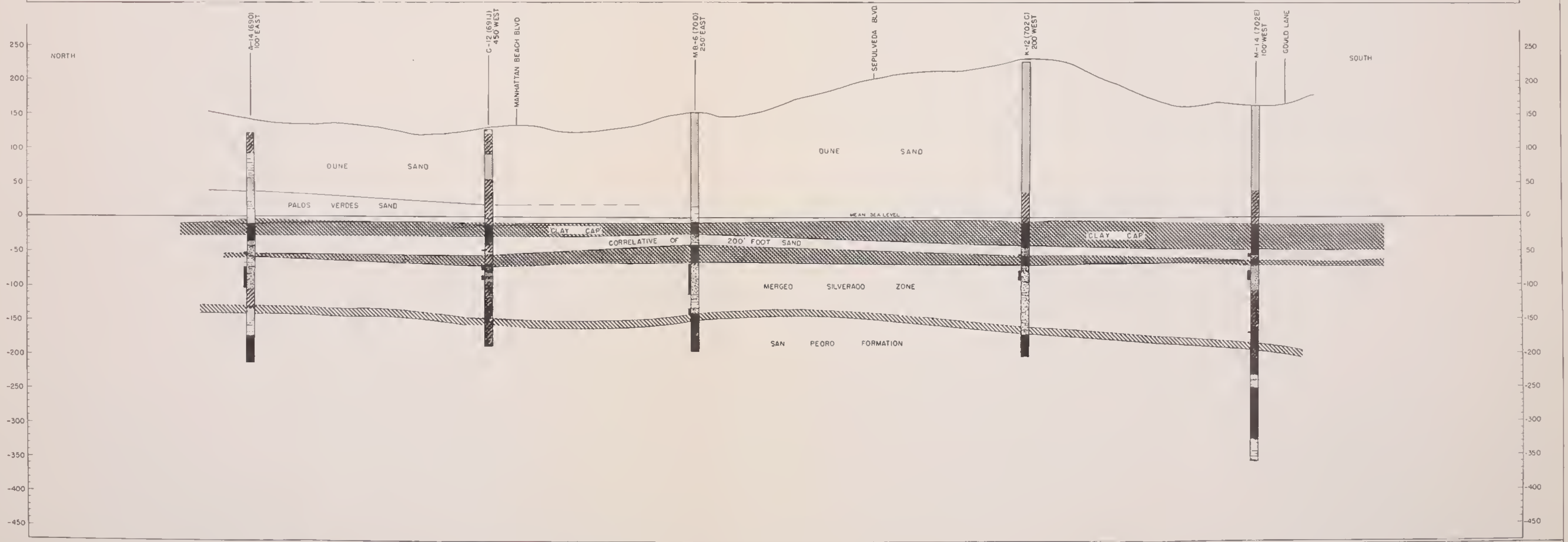


PLATE 6
WEST
EAST
MANHATTAN AVENUE
RECHARGE LINE
CENTER ST
SEPULVEDA BLVD
DUNE SAND
MEAN SEA LEVEL
CLAY CAP
CORRELATIVE OF 200 FOOT SAND ZONE
UPPER PLEISTOCENE DEPOSITS
MERGED SILVERADO ZONE
SAN PEDRO FORMATION
GEOLOGIC SECTION C-C
PACIFIC OCEAN
AVIATION BLVD
200 FOOT SAND
GEOLOGIC SECTION K-K
LEGEND
CASING PERFORATION
GRAVEL
GRAVEL & SAND
SAND
FINE TO VERY FINE SILTY SAND
SILT OR CLAY
SILT OR CLAY PHASES
RELATIVELY IMPERVIOUS STRATA
BASE OF MERGED SILVERADO ZONE
HORIZONTAL SCALE
IN FEET
ELEVATIONS IN FEET
USGS DATUM
LOCATION OF SECTIONS
SEE PLATE I
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
GEOLOGIC SECTIONS
C-C AND K-K

PLATE 7



GEOLOGIC SECTION 4-4



LEGEND

Casing Perforation · Gravel · Gravel & Sand · Sand · Fine To Very Fine Silty Sand · Silt or Clay · Silt or Clay Phases · Relatively Impervious Strata · Base of Merged Silverado Zone

HORIZONTAL SCALE

200 0 400 800

IN FEET

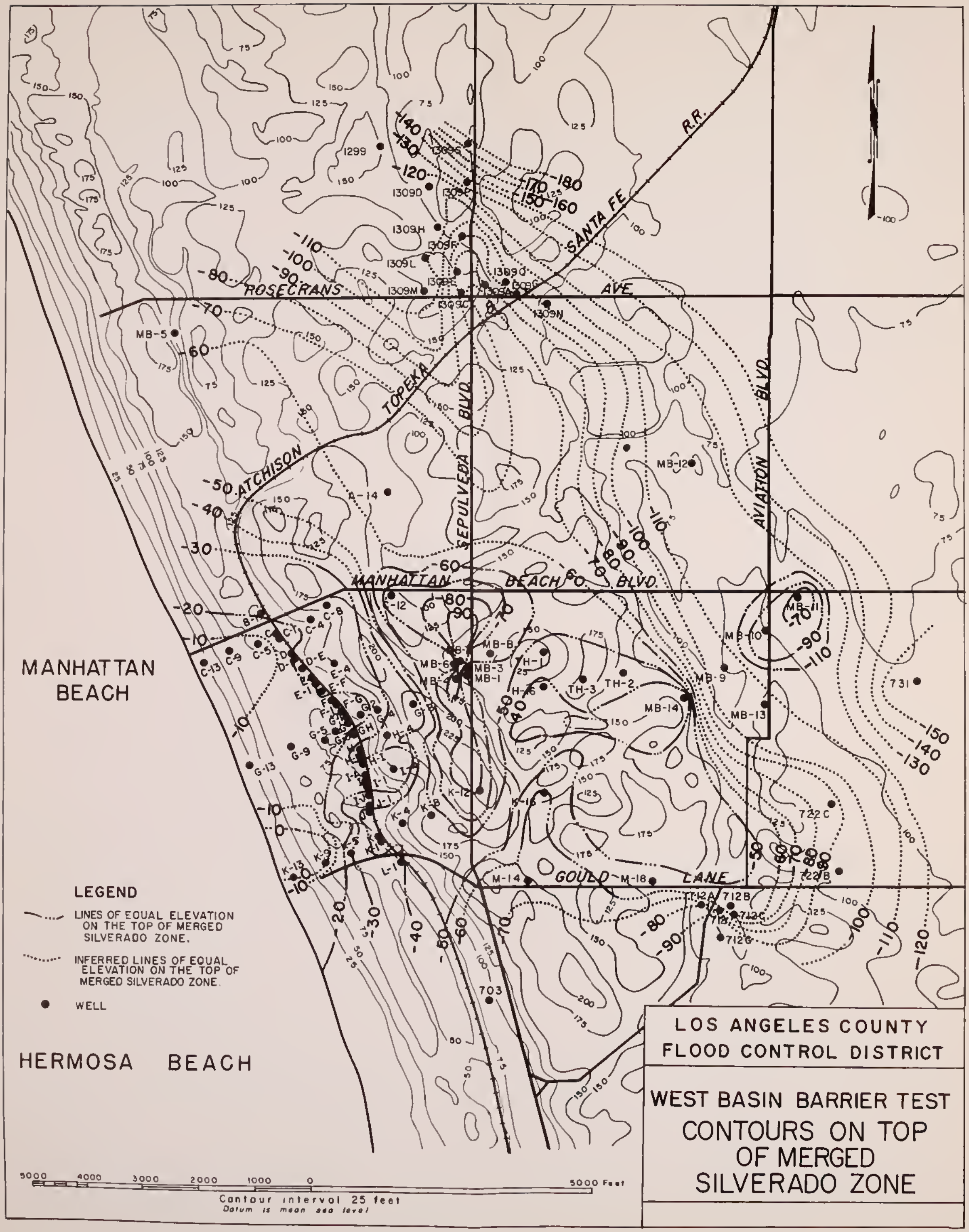
ELEVATIONS IN FEET
USGS DATUM

LOCATION OF SECTIONS
SEE PLATE 1

GEOLOGIC SECTION 14-14

LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
GEOLOGIC SECTIONS
4-4 AND 14-14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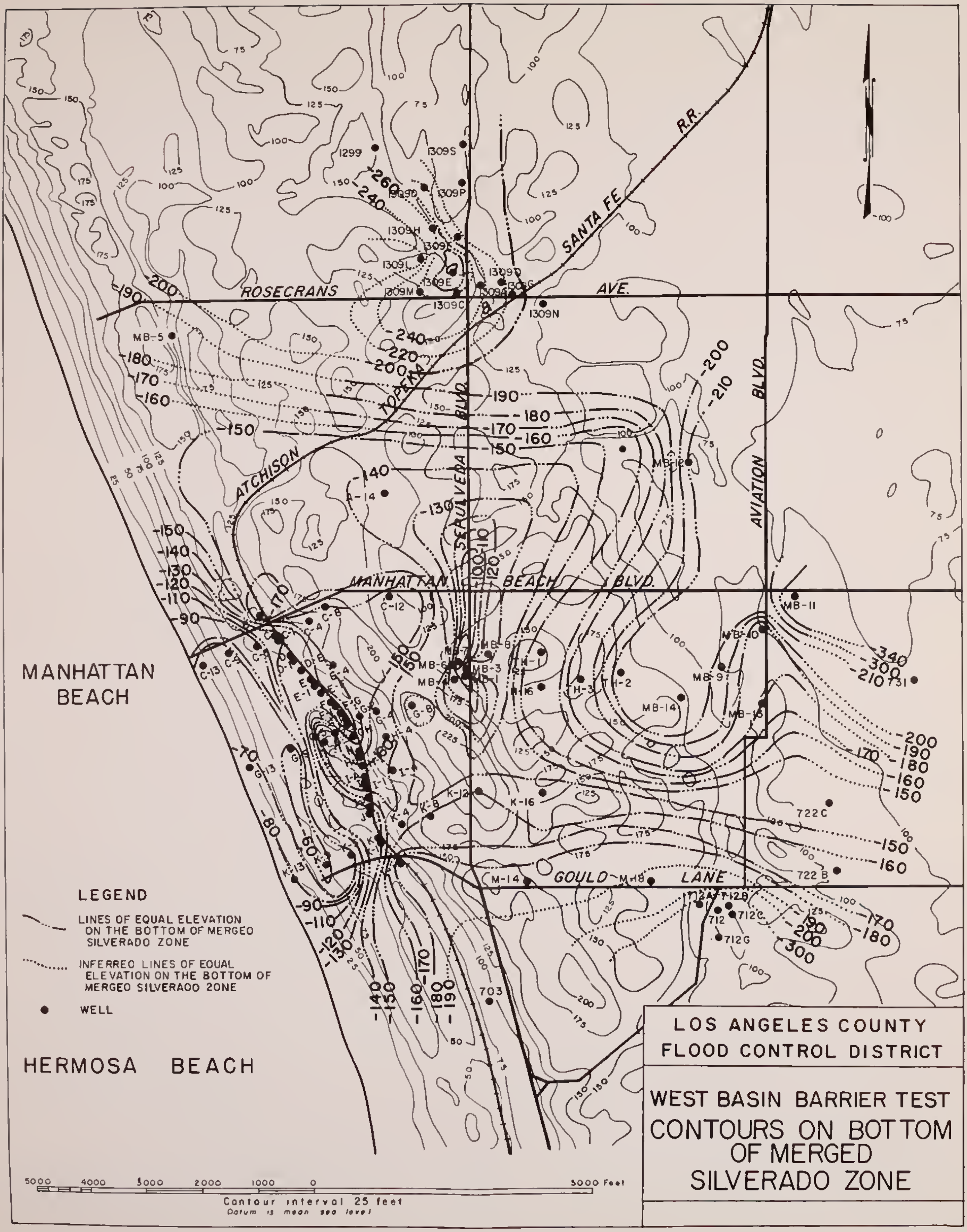
SANTA FE R.R.
ATCHISON TOPEKA
SEPULVEDA BLVD.
AVIATION BLVD.
MANHATTAN BEACH BLVD.
GOULD LANE
MANHATTAN BEACH
HERMOSA BEACH
LEGEND
LINES OF EQUAL ELEVATION ON THE TOP OF MERGED SILVERADO ZONE.
INFERRED LINES OF EQUAL ELEVATION ON THE TOP OF MERGED SILVERADO ZONE.
WELL
Contour interval 25 feet
Datum is mean sea level
5000 4000 3000 2000 1000 0 5000 Feet
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
CONTOURS ON TOP OF MERGED SILVERADO ZONE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GOULD LANE
MANHATTAN BEACH
HERMOSA BEACH
LEGEND
LINES OF EQUAL ELEVATION ON THE BOTTOM OF MERGED SILVERADO ZONE
INFERRED LINES OF EQUAL ELEVATION ON THE BOTTOM OF MERGED SILVERADO ZONE
WELL
5000 4000 3000 2000 1000 0 5000 Feet
Contour interval 25 feet
Datum is mean sea level
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
CONTOURS ON BOTTOM OF MERGED SILVERADO ZONE

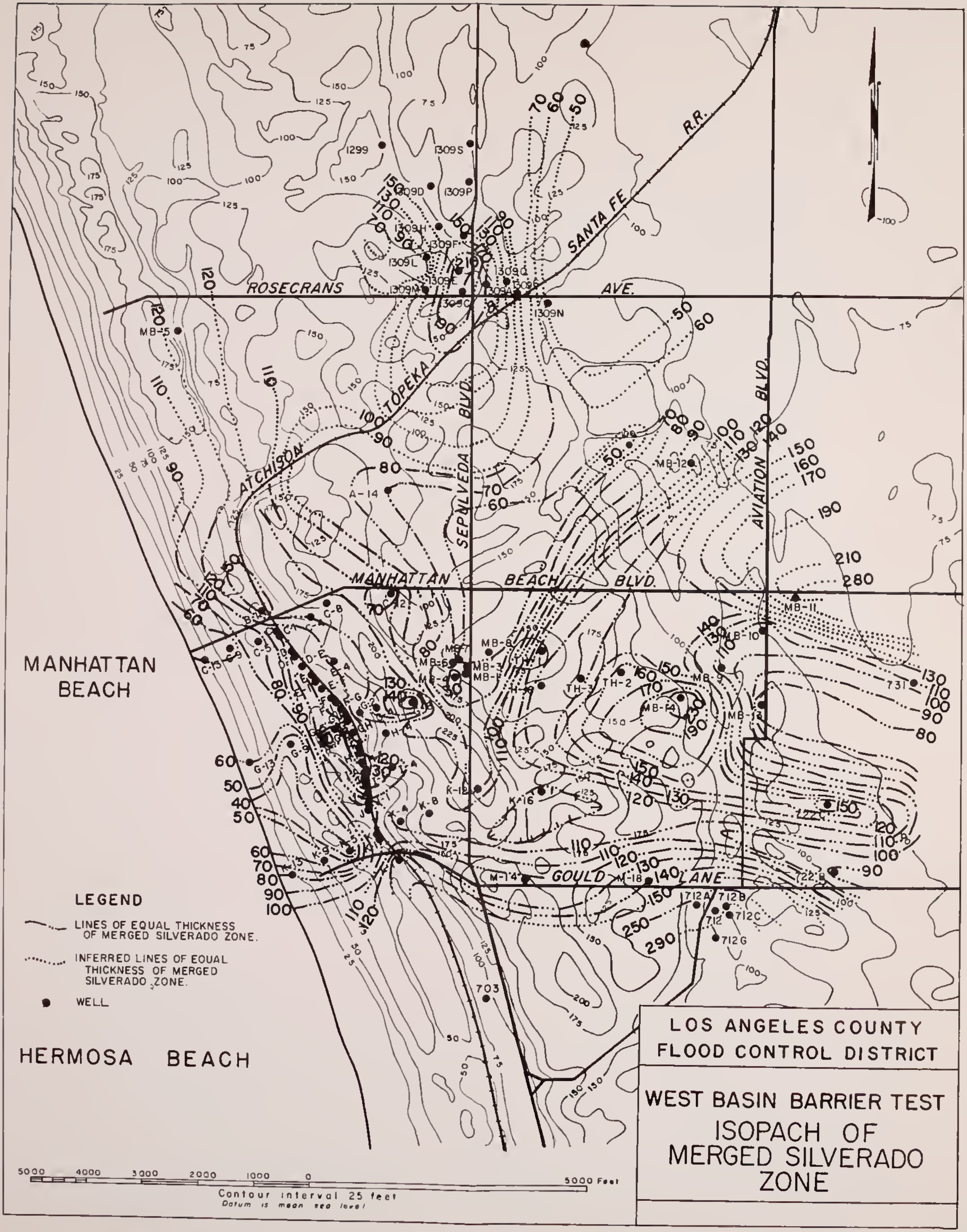

MANHATTAN BEACH
HERMOSA BEACH
ROSECRANS AVE.
MANHATTAN BEACH BLVD.
GOULD LANE
SEPULVEDA BLVD.
AVIATION BLVD.
ATCHISON TOPEKA SANTA FE R.R.
LEGEND
LINES OF EQUAL THICKNESS OF MERGED SILVERADO ZONE.
INFERRED LINES OF EQUAL THICKNESS OF MERGED SILVERADO ZONE.
WELL
5000 4000 3000 2000 1000 0 5000 Feet
Contour interval 25 feet
Datum is mean sea level
LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
ISOPACH OF MERGED SILVERADO ZONE

TABLE I

PEBBLE COUNT OF AQUIFER GRAVELS—WEST BASIN BARRIER TEST

Upper Brown Phase Well Number	K-8	A-14	K-12	M-14	K-16	Accum. Avg.
Elevation	—73	—62	—79	—53	—75	
Rock type*						
Granitoids	53.7	61.6	50.0	55.3	76.8	56.5
Volcanics	6.0	6.8	8.4	12.0	2.5	8.3
Metamorphics	39.8	29.1	41.6	29.9	30.6	34.0
Sediments	0.0	2.6	0.0	3.0	0.0	1.1
Total %	99.5	100.1	100.0	100.2	99.9	99.9

Lower Gray Phase Well Number	C-12	A-14	K-12	M-14	H-16	M-18	Accum. Avg.
Elevation	—73	—84	—94	—83	—159	—68	
Rock type*							
Granitoids	67.2	56.2	74.0	74.2	26.6	38.1	64.3
Volcanics	1.4	5.5	4.6	5.7	14.0	10.5	5.4
Metamorphics	31.5	38.4	20.5	19.3	41.1	51.4	27.6
Sediments	0.0	0.0	0.9	0.6	18.3	0.0	2.4
Total %	100.1	100.1	100.0	99.8	100.0	100.0	100.1

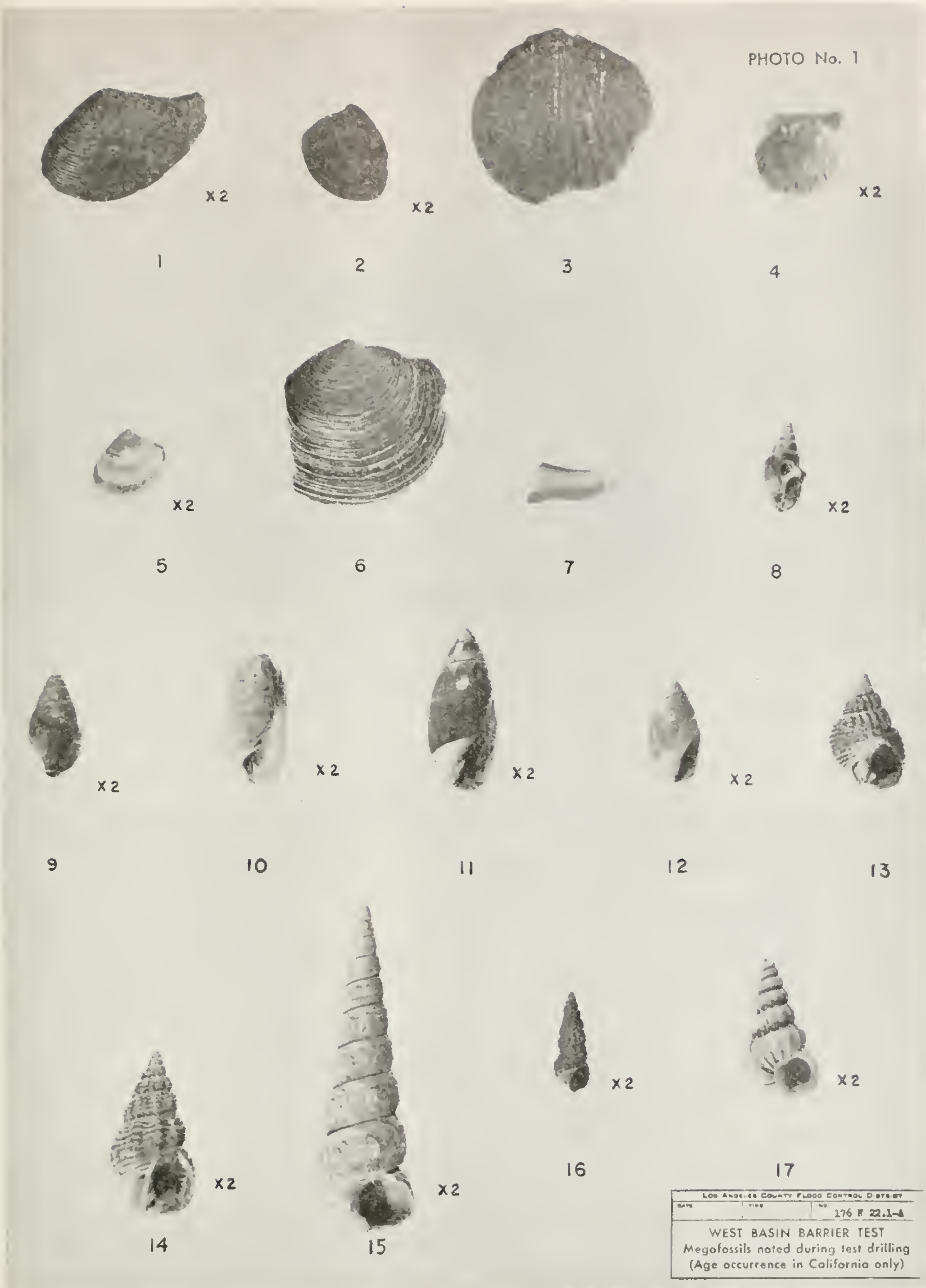
Lower Gray Phase—Continued Well Number	C-8	C-12	H-16	M-18	Accum. Avg.
Elevation	—76	—89	—206	—141	
Rock type*					
Granitoids	65.0	77.2	83.5	80.8	Included above
Volcanics	1.1	1.6	3.2	6.5	
Metamorphics	31.9	21.3	8.9	11.5	
Sediments	2.1	0.0	4.5	1.3	
Total %	100.1	100.1	100.1	100.1	

* Granitoids include monzonite, granite, granodiorite, diorite, gabbro, and vein and pegmatitic quartz. Volcanics include basalt, rhyolite and felsite. Metamorphics include gneiss, schist, and quartzite. Sediments include shale, sandstone, and chert.

MEGAFOSSILS NOTED DURING TEST DRILLING

(Age occurrence in California only)
(See Photo 1)

	Species	*Range*
1.	Nuculana taphria (Dall)	Miocene—Recent
2.	Acila (Truncacila) castrensis (Hinds)	Miocene—Recent
3.	Pecten (Patinopecten) caurinus Gould	Pliocene—Recent
4.	Pecten (Pecten) caurinus Gould	Upper Miocene—Recent
5.	Transennella tantilla (Gould)	Pleistocene—Recent
6.	Lucina (Myrtea) acutilineata Conrad	Miocene—Recent
7.	Solen sicarius Gould	Miocene—Recent
8.	Mitrella carinata (Hinds) var. gausapata (Gould)	Pliocene—Pleistocene
9.	Mitrella tuberosa (Carpenter)	Upper Miocene—Recent
10.	Retusa (Acteocina) culcitella (Gould)	Pliocene—Recent
11.	Olivella pedroana (Conway)	Miocene—Recent
12.	Olivella biplicata (Sowerby)	Pliocene—Recent
13.	Nassarius (Schizopyga) perpinguis (Hinds)	Miocene—Recent
14.	Nassarius (Schizopyga) mendicus (Gould)	Miocene—Recent
15.	Turritella cooperi Carpenter	Upper Miocene—Recent
16.	Bittium (Semibittium) rugatum Carpenter	Pliocene?—Recent
17.	Epitonium catalinae Dall	Pliocene?—Recent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LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
176 F 22.1-A
WEST BASIN BARRIER TEST
Megofossils noted during test drilling
(Age occurrence in California only)

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
TESTING DIVISION

APPENDIX B

LABORATORY STUDIES AND RESEARCH RELATIVE TO INVESTIGATIONAL WORK FOR THE PREVENTION AND CONTROL OF SEA WATER INTRUSION

December 20, 1954

By CHARLES GREEN

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Recommended by:
PAUL BAUMANN
Assistant Chief Engineer

Approved by:
H. E. HEDGER
Chief Engineer

TABLE OF CONTENTS

LABORATORY STUDIES AND RESEARCH RELATIVE TO INVESTIGATIONAL WORK FOR THE PREVENTION AND CONTROL OF SEA WATER INTRUSION

A. SUMMARY

This Appendix is a report on the laboratory studies and research conducted by the Testing Division of the Los Angeles County Flood Control District as a part of its responsibility in formulating plans and design criteria for the correction or prevention of damage to underground water by sea-water intrusion in the West Coast Basin of Los Angeles County.

The tests procedures are described, results indicated and conclusions presented. The summary of conclusions being:

1. The tests establish the fact that the source of saline pollution of the groundwater at the West Basin Barrier Test site is sea-water.

2. The saline water has converted the soil in contact with it to a high sodium soil through cation exchange. The degree of conversion is in proportion to the concentration of the saline water and therefore decreases with increased distance from the shore line.

3. As a consequence of recharge, the sodium saturated soil has given up its exchangeable sodium to saline water as its dilution with recharge water increases. As the recharge water displaces the saline water completely this exchange continues as long as enough exchangeable sodium is available in the soil.

4. The changes in permeability resulting from the above cation exchange processes in the soil are obscured by other factors difficult to evaluate.
5. The optimum chlorine dosage was best determined by the maintenance of maximum acceptance rather than by chlorine demand evaluations or bacteriological analyses.

6. The effect of suspended solids in the recharge water on acceptance rate could not be evaluated.

B. INTRODUCTION

The purpose of this report is to present information obtained from the laboratory research studies of the general program conducted by the Los Angeles County Flood Control District. This program was undertaken as a result of an agreement, entered into by the State Board of Water Resources and the Los Angeles County Flood Control District, to perform investigational work for the prevention and control of sea-water intrusion.

C. SCOPE OF REPORT

The laboratory research studies involved the conduction of experimental tests and investigation of phenomena resulting from prototype application of injection wells toward creating a pressure ridge in confined aquifers to prevent and control this sea-water intrusion. The data obtained has been tabulated, assembled and evaluated where feasible for the items that were investigated. A brief outline of these investigations follows:

1. Evaluation of Water Quality
   - a) Pre-recharge analyses
   - b) Quality changes resulting from recharge

2. Origin of Various Types of Groundwater Present in the Area
   - a) Criteria used for determining pollution sources
   - b) Special tests

3. Effects of Deleterious Constituents in Ground and Recharge Waters
   - a) Iron
   - b) Calcium carbonate (sludge)
   - c) Bacteria
   - d) Suspended solids

     Under this heading is included deleterious microbiological and chemical constitutents natively present in the merged aquifer and also in the recharge water.

4. Ion Exchange

   Ion (base) exchange reactions will be covered in the "Evaluation of Water Quality" section of this report and in other sections where the subject is pertinent.

5. Laboratory Soil Permeability Tests

   This consists of a compilation of laboratory permeability and supplementary test data. Evaluation is presented where permeability is related to cation exchange.

6. Control of Chlorine Dosage

   An evaluation of methods for control of chlorine dosages is presented. This evaluation is based on an investigation of the effect of suspended solids and micro-organisms in water, on the percolation rate through Ottawa Sand.

D. DESCRIPTION OF TESTS AND METHODS

1. Water Quality Analyses

a) Routine Analyses

Samples received from the West Coast Basin Barrier Test were subjected, in the greater majority of the cases, to complete analyses which included the determination of the following items:

(1) Calcium
(2) Magnesium
(3) Sodium with Potassium (by calculation)
(4) Carbonate
(5) Bicarbonate
(6) Chloride
(7) Sulfate
(8) Ammonium
(9) Nitrates
(10) Total dissolved solids by evaporation at 103°C
(11) Total dissolved solids by summation of items (1) to (9) inclusive
(12) Hardness
(13) pH
(14) Electrical conductivity—micromhoes per cm at 25° C.
(15) Description of sample: turbidity, sediment, color and odor.

Occasionally where a complete analysis was thought unnecessary a partial analysis was made in which only items 4, 5, 6, 7, 10, 12, 13, 14 and 15 were determined.

b) Special Analyses

Certain constituents not included in the above list of routine determinations were analized in specific instances.

(1) Bromide, borate and iodide ions were determined in samples from Wells C-12, Manhattan Beach #4 and K-12 with the view to determining polluting source, i.e., connate oil brine versus sea-water.

(2) Iron was determined in some water samples in order to evaluate effects of corrosion.

(3) Dissolved oxygen determinations were made on samples of recharge water and pumped native water in order to evaluate the degree of air entrainment. These tests were conducted at Manhattan Beach.

(4) Other tests on the recharge water, performed at Manhattan Beach, were pH, chlorine residual and chlorine demand. Wherever possible the "project personnel" were instructed in the methods of performing the above tests, and the data obtained by them are not included in this report though reference may be made to these data.

(5) Analyses were made of residues and deposits taken from valves, meters, chlorinator equipment and other locations.

2. Method of Reporting Analytical Results

The "water quality reports" on the West Coast Basin Barrier Test water samples, submitted periodically, presented the above itemized constituents, where applicable, in terms of parts per million by weight (ppm) and as percent equivalents per million (%epm). To complete the information in terms of equivalents per million (epm), the sum of the major ions as equivalents per million has been recently included in the water quality reports.

The purpose of reporting percent epm is that it reduces the analyses to a form which may be plotted on a tri-linear chart. The value of plotting the data on a tri-linear chart graphically, to determine the source of pollution or dilution, is described in more detail later. Percent epm further provides a means for quantitatively measuring cation exchange in water.

3. Constant Head Permeameters

Constant head permeameters were used to determine permeabilities of soil samples obtained during drilling of the West Coast Basin Barrier Test wells.

Initially the permeameters were machined from steel and were either black oxide finished or chrome plated to inhibit corrosion. This protection was soon found to be inadequate. The products of corrosion formed in the presence of the highly saline water in the samples tended to clog the sample and give decreasing values of permeability. In addition, it was found that the porous plate supporting the sample eventually became clogged either with the products of corrosion or suspended material in the water. The permeability values reported are the first values obtained in the test after correction was made for porosity of the porous plates.

In order to correct the deficiencies apparent in the permeability apparatus as originally designed, the permeameters were redesigned along the lines indicated in Figure 1. The features which have been incorporated in the new design are:

a) All plastic construction

b) Piezometer connections at one-inch intervals along the length of the permeameter cylinder

c) Means for evacuating the sample to prevent decreased permeabilities due to entrained air

Sea-water, filtered through diatomaceous earth under vacuum was used in the tests. The water was filtered to minimize the effect of suspended solids, micro-organisms and air in the water. Sea-water was used to duplicate the conditions existing in the aquifer prior to recharge.

In conjunction with permeability tests, ion exchange phenomena were quantitatively determined by chemical analyses of the calcium, magnesium and chloride ions in the influent and effluent. Two fluids were used in these determinations: filtered sea-water followed by filtered Metropolitan Water District water, both obtained at Manhattan Beach. Changes in permeability were noted as sea-water was replaced by M.W.D. water.

4. Falling Head Permeameters

Two percolation tubes, each consisting of a 1½″ I.D. lucite cylinder 2 feet long, closed at the bottom with a fine screen and approximately half filled with a weighed amount of screened, sterile Ottawa Sand, were set up in the chlorine storage shed at Manhattan Beach.

Water taken from the line leading to the chlorinator (pre-chlorinator water) passed continuously through one cylinder; and, water taken from the line leading away from the chlorinator (post-chlorinator water) passed through the second cylinder. The cylinders were supported on a board with a third cylinder placed between them similarly loaded but closed at the bottom and filled with water. This cylinder was included as part of the apparatus arrangement to furnish a visual comparison with the other cylinders.

Permeabilities were determined periodically by considering the percolation tubes as falling head permeameters. Although this assumption is not entirely valid, the results obtained are sufficiently close to the true values to be comparable with each other.

E. PRESENTATION OF DATA AND RESULTS

1. Water Quality Evaluation

a) Pre-recharge conditions

The selection of analyses to represent pre-recharge conditions for each observation and recharge well was determined by the following considerations:

(1) Proximity of sample date, to date of recharge in the nearest recharge well so that the sample is not influenced by recharge.

(2) "Typicalness" of sample as determined by comparison with analyses of other samples from the same well taken at approximately the same time.

(3) Proximity of the chloride concentration of the sample with the chloride concentration selected as representative of pre-recharge concentrations.

(4) Availability of analyses that would fill the above conditions. In case one was not available a United States Geological Survey analysis was taken to represent the pre-recharge state.

Pre-recharge analyses are tabulated in Table I. The calculated and analysed percent epm for each well are given in order to determine the loss or gain of cations due to cation exchange. This ininformation is listed in the three columns under cation exchange. Negative values indicate migration of cations from water to soil* and positive values the reverse. Percent dilution in the fifth column is determined from the chloride concentrations of native and sea-water as shown in Table I. Percent dilution in turn was used to calculate cation percent epm. The native water analyses given in Table I is the mean of the analyses of samples taken from wells G-8, H-16, K-12, Test Hole No. 2 and General Chemical Well No. 4.

In order to determine the degree of saturation of the soil with sea-water at each well, the Sodium Absorption Ratio (SAR) is first calculated from the relation:[1]

$$SAR = \frac{Na}{\sqrt{(Ca + Mg)\ /2}}$$

where Na, Ca and Mg are the analysed epm of sodium, calcium and magnesium in the water in contact with the soil. The above relationship combines the effect of total cation concentration and the ratio of sodium to calcium plus magnesium, each of which influence the equilibrium reached between the soil and the soluble cations in the water. Since a linear relationship exists between SAR and the ratio of exchangeable sodium to exchangeable calcium plus magnesium pres-

* The term "soil" as used in this report refers to the unconsolidated deposits present in the merged Silverado Zone which have cation exchange properties.

[1] See Bibliography.

ent in the soil,[1] a similar relationship exists between *SAR* and the degree to which the soil is saturated with exchangeable sodium.

Therefore one hundred percent saturation of the soil is arbitrarily taken as the condition where the exchange reaction has ceased and the intruded sea-water is unaltered; zero percent saturation is where only native water exists. Between these two limits the percent saturation existing at each well is interpolated using the *SAR* values for each well.

The analysed percent epm data in Table I are plotted on a tri-linear chart (Figure 2). Zones delineated in the tri-linear chart, representing different ranges of water quality distinct from concentration are then geographically represented on the map of the West Coast Basin Barrier Test (Figure 3). The chloride concentration, tabulated both in ppm and as a decimal ratio to sea-water is indicated on this map as the decimal ratio. The groundwater-sea-water chloride ratio referred to, has been selected as a parameter rather than the chloride concentration in ppm, since it is then possible to correlate and amplify concentration data with a similar ratio of electrical conductivity.

b) Effect of Recharge

Well GH and the G line of wells, G-2, G-4 and G-8 have been selected to show change of water quality and concentration with recharge. The data tabulated in Table 2-A for Well GH are plotted in Figure 4. The recharge rates in adjacent wells G and H are plotted to show their influence on concentration, degree of dilution and water quality. The difference between calculated and analysed percent epm of the cations are plotted to show cation exchange effects. The tri-linear plotting of analysed percent epm is shown in Figure 5. This plot shows the water quality "path" taken by the water in well GH as recharge progressed. The data tabulated in tables 2B, 2C and 2D for wells G-2, G-4 and G-8 are plotted in Figures 6 and 7. Figure 6 shows the change in the concentration (as epm) of the anions and cations in each well with increased dilution. Each triangle can be considered as an integration of several bargraphs with the anion and cations bars separated. Figure 7 shows the variation in cation exchange, as percent epm, with dilution for each well. Dilution is represented in absolute terms of equivalent per million so that the products of ordinate and abscissa equals exchange as equivalents per million.

Tri-linear chart (Figure 8) is drawn to show the sources of dilution in wells K-8 and K-12.

c) Results of Special Analyses

(1) Source of Pollution

Table 3 presents the results of special analyses conducted to determine polluting source in Wells C-12, MB-4 and K-12.

For comparison, analyses of sea-water and connate waters are included.[2]

(2) Dissolved Oxygen in Groundwater

Table 4 lists the results of dissolved oxygen determinations on pump discharges.

(3) On four occasions recharge water was analysed in the field for pH, dissolved oxygen and residual chlorine. The results are tabulated in Table 5.

(4) Iron in Ground and Recharge Waters

A series of tests involving the detection of iron in ground and recharge waters was conducted during the course of the West Coast Basin Barrier Test.

One test was to determine the source of iron present in Well G following the cave-in. Table 6 presents the data obtained during this test.

A second test was conducted when the following reaction was noted during the course of a field chlorine demand test. A 160 ppm chlorine solution precipitated an iron hydroxide floc when added to clear thiefed samples. The samples were filtered and sent to the laboratory with samples of unfiltered (raw) water from the same sources. The results of the iron determinations on the samples are shown in Table 7.

Samples of M.W.D. water were taken from the "vault" at Manhattan Beach Boulevard and Redondo Boulevard and at the Field House in order to determine the iron picked up in the feeder line. The results of the test are presented in Table 8.

[1] See Bibliography.

[2] See Bibliography.

(5) Analyses of Deposits and Sludges

The laboratory has received samples of deposits and sludges found on equipment and in the feed lines. The results of the analyses of these are shown in Tables 9, 10 and 11. Table 12 contains the analyses of the water found in the chlorinator bell-jar from which the sludge reported in Table 11 was taken. For comparison an analyses of the recharge water is included.

2. Constant Head Permeameter

a) The permeability results of soil samples de-determined in the steel permeameters are reported in Table 13. Those determined in the lucite permeameters are reported in Table 14. The tests performed in the lucite permeameters were conducted using sea-water. M.W.D. water was used in some of the tests after the application of sea-water.

The permeability values listed in these tables are based on a hydraulic gradient of unity.

b) Influent and effluent fluids were chemically analysed in some of the tests reported in Table 14 to determine relationship of permeability to ion exchange.

The complete analytical and permeability data for the soil sample from Well G-8 are tabulated in Table 15 and indicated graphically in Figure 9.

3. Supplementary Soil Test Data

Table 16 summarizes all the data (other than permeability) obtained from tests on disturbed and undisturbed soil samples. These data include, percent moisture, dry density, specific gravity, porosity, void ratio and sieve analyses.

4. Falling Head Permeameter

The results of the permeability runs made on the "pre-chlorinator" and "post-chlorinator" percolation tubes installed at Manhattan Beach are reported in Table 17 and shown in Figure 10.

F. DISCUSSION OF RESULTS

1. Preliminary discussion of geochemical changes.

Before entering the discussion of the geochemical changes that apply to the intrusion and recharge phases of the West Basin Barrier Test, a brief discussion of the geochemical changes that occur in groundwater is in order.

The following has been taken from various sources.[3,6] The geochemical changes that occur in groundwater result from:

(1) Evaporation near the surface
(2) Solution of soil minerals
(3) Precipitation of salts
(4) Ion exchange between cations in water and in the soil
(5) Sulfate reduction from microbiological activity
(6) Admixture with waters of different types and concentrations

a) Evaporation, Solution and Precipitation

These processes affect the amounts of anions and cations present. These processes are not important in the geochemical changes occurring during sea-water infiltration or recharge.

b) Ion Exchange

This process affects only the cations present in water. The sum of the cations as equivalents per million will not be influenced, though the amounts of each cation will be altered because of ion exchange. The relative ion exchange activities of the three dominant cations in water are: calcium > magnesium > sodium, that is, when the concentration of each cation is relatively low, the tendency is for the calcium to leave the water and enter the soil and sodium to enter the water from the soil. The reverse will occur when sodium is present in water in great excess as it is in sea-water. Both these tendencies are readily apparent in the analyses of the West Coast Basin Water Samples, the first during recharge and the second prior to recharge. The influence of cation exchange on permeability will be included in the discussion of permebility.

c) Sulfate Reduction

This process which affects only the anions, sulfate and bicarbonate (the latter increasing at the expense of the former), requires the presence of both sulfur reducing bacteria and organic matter. This process is not active in the geochemical changes observed in the West Coast Basin Barrier Test.

d) Mixture of Waters of Different Types and Concentrations

When groundwaters mix, little if any reaction will occur between them and the only change will be that of concentration. When two waters of different types are plotted on

[3, 6] See Bibliography.

tri-linear charts (see Figures 5 and 8) a line drawn between them will be the locus of all waters that could result from mixing the two waters. In Figure 5 the recharge (M.W.D.) water is represented by point "R" and the pre-recharge analyses of Well G-H by point "1". The dotted line connecting these two points contains all the mixtures that could result from mixing "R" and "1". However this relationship is valid only when geochemical changes resulting from cation exchange (affecting only cations) and sulfate reduction, (affecting only anions), do not occur. In fact any deviation from this straight line relationship is an indication as well as a measure of the geochemical changes that result from ion exchange, sulfate reduction, precipitation and solution. This straight line relationship between "original" and "diluting" waters is also used to determine which one of two or more "original" (or "diluting") waters when mixed with a known "diluting" (or "original") water yields a known mixture. Where sulfate reduction is known to have little or no effect on the water the anion triangle can be used for this purpose. By this means it is possible to determine whether a known connate or sea-water is responsible for an increase in chlorides, or whether a reduction in chlorides has resulted from dilution by native or recharge water.

Although a mixture may not be altered by sulfate reduction, if one of the "original" waters has been so altered the resulting mixture may deviate from the straight line established by the "original" waters when plotted on the tri-linear graph. The reason for this is that connate waters which contain very little to no sulfate ions due to sulfate reduction may pick up by solution, appreciable quantities (24 to 142 ppm) [2] of barium. Because of the high degree of insolubility of barium sulfate it is not usually present in waters containing any sulfate ions. When connate water containing sufficent barium ions is mixed with normal groundwater containing sulfate ions, barium sulfate will precipitate thus depleting the concentration of sulfate ions. This depletion will be reflected as a deviation from the mixture line on the anion side of the tri-linear chart.

Of the 6 major ions in groundwater, calcium, magnesium, sodium, bicarbonate, sulfate and chloride, only the chloride ion is unaffected by either ion exchange or sulfate reduction and thus becomes a reliable parameter for measuring sea-water intrusion or effect of recharge.

The tri-linear chart referred to, is a means of graphically characterizing water quality. This form of tri-linear plotting is described in a paper by Raymond A. Hill previously referred to.[6] Cations in %epm can be represented as a point on the cation triangle and similarly, the anions on the anion triangle. The combination of anion and cation is represented on the "diamond" as a point of intersection of lines drawn from the cation and anion points parallel to the sodium and chloride base lines respectively. This is shown on Figure 2 for Well TH-3.

2. Geochemical Changes in Groundwater and Soils Resulting from Sea-Water Intrusion

The quality of the groundwater at the Manhattan Beach site of the West Coast Basin Barrier Test has been altered as the result of the admixture of native groundwater and sea-water, and also cation exchange. The concentration as determined from chloride analyses ranges from essentially sea-water near the ocean to slightly polluted water near Sepulveda Boulevard. Parallel and indirectely related to the change in concentration is the change in water quality resulting from cation exchange. Cation exchange was evident; though slight near the ocean it increased rapidly landward with the decrease in concentration of chlorides. This change is reflected in the decrease in sodium and increase in calcium in the water and the decrease in percent saturation of the soil. Seaward the soil containing montmorillonitic minerals responsible for cation exchange, have by long exposure to sea-water become saturated with exchangeable sodium. Recently intruded sea-water exposed to the sodium saturated soil will show little or no exchange of sodium in the only direction possible for sea-water, i.e., water to soil. However inland the soil becomes progressively less saturated with exchangeable sodium thus permitting cation exchange to increase landward. This cation exchange phenomenon is demonstrated in Table 1 and Figure 2. The zones indicated in Table 1, Figure 2 and the map in Figure 3 were arbitrarily defined on the basis of %epm of calcium as follows:

Zone I, to 2.50 %epm calcium, includes 13 wells and sea-water, % saturation 89-100
Zone II, 2.5 to 4.0 %epm calcium includes 9 wells, % saturation 85-94
Zone III, 4-10 %epm calcium includes 7 wells, % saturation 52-79

[2] See Bibliography.

[6] See Bibliography.

Zone IV, 10-20 %epm calcium includes 8 wells, % saturation 7-27

Zone V, 20% plus epm calcium includes 7 wells, % saturation 0-9

When the analyses (in terms of %epm) of the water in the wells in each zone are plotted on the tri-linear chart of Figure 2, the boundaries of each zone are well defined with certain exceptions as follows:

a) Zone 1-A—Well K-9; geographically this well is in Zone 1; on the basis of the chloride concentration it should be in Zone V. It is the only well where cation exchange is in the reverse direction, that is the direction of sodium is soil to water.

This well contains water with a sodium concentration as ppm about 8 times that of its combined calcium and magnesium concentration. The reason for this is that the water has been exposed to a soil saturated with sodium. A high sodium soil, as has already been stated, is indigenous geographically, to Zone I. The source of the water originally was meteorological, garden or lawn or possibly septic tank or cess-pool. The low nitrate and ammonium content renders the last two possibilities doubtful.

b) Zone IV-A, Wells TH-2, TH-3, H-16; these 3 wells on the bases of water quality, geographic contiguity and chloride concentration, belong in one group. They are quite distinct from the cluster of six Zone V wells to the northwest. They contain native water in two of the wells and slightly polluted native water in the third.

c) Zone V-A, Well M-18; this well is very closely associated to the ones in Zone V. It is however, less polluted with sea-water than the Zone V wells. The bicarbonate content is correspondingly higher and is responsible for setting this well slightly apart from the Zone V wells.

Examination of Table I and Figure 2 yields the following additional information concerning the groundwater in this area:

a) Although sodium in the water shows a definite tendency to replace the calcium in the soil, magnesium (with sodium) may replace the calcium at one location and be replaced by sodium at another location. The reason for this inconsistent behavior of magnesium is because of its relatively high concentration in sea-water compared to calcium (calcium (ppm) : magnesium (ppm) = 1:3). It will therefore have the tendency under certain conditions to be absorbed by the soil minerals along with sodium at the expense of calcium. Experiments in which soil was mixed with sea-water showed the absorption of both sodium and magnesium.[7] The results are presented in the following table.

	Percent Exchangeable Cations in Soil			
	Calcium	*Magnesium*	*Potassium*	*Sodium*
Before Treatment	71.7	19.8	4.5	4.0
After Treatment	20.3	37.1	6.7	35.9
Percent Change	—51.4	+17.3	+2.2	+31.9

It is apparent that there is as much exchangeable magnesium present as exchangeable sodium in the treated soil, and that the increase in percent, of exchangeable magnesium (17.3%) is appreciably less than the gain or loss of exchangeable sodium and calcium (31.9%, 51.4%). This explains why magnesium does not show as much exchange activity as the other two cations. The saturation of soil with magnesium occurs before saturation of the soil with sodium, and as the absorption of sodium progresses the absorption of magnesium decreases to zero, and magnesium may then be displaced by sodium. Table I illustrates the changes in the direction that magnesium takes with changes in degree of saturation. In more highly saturated Zone I the tendency is for magnesium to be displaced by the sodium. In less saturated Zones II and III the proportion of wells in which magnesium displaces calcium increases. Zone V wells are the exception. The magnesium is displaced by sodium in all the wells. During the 1950 pilot recharge test one of the wells in Zone V, M.B. No. 7 was used as the recharge well, and four of the other wells in this zone were used as observation wells. This might have altered the exchangeable cation status of the soils in Zone V.

b) The sulfates show a narrow range from 1.85 %epm to 5.60 %epm. In the higher concentrations this range is even narrower. This slight variation in %epm of sulfate is an indication that the polluting source is sea-water rather than connate water in all the wells listed in Table I.

c) The concentration of wells into small contiguous zones on the anion side of Figure 2 as compared with the much greater spread on the cation side is an indication as to the relative changes in water quality resulting from dilution of sea-water with and without the influence of ion exchange.

[7] See Bibliography.

The source of the pollution in all the wells is believed to be sea-water as indicated in the above paragraph. The analyses of the trace elements as borate, iodide and bromide (see Table 3), in Wells C-12, MB-4 and K-12 confirms this to some degree.

The amount of borate ion in sea-water is within the connate range but close to the minimum. The borate found in Wells C-12, MB-4 and K-12 is even less than what should be expected in sea-water when diluted to the chloride content in each sample. This is not definite proof of sea-water pollution as against connate water pollution.

The iodide content in sea-water is considerably below the range found in connate waters. The iodide found in Wells MB-4 and K-12 within the limits of analyses, corresponds to the theoretical iodide concentration for sea-water pollution. The iodide in Well C-12 is above this theoretical value but still well below the connate range. This is a more conclusive proof of sea-water pollution.

The bromide concentration in sea-water like the borate is within the connate range but toward the lower limit. Considering the range of bromide possible if connate water was the source, good correlation therefore exists between theoretical sea-water concentration and actual concentration particularly in Well C-12.

The %epm for sulfate in these wells is well above the connate range and approaches the %epm for sea-water. Very good correlation exists in the chloride %epm between the well analyses and sea-water.

3. Geochemical Changes in Groundwater and Soils Resulting from Recharge

a) Well G-H

The quality of the water in Well G-H was altered by the recharge in the adjacent Wells G and H. The rapid decrease in concentration shown in Figure 4 between June and September of 1953 reflects the increase in recharge rates in this period. The fluctuation in concentration between September of 1953 and February, 1954, was caused by the temporary cessation of recharge in Well H and a subsequent decrease in Well G. It is apparent that an appreciable lag exists between changes in recharge rates and concentration. The increase in concentration in November and December of 1953 resulted from the return of the more saline water which earlier had been pushed toward Well H. The resumption of a steady rate of recharge eventually succeeded in slowly pushing this saline water either seaward or inland.

With recharge the cation exchange reverses direction. At first it is mainly between sodium and calcium with the calcium entering the soil at the expense of the sodium. At a concentration of about 9000 ppm of chlorides, magnesium enters into the exchange displacing the sodium. With further dilution the exchange increases, with magnesium showing greater activity than calcium. The direction taken by magnesium (water to soil) is the same for recharge and pre-recharge conditions (except, as noted, for saturated conditions, i.e., Zone I) with the greater activity displayed during recharge. This unidirectional activity of magnesium is explained by the fact that sodium is no longer competing with magnesium in the exchange reaction but, in fact, is now on the opposite side of the exchange equation, viz:

Pre-recharge—

$$Na^{+} + Mg^{++} + CaX \rightleftharpoons NaX + MgX + Ca^{++}$$

Post-recharge—

$$Ca^{++} + Mg^{++} + NaX \rightleftharpoons CaX + MgX + Na^{+}$$

where cations in solution are represented as Na^{+} etc. and cations combined with soil is represented as NaX etc. Magnesium, being the more abundant cation of the two, enters into the exchange reaction to a greater degree than calcium.

The tri-linear chart Figure 5, shows the water quality "path" taken by the ions as recharge progresses. The deviation from the straight line defined by the original water in G-H and the recharge water (points "1" and "R" in the cation triangle) clearly shows the influence of cation exchange. No such deviation appears on the anion side, the points following very closely the mixing line as anticipated.

b) G-Wells

The analyses of the change in the quality of the water with dilution in the G Wells as shown in the integrated barograph, (Figure 6), demonstrates that the degree of cation exchange increases as the recharge water progresses inland from the recharge wells. This is indicated by the increase in the separation of the calculated from analysed dilution lines from Well G-2 to Well G-8. This separation occurs only on the cation side of the barograph. On the anion side the two lines coincide indicating dilution is the only factor involved on this side. This increase in cation exchange landward is elaborated in Figure 7. Well G-2, the one nearest the recharge well, shows an increase in cation exchange only at extreme dilution (concentration 20 epm). The next well, G-4, shows a peak exchange at 95% dilution (concentration 74 epm) and Well G-8 has its peak at 35% dilution (concentration 200 epm). In terms of equivalents per million of exchange these peaks are represented by the following quantities:

	Calcium	*Magnesium*	*Sodium*
G-2	—1.2	—0.6	+1.8
G-4	—1.1	—3.2	+4.3
G-8	—2.0	—7.0	+9.0

Subsequent to the peaks observed in wells G-4 and G-8 cation exchange falls off rapidly with dilution so that no (or very little) cation exchange will be evident by the time the unpolluted recharge water reaches these wells.

The following discussion is presented to explain the difference in cation exchange activity from Well G-2 to Well G-8 as evidenced in Figures 6 and 7 and discussed in the preceding paragraphs. As recharge water displaces the sea-water inland, the sea-water will become increasingly diluted with the recharge water at the interface. Very slight exchange activity is observed until a concentration of 200 epm to 300 epm is reached at which point the concentration of sodium in the water has been reduced enough to permit the reaction:

$$Ca^{++} + Mg^{++} + NaX \rightleftarrows CaX + MgX + Na^{+}$$

to advance to the right. The amount of exchange that will then take place naturally depends on both the quantity of sodium saturated soil the water is exposed to, the degree of saturation and the time in which the water is in contact with the soil. The water reaching Well G-8 has been exposed to more high-sodium soil than water reaching Well G-4 although the soil between Well G-4 and Well G-8 shows a reduction in exchangeable sodium saturation from 82% to 27%. The same thing can be said of the water reaching Well G-4 compared to Well G-2, although here the sodium saturation decreases slightly, from 84% to 82%. Following the increase in exchange activity the soil becomes gradually depleted in exchangeable sodium and the exchange activity lessens. At Well G-8 the fall-off in exchange activity starts at a higher concentration than G-4. This is because of the reduction in exchangeable sodium saturation of the soil between these wells.

The groundwater which reached Well G-8 as result of recharge, first showed a decrease in concentration probably due to the displacement of a fresher body of water existing seaward of Well G-8. As the saline wave proceeded inland the concentration increased and as the wave passed Well G-8 the concentration decreased again. Therefore waters of the same concentration reached Well G-8 ahead of the saline wave and then later following the wave. The water following the wave shows a greater degree of exchange than the initial water. The curves for Well G-8 in Figure 7 show that this increase amounts to 3% epm at a concentration of 220 epm. Evidently the water following the wave was in contact with a greater mass of sodium saturated soil than the water which preceded the wave which of course was the case.

The water at G-2 unlike the water at G-4 or G-8 shows no fall-off in exchange activity although the water at this well is unpolluted recharge water. The reason for this is twofold, (1) the higher level of exchangeable sodium saturation in the soil, (94% to 84%) between Wells G-1 and G-2, and (2) a more rapid displacement of the water at those concentrations that would permit release of much of the sodium from the soil at this location (see Table 2B); thus the contact period required for cation exchange is relatively brief. The soil therefore, between Wells G-1 and G-2 has not been depleted in exchangeable sodium to a degree that would inhibit exchange, particularly with diluted water that exists between these wells. Just how long this exchange capacity will be maintained can only be determined from subsequent analyses.

c) Wells K-8 and K-12

The pre-recharge and latest analyses for Wells K-8 and K-12 are plotted on the trilinear chart (Figure 8). The analyses of the two possible sources of dilution of the recharge water, (viz: native and recharge water) are also plotted. As stated previously (see page 95) one of the lines drawn down between the pre-recharge analyses (on the anion triangle) and the diluting water analyses will contain the point representing the latest analyses. For Well K-8 that line is the one connecting recharge water analysis with pre-recharge analysis. For Well K-12 the line connects pre-recharge water analysis with native water analysis. Therefore the water in K-8 is being diluted by recharge water and the water in K-12 by native water. The pre-recharge analyses for Well K-12 falls on the K-8 native water mixing line as would be expected.

4. Control of Chlorine Dosage

Since chlorine is added to the recharge water to prevent bacterial slimes from accumulating at the well perforations, a criterion for controlling the dosage to achieve this should be developed. The criteria used at Manhattan Beach were:

(a) Adding sufficient chlorine to the recharge water to obtain a minumum residual in the adjacent "20 foot" well.
(b) Adding sufficient chlorine to obtain a minimum bacterial count in the "20 foot" well.
(c) Adding sufficient chlorine to obtain maximum acceptance of the recharge water.

The chlorine dosage was gradually reduced from the initial 20 ppm dosage to 1.5 ppm, the lowest concentration that would satisfy the first two criteria. However the recharge well head (at constant recharge rate) started to increase at this low dosage. The dosage was then increased to 5 ppm which resulted in an appreciable decrease in the recharge well head. A low bacterial count and a detectible residual chlorine in the "20 foot" well are therefore not reliable criteria for controlling chlorine dosage.

The following discussion is presented for a better understanding of the factors responsible for the inadequacy of the above two criteria and to shed some light on this problem.[8]

When chlorine gas is added to water it combines with the water to form hypochlorous and hydrochloric acid. The hypochlorous acid further dissociates into hydrogen and hypochlorite ions. The proportions of these components present depends on the pH as follows:

Below pH 5 —molecular chlorine
pH 5-6 —hypochlorous acid
pH 6-7.5 —hypochlorite ions increase
Above pH 7.5—hypochlorite ions predominate

The recharge water with a chlorine dosage up to 20 ppm will therefore have the chlorine as hypochlorite ions, (See Table 5). Chlorine in this form is defined as free available chlorine.

Chlorine in this form is highly reactive and will either combine with inorganic matter or oxidize inorganic matter, (See Table 7 for its effect on iron).

[8] See Bibliography.

Chlorine will also combine with and oxidize organic matter, and in so doing it can, by coagulation or precipitation, alter the physical state of organic compounds. Chlorine in the combined form is defined as combined available chlorine.

When a small dose of chlorine is added to water the reaction is mainly that of combination. As the dosage increases free available chlorine appears. When a sufficiently large dose is added, the free available chlorine predominates and oxidation occurs.

The oxidation reaction is desired in the recharge water to kill bacteria and to effect a beneficial change in the physical state of the organic slime, at the perforations. The amount of chlorine required to assure an oxidation reaction depends on, (1), the chlorine demand, i.e. the amount of inorganic and organic matter in the recharge water and in the well itself that will combine with or be oxidized by the chlorine, (2), time of contact and, (3), the water temperature.

Since these factors, particularly the first one, are difficult to evaluate no direct method for the determination of the optimum chlorine dosage is therefore possible. An indirect method, the 3rd criterion of adding sufficient chlorine to permit maximum acceptance would appear to be the logical one to use to adequately control chlorine dosages.

The one difficulty inherent in this method is that even an optimum dosage as determined by the third criterion may permit an insiduous bacterial or organic growth to collect at the perforations without it being reflected in a reduction in acceptance. By the time the reduction is noticed a sufficient quantity of slime may have accumulated that would be difficult to remove even by increasing the dosage in small increments. However by giving the recharge well a "shock" treatment of 20 ppm or more, it may be possible to remove most of the accumulated slime.

5. Relationship of Permeability to Cation Exchange and Concentration

Sodium soils are more impermeable than calcium soils.[9] It would therefore be expected that a soil in contact with saline water would on conversion to a sodium soil become more impermeable, and the reverse would occur when the saline water was displaced by a recharge water containing a normal amount of calcium. This change in permeability, resulting from cation exchange, is appreciably influenced by the concentration of salts in the water to which the soil is exposed. The high concentration of salts in

[9] See Bibliography.

sea-water may offset the reduction in permeability from cation exchange, due to the flocculating effect of the charged ions. When the sea-water is displaced by recharge water any increase in permeability resulting from ion exchange will be offset again by the removal of the flocculating effect of the charged ions.

This has been demonstrated in the article on the "Sealing of the Lagoon Lining at Treasure Island with Salt" by Charles H. Lee.[10] When fresh water was pumped into the clay lined lagoon, the seepage rate was 0.90 inches per day, which was excessive. Sea-water then replaced the fresh water to convert the clay to the sodium type. With sea-water the seepage rate fell to 0.60 inches per day. When fresh water replaced the sea-water in the lagoon, the seepage rate fell to 0.10 inches per day in 3 months. Only a 33⅓% decrease resulted in the conversion of the soil to the sodium type in the presence of sea-water, but when the sea-water was displaced with fresh water an additional 83⅓% decrease ensued, making a total decrease of 88.9%.

The same type of decrease, resulting from displacing sea-water with fresh water, was observed in a laboratory permeability test of sample #51 from Well G-8, (see Table 15 and Figure 6). The mean permeability with sea-water was 19.9 feet per day. Following the change to M.W.D. recharge water, the mean permeability fell to 16.8 feet per day. After the soil was exposed to the fresh water over the week end the mean permeability rose to 19.3 feet per day. This final increase in permeability was due to conversion of the soil to the more permeable calcium type. Though most of the conversion to the calicum soil occurred the first day, (see Table 15, last 3 columns) some additional time was required for the physical change to occur. The maximum displacement of the sodium ion from the soil (7.14 %epm) occurred at 10:03 AM, one and one-half hours after the influent was changed to fresh water. At this time the permeability was the maximum for the first day, i.e., 18.6 feet per day, but the effluent still showed an appreciable proportion of sea-water. For the remainder of the day the permeability fluctuated above and below a mean of 16.7 foot per day and the amount of sea-water in the effluent decreased rapidly until practically none was present. The increase of chlorides (see Table 15 and Figure 6) in the effluent, following the week end standover, probably resulting from the interim diffusion and mixing of the more saline water trapped in "dead spots" during the first day's run.

6. Presence of Deleterious Substances and Their Effect on Recharge

a) Dissolved Oxygen in the Recharge Water

The average temperature of the recharge water is about 18° C which would permit a dissolved oxygen content of 9.6 ppm. Table 5 shows that in some cases, particularly on September 15, 1953 and January 11, 1954, the concentration exceeded the maximum solubility of oxygen. The excess was apparent in the entrapped air bubbles observed during sampling. Trapped air is considered deleterious because of its detrimental effect on permeability of soils.

b) Iron in the Recharge Water

Table 8 shows an appreciable increase of iron in the recharge water resulting from its passage through the 20″ asphalt coated iron pipe. Although the amount of this iron pickup is too small to adversely affect the recharge water it does indicate corrosion in the line.

Tables 6 and 7 on the other hand show the presence of sufficiently large amounts of iron in the recharge and observation wells to indicate a high degree of corrosion of the casing. The formation at the perforations of a gelatinous iron hydroxide which is encouraged by the presence of oxidizing substances, chlorine and oxygen, would tend to reduce the acceptance of the recharge water.

In addition to the damage to project facilities and reduction of permeability resulting from corrosion, the accumulation of corrosion products in back pressure valves and metro valves (see Tables 9 and 10) resulting from the corrosion of aluminum as well as iron may seriously affect the function of the equipment.

c) Bacterial Slimes and Organic Matter

The effect of these substances on permeability has already been discussed. Another effect of the bacterial slime, already referred to but not explained, is its filtering action on bacteria. The low bacterial count that has been observed in the water samples taken from the adjacent "20 foot" well, may have resulted from this filtering action rather than the bactericidal action of the chlorine. This explains why the second criterion for control of chlorine, i.e., a low bacterial count, is not a reliable one. This filtering action of the slimes has been observed during pollution-travel studies conducted by the University of California at its Richmond station.

The results of the tests involving the two percolation tubes that were installed at Manhattan Beach show an appreciable difference in the clogging action of chlorinated and nonchlorinated water (see Table 17 and Figure 9). Water without chlorine left a brown deposit on the top layer of Ottawa Sand at the outset, which was reflected in a persistent decline in permeability. The deposit in the other tube appeared later and was never as great. The permeability in this tube after the initial drop fluctuated about a mean value appreciably above that found in the other tube. No attempt was made to prevent the suspended solids in the water from contributing to the decrease in permeability so that the changes in permeability reflect the presence of suspended solids in the water as well as the slime forming bacteria.

d) Presence of Sludge in the 6″ Line and in the Chlorinator Bell-Jar

Appreciable amounts of a white powdery sludge have appeared in the Chlorinator Bell-Jar and in the 6″ line. Samples of the material taken from these sources indicate that the sludge is primarily calcium carbonate (see Table 11). This sludge has been brought in by the recharge water and may have resulted from the erosion of the concrete M.W.D. feed lines. It could be considered a deleterious substance for two reasons: (1), It may affect the operation of project equipment, such as the chlorinator, if permitted to accumulate to a sufficient degree; (2), when combined with bacterial and iron hydroxide slimes it may augment the clogging action of these substances.

G. CONCLUSIONS

1. Source of Groundwater Pollution

No evidence has been found to indicate that the source of pollution at the Manhattan Beach site of the West Basin Barrier Test is other than the ocean.

2. Effect of Recharge on Soil and Groundwater

a) Soil

As the fresh water displaces the sea-water landward a gradual conversion of soil will occur. The sodium will be exchanged for the calcium in the soil and the permeability will tend to decrease at first but then increase as the sea-water is completely displaced. These changes in permeability may be obscured by other factors difficult to evaluate.

b) Groundwater

(1) Recharge water is altered by cation exchange when it contacts soil which previously had been exposed to highly saline water. This cation exchange activity which softened the water is appreciable within 300 feet of the recharge well but diminishes further inland.

(2) The dilution of water observed in Well K-12 was found to be caused by admixture with native water rather than recharge water.

3. Control of Chlorine Dosage

The chlorine demand studies and quantitative bacterial analyses have been proven an unreliable means of controlling the chlorine content of the recharge water. Maintenance of a maximum acceptance is a more reliable method as long as an excess of chlorine is present to prevent insiduous growth of bacterial slime. Any change in the recharge well head would not be sensitive enough to reflect such a growth, until too late. An Ottawa Sand percolation tube similar to the ones installed in the chlorine storage shed at Manhattan Beach is a more sensitive indicator. Periodic determinations of the permeability are sufficient to indicate any tendency toward the formation of slimes. For confirmation, a microscopic examination and bacterial analyses of a few surface grains could be made. Precautions should be taken to avoid passage of sludge and rust particles into the percolation tubes which would also reduce the permeability. This could be accomplished by introducing traps in the line feeding the percolation tubes.

4. Suspended Solids in the Recharge Water

Suspended solids which may include sand, calcium carbonate, rust particles and zeolite particles will not be apparent in a closed system except where some may be caught in the chlorinator bell-jar

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I. TABLES

Table Number

J. CHARTS

Figure Number

TABLE 1
PRE-RECHARGE ANALYSES

Source	Chlorides		Total Dis. Solids epm	% Dil.	Analyzed % epm						Calculated % epm			Cation Exchange % epm			SAR[1]	% Sat.
	ppm	Sea Water Ratio			Cal.	Mag.	Sod.	Bicarb.	Sulf.	Chlor.	Cal.	Mag.	Sod.	Cal.	Mag.	Sod.		
Sea	18,760	1.00	1,172	0	1.69	8.98	39.33	.17	4.70	45.13							58.3	100
Native	124		3.5	100.0	19.60	9.23	21.16	19.78	5.14	25.07							2.3	0
Zone 1																		
C-9	18,720	1.00	1,169	0.2	1.73	8.99	39.28	.20	4.63	45.17	1.69	8.98	39.33	+0.04	+0.01	—0.05	58.0	99
G-5	18,600	.99	1,168	0.9	1.82	9.34	38.84	.35	4.73	44.91	1.69	8.98	38.33	+0.13	+0.36	—0.49	56.4	96
B-1	18,600	.99	1,162	0.9	1.77	9.06	39.18	.23	4.65	45.12	1.69	8.98	39.33	+0.08	+0.08	—0.15	57.4	98
D	18,440	.98	1,155	1.7	1.66	9.22	39.11	.20	4.77	45.03	1.69	8.98	39.33	—0.03	+0.24	—0.22	57.0	97
F	18,440	.98	1,151	1.7	2.17	8.73	39.10	.14	4.69	45.17	1.69	8.98	39.33	+0.48	—0.25	—0.23	57.0	96
C-5	18,400	.98	1,149	1.9	1.72	9.14	39.14	.20	4.64	45.16	1.69	8.98	39.33	+0.03	+0.16	—0.19	57.0	97
G-9	18,300	.98	1,149	2.5	1.70	9.34	38.96	.18	4.66	45.16	1.70	8.98	39.32	0.00	+0.36	—0.36	56.3	96
C	18,000	.96	1,135	4.1	1.79	8.73	39.48	.17	4.71	45.21	1.70	8.98	39.32	+0.09	—0.25	+0.16	58.0	99
E-4	18,000	.96	1,128	4.1	2.27	8.95	38.78	.20	4.65	45.15	1.70	8.98	39.32	+0.57	+0.03	—0.60	55.0	93
G-3	17,640	.94	1,113	6.0	2.46	8.99	38.55	.21	4.71	45.09	1.70	8.98	39.31	+0.76	+0.01	—0.76	53.7	91
K-5	17,360	.93	1,085	7.5	1.76	8.46	39.79	.28	4.78	44.91	1.71	8.98	39.31	+0.05	—0.52	+0.48	58.0	99
L-1	17,100	.91	1,085	8.9	2.27	9.35	38.38	.20	4.72	45.07	1.71	8.98	39.31	+0.56	+0.37	—0.93	52.5	89
Zone 1-A																		
K-9	1,920	.10	133	90.4	1.33	7.94	40.73	4.22	5.01	40.77	3.44	9.01	37.55	—2.11	—1.07	+3.18	21.8	34
Zone II																		
G-1	17,200	.92	1,071	8.4	2.67	8.14	39.19	.21	4.65	45.14	1.72	8.98	39.30	+0.95	—0.84	—0.11	55.1	94
G-H	16,600	.88	1,040	11.6	3.34	9.13	37.53	.19	4.79	45.02	1.72	8.98	39.30	+1.62	+0.15	—1.77	48.6	82
E-1	16,600	.88	1,040	11.6	2.58	8.75	38.68	.28	4.72	45.00	1.72	8.98	39.30	+0.86	—0.23	—0.62	52.5	89
G-2	16,000	.85	1,001	14.8	2.82	9.02	38.15	.21	4.66	45.10	1.73	8.98	39.29	+1.09	+0.04	—1.14	49.6	84
H	15,960	.85	996	15.0	3.84	9.18	36.98	.23	4.58	45.19	1.73	8.98	39.29	+2.11	+0.20	—2.31	45.7	77
G-4	15,820	.84	988	15.8	3.34	8.68	37.98	.25	4.64	45.11	1.73	8.98	39.29	+1.61	—0.30	—1.31	48.2	82
K-4	15,700	.84	995	16.4	3.39	9.18	37.42	.24	4.65	45.11	1.73	8.98	39.29	+1.66	+0.20	—1.87	47.1	79
G-G	15,500	.83	975	17.5	3.27	8.96	37.76	.25	4.60	45.14	1.74	8.98	39.28	+1.53	—0.02	—1.52	47.7	80
J	15,400	.82	966	18.0	3.00	8.33	38.67	.24	4.81	44.96	1.74	8.98	39.28	+1.26	—0.65	—0.61	50.5	85
Zone III																		
I-1	17,200	.92	1,077	8.3	4.04	9.14	36.82	.17	4.69	45.03	1.71	8.98	39.31	+2.33	+0.16	—2.49	47.1	79
C-4	15,700	.84	988	16.4	4.12	8.94	36.93	.23	4.53	45.23	1.73	8.98	39.29	+2.39	—0.04	—2.36	45.4	76
H-4	15,520	.83	890	17.4	4.08	9.10	36.82	.24	4.66	45.10	1.74	8.98	39.28	+2.34	+0.12	—2.46	42.8	72
C-8	14,200	.76	898	24.5	5.39	8.72	35.89	.26	4.64	45.10	1.76	8.98	39.26	+3.63	—0.26	—3.37	40.4	68
K-8	12,500	.67	792	33.6	5.91	8.72	35.35	.32	4.64	45.01	1.80	8.98	39.22	+4.11	—0.26	—3.87	36.8	61
I-4	12,320	.66	770	34.6	7.17	8.51	34.33	.23	4.65	45.12	1.80	8.98	39.22	+5.37	—0.47	—4.88	33.9	56
K-1	10,480	.56	660	44.5	6.63	8.98	34.40	.40	4.73	44.86	1.86	8.98	39.16	+4.77	0.00	—4.76	31.6	52
Zone IV																		
C-12	7,080	.38	445	62.7	17.58	7.85	24.57	.66	4.48	44.86	2.04	8.98	38.97	+15.54	—1.13	—14.40	14.6	22
G-8	4,980	.27	315	74.0	10.35	9.05	30.60	.83	4.62	44.55	2.28	8.99	38.73	+8.07	+0.06	—8.13	17.5	27
K-12	2,580	.14	164	86.9	17.14	11.99	20.86	2.06	4.20	43.73	3.01	9.00	37.99	+14.13	+2.99	—17.13	6.8	8
A-14	2,320	.12	142	88.3	14.53	8.99	26.49	1.19	4.34	44.47	3.18	9.00	37.82	+11.35	—0.01	—11.33	9.0	12
M-14	1,510	.08	104	92.5	18.36	8.34	23.30	5.54	3.59	40.87	4.00	9.01	36.99	+14.36	—0.67	—13.69	6.5	7
Zone IV-A																		
TH-3	220	.01	22.4	99.4	11.08	6.12	32.80	18.45	3.84	27.70	13.62	9.14	27.24	—2.54	—3.02	+5.56	8.3	11
H-16	136	.01	15.4	99.7	17.45	6.84	25.72	19.40	5.60	25.00	16.00	9.23	24.77	+1.45	—2.39	+0.95	3.0	1
TH-2	140	.01	16.0	99.7	15.94	9.78	24.28	22.95	3.20	23.85	16.00	9.23	24.77	—0.06	+0.55	—0.49	2.7	1
Zone V																		
MB-4	4,090	.21	245	79.0	20.58	10.68	18.74	.89	3.83	45.28	2.46	8.99	38.54	+18.12	+1.69	—19.80	7.4	9
MB-7	1,920	.10	133	80.3	21.89	12.51	12.64	2.90	3.69	43.40	3.49	9.00	37.51	+21.40	+3.51	—24.87	3.4	2
K-16	1,760	.09	116	91.1	21.39	9.86	18.75	3.11	3.96	42.93	3.64	9.01	37.34	+17.75	+0.85	—18.59	5.1	5
MB-8	1,616	.09	105	91.9	24.28	11.94	13.78	3.35	3.18	43.47	3.84	9.01	37.15	+20.44	+2.93	—23.37	3.2	2
MB-6	972	.05	64.3	95.3	22.64	12.36	15.09	3.45	1.85	44.70	5.20	9.03	35.77	+17.44	+3.33	—20.77	2.8	1
TH-1	920	.05	65.9	95.6	25.02	12.56	12.42	6.74	3.87	39.39	5.40	9.03	35.57	+19.62	+3.53	—23.15	2.3	0
Zone V-A																		
M-18	386	.02	31.0	98.5	24.11	9.17	16.72	11.52	3.33	35.15	9.61	9.10	31.30	+14.50	+0.07	—14.58	2.3	0

[1] Sodium absorption ratio.

TABLE 2A

EFFECT OF RECHARGE—WELL GH

	Days of Rechg.	Chlor.	% Dil.	Chlor. Ratio	Total Dis. Solids epm	Analysed % epm						Calculated % epm			Cation Exchange % epm		
						Ca.	Mg.	Na.	Bicarb.	Sulf.	Chlor.	Ca.	Mg.	Na.	Ca.	Mg.	Na.
Pre-Recharge	0	16,600	0	.89	1,040	3.34	9.13	37.53	.10	4.79	45.02						
Recharge		88	100.0		22	6.93	4.04	39.03	12.21	26.40	11.39						
Date																	
7/ 2/53	128	14,520	12.5	.78	913	2.36	8.86	38.78	.25	4.90	44.85	3.35	9.11	37.54	—0.99	—0.25	+1.24
8/20/53	177	9,760	41.4	.52	619	2.01	9.20	38.79	.37	5.16	44.47	3.39	9.06	37.55	—1.38	+0.14	+1.24
9/10/53	198	6,480	61.3	.35	416	2.56	7.86	39.58	.55	5.52	43.93	3.47	8.97	37.57	—0.91	—1.11	+2.01
10/ 8/53	226	4,360	74.1	.23	282	3.45	7.44	39.11	.85	5.61	43.54	3.54	8.84	37.61	—0.09	—1.40	+1.50
10/27/53	245	3,320	80.4	.18	219	3.75	7.11	39.14	.88	6.40	42.72	3.62	8.73	37.64	+0.12	—1.62	+1.50
11/12/53	261	4,080	75.8	.22	267	3.32	7.54	39.14	.92	6.00	43.08	3.56	8.82	37.62	—0.24	—1.28	+1.52
12/29/53	308	6,920	58.6	.37	440	2.75	7.04	40.21	.50	5.15	41.35	3.41	8.99	37.57	—0.69	—1.95	+2.61
1/13/54	323	7,120	57.4	.38	453	2.49	7.65	39.86	.49	5.16	41.36	3.44	8.99	37.57	—0.95	—1.34	+2.29
1/28/54	338	6,240	62.7	.33	400	2.87	6.66	40.47	.55	5.39	44.06	3.46	8.96	37.58	—0.59	—2.30	+2.89
3/10/54	379	5,080	69.8	.27	325	2.39	6.78	40.83	.68	5.66	43.66	3.49	8.99	37.51	—1.10	—2.21	+3.32
4/ 8/54	408	4,440	73.6	.24	290	2.66	6.55	40.78	.80	6.02	43.18	3.54	8.85	37.61	—0.88	—2.30	+3.17
10/ 5/54	588	2,800	83.6	.15	191	1.93	5.26	42.81	1.22	6.91	41.45	3.68	8.64	37.68	—1.75	—3.38	+5.13

TABLE 2B

EFFECT OF RECHARGE—WELL G-2

	Days of Rechg.	Chlor.	% Dil.	Chlor. Ratio	Total Dis. Solids epm	Analysed % epm						Calculated % epm			Cation Exchange % epm		
						Ca.	Mg.	Na.	Bicarb.	Sulf.	Chlor.	Ca.	Mg.	Na.	Ca.	Mg.	Na.
Pre-Recharge	0	16,000	0	.85	1,001	2.82	9.02	38.15	.21	4.66	45.10						
Recharge		88	100.0		22	6.93	4.04	39.03	12.21	26.40	11.39						
Date																	
7/22/53	148	8,320	48.3	.44	529	3.10	7.98	38.92	.39	5.09	44.32	—2.91	—8.92	—38.17	+0.19	—0.94	+0.75
7/30/53	156	6,160	61.9	.33	395	3.37	7.43	39.20	.60	5.42	43.98	—2.96	—8.85	—38.19	+0.41	—1.42	+1.01
8/ 6/53	163	4,320	73.4	.23	282	3.31	6.78	39.91	.57	5.66	43.20	—3.05	—8.74	—38.20	+0.26	—1.96	+1.71
8/25/53	182	1,260	92.6	.07	93	2.78	6.32	40.90	2.44	9.25	38.31	—3.70	—7.95	—38.35	—0.92	—1.63	+2.55
9/ 6/53	194	704	96.1	.04	58	2.45	4.52	43.04	4.12	11.64	34.24	—4.25	—7.29	—38.46	—1.80	—2.77	+4.58
9/27/53	215	272	98.9	.01	32	2.93	2.25	44.81	7.19	19.14	23.67	—5.53	—5.72	—38.75	—2.60	—3.47	+6.06
6/ 4/54	465	94	100.0	.00	22	1.04	1.35	47.61	9.62	26.02	11.97	—6.89	—4.12	—38.98	—5.85	—2.77	+8.63

TABLE 2C

EFFECT OF RECHARGE—WELL G-4

	Days of Rechg.	Chlor.	% Dil.	Chlor. Ratio	Total Dis. Solids epm	Analysed % epm						Calculated % epm			Cation Exchange % epm		
						Ca.	Mg.	Na.	Bicarb.	Sulf.	Chlor.	Ca.	Mg.	Na.	Ca.	Mg.	Na.
Pre-Recharge	0	15,820	0	.84	988	3.34	8.68	37.98	.25	4.64	45.11						
Recharge		88	100.0		22	6.93	4.04	39.03	12.21	26.40	11.39						
Date																	
7/ 9/53	135	15,560	2.2	.83	975	3.29	8.88	37.83	.20	4.68	45.12	3.34	8.68	37.98	—0.05	+0.20	—0.15
8/ 5/53	162	14,400	9.6	.77	901	3.06	8.46	38.48	.25	4.67	45.08	3.34	8.67	37.99	—0.28	—0.21	+0.49
9/ 1/53	189	11,920	25.3	.63	750	2.87	8.39	38.74	.33	4.83	44.84	3.35	8.65	38.00	—0.48	—0.26	+0.74
9/29/53	217	9,560	40.1	.51	604	3.14	8.05	38.81	.30	4.91	44.61	3.39	8.61	38.00	—0.25	—0.56	+0.81
11/ 4/53	253	6,800	57.6	.36	434	3.47	7.64	38.89	.60	5.23	44.19	3.44	8.55	38.01	+0.03	—0.91	+0.88
12/14/53	293	4,280	73.5	.23	280	3.54	6.77	39.69	.90	5.98	43.12	3.53	8.42	38.06	+0.01	—1.65	+1.63
2/17/54	358	1,760	89.4	.09	126	2.04	4.71	43.24	1.74	8.74	39.31	3.90	7.95	38.15	—1.86	—3.24	+5.09
4/15/54	415	932	94.7	.05	74	2.72	2.79	44.49	3.22	11.36	35.42	4.35	7.37	38.28	—1.63	—4.58	+6.21
6/22/54	483	564	97.0	.03	50	4.07	2.12	43.81	4.91	13.34	31.75	4.83	6.74	38.44	—0.76	—4.62	+5.37
8/24/54	546	200	99.3	.01	29	5.87	2.29	41.84	6.83	22.12	19.25	6.06	5.15	38.79	—0.19	—2.86	+3.05

TABLE 2D

EFFECT OF RECHARGE—WELL G-8

	Days of Rechg.	Chlor.	% Dil.	Chlor. Ratio	Total Dis. Solids epm	Analysed % epm						Calculated % epm			Cation Exchange % epm		
						Ca.	Mg.	Na.	Bicarb.	Sulf.	Chlor.	Ca.	Mg.	Na.	Ca.	Mg.	Na.
Pre-Recharge	0	4,980	0	.27	315	10.35	9.05	30.60	.83	4.62	44.55						
Recharge		88	100.0		22	6.93	4.04	39.03	12.21	26.40	11.39						
Date																	
10/15/53	233	3,340	33.7	.18	211	9.67	7.94	32.39	1.22	4.23	44.55	10.23	8.88	30.89	—0.56	—0.94	+1.50
10/29/53	247	4,220	15.7	.22	267	8.87	7.74	33.39	.94	4.29	44.77	10.30	8.99	30.71	—1.43	—1.25	+2.68
12/11/53	290	3,800	24.3	.20	241	7.49	7.67	34.84	1.04	4.56	44.39	10.27	8.94	30.79	—2.78	—1.27	+4.05
1/12/54	322	3,360	33.3	.18	215	7.02	7.22	35.76	1.17	4.70	44.13	10.23	8.88	30.89	—3.21	—1.66	+4.87
3/ 3/54	372	2,760	45.5	.15	179	6.14	8.31	35.55	1.12	4.79	43.50	10.16	8.78	31.07	—4.02	—0.47	+4.48
6/ 2/54	463	1,800	65.1	.10	119	6.53	8.35	35.12	1.79	5.03	42.73	9.96	8.46	31.58	—3.43	—0.11	+3.54
8/ 5/54	527	1,300	75.3	.07	88	8.43	7.27	34.30	2.51	5.66	41.83	9.75	8.18	32.07	—1.32	—0.91	+2.23

TABLE 3

SOURCE OF POLLUTION

Source		Sea Water	Connate Water		Wells		
			Max.	Min.	C-12	MB-4	K-12
Date of Sample					6/18/52	5/29/52	6/17/52
Depth					219′	311′	248′
Chlor. ppm		18,980	13,319	3,384	4,140	2,440	3,220
Borate ppm		25	386	0	1.6	.8	1.5
Iodide ppm		.05	80	30	2.0	<1	<1
Bromide ppm		65	200	25	13.0	14	8
Calcium % epm		1.7	6.6	1.0	15.6	23.5	20.6
Magnesium % epm		8.8	4.8	0.1	9.1	12.0	9.2
Sodium % epm		39.5	47.9	41.8	25.3	14.5	20.2
Bicarbonate % epm		.2	12.3	.2	1.6	1.4	1.3
Chloride % epm		45.2	49.8	37.5	44.0	45.1	44.6
Sulfate % epm		4.7	.7	0	4.4	3.5	4.1
*Theor. Bor.	C-12	5.5	120	0			
	MB-4	3.2	71	0			
	K-12	4.2	92	0			
*Theor. I.	C-12	.010	25	9			
	MB-4	.006	15	6			
	K-12	.009	19	7			
*Theor. Br.	C-12	14	62	8			
	MB-4	8	37	5			
	K-12	11	49	6			

* Theoretical concentration based on a dilution ratio calculated from chloride concentrations.

TABLE 4

DISSOLVED OXYGEN—GROUND WATER

Well	Date	Dissolved Oxygen			Remarks
		Max.	Min.	Mean	
E	12/ 5/52	2.6	1.3	2	Turbulence caused a steady increase in dissolved oxygen.
H	2/12/53	.8	.6	.7	
I	2/ 4/53	1.1	.6	.8	
G-2	1/11/54	1.8	1.5	1.7	

TABLE 5

FIELD ANALYSES OF RECHARGE WATER

Date	Source	pH	Dissolved Oxygen ppm	Residual Chlorine ppm	Remarks
6/23/53	Pre-Chlor	8.35	8.8	----	All analyzed in field house. Sampling technique at Wells caused air to be entrapped during sampling.
6/23/53	Post-Chlor	7.45	7.0	----	
6/23/53	Well E	7.60	4.6	12.8	
6/23/53	Well G	7.64	7.4	10.4	
6/23/53	Well H	7.44	8.1	12.2	
6/23/53	Well I	7.50	8.4	9.2	
6/23/53	Well K	7.65	4.6	7.6	
7/ 1/53	Pre-Chlor	8.53	9.8	----	All analyzed at point of sampling. No air entrapped during sampling.
7/ 1/53	Post-Chlor	7.51	7.0	10.3	
7/ 1/53	Well E	7.50	10.2	7.3	
7/ 1/53	Well G	7.55	8.7	7.6	
7/ 1/53	Well H	7.40	7.7	7.6	
7/ 1/53	Well I	7.51	12.1	8.0	
7/ 1/53	Well K	7.58	8.4	9.7	
9/15/53	Pre-Chlor	8.40	11.6	----	Same as for July 1, 1953.
9/15/53	Post-Chlor	7.94	9.4	4.8	
9/15/53	Well E	7.85	11.2	5.1	
9/15/53	Well H	7.85	10.3	5.0	
9/15/53	Well G	7.85	11.0	5.2	
9/15/53	Well K	7.85	10.0	5.2	
1/11/54	Pre-Chlor	8.50	11.1	----	Same as for July 1, 1953.
1/11/54	Well E	7.80	10.9	----	
1/11/54	Well K	7.85	10.1	----	

TABLE 6

IRON IN WELL G AFTER CAVE-IN

Source	Date	Time	Depth (Ft.)	Conductivity ECX10[6]	Iron, ppm
G	3-18-53	1250	188	1046	3.8
G	3-18-53	1305	189	1048	3.8
G	3-18-53	1315	155	1050	3.5
G	3-18-53	1325	145	1049	4.0
MWD	Pre-Chlorination			1037	0.1
FG	3-27-53	1215	193		0.5
GH	3-27-53	1515	144		0.5
G-1	3-19-53	1450	158		0.2
G-2	3-19-53	1130	193		2.0
G-3	3-20-53	1520	185		0.2

TABLE 7

IRON PRECIPITATED IN PRESENCE OF CHLORINE

Source	Date	Depth (Ft.)	Iron (Raw)	Iron Filtered
FG	11-25-53	----	----	0.6, 0.2, >0.1
K-1	12- 7-53	152	3.0	>0.1
E-1	12- 7-53	160	3.5	0.6

TABLE 8

IRON PICKUP IN FEEDER LINE

Source	Date	Iron ppm	Remarks
Vault	4-28-54	0.05	
Field House	4-28-54	0.20	Chlorinated
Pickup		0.15	In Feeder Line

TABLE 9

DEPOSIT INSIDE BACK PRESSURE VALVE

Source:	Well G
Date:	April 1953
Description:	Red Brown Powder
Iron and Al. Oxide:	75.0%
Silica	1.5%

TABLE 10

DEPOSIT FOUND INSIDE METRO VALVE

Source:	Well E
Date:	August 1953
Description:	White Powdery Crust
Aluminum:	About 20%

TABLE 11

SLUDGE FOUND IN CHLORINATOR BELL-JAR AND 6" FEED LINE

	Chlorinator Bell-Jar	6 Inch Feed Line
Calcium Carbonate	97.8%	*89.3%
Calcium Carbonate	96.0%	†88.1%
Silica	2.6%	1.9%
Iron & Al. Oxides	0.6%	.1%
Magnesium	0.0%	.2%
Combined Water	3.2%	4.7%

* Calculated from analyses of calcium.
† Calculated from loss on ignition.

TABLE 12

ANALYSIS OF WATER IN CHLORINATOR BELL-JAR CONTAINING SLUDGE

		Typical MWD Water
Chlorine	860 ppm	----
pH	2.6	8.5
Calcium	313 ppm	33.7 ppm
Magnesium	5.2 ppm	10.5 ppm
Sulfate	306 ppm	348 ppm

TABLE 13

LABORATORY PERMEABILITY RESULTS—METAL PERMEAMETERS

Well	Depth	Field Description	Date Run	Permeability Ft/Day
C-1	96.4- 96.9	Light brown sandy silt & brown silty fine sand	5-20-52/ 5-23-52	5.0
C-4	208.3-208.4	Bl Gry Vy Fine Sand, Woodclips, Vy scat. ¼" grav.	8-19-52/ 8-22-52	3.45
*C-9	116.0-118.0	Yel brn fine to vy coarse sand & grav. to 1", (average ½"), occasional pebbles to 2"	4-15-53/ 4-28-53	21.55
*C-9	140.0-140.5	Dark gray gravel to ½" (average ¼") with fine sand, thin silty clay layers abund., shells	3-26-53/ 4- 8-53	6.70
*E-4	236.1-236.5	Gray fine to coarse sand & grav. ⅛"-2"	5-12-53/ 5-13-53	30.38
G	138.9-139.2	Light brown fine to med. sand with scattered gravel to ⅜"	8-22-52/ 8-27-52	2.80
G-1	158.3-158.6	Tan fine to coarse sand	6-12-52/ 6-20-52	20.1
J	134.7-135.6	Gray brown fine to coarse sand & gravel to ½"	8-29-52/ 9- 2-52	18.4
K-12	305.7-306.0	Yellow brown very coarse to medium sand	7- 2-52/ 7- 9-52	57.5

* These tests run on the portion of the sample passing No. 4 sieve (less than one-quarter inch).

TABLE 14

LABORATORY PERMEABILITY RESULTS

Well	Sample Number	Depth (Ft.)	Field Description	Dates of Test	Permeability Ft./Day at 60° F Sea-Water Influent			Permeability Ft./Day at 60° F Fresh-Water Influent			Remarks
					Max.	Min.	Mean	Max.	Min.	Mean	
I	34	202.5 to 202.8	Light gray medium to very coarse sand with scattered gravel—½"	4-21-54 to 4-28-54	45.6	27	37.4	44	27	34.4	Run intermittently. Partially analyzed influent & effluent
K-4	27	255.7 to 256.0	Blue gray medium to fine sand	4-21-54 to 4-28-54	18.5	11.0	15.0	14.0	12.0	12.9	Run intermittently. Partially analyzed influent & effluent
G-1	13	133.7 to 134.0	Gray silty fine sand scat. coarse sand and pebbles—⅜"	4-12-54 to 4-15-54	4.4	1.5	2.7				Run intermittently. Partially analyzed influent & effluent
L-1	18	124.0 to 124.5	Gray silty fine sand	4-12-54 to 4-15-54	6.0	.24	1.4				Run intermittently. Partially analyzed influent & effluent
F-G	12	140.8 to 141.1	Gray brn silty fine crse sand w/scat. grav. ⅛"—¼"	3-18-54 to 3-19-54	111	51	91	107	26	59.4	Run continuously. Partially analyzed influent & effluent. Rust in sample
K	20	194.9 to 195.2	Blue gray fine very crse sand w/scat. gravel—⅛"	3-20-54 to 3-23-54	17.4	14.2	15.4	20.4	14.6	16.6	Run continuously. Partially analyzed influent & effluent. Rust in sample
M-18	13	225.3 to 225.6	Gray medium to very coarse sand and gravel—1"	5- 6-54 to 5-11-54	58.0	39.0	46.3	61	41	52.7	Run intermittently. Analyzed influent & effluent. Rust in sample; anthrone sugar test
G-8	51	279.9 to 280.2	Gray medium to coarse sand	5-25-54 to 6- 1-54	22.6	19.0	20.6	21.4	15.4	18.0	Run intermittently. Analyzed influent & effluent

TABLE 15

LABORATORY PERMEABILITY TEST ON SAMPLE No. 51 FROM WELL G-8, DEPTH 279.9-280.2 FEET, GRAY MEDIUM TO COARSE SAND

Date	Time	Permea- bility Ft./Day at 60° F	Analyses of Influent and Effluent ppm			Theoretical ppm[1]		Difference ppm[2]		Difference epm		
			Chlor.	Cal.	Mag.	Cal.	Mag.	Cal.	Mag.	Cal.	Mag.	Sodium
5-25	11:20	------	18640	402	1258		Start 1st	Bottle-Sea	water Influ	ent		
*5-25	11:35	------	14960	665	1012							
5-25	12:13	22.2	18360	466	1255	------	------	+64	—3	+3.2	—.25	—2.95
5-25	2:00	22.6	18680	397	1261	------	------	—5	+3	—.25	+.25	0
5-25	4:00	22.3	18680	393	1271	------	------	—9	+13	—.45	+1.7	—1.25
*5-26	8:00	------	18720	426	1272	------	------	+29	—7	+1.4	—.58	—.82
5-26	8:40	20.9	18680	399	1275	------	------	+2	—4	+.10	—.33	+.23
5-26	10:30	19.7	18700	393	1279	------	------	—4	—3	—.20	—.25	+.45
5-26	12:00	------	18760	397	1279		Start 2nd	Bottle-Sea	water Influ	ent		
5-26	2:00	20.5	18740	396	1287	------	------	—1	+8	—.05	+.65	—.60
5-26	4:00	19.8	18800	390	1281	------	------	—7	+2	—.35	+.16	+.19
*5-27	8:00	------	18760	394	1289	------	------	—3	+10	—.15	+.82	—.67
5-27	8:45	19.8	18880	390	1294	------	------	—7	+15	—.35	+1.2	—.85
5-27	10:45	19.3	18600	402	1263	------	------	+5	—16	+.25	—1.3	+1.05
5-27	1:30	19.0	18580	392	1270	------	------	—5	—9	—.25	—.74	+.99
*5-28	8:15	17.7	18795	409	1290	------	------	+12	+11	+.60	+.91	—1.51
5-28	8:45	15.4	18760	390	1270	------	------	—7	—9	—.35	—.74	+1.09
5-28	8:46	------	80	32.6	12.7		Start 3rd	Bottle-M	WD Water	Influent		
5-28	9:50	16.5	17980	377	1220	381.4	1226.5	—4.4	—6.5	—.22	—.53	+.75
5-28	9:57	------	15440	318	1000	331.8	1054.3	—13.8	—54.3	—.69	—4.5	+5.19
5-28	10:03	16.3	11020	229	688	246.5	754.3	—17.5	—66.3	—.87	—5.5	+6.37
5-28	10:05	------	7400	147	439	175.8	508.7	—28.8	—69.7	—1.44	—5.7	+7.14
5-28	10:15	18.6	4300	79.5	250	114.9	298.8	—35.4	—48.8	—1.77	—4.0	+5.77
5-28	10:28	------	2102	37.7	118	72.1	149.7	—34.4	—31.7	—1.72	—2.6	+4.32
5-28	10:45	16.6	1130	22.3	79	53.1	83.9	—30.8	—4.9	—1.54	—.40	+1.94
5-28	11:00	------	680	16.2	53	44.3	53.4	—28.1	—.4	—1.40	—.03	+1.43
5-28	11:18	16.4	440	12.8	45	39.6	37.1	—26.8	+7.9	—1.34	+.65	+.69
5-28	11:32	------	385	12.0	29.4	38.5	33.3	—26.5	—3.9	—1.32	—.32	+1.64
5-28	11:50	16.5	312	14.2	22.9	37.1	28.4	—22.9	—5.5	—1.14	—.45	+1.59
5-28	12:17	------	192	11.5	17.7	34.6	19.8	—23.1	—2.1	—1.15	—.17	+1.32
5-28	12:50	17.1	182	15.0	22.9	34.6	19.7	—19.6	+3.2	—.98	+.26	+.72
5-28	1:15	------	134	15.3	16.1	33.7	16.4	—18.4	—.3	—.92	—.02	+.94
5-28	1:45	16.7	124	16.0	18.1	33.4	15.7	—17.4	+2.4	—.87	+.20	+.67
5-28	2:17	------	116	18.1	15.7	33.3	15.1	—15.2	+.6	—.76	+.05	+.71
5-28	2:45	16.5	108	19.0	15.5	33.2	14.6	—14.2	+.9	—.71	+.07	+.64
5-28	3:15	------	106	19.7	15.8	33.1	14.5	—13.4	+1.3	—.67	+.11	+.56
5-28	3:45	17.0	100	20.0	17.5	33.0	14.1	—13.0	+3.4	—.65	+.28	+.37
*5-31	------	------	402	28.2	30.0	38.9	34.5	—10.7	—4.5	—.53	—.37	+.90
5-31	9:00	21.4	196	20.1	16.6	34.9	20.6	—14.8	—4.0	—.74	—.33	+1.07
5-31	10:00	17.5	150	22.5	16.3	34.0	17.4	—11.5	—1.1	—.57	—.09	+.66
5-31	11:00	19.3	114	23.5	15.6	33.3	15.0	—9.8	+.6	—.48	+.05	+.43
5-31	12:00	19.7	90	23.6	17.1	32.8	13.3	—9.2	+3.8	—.46	+.31	+.15
5-31	1:00	20.5	86	25.7	17.1	32.7	13.1	—7.0	+4.0	—.35	+.33	+.02
5-31	2:00	20.4	84	26.7	16.3	32.7	13.0	—6.0	+3.3	—.30	+.27	+.03
5-31	3:00	20.1	84	27.6	14.9	32.7	13.0	—5.1	+1.9	—.25	+.16	+.09
5-31	3:38	------	80	27.1	15.2	------	------	------	------	------	------	------
*6- 1	------	------	158	26.6	22.9	34.2	18.1	—7.6	+4.8	—.38	+.40	—.02
6- 1	11:00	19.2	90	27.8	14.6	32.8	13.3	—5.0	+1.3	—.25	+.11	+.14
6- 1	12:45	18.8	112	28.6	9.4	33.2	16.3	—4.5	+3.7	—.22	+.30	—.08
6- 1	4:00	18.3	88	28.5	15.2	32.8	13.2	—4.3	+2.0	—.21	+.16	+.05

* First effluent after starting permeameter runs, represents, (1) influence of original soil water, or, (2) water left in sample overnight or over week-end.

[1] Calculated from dilution ratio based on chloride concentration.

[2] With sea water as influent—effluent minus influent ppm; with MWD water as influent—Theoretical minus Analytical.

TABLE 16

SUMMARY OF SOIL TEST DATA

Well No.	Depth	% Moist.	Dry Density p.c.f.	Spec. Grav.	Sieve Analyses			Porosity	Void Ratio
					% Silt or Clay	% Sand	% Gravel		
*C-9	116.0-118.0	8.0	115.0	2.67	1	82	17	.31	.45
*C-9	140.0-140.5	10.1	120.2	2.67	6	55	39	.28	.38
C-4	208.0-208.3	10.0	131.8	2.70				.22	.28
C	172.0				0	55	45		
*C-1	96.4- 96.9	22.6	99.6	2.66	0	100	0	.40	.69
C-1	162.8-163.2	36.1	85.4						
C-1	169.4-169.7	12.0	128.0						
D	197.0				0	47	53		
E	156.0				1	64	35		
E-1	157.0				0	65	35		
E-4	236.5-237.0	8.0	116.6	2.68	2	78	20	.30	.43
*E-4	236.1-236.5	6.8	110.1	2.73	1	50	49	.33	.55
F	156.0				0	57	43		
F-G	150.0				0	81	19		
*G-1	158.3-158.6	15.0	118.0	2.71	2	65	33	.30	.43
G-2	169.0-171.0				0	64	36		
G-2	177.0-181.0				1	38	61		
G-4	204.0-209.0				0	53	47		
G-4	211.0-215.0				0	22	78		
G-4	218.0-224.0				0	23	77		
G-4	225.0-230.0				0	59	41		
G-4	230.0-234.0				0	68	32		
G-4	248.0-250.0				3	48	49		
*G	138.9-139.2	15.0	114.9	2.71	3	85	12	.32	.47
G-H	134.0				0	30	70		
H	155.0				1	48	51		
I	201.0				0	42	58		
J	136.0				3	48	49		
*J	134.7-135.0	12.0	116.0	2.75	3	89	8	.32	.48
K	136.0-138.0				1	71	28		
K-1	149.3-149.6	31.6	90.9						
K-1	183.4-183.7	21.1	105.2						
K-1	210.7-211.1	23.6	101.2						
K-1	259.8-260.2	21.9	103.8						
*K-12	305.7-306.0	14.0	112.8	2.68	0	91	9	.32	.48

* Sieve analysis run after permeability test.

TABLE 17

FALLING-HEAD PERMEAMETER TEST AT MANHATTAN BEACH

Date	Time	Permeability Ft./Day at 60° F		Remarks
		Pre-Chlorinator Water	Post-Chlorinator Water	
3-30-54	1335	315	317	
3-30-54	1400	277	242	
3-31-54	0803	177	132	
3-31-54	1600	165	132	Buff colored precipitate in "Pre" tube above sand.
4- 1-54	0810	146	119	
4- 2-54	0805	129	124	Slight darkening of top ½'' of sand in "Pre" tube.
4- 3-54	1400	92	86	
4- 4-54	1452	55	68	
4- 5-54	1110	56	87	
4- 6-54	1050	44	90	
4- 7-54	1300	46	84	
4- 8-54	1325	47	90	Sand in "Pre" tube darkening. Sand compaction—5/8'' Pre ¼'' Post.
4- 9-54	1125	43	86	Top ½'' "Post" sand starting to turn brown.
4-10-54	1340	42	88	
4-11-54	1200	39	90	
4-12-54	1535	49	84	Sand fading in "Pre" tube.
4-13-54	1330	57	90	
4-14-54	1455	58	87	
4-15-54	1119	41	93	
4-15-54	1552	41	96	
4-16-54	0815	38	91	
4-17-54	0935	30	91	
4-18-54	0950	28	89	
4-19-54	0815	26	87	
4-20-54	0810	25	83	
4-21-54	1252	23	82	Sand getting darker in both samples.
4-22-54	1303	22	83	
4-23-54	1153	21	78	
4-24-54	1339	22	84	
4-25-54	1258	22	86	
4-26-54	1551	20	86	
4-27-54	1100	20	83	
4-28-54	1045	24	81	
4-29-54	1414	25	88	
4-30-54	1115	19	80	
5- 1-54	1540	21	82	
5- 3-54	1257	16	72	End of Test—Sludge from lines deposited on sand.

FIGURE I

LTERED WATER
3OTTLE

EVACUATE SPECIMEN:
Close all valves except no.6, allow specimen to evacuate completely (approximately 1hr).

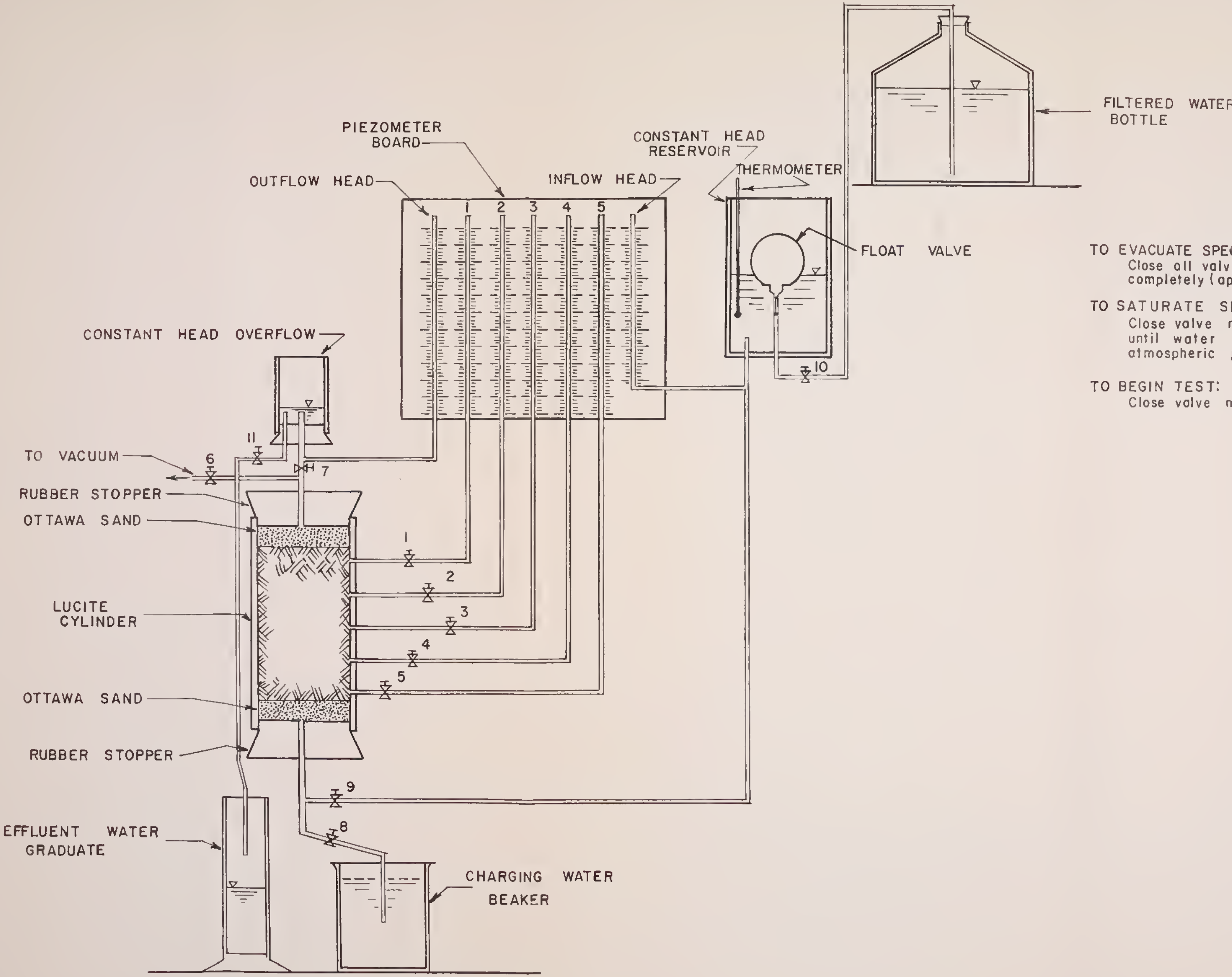
SATURATE SPECIMEN:
Close valve no. 6, open valve no. 8 to charging water until water rises to valve no. 7 & specimen is at atmospheric pressure.

BEGIN TEST:
Close valve no. 8, open all others except no. 6.

LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
CONSTANT HEAD
PERMEAMETER

FIGURE 1



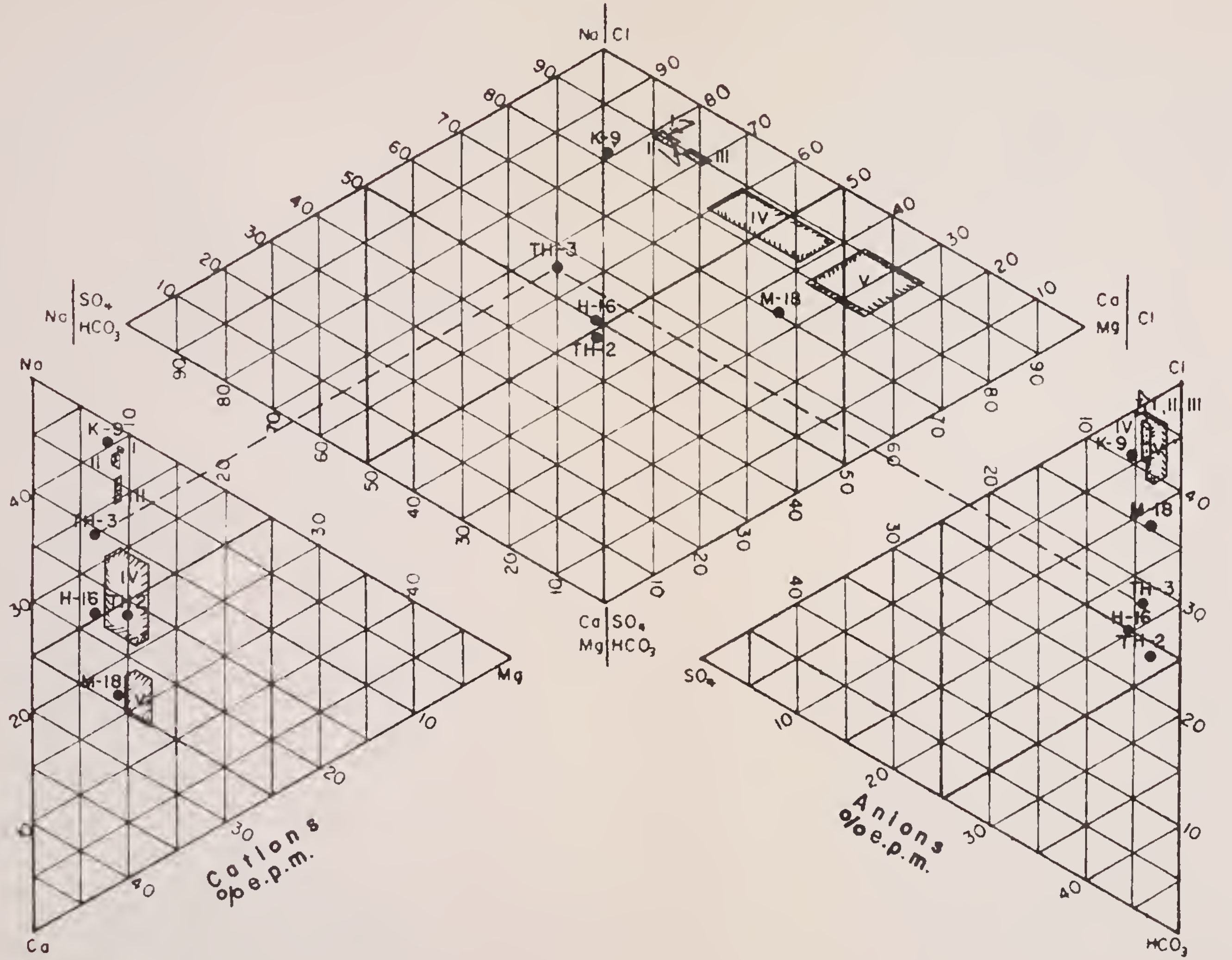
TO EVACUATE SPECIMEN:
Close all valves except no.6, allow specimen to evacuate completely (approximately 1 hr).

TO SATURATE SPECIMEN:
Close valve no. 6, open valve no. 8 to charging water until water rises to valve no. 7 & specimen is at atmospheric pressure.

TO BEGIN TEST:
Close valve no. 8, open all others except no. 6.

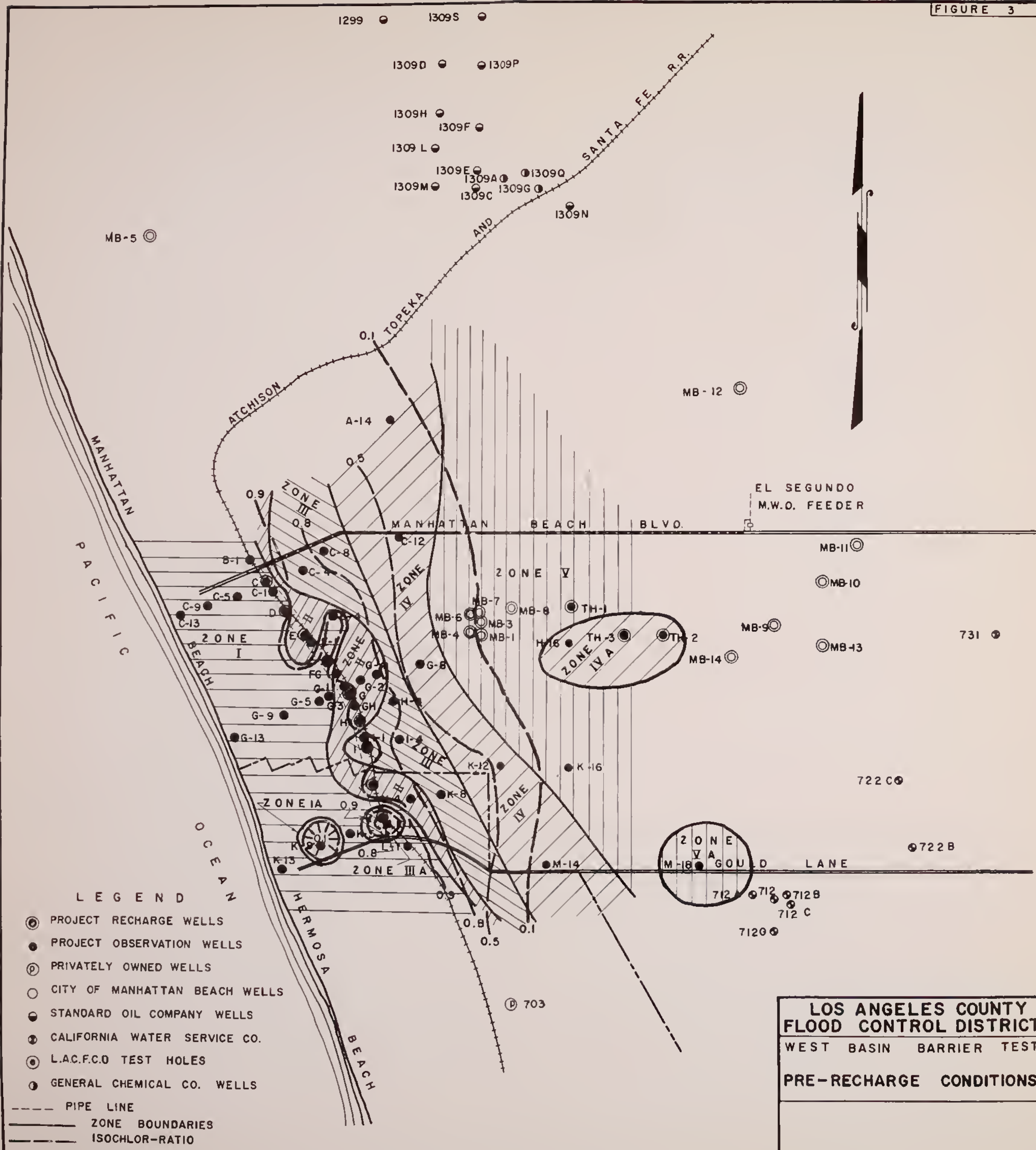
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
CONSTANT HEAD PERMEAMETER



LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
PRE-RECHARGE CONDITIONS



LEGEND

- PROJECT RECHARGE WELLS
- PROJECT OBSERVATION WELLS
- PRIVATELY OWNED WELLS
- CITY OF MANHATTAN BEACH WELLS
- STANDARD OIL COMPANY WELLS
- CALIFORNIA WATER SERVICE CO.
- L.A.C.F.C.D TEST HOLES
- GENERAL CHEMICAL CO. WELLS
- PIPE LINE
- ZONE BOUNDARIES
- ISOCHLOR-RATIO

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

PRE-RECHARGE CONDITIONS

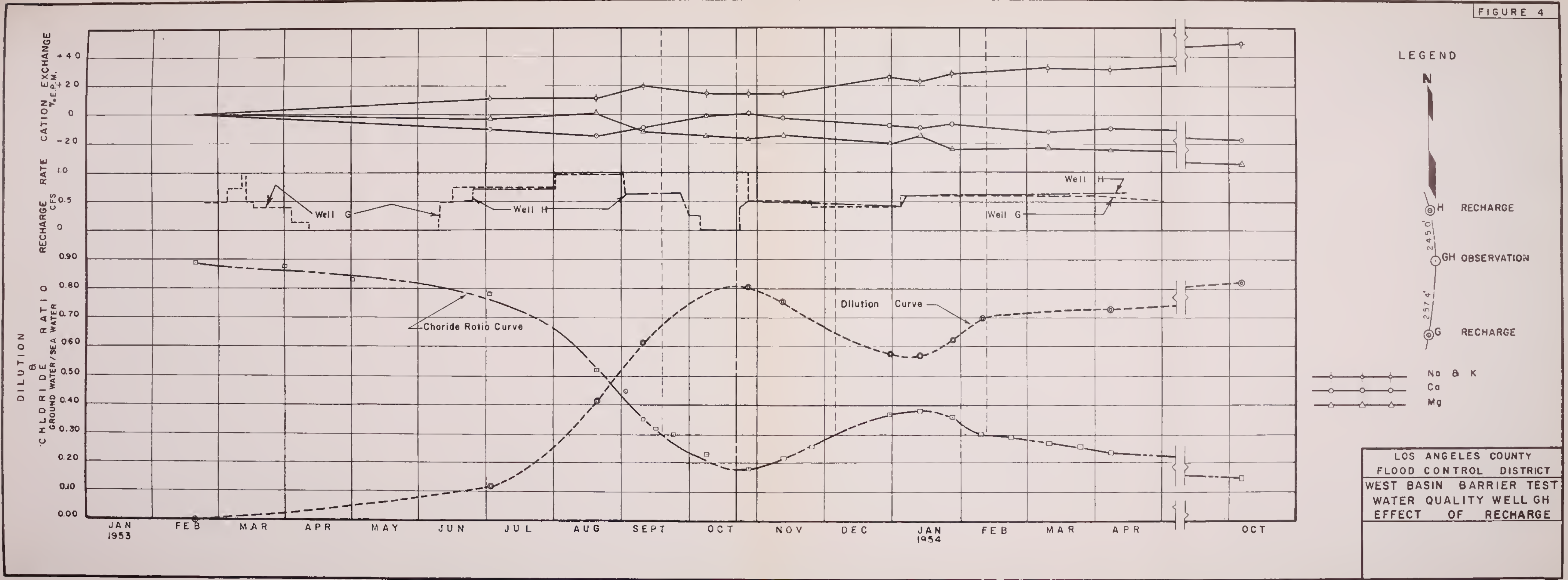
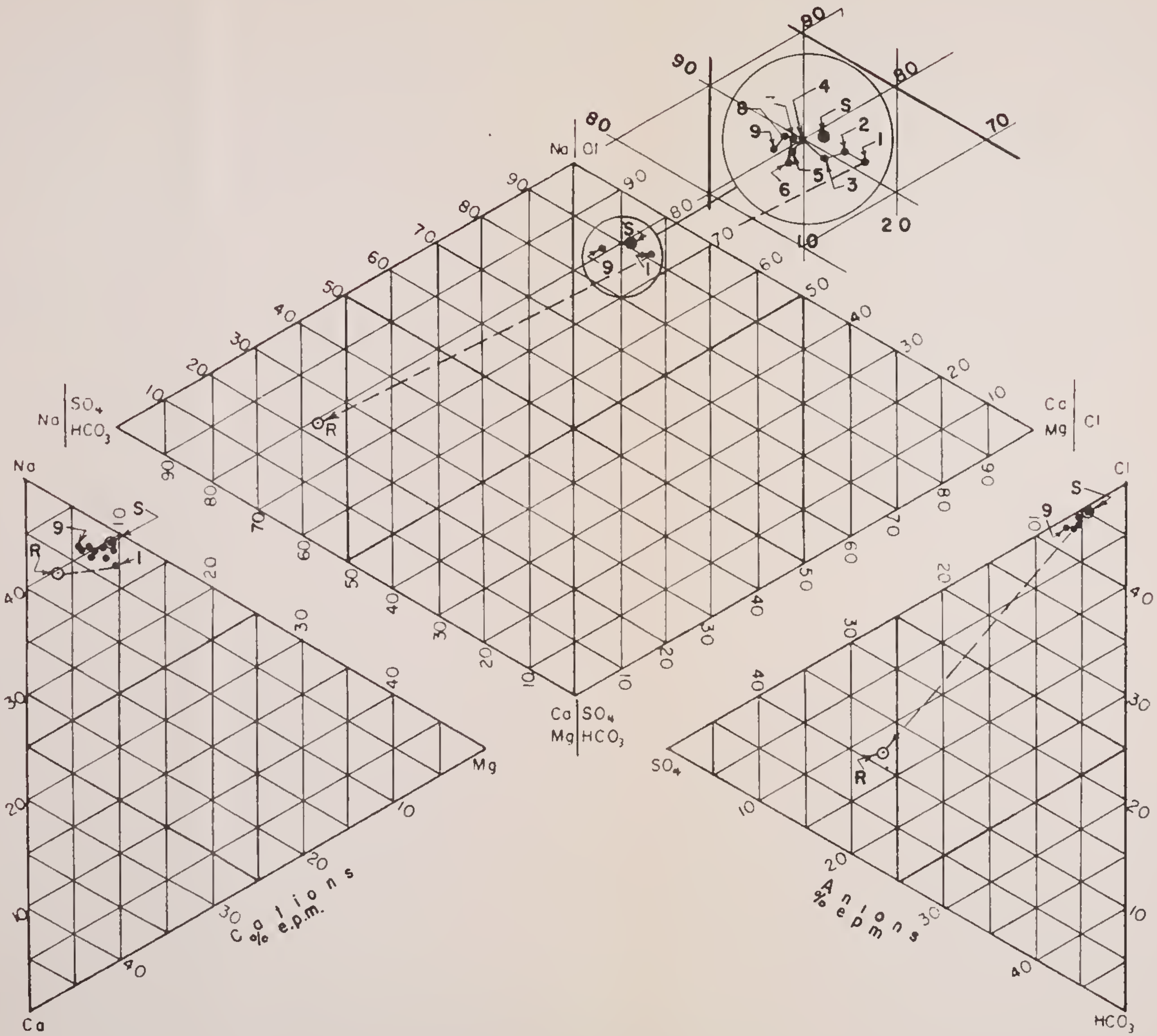


FIGURE 4
CATION EXCHANGE % E.P.M.
RECHARGE RATE CFS
DILUTION & CHLORIDE RATIO GROUND WATER/SEA WATER
Well G
Well H
Choride Ratio Curve
Dilution Curve
JAN 1953
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEPT
OCT
NOV
DEC
JAN 1954
FEB
MAR
APR
OCT
LEGEND
N
H RECHARGE
GH OBSERVATION
G RECHARGE
2450'
2574'
Na & K
Ca
Mg
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
WATER QUALITY WELL GH
EFFECT OF RECHARGE

FIGURE 5

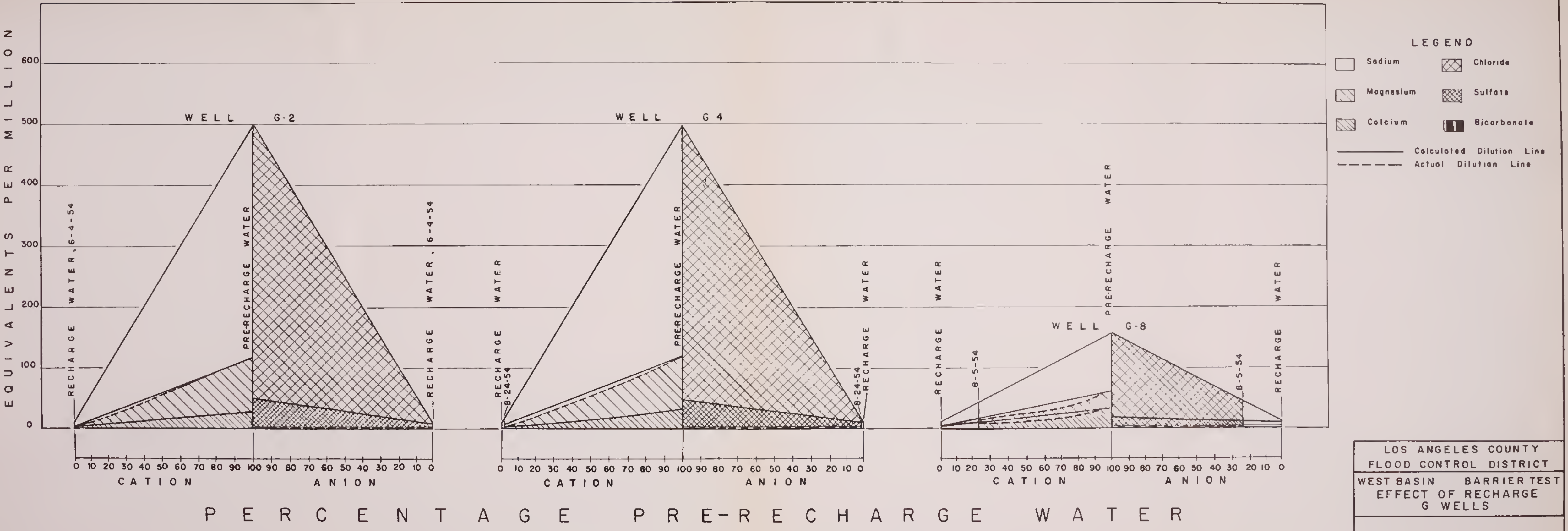


LEGEND

S SEAWATER
R RECHARGE WATER
SAMPLES FROM WELL GH
See Table 2A

LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
EFFECT OF RECHARGE
ON WELL GH



LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
EFFECT OF RECHARGE
G WELLS

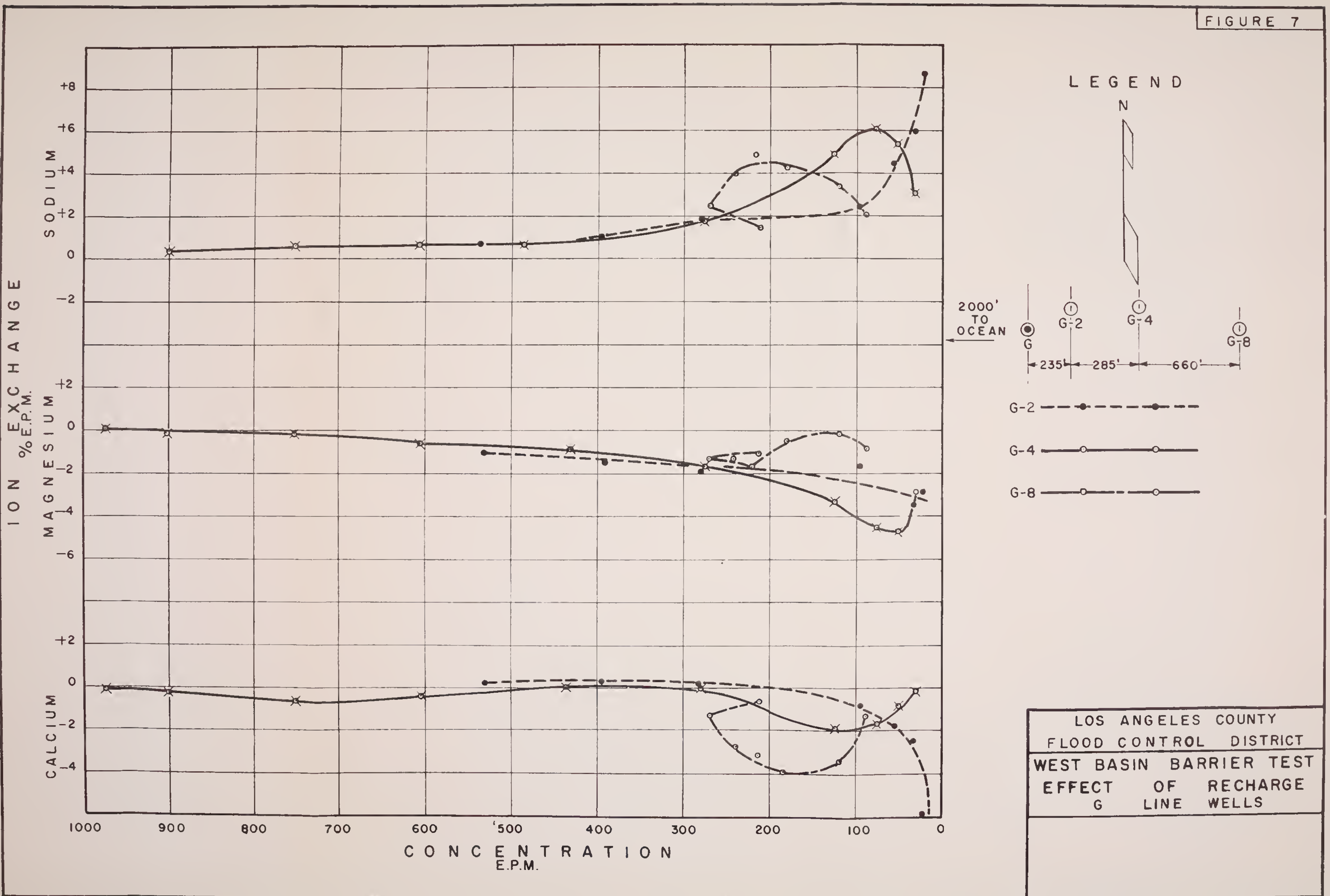
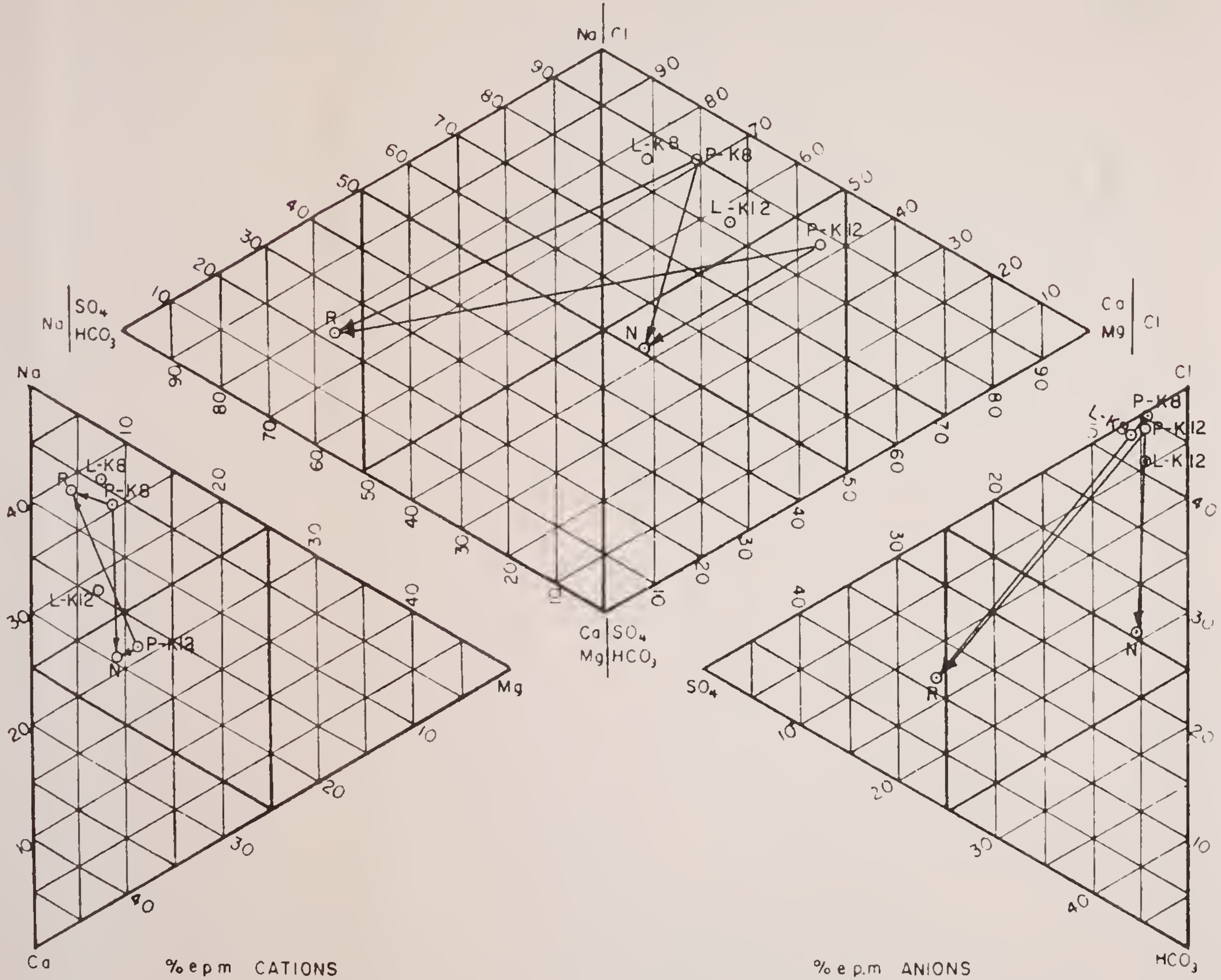


FIGURE 7
LEGEND
N
2000' TO OCEAN
G
G-2
G-4
G-8
235'
285'
660'
G-2
G-4
G-8
SODIUM
MAGNESIUM
CALCIUM
ION EXCHANGE % E.P.M.
+8
+6
+4
+2
0
-2
-4
-6
1000
900
800
700
600
500
400
300
200
100
0
CONCENTRATION E.P.M.
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT
WEST BASIN BARRIER TEST
EFFECT OF RECHARGE
G LINE WELLS

FIGURE 8



LEGEND

P	PRE-RECHARGE	WATER
N	NATIVE	WATER
R	RECHARGE	WATER
L	LATEST ANALYSES, 9/54	

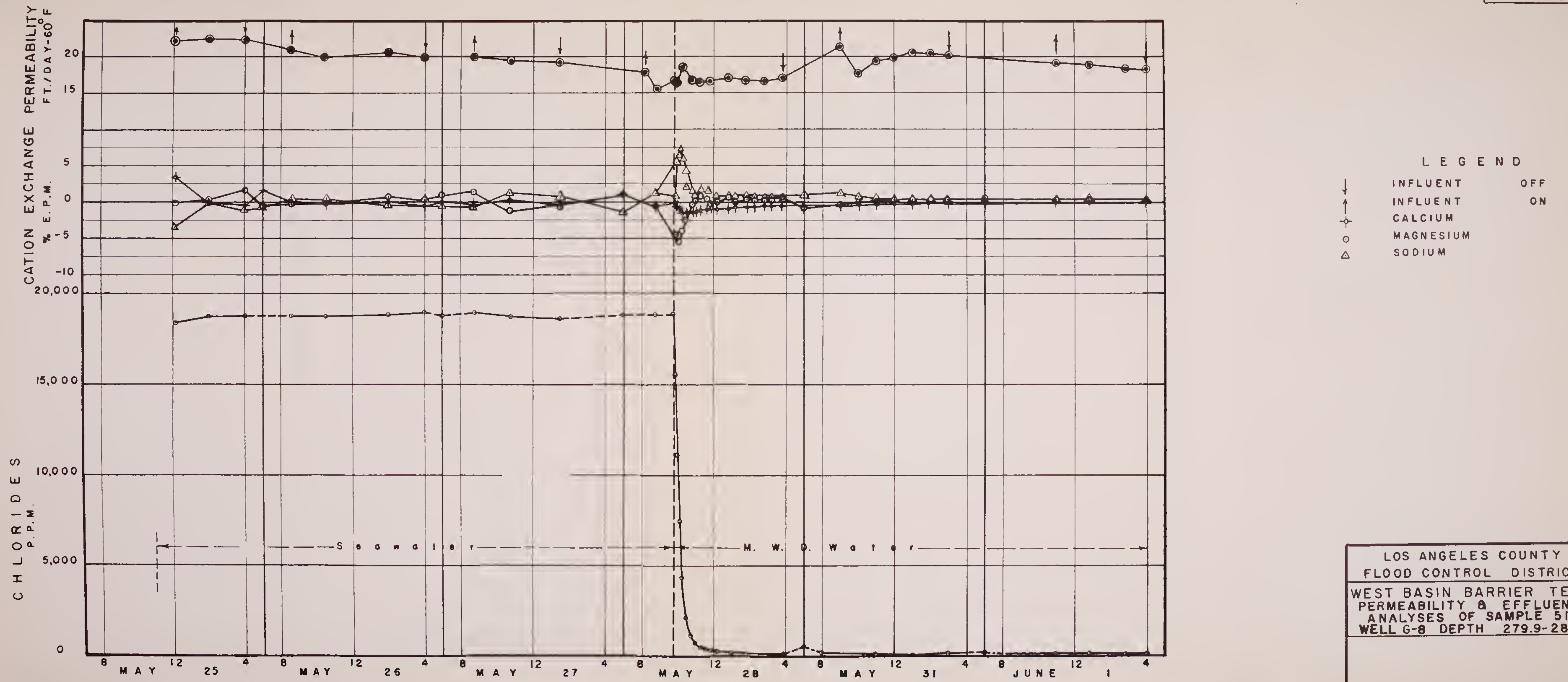
LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

SOURCE OF DILUTION

WELLS K-8 & K-12

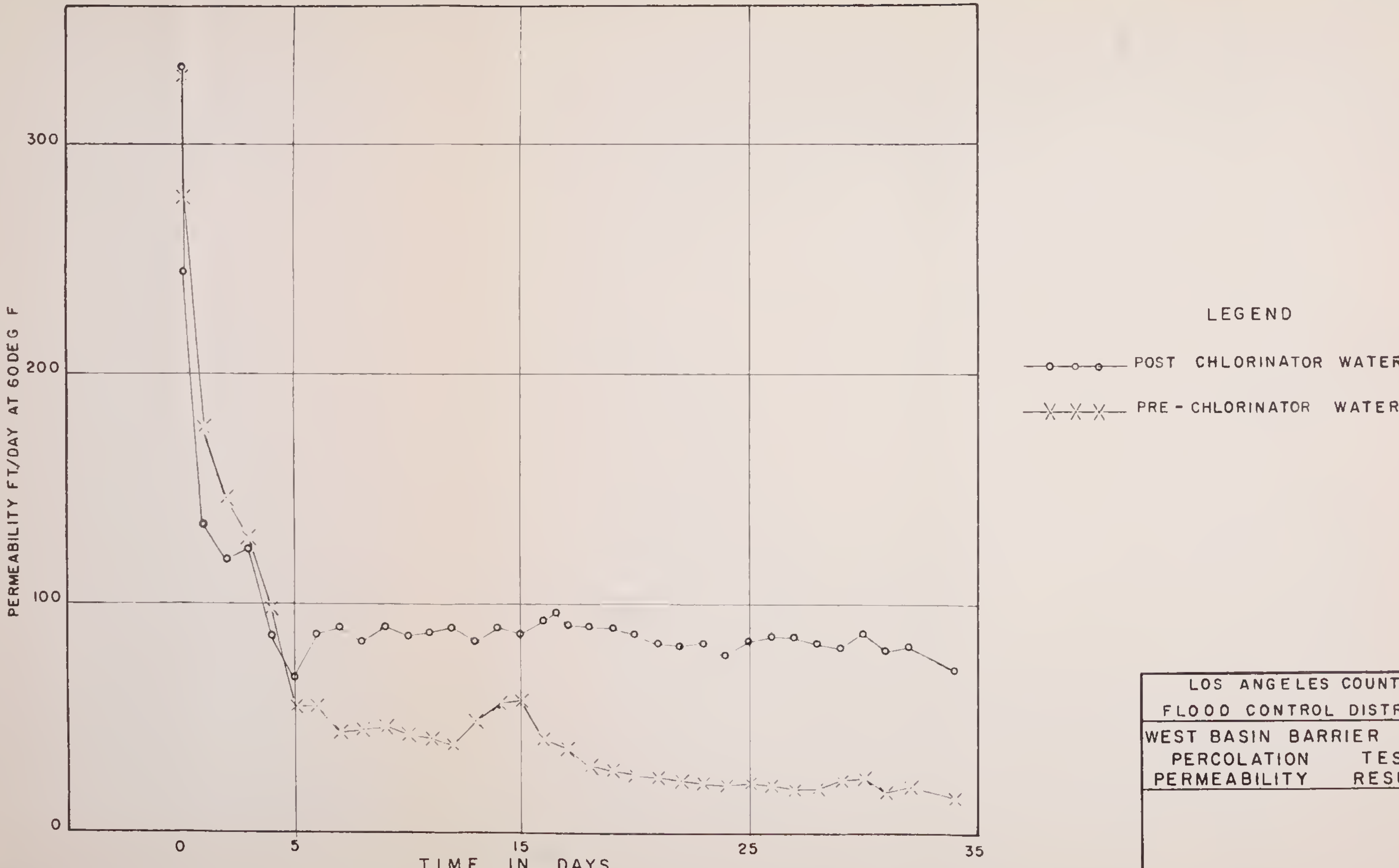
FIGURE 9



LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
PERMEABILITY & EFFLUENT
ANALYSES OF SAMPLE 51
WELL G-8 DEPTH 279.9-280.2

FIGURE 10



LOS ANGELES COUNTY
FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST
PERCOLATION TESTS
PERMEABILITY RESULTS

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
TESTING DIVISION

APPENDIX C

SPECIAL TESTS ON CLAY CAP MATERIALS RELATIVE TO INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA WATER INTRUSION

December 10, 1954

By IRVING SHERMAN

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Chemical Section Head

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Recommended by:
PAUL BAUMANN
Assistant Chief Engineer

Approved by:
H. E. HEDGER
Chief Engineer

9—52568

SPECIAL TESTS ON CLAY CAP MATERIALS RELATIVE TO INVESTIGATIONAL WORK FOR PREVENTION AND CONTROL OF SEA WATER INTRUSION

SUMMARY

This Appendix is a report describing several tests undertaken by the Testing Division in investigating the possible causes for the observed instances of destruction of the clay cap, in the West Coast Basin Barrier Project, which is a field test of ground-water recharge by wells.

The possible causes were considered in two groups, as follows:

1. Chemical causes, due to the nature of the recharge water used.
   - (a) A change in the plasticity of the clay cap, resulting in flow of the clay by failure in the plastic state.
   - (b) An increase in the erodability of the clay cap, due to dispersion, so that dispersed particles are carried through voids in the surrounding granular materials.
2. Physical causes, arising from the type of drilling operations used, and the methods of development of the wells.
   - (a) Erosion of the clay cap by lateral flow from the well.
   - (b) Erosion of the clay cap by vertical percolation.
   - (c) Erosion of the clay cap by flow through voids adjacent to the well casing.
   - (d) Physical collapse of the clay cap into adjacent large voids due to gravity alone.
   - (e) Collapse of the clay cap due to sudden pressure changes associated with well operation.

The tests made involved six different waters of varying chemical composition. The chemical causes were investigated by comparative tests of settling velocity, plasticity, and permeability of the clay with each water. These tests indicated the chemical effects to be relatively unimportant.

The physical causes were investigated by means of hydraulic models of the recharge wells, using "undisturbed" samples of the clay cap. By controlling the conditions of these models, it was found possible to reproduce the physical failures listed under 2(a), (c), and (e) above, but only the latter two were considered serious.

Recommendations are made for drilling any future wells so as not to create large voids adjacent to the casing, and for sealing the clay cap to the well casing by grouting.

Introduction

The West Coast Basin Barrier Project is essentially a field trial of a proposed method of halting the intrusion of sea water into the fresh water aquifer along the coast. The method consists of creating a barrier or ridge of fresh water, by pumping (recharging) fresh water into the aquifer through a line of recharge wells approximately parallel to the sea coast.

The success of the operation depends in part on the existence of a relatively impermeable stratum, generally called the clay cap, which is seven to twenty feet thick. This material lies immediately above the aquifer. In the vicinity of the recharge wells it is seven to twenty feet thick and lies at a depth of eighty to one hundred feet. This clay cap, being quite impermeable prevents upward movement of the recharged water and thus forces it to spread laterally and form a continuous body rather than a series of disconnected water "mounds" at the wells.

Shortly after the start of the recharge operations, it was found that the clay cap had been greatly disturbed in the vicinity of two of the recharge wells. This disturbance was evidenced by two phenomena: (1) the appearance of recharge water in the normally dry dune sands lying above the clay cap, and (2) subsidence of the ground surface around the well.

The Chemical Section of the Testing Division was asked to investigate the possible causes of destruction of the clay cap, and if possible, to recommend methods of restoring some impermeable continuous layer in the clay cap region. This report summarizes the investigations and results thereof.

Hypotheses of the Destruction Mechanisms

Several possible causes for the clay cap failure have been proposed. Generally speaking, they can be divided into two groups: (1) Chemical causes, due to the chemical composition of the recharge water, and (2) physical causes, related to the hydraulic conditions which exist at the wells. A more detailed listing and description follows:

1. *Chemical Causes*

The recharge water is obtained from the Metropolitan Water District. Being zeolite-treated and chemically softened water from the Colorado River, it is high in both total dissolved solids and percent of sodium. It is known that sodium tends to disperse clays and change their physical properties.

Two such changes considered were:

(a) A change in the plasticity of the clay cap resulting in an increased tendency for the material to flow in the direction of the hydraulic gradient by failure of the mass in the plastic state.

(b) An increase in the erodability of the clay cap due to the dispersion by sodium ions. It has been shown that dispersed material, acting as individual particles rather than aggregations of particles, may be more easily eroded by flowing water. This hypothesis visualized dispersed particles being carried through voids in the dune sand above or the "Silverado zone" aquifer below, though the voids were too small to permit passage of particle aggregations.

2. *Physical Causes*

The well holes were drilled by a cable tool rig rather than a rotary drill rig. It is known that the holes thus created were somewhat larger in diameter with irregular voids outside of the casing because of the type of action of the drill tool.

Prior to the observed destruction of the clay cap, no measures were taken to seal any gaps which may have existed at the clay cap.

Necessary pumping to develop the wells and remove fines from the aquifer immediately adjacent to the well created some voids, of unknown size and distribution. Unavoidable interruptions of the pumping or recharge operations resulted in rapid changes of pressure conditions within the system.

Hypotheses of primarily physical destruction included:

(a) Erosion of clay cap material by water flowing horizontally outwards from the well along the bottom of the clay cap.

(b) Erosion by water flowing upwards through the clay cap mass under the unnaturally high hydraulic gradient produced by high rates of recharge.

(c) Erosion by water flowing upwards through the voids in the clay cap adjacent to the well casing, removing particles from the exposed clay surface.

(d) Physical collapse due to the action of gravity alone, of the clay cap into adjacent large voids in the aquifer, which were created during development of the wells.

(e) Physical collapse of the clay cap into adjacent large voids due to sudden pressure changes resulting from rapid fluctuation in rates of recharge, or shut-down.

It is here emphasized that the existence of any one condition tending to cause failure does not rule out the existence of other conditions. Two or more may conceivably occur simultaneously.

Tests and Investigations

The first tests performed were intended to test the chemical hypotheses. To a large degree the tests were improvised, as previous standards for such procedures were largely lacking.

The chemical tests involved the use of several different waters, as follows:

(1) Distilled water

(2) MWD treated water (used in the recharge operations at the West Basin)

(3) MWD water before treatment—a high calcium water, high total dissolved solids

(4) San Gabriel River water—low in total dissolved solids, but high in percentage of calcium

(5) Sea water—very high total dissolved solids, high percent sodium

(6) MWD treated water saturated with calcium sulfate so as to make it a high calcium water again

Data on these waters is presented in Table I.

TABLE I

CHEMICAL COMPOSITION OF TEST WATERS

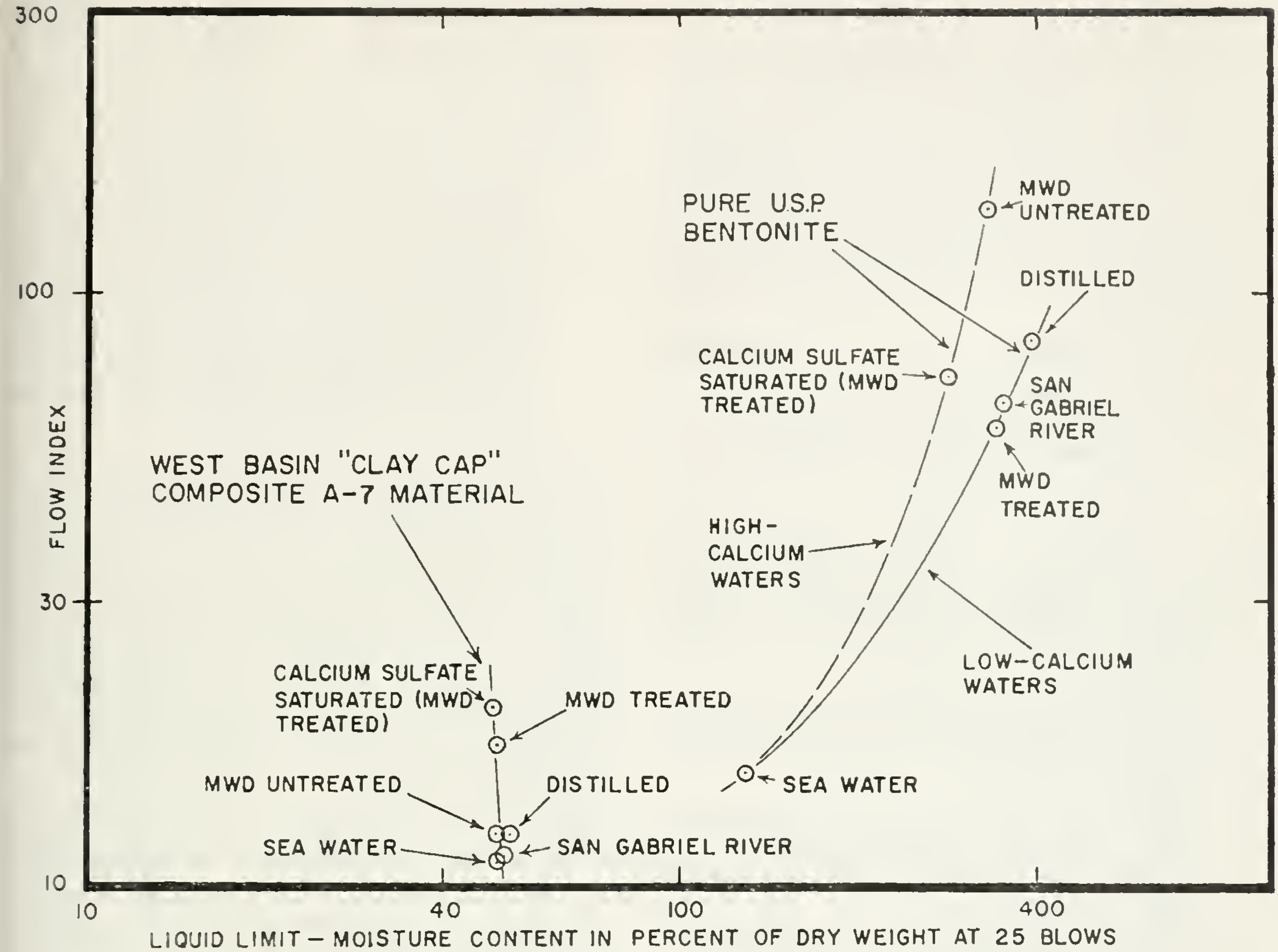
Water	Total Dissolved Solids, epm	Total Hardness, epm	Na & K epm %	Ca epm %	$\frac{Na+K}{Ca}$
MWD treated (at Well G)	674	134.3	37.12	9.48	3.91
MWD untreated	636	301.0	20.95	18.87	1.11
San Gabriel River	297	174.0	8.79	30.76	0.29
Sea Water	36,360	6,539.9	38.8S	1.72	22.6
MWD treated Ca SO_4 Saturated	2,7S3	1,554.9	9.68	39.19	0.25

A number of disturbed samples of clay cap material were available for the chemical tests. All were classified by means of the Atterberg tests, (ASTM Designations D-423-39 and D-424-39) and found to fall into three general classes according to the BPR system—A4, A6, and A7. Samples belonging to each class were then mixed together to give three composite samples which were used for further tests.

The Atterberg tests were then performed on composite sample A7, using distilled water and also the

FIGURE I

EFFECTS OF WATER ON PLASTICITY



waters listed in Table I. The purpose was to investigate possible changes in plasticity due to water composition. The tests were made only on the A7 sample because it contained a higher percentage of clay than the other two, and thus would be more sensitive to chemical effects. Test results are presented in Table II and Fig. I.

Details of the test procedure are given in the supplement following this report. Pure bentonite was also tested, because it is known to be sensitive to chemical changes and thus was used as a standard.

The "Flow Index" shown in the fourth column of Table II measures the slope of the Liquid Limit curve, and is defined as the change in moisture content required to change the number of blows in the Liquid Limit Test from ten to one hundred. A low flow index indicates a rapid increase in fluidity with increasing moisture content; a high flow index indicates a slow increase.

TABLE II

EFFECTS OF WATER ON ATTERBERG LIMITS

Water	Liquid Limit	Plastic Limit	Plasticity Index	Flow Index
(a) West Basin A7 Clay Cap				
Distilled	52.0	25.9	26.1	12.2
MWD treated	49.8	27.3	22.5	17.2
MWD untreated	49.6	27.4	22.2	12.2
San Gabriel River	50.7	27.0	23.7	11.2
Sea Water	49.2	25.3	23.9	10.8
MWD treated, Ca SO_4 Saturated	48.8	25.2	23.6	19.8
Mean	50.0	26.4	23.7	13.9
Range	3.2	2.2	3.9	9.0
(b) Bentonite, USP				
Distilled	393	48	345	83
MWD treated	341	45	296	59
MWD untreated	332	46	286	139
San Gabriel River	351	45	306	65
Sea Water	129	48	81	15.4
MWD treated, Ca SO_4 Saturated	283	46	237	72
Mean	305	46.3	256	72
Range	264	3	264	123.6

It will be seen from this data that the bentonite is far more sensitive than the A7 material. For example, using $Ca\ SO_4$ saturated water instead of distilled water changes the liquid limit of bentonite by 28%, as compared with only 6.2% for the A7 material. This is to be expected from the lower clay content of the A7 material (see mechanical analysis results below) as contrasted with the 100% clay content of the bentonite. However, on the absolute scale, the plasticity effects on the A7 material are small. It is therefore concluded that the "plastic flow" hypothesis is not supported by the evidence.

Experimental hydrometer analyses were also made for three purposes:

(a) To test the sensitivity of the clay to chemical treatment when in the form of a clay suspension rather than a paste.[1]
(b) To evaluate the effect of each water, and
(c) To obtain data which might indicate the relative erosion characteristics.

The general plan of this phase was taken from previous work by the writer (See Sherman, I., "Flocculent Structure of Sediment Suspended in Lake Mead," Transactions, Amer. Geophys. Union, Vol. 34, No. 3, pp. 394-406, June 1953). Again the A7 clay was used.

Duplicate hydrometer analyses were made on the A7 composite samples using the following waters in the hydrometer jars:

(1) Distilled water only
(2) Distilled water with sodium silicate dispersing agent, (Standard laboratory procedure).
(3) Distilled water with sodium hexa-metaphosphate dispersing agent.
(4) MWD treated water as received at Well G
(5) MWD untreated water
(6) San Gabriel River water
(7) Sea water

Details of the hydrometer technique are given in the supplement following this report. The test results are presented in Figures 2 and 3 and Table III:

TABLE III

EFFECT OF WATER ON CLAY BEHAVIOR

Water	Mean Settling Velocity, cm/sec	Mean Floccule Density gm/cm³	Mean Floccule Diameter mm
MWD treated	.0205	1.49	.0277
MWD untreated	.0228	1.43	.0312
San Gabriel River	.0254	1.48	.0312
Sea Water	.0239	1.49	.0319

[1] The sensitivities may not be the same under both conditions. See R. E. Grim, in "Symposium on Exchange Phenomena in Soils," ASTM Spec. Pub. No. 142, 1953, p. 7.

Floccule data are presented only on the natural waters. It was found that in all of the distilled-water treatments the samples were essentially dispersed, so that floccules did not exist to any great extent. The data on all treatments are presented graphically in Figures 2 and 3.

The hydrometer results are consistent with chemical theory and lead to the following conclusions:

(1) The clay cap material is sensitive to chemical dispersion treatments, and appears to be probably a sodium clay in the original state.
(2) Waters with low Na/Ca ratios and/or high total dissolved solids tend to flocculate the clay.
(3) The quantitative differences between the diameters of floccules produced by the different waters are probably not large enough to have any significant effect on the phenomena at the wells.

Computations were made of the pore diameters in the aquifer from the mechanical analyses of aquifer samples, as compared to the floccule diameters in the hydrometer test. It was found that even the largest floccules formed could individually pass through the aquifer pores, which naturally were of the order of several times the floccule diameters, even without the presence of large voids caused by well development. This indicates that further flocculating the clay by a change in water composition by calcium sulfate treatment would not have a large beneficial effect on preserving the clay cap.

A third set of tests involved percolating each of the different waters through a specimen of clay cap material, to measure resulting differences in permeability and rates of erosion. The apparatus was patterned after that of J. E. Christiansen ("Some Permeability Characteristics of Saline and Alkali Soils," Agric. Eng. Vol. 28, No. 4, pp. 147-150, April, 1947). However, in order to get appreciable rates of flow it was necessary to "dilute" the clay cap material with Ottawa sand. The final mixture consisted of 15% of A7 clay cap composite sample, 85% of Ottawa sand, compacted by a modified miniature Proctor method between layers of pure Ottawa sand. In order of decreasing rates of clay erosion, the different waters are:

(1) Distilled
(2) MWD treated as received at Well G
(3) $Ca\ SO_4$—Saturated MWD treated water
(4) MWD untreated water
(5) San Gabriel River water (No erosion)
(6) Sea Water (No erosion)

The coefficient of permeability was the same with all waters except the $Ca\ SO_4$ saturated water, which appeared to flocculate the clay extremely and which produced a higher permeability. The increased volume of flow resulted in appreciable erosion, as contrasted with other flocculating waters.

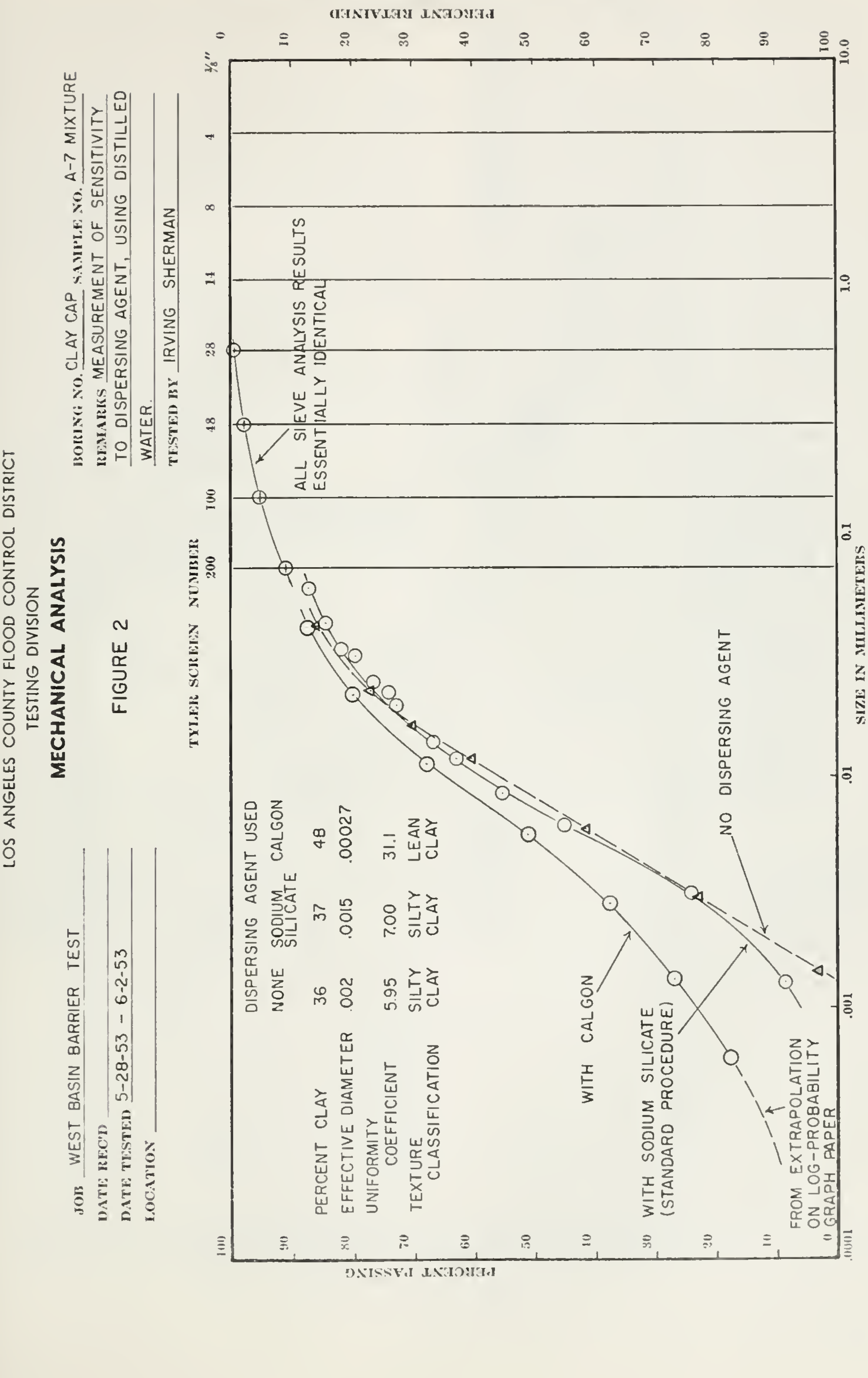
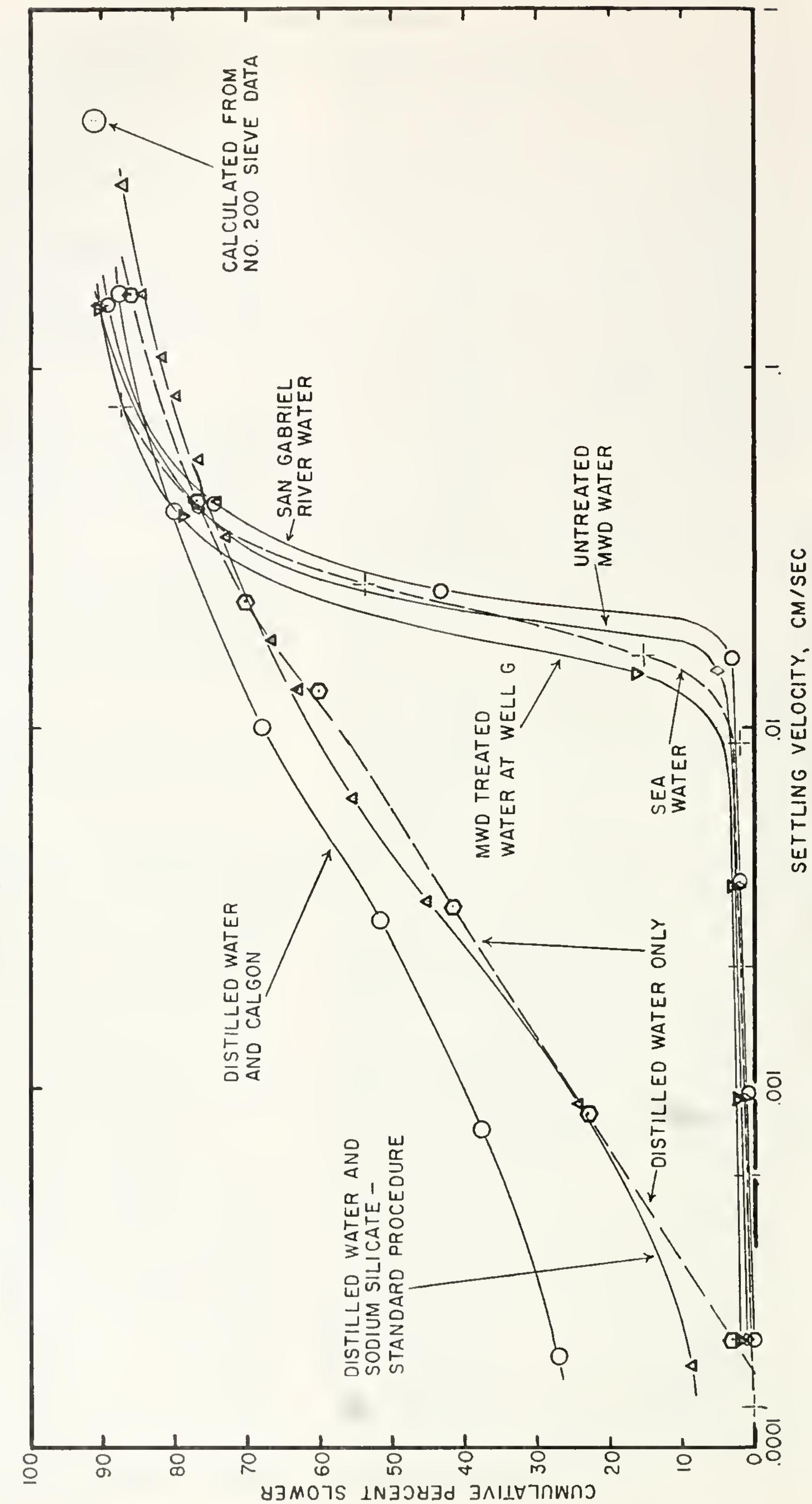

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
TESTING DIVISION
MECHANICAL ANALYSIS
FIGURE 2
JOB WEST BASIN BARRIER TEST
DATE REC'D
DATE TESTED 5-28-53 - 6-2-53
LOCATION
BORING NO. CLAY CAP SAMPLE NO. A-7 MIXTURE
REMARKS MEASUREMENT OF SENSITIVITY TO DISPERSING AGENT, USING DISTILLED WATER.
TESTED BY IRVING SHERMAN
TYLER SCREEN NUMBER
⅜" 4 8 14 28 48 100 200
PERCENT RETAINED
PERCENT PASSING
SIZE IN MILLIMETERS
10.0 1.0 0.1 .01 .001 .0001
ALL SIEVE ANALYSIS RESULTS ESSENTIALLY IDENTICAL
DISPERSING AGENT USED
NONE SODIUM SILICATE CALGON
PERCENT CLAY 36 37 48
EFFECTIVE DIAMETER .002 .0015 .00027
UNIFORMITY COEFFICIENT 5.95 7.00 31.1
TEXTURE CLASSIFICATION SILTY CLAY SILTY CLAY LEAN CLAY
NO DISPERSING AGENT
WITH CALGON
WITH SODIUM SILICATE (STANDARD PROCEDURE)
FROM EXTRAPOLATION ON LOG-PROBABILITY GRAPH PAPER

FIGURE 3

WEST BASIN BARRIER TEST

EFFECT OF WATER ON BEHAVIOR OF CLAY CAP MATERIAL (BPR CLASS A-7) IN THE HYDROMETER ANALYSIS



Although the results of the Atterberg, Hydrometer and Percolation Tests are indicative, it is hazardous to extend the results to the West Basin problem because the tests mentioned above do not actually duplicate field conditions. So a fourth set of tests was made in which approximate hydraulic models of a West Basin recharge well were subjected to varying conditions. A drawing of the apparatus is given in Fig. 4.

Four models were set up. The essential differences were in the nature of the "clay cap" specimens which were as follows:

Model No. 1—Well G, Sample #26, depth 101.0′-101.3′, field description: very fine sandy silt. Specimen placed with essentially no disturbance.

Model No. 2—Well G, Sample #20, depth 92.2′-92.5′, field description: clayey silt. Specimen seriously disturbed at periphery during placement. (G-20-0.)

Model No. 3—Same as No. 2, but specimen less seriously disturbed. (G-20-1.)

Model No. 4—Same as Nos. 2 and 3, but specimen allowed to swell by access to distilled water before placement. Specimen placed with no obvious disturbance.

Table IV summarizes some of the soil data for the hydraulic models:

Model No. 1 was abandoned as being too permeable; No. 2 was abandoned because of the excessive peripheral disturbance.

The peripheral voids in the specimen in Model No. 3 enlarged progressively until a continuous channel through the specimen had been created. At that time the flow of water into the "Well" was shut off, thus reversing the direction of the hydraulic gradient. This caused the "clay cap" to collapse and sand from the overlying stratum fell into the voids thus created.

No large peripheral voids existed in Model No. 4. Some small voids originally present persisted without enlargement. The permeability increased, but surge tests failed to collapse or visibly damage the specimen. Six such tests were performed. During the tests of the model, "clay cap" material in measurable quantities was deposited in the overflow beaker, and more "clay cap" material was subsequently found in the "Silverado Zone" after disassembly of the model. The only possible source for this material seemed to be the lower surface of the "clay cap." The mechanism of removal could have been erosion by lateral flow. The process of erosion was not directly observable however.

TABLE IV

	Model No's.			
	1	2	3	4
Depth below ground surface	101.0-101.3	92.2-92.5		
Soil Classification	Very fine Sandy Silt	Clayey Silt		
Original Dry Density, p.c.f.	92.4	98.1		
Original Moisture Content	44.3%	26.9%		
Dry Density in experiment, p.c.f.	92.4	98.1	98.1	93.4
Co-efficient of Permeability, Ft/day	High but not measured	2.8×10^{-3}	1.8×10^{-2} 1.4×10^{-2}	5.2×10^{-4} (Initial) 3.0×10^{-3} (Final)
Length of test, days	1	1	13	32
Result of surge tests	Not tried	Not tried	Spec. failure	No failure

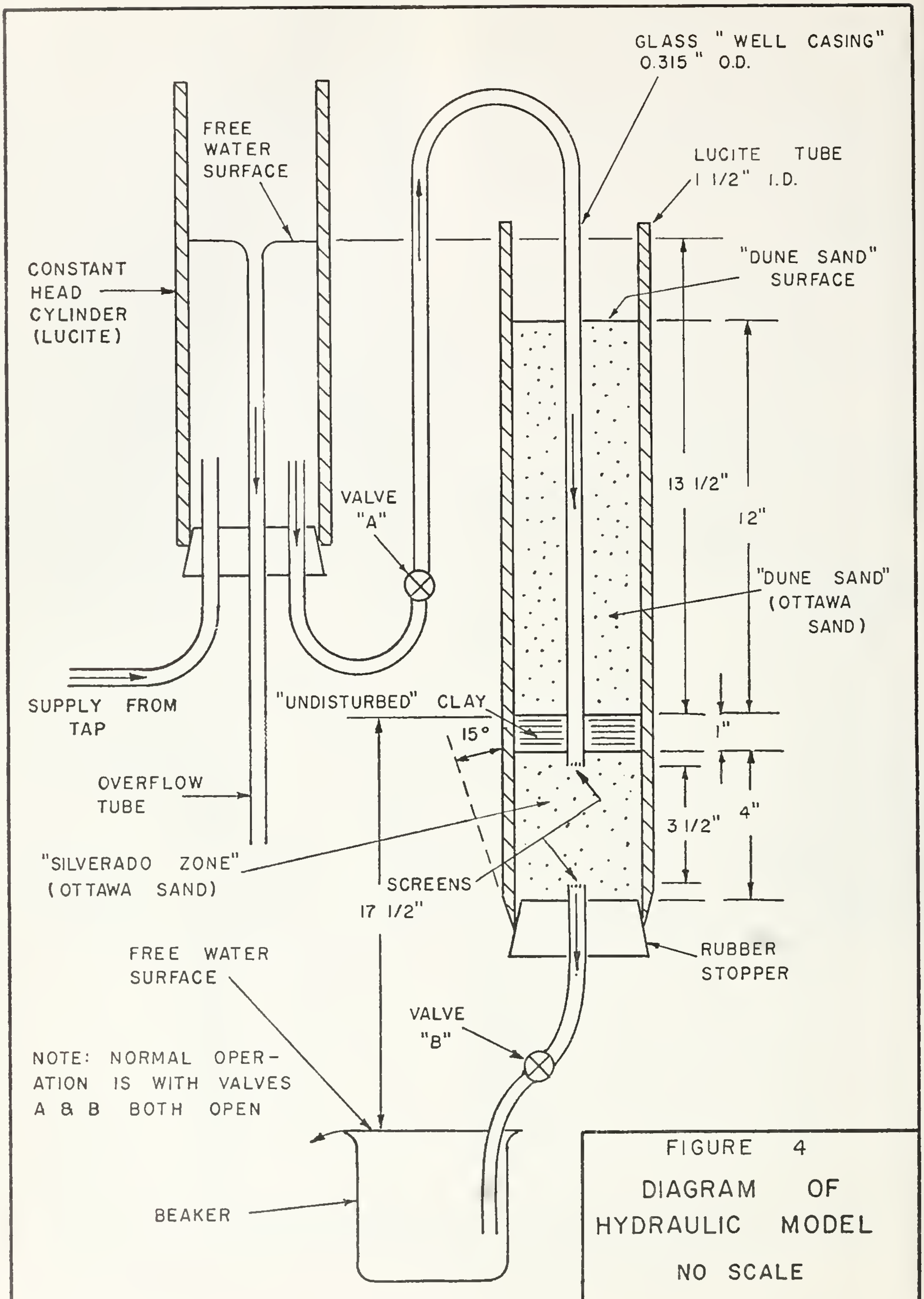


FIGURE 4

DIAGRAM OF HYDRAULIC MODEL

NO SCALE

The following is the material balance for the destroyed "clay cap" specimens of Models Nos. 3 and 4:

TABLE V

CLAY CAP MATERIAL BALANCE

	Model No. 3		Model No. 4	
Before destruction				
Original volume of specimen, cm^3	29.25		29.25	
Weight in grams of undistrubed volume	46.0		43.7	
Estimated volume of peripheral voids, percent	5		1	
Estimated weight of specimen in the model, gms	43.7		43.2	
	Percent	Grams	Percent	Grams
After destruction or dis-assembly				
Weight of specimen remnant in place	81.8	35.72	91.9	39.68
Weight eroded upwards into "dune sand"	4.1	1.80	0.2	0.10
Weight found in "Silverado Zone"	7.8	3.41	3.0	1.29
Weight found in overflow beakers	5.3	2.32	1.1	0.48
Actual total recovered	99.0	42.25	96.2	41.55
Estimated weight which passed through overflow beakers, based on sieve analyses of original and material found in overflow beakers	1.0	0.45	0.4	0.20
Estimated total	100	43.70	96.6	41.75

Conclusions and Recommendations

1. The quality or chemical composition of the water used probably has relatively little effect on what has happened at the West Basin.

2. Any deleterious effect of MWD-treated water would probably not be reversed by using Ca SO_4—saturated water.

3. If the clay cap is not disturbed by large unsealed voids, it will not be destroyed by well operation. But this excludes large void spaces around the casing such as are produced by cable drilling tools.

4. Any future wells should preferably be drilled by equipment which does not create the disturbances, impacts, and large voids produced by the cable tool, and the clay cap sealed to the casing by grout.

5. There may be a certain amount of erosion of the lower surface of the clay cap due to horizontal flow through the aquifer during recharge. The model tests indicate that such erosion will not be serious if there are no large voids in the clay cap where turbulence could become a factor.

SUPPLEMENT

DESCRIPTION AND DISCUSSION OF TECHNIQUES

I. Hydrometer Analysis

The standard procedure used at the Testing Division was basically that of ASTM Designation D-422-39 (see "Procedures for Testing Soils" 1950). Procedures used in this experiment follow that standard method except as noted below. The paragraph numbers follow the numbering scheme of Method D-422-39. ASTM methods were followed where paragraph numbers are omitted. The notation (FCTDS) indicates a procedure which was standard at the Testing Division.

2. *Apparatus*

(f) Sieves. Those used were as follows:

Tyler No. 14	(1.168 mm)
U. S. No. 30	(0.59 mm)
Tyler No. 40	(0.38 mm)
Tyler No. 48	(0.295 mm)
Tyler No. 100	(0.147 mm)
Tyler No. 200	(0.074 mm)

(FCTDS)

3. *Sample*

All portions for different treatments were obtained by use of a splitter for small samples. The weight of each portion was intended to be approximately 40 grams air-dry, in order to conserve the small amount of sample available. Actual weights of the portions varied between 37.13 gm and 45.70 gm.

One portion was oven-dried and then weighed. (FCTDS)

All portions were taken from material passing the Tyler No. 14 sieve.

4. *Hygroscopic Moisture*

The determination was made in duplicate on portions of approximately 14 grams each, obtained by splitting.

5. *Dispersion of Soil Sample*

All water was filtered. The seven treatments used were as follows:

(a) ASTM Standard with sodium silicate. This was used on the portion which had been oven-dried.

(b) 50 cc of 5% Calgon solution (essentially sodium metaphosphate, buffered to pH 9 with sodium carbonate) instead of sodium silicate; added at the start of the soaking period instead of at the end. This is essentially the new ASTM specification not yet published.

(c) No dispersing agent. Distilled water.

(d) No dispersing agent. Treated MWD water as received at Well G used instead of distilled water.

(e) No dispersing agent. Untreated MWD water used instead of distilled water.

(f) No dispersing agent. San Gabriel River water (from the reservoir behind San Gabriel Dam) used instead of distilled water.

6. *Hydrometer Test—time of readings*

(a) For the portion treated with sodium silicate, hydrometer readings were taken at the following times: ½, 1, 1½, 2, 3, 4, 5, 10, 15, 30, 60, 240, and 1440 minutes. Because of the close spacing of the first readings it was not possible to remove the hydrometer after each reading until after the five minute reading. (FCTDS).

(b) For all other portions, hydrometer readings were taken at the following times: 1, 4, 16, 64, 256, and 1260 minutes. The main purpose of taking fewer readings was to save time and speed the work.

(c) The portion treated with Calgon, because it settled most slowly, was also read at 93 hours in an effort to approach the "ten percent finer" point.

(d) The portions treated wtih sea water, San Gabriel River water and distilled water only, were also read at 8 minutes.

7. *Sieve Analysis*

After the last hydrometer reading, each portion was treated as follows:

(a) The portions treated with San Gabriel River and sea water were washed on the No. 40 sieve, using their respective waters. This was done with the intention of collecting the material passing the No. 40 sieve for Atterberg tests. The material retained was then oven-dried and weighed.

(b) All other portions were washed on the No. 200 sieve, oven-dried, and weighed. They were then dry-sieved through the sieves listed in paragraph 2 (f) above. (FCTDS except for the Tyler No. 40). The Tyler No. 40 sieve was included for comparison with the San Gabriel River and sea water portions.

Calculations

10. *Percentage of Soil in Suspension*

(a) Temperature corrections were all obtained from a graph similar to Fig. 6a of D-422-39, but reading in degrees centigrade. (FCTDS).

(c) The percentage of soil in suspension represented by a hydrometer reading was calculated from the formula:

$$W\% = \frac{(R_H + C_M + M_T + C_D)100}{W_s} \text{ (FCTDS)}$$

in which

W_s = original weight of sample, oven-dry basis, grams

$W\%$ = percentage of soil in suspension

R_H = hydrometer reading at the top of the meniscus, gm/l

C_M = meniscus correction

M_T = temperature correction

C_D = correction for the density of the suspension fluid, as compared with distilled water.

The values of C_D obtained for the treatments, in terms of Type A hydrometer readings, were

(a) Standard sodium silicate:	$C_D =$ 0.9 gm/l
(b) Calgon:	$C_D =$ 3.9 gm/l
(c) Distilled water only:	$C_D =$ 0.0
(d) MWD treated water (Well G):	$C_D =$ 1.0 gm/l
(e) MWD untreated water:	$C_D =$ 1.3 gm/l
(f) San Gabriel River water:	$C_D =$ 0.1 gm/l
(g) Sea water:	$C_D =$ 40.4 gm/l

11. *Diameter of Soil Particles in Suspension*

(a) The maximum diameter of particles in suspension, corresponding to the percentage indicated by a given hydrometer reading, was calculated from Stokes Law[2] as given in ASTM method D-422-39. The values inserted in the equation were as follows:

n = coefficient of viscosity of the suspending medium, taken from the nomogram as a function of temperature (FCTDS) except for sea water; the value of η for sea water was the nomogram value multiplied by 1.08, which is the viscosity of sea water in centipoises at 20° C.

L = distance in centimeters through which soil particles settle. Taken as the effective depth of the hydrometer at the reading in question. Effective depths

[2] Stokes Law $d = \sqrt{\frac{30\ L\ n}{980\ (G-G_1)\ T}}$

were determined by calibrating the hydrometer according to a modification of the method proposed by Edward E. Bauer (see p. 148, "Procedures for Testing Soils," 1950). The modification consisted of actually determining the center of volume of the hydrometer bulb, which was asymmetrical, instead of assuming it to be at the midpoint of the length of the bulb. (see p. 79, "Procedures for Testing Soils," 1950).

G = specific gravity of soil particles. Assumed as 2.70. The actual value was not determined because of lack of sample.

G_1 = specific gravity of the suspending liquid. Assumed as 1.0 on the nomogram (FCTDS). Corrected to 1.025 for the sea water treatment.

T = settling time in minutes.

D = particle diameter in millimeters.

It should be noted that the assumption of 2.70 as the value of G was made only for the three portions treated with distilled water, which appeared dispersed or nearly so in suspension. The four portions treated with "natural" waters were obviously flocculated, and it has been shown that the effective density of soil floccules in suspension is less than the specific gravity of the soil itself. The effective density is unknown except as an average value for the entire flocculated sample, as explained below. For that reason, diameters of soil floccules in suspension in the natural waters were calculated only for mean floccule diameters corresponding to the "50% slower" point on the curve relating settling velocity to percent of soil in suspension.

13. *Plotting*

For purposes of comparison between treatments, the results of all seven treatments were plotted as cumulative percent of soil in suspension versus the settling velocity (cm/sec = L/60T), uncorrected for temperature, viscocity, or specific gravity of the suspending liquid.

The original plot was made on logarithmic probability paper, which has the effect of tending to straighten out the usual S-shaped cumulative curve and facilitate drawing a smooth curve. The curves for "natural" water treatments would have been other-wise almost impossible to draw with any degree of accuracy or reproducibility. The curves were then carefully transferred to an ordinary semi-logarithmic graph by noting their intersections with each percentage line.

The results for the three treatments in distilled water were also plotted on semi-logarithmic graph paper in the usual manner, showing particle diameter vs. cumulative percent finer. The Calgon results were plotted on log probability paper and extrapolated to the 10% finer point.

14. *Report*

The results for the three distilled water treatments were reported as follows (FCTDS):

(a) percent clay = percentage by weight of particles finer than .005 mm
(b) effective diameter = D_{10} (diameter corresponding to 10% finer)
(c) uniformity coefficient = D_{60}/D_{10}
(d) texture classification—taken from a texture triangle based on percents of clay, silt (.005 mm to .05 mm) and sand (> .05 mm)

The results for the four treatments with "natural" waters were reported as follows:

(a) Median settling velocity
(b) Mean floccule density in suspension
(c) Mean floccule diameter, mm

II. Calculations for Flocculated Sediments in "Natural" Waters

This entire experiment, using different waters for the hydrometer test, was patterned after some similar work previously done by the writer (see Sherman, I—"Flocculent Structure of Sediment Suspended in Lake Mead," Transactions, American Geophysical Union, June, 1953). The methods of calculation here are taken directly from that work.

The paper referred to shows that when sediment settles in the flocculated state, the floccules are both porous and impermeable. Therefore, the effective density of the sediment floccules in suspension depends on the floccule porosity. The porosity of any individual floccule is unknown, but the average porosity of the floccules comprising a sample can be calculated from the porosity of the deposit formed by the floccules as they settle out.

The technique used in this experiment involved a careful measurement of the volume of the deposit at the bottom of the hydrometer jar after 24 hours. This volume included, in the case of the four flocculated portions, the entire weight of sample originally placed in the jar. The equations for the calculations which are made are given below:

(1) $\varrho_a = W_s/V_s$
(2) $e_t = 1 - \varrho_a/\varrho_r$
(3) $\varrho_f = (\varrho_a + e_t - e_d)/(1 - e_d)$

in which

ϱ_a is the specific weight (or apparent density) of the deposit in the hydrometer jar gm/cm³

W_s is the weight of soil in the deposit, gms
V_s is the volume of the deposit, cm^3
c_t is the decimal porosity of the deposit
ϱ_r is the specific gravity of the soil (assumed as 2.70 in this experiment)
ϱ_f is the mean effective density of the floccules in suspension, gm/cm^3
c_d is the decimal porosity around the floccules (as contrasted to within the floccules) in the deposit in the hydrometer jar. c_d was assumed as 0.400 for all deposits.

Those interested in the assumptions and derivations of the above are referred to the paper previously mentioned.

The value of ϱ_f is then inserted in the equation of Stokes Law instead of the value of G (see paragraph 11, under "Hydrometer Analyses" above) in conjunction with the median settling velocity (50% slower point). This gives a mean floccule diameter for the entire sample.

III. Atterberg Limits

A. *Liquid Limit of Soils* (ASTM Designation D-423-39)

This test was performed as specified in the ASTM method except as listed below. The paragraph numbers follow those in the ASTM specifications.

7. *Procedure*

The mechanical liquid limit device was used throughout.

(a) The water used in the determination was was varied, a different water being used with each representative portion. The waters used were:
  (1) Distilled
  (2) MWD before treatment
  (3) MWD after treatment, as delivered at Well G
  (4) MWD after treatment, saturated with Ca SO_4
  (5) San Gabriel River water
  (6) Sea water

(b) In order to ensure equilibrium in any chemical reaction between soil and water, an excess of the water was added so as to produce a highly saturated paste, which was then allowed to stand overnight. The resulting mixture was above the liquid limit. The moisture content was reduced during the test by allowing evaporation to occur until determinations both above and below the liquid limit had been made.

8. *Preparation of Flow Curve*

In such cases where considerable "scatter" of points made the drawing of the flow curve unduly subject to human error or personal judgment, the flow curve was calculated as a semi-logarithmic least-squares line of regression by the usual statistical procedure.

(B) *Plastic Limit and Plasticity Index of Soils* (ASTM Designation D-424-39)

Except as noted below, the ASTM procedure was used throughout:

3. *Sample*

The sample consisted of a portion of the material remaining from the liquid limit test described above.

4. *Procedure*

The water used in the determination was the same as was used in the previous liquid limit determination.

IV. Analysis of Clay Cap Disintegration in Hydraulic Models

The model was dis-assembled after the test and the contents divided into four portions:

(1) The "dune sand" region, including any clay cap material eroded upwards.
(2) The "clay cap" region, including any "dune sand" which had fallen into voids created during the test.
(3) The "Silverado" region including any clay cap material eroded into it.
(4) The solids carried through the "Silverado Zone" into the overflow beaker.

Each portion was dried and weighed, then wet-screened on the No. 325 screen, re-weighed, and dry-screened through a nest of sieves. It was found that all of the clay cap material passed the Tyler No. 40 sieve, and all of the Ottawa sand was retained on the U. S. No. 50 sieve, with only a small fraction of one percent in the overlap range. This made it possible to separate out the two materials in each portion, and calculate the weight of clay cap material in each.

The weights of material in the overflow beakers were very small, and the samples would have been "lost" on the usual 8″ diameter sieves. Those very small samples were sieved on 3″ diameter sieves made available through the courtesy of the Arcadia Soil Laboratory of the U. S. Forest Service.

For comparison with the portions listed above a standard sieve analysis was made on an undisturbed portion of the clay cap which had not been placed in a hydraulic model. Comparison of the sieve analysis results of the undisturbed sample and of the portion retained in the overflow beakers made possible the computation of the weight of silt and clay washed out of the overflow beakers and thus not directly measurable.

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
HYDRAULIC DIVISION

APPENDIX D

LETTER REPORTS ON GROWTH OF MICROORGANISMS IN THE AQUIFER

By DR. CARL WILSON

June 19, 1953

Mr. H. E. Hedger, *Chief Engineer,*
Los Angeles County Flood Control District,
Box 2418 Terminal Annex,
Los Angeles 54, California.

Attention: Mr. Finley B. Laverty

Dear Sir:

Fearing that heavy chlorination of water which you are injecting into the underground basin at Manhattan Beach might be increasing the possibilities of corrosion of pumps and well casings, you sent me under date of June 15th two samples of the water in question for determination of the increase in hydrogen ion concentration caused by the present chlorine dosage of 15 parts per million. That work was completed the day the samples were received and the results were reported by telephone to Mr. Zielbauer.

The first sample was taken from the Metropolitan District inlet line, and represented water before chlorination. Its pH was found to be 8.30.

The second sample, taken five minutes later from the office tap, represented water after chlorination at the dosage of fifteen parts per million: its pH was found to be 7.55.

The free carbon dioxide content of these waters, a constituent which may be actively corrosive under certain conditions, was found to be 1.5 parts per million in the first sample and 8 parts per million after chlorination; not a very significant increase.

My belief, based upon these samples, is that chlorination at a dosage of 15 parts per million has not increased the aggressiveness of this water to any significant degree. However, because the acceptance rate at the injection wells has not been dropping, I suggested that the chlorine dosage be dropped to 12 parts per million in the interest of economy. This change was made at once, but should the acceptance rate start to drop it may be desirable to restore the chlorine dosage to its previous value.

I would further suggest that similar samples be taken to check the pH drop under the present dosage of 12 parts per million.

Respectfully submitted,

Carl Wilson /s/

June 29, 1953

Mr. H. E. Hedger, *Chief Engineer,*
Los Angeles County Flood Control District,
Box 2418 Terminal Annex,
Los Angeles 54, California.

Attention: Mr. F. B. Laverty

Dear Sir:

Under date of June 19th 1953 you sent us six samples of water for examination in the hope of affording additional light upon the corrosive properties of Colorado River water when heavily chlorinated for injection into the underground. This work was promptly completed and the results were telephoned to Mr. Zielbauer. This memorandum will confirm that report and provide copies for your files. All samples were designated: "West Basin B Test." Our findings were as follows:

Sample No.	*pH*	*Phenol alk*	*Methyl Orange alk*	*Carbon dioxide*
1	8.20	11	146	2
2	7.70	0	135	6
3	7.45	0	135	10
4	7.70	0	135	6
5	7.60	0	130	7
6	7.70	0	135	6

Identification of the samples is as follows:

No. 1, Metropolitan Water District water prior to chlorination taken at office, June 19th, at 9:30 A.M. Temperature 18° C.

No. 2, Metropolitan Water District water obtained at Well E, June 19th, at 9:50 A.M. Temperature 18° C.

No. 3, Metropolitan Water District water after chlorination, taken at office, June 19th, 9:53 A.M. Temperature 18° C.

No. 4, Metropolitan Water District water from Well G, sampling tap. Taken at 9:55 A.M. June 19th. Temperature 18° C.

No. 5, Metropolitan Water District water taken at Well I, at 10:03 A.M. June 19th. Temperature 18° C.

No. 6, Metropolitan Water District water from Well K, taken at 10:10 A.M. June 19th. Temperature 18° C.

The least benign of these waters is that represented by Sample No. 3, but even here the pH of 7.45 and a free carbon dioxide content of 10 parts per million seems, in my opinion, unlikely to be actively corrosive. Samples Nos. 2, 4, 5 and 6 are not aggressive, while the water of Sample No. 1, is moderately protective.

Conclusion: Chlorination at a dosage of 12 parts per million appears to impose no especial hazard of corrosion.

Recommendation:

It seems desirable to repeat these samplings at intervals of two weeks, for a multiplicity of readings would give more dependable information than can be deduced from only one series.

Respectfully submitted,

Carl Wilson /s/

August 20, 1953

Mr. H. E. Hedger, *Chief Engineer,*
Los Angeles County Flood Control District,
Box 2418 Terminal Annex,
Los Angeles 54, California.

Attention: Mr. Finley B. Laverty

Dear Sir:

I have had the privilege and pleasure of reading the report by A. F. Bush and S. F. Mulford covering some of your problems, and now ask your permission to comment upon one remark which it seems to me might easily lead to confused ideas. I make reference to the second paragraph on page 9.

There the statement is made; "No Myxomyces (slime-producing bacteria) were noted."

Since we have done much talking about slime-forming bacteria this remark could be regarded as a refutation of some of my own statements, but actually it is not, for it refers to a group of organisms (the Myxomyces) which has no relation to our problem, and ignores the great class of slime forming saprophytic bacteria which are normal inhabitants of water and soils, and which, in my opinion, play an active part in retarding the continued introduction of water to an aquifer. As far as is known only one species of Myxomyces is found in water, where it is parasitic on Cladophora, one of the larger attached algae. The others are confined to wood, dung and soils of high organic content. One of the best general discussions of the group is to be found in "Fundamentals of Bacteriology," by Martin Frobisher, Jr., 4th Edition, pages 419 et seq., published by Saunders.

The "slime-forming bacteria" with which we are concerned are those organisms, and their number is legion, covering many genera and species, which secrete pectins (slimes) with which to anchor themselves to the substrate, sand, gravel, and even well casings, pump bowls and runners, etc. They have been present in all of your samples, and we have estimated their numbers relative to the total count in many instances. This we have been able to do by taking advantage of their slime-forming proclivities. A clean and sterile micro slip, one inch by three inches, is immersed in the sample and left for twenty-four hours, during which period the organisms impinge upon the glass, where they attach themselves with a quantum of pectin. By appropriate staining methods we are able to recognize the pectin (slime). A large number of these culture slides from your earlier samples showed numbers of slime-forming bacteria nearly as great as the total count on standard nutrient agar incubated for 72 hours at 20° C.

I will repeat this work on some of your current samples and furnish you with some actual figures. Perhaps I can make a photograph of a culture slip to show the slime, and if so I will send you a print.

Trusting you will pardon this long and uninvited note, I am,

Yours faithfully,

Carl Wilson /s/

August 20, 1953

Mr. H. E. Hedger, *Chief Engineer,*
Los Angeles County Flood Control District,
Box 2418 Terminal Annex,
Los Angeles 54, California.

Attention: Mr. Finley B. Laverty

Dear Sir:

You recently sent us two samples of water from your Manhattan Beach well 8, taken August 12th, with a request for a special examination to determine the nature of the yellow color which this water exhibited after standing for a while. A report on the bacterial findings was sent to you under date of August 19th. It remains now to report on the microscopical and chemical investigation.

The microscope revealed the presence of iron oxide floc embedded in bacterial slime. We found no organisms of specific significance.

The chemical examination confirmed the presence of iron oxide and showed manganese to be absent.

Iron in water in insoluble forms, usually as oxides, but often as sulphides, is found in most alluvial fills, and it is taken into solution as ferrous bicarbonate (colorless) by the carbon dioxide released by bacterial activity. When such water comes into contact with air the iron is oxidized to a basic ferric carbonate, which shortly becomes converted to hydroxide (insoluble in water) shortly to be precipitated as the red-brown ferric oxide (Fe_2O_3). This has happened in your samples, giving rise to the phenomenon which attracted the attention of your engineers.

Trusting this gives you the desired information, I am,

Yours faithfully,

Carl Wilson /s/

December 24, 1954

Mr. H. E. Hedger, Chief Engineer,
Los Angeles County Flood Control District,
Los Angeles 54, California

Attention: Mr. Finley B. Laverty

Dear Sir:—

In connection with the recharge operations at Manhattan Beach it was assumed that the steadily diminishing rate at which water could be introduced into an aquifer was principally due to the growth of micro-organisms in the sands and gravels of the aquifer.

It is a well established fact that virile bacteria, and some other micro-organisms, are always present in subsoils and sands and gravels, where they persist indefinitely in small numbers in static equilibrium with the environment. Should conditions within the environment become more favorable these dormant organisms would burst into activity. The introduction of an impounded surface water, like the Colorado river water used to recharge the underground basin, which is relatively high in organic matter, would provide a powerful stimulant to the growth and multiplication of the native organisms at the same time that it brought with it a foreign flora to complicate the situation.

To control such growths, and by doing so to preserve the original porosity of the aquifer, recourse was had to the bactericidal and bacterio-static properties of chlorine as used in water treatment. The dose initially used was 20 parts per million, and this was found sufficient to maintain the input rate at a constant level. Realizing that this dosage might be in excess over that actually required to yield a constant acceptance rate the dosage was reduced from time to time, being held for significant periods at, 15, 10, 8 and even 5 parts per million. In each case the acceptance rate remained constant, showing no material reduction. From this it would seem at a glance that 20 parts per million had been an unnecessarily high dosage, but one must not jump at conclusions. Each reduced dosage was applied to water entering a portion of an aquifer which had previously been subjected to heavy dosages, hence having a low chlorine demand. It is not to be assumed that an initial dosage of five or ten parts per million applied to another aquifer, even in the same vicinity, would suffice to stabilize injection rates. In each new location it will always be necessary to first destroy, or at least inhibit the activities of the organisms which are indigenous to the aquifer, and to saturate the chlorine demand of all the organic matter contained in the affected area. This can be accomplished only by heavy chlorine dosage. Experience at Manhattan Beach seems to point strongly toward twenty parts per million as a proper initial dosage to prepare the aquifer vestibule to a sufficient distance to permit injection at the desired rate. When this end has been accomplished is to be recognized by the attainment of a constant acceptance rate. After stability has been attained and maintained for a period of perhaps, a week, then the dosage may be reduced twenty-five per cent. Experience leads to the belief that after stabilization has been obtained a dosage of eight to ten parts per million will be required to maintain a constant injection rate. If the acceptance rate should be found to still be not affected then further reductions in dosage may be tried until a point is reached where the input can no longer be maintained at the desired level. Any attempt to control growths in a subterranean environment through the use of chlorine must always be carried out empirically, for no two environments will be alike. The best criterion of chlorine sufficiency is undoubtedly found in a constant acceptance rate, but in this connections within the aquifer will always limit the rate at which water may be injected into it, and when this limit is reached no increase in chlorine dosage can further augment the acceptance rate.

The dominant organisms encounted in unchlorinated ground water throughout the study were colonial slime-formers, excreting a pectin-containing jelly in which the individual bacteria were embedded. Sulphate-reducers were also present, and these were believed responsible for the faint odor of hydrogen sulphide observed in some raw water samples.

When chlorine was added to the recharge water many samples from injection and observation wells alike produced no colonies on standard nutrient agar incubated for 24 hours at 37° C., indicating absence of intestinal bacteria. Incubated for 72 hours at 20° C., the total count was generally less than 10 colonies per milliliter.

The surprising feature of this study was the absence in all samples of the iron bacteria, Crenothrix and Gallionella, which for years have afflicted many of the Manhattan Beach Water Department's Wells, and which are a constant source of trouble in so many wells all over the Los Angeles coastal plain. The explanation of this seemingly anomely is to be sought in the high saline content of sea water which had found its way into the recharge area, for salt is known to inhibit these organisms.

The results thus far obtained at Manhattan Beach have shown that chlorine is of great value in helping to maintain constant injection rates.

Respectfully submitted,

Carl Wilson /s/

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
HYDRAULIC DIVISION

APPENDIX E

DERIVATION OF THE EQUATION FOR THE QUANTITY OF FRESH WATER FLOWING SEAWARD AS A RESULT OF AQUIFER RECHARGE

By PAUL BAUMANN and M. F. BURKE

Reference is made to the attached Plate 1 for nomenclature and quantities necessary to the discussion of the problem.

In order to prevent landward movement of the sea water wedge past the Recharge Line by means of a counteracting fresh water head, it is necessary to maintain at least an equalizing pressure at the lowest point in the aquifer which must be protected. This means that the pressure due to fresh water at the point $x = O$, $z = O$, must be equal to the pressure of sea water at the same point. Calling the specific gravity of fresh water $\frac{S_f}{HS} = 1$, and the specific gravity of sea water S_s, then $HS_S = (H+y_o)\ S_f$ or $\frac{HS_S}{S_f} - H = y_o$, or

$$y_o = H\ (S\text{-}1) \qquad (1)$$

in which S = the ratio of specific gravity of sea water to fresh water, $\frac{S_s}{S_f}$. This is the head of fresh water necessary to balance the pressure of sea water at the depth H.

Under this condition of balanced pressure, the toe of the sea water wedge will be stabilized at the section where the injection is occuring. Furthermore, with the sea water wedge stabilized, an interface between the sea water wedge and the fresh water will be formed of somewhat the shape shown (except for the distortion of H with respect to L in the sketch), and being characterized by balanced pressure on each side of the interface. This balance of pressures is expressed by the equation $(H\text{-}z)\ (S\text{-}1) = y$, similarly to the balance of pressures at the toe of the wedge. At the discharge section, $x = L$, the pressures at the interface are still balanced, represented by the equation $(H-M+h)\ (S-1) = y_L$.

Since the piezometric line, characterized by the values of y_o and y_L, drops in elevation from y_o to y_L, a flow of fresh water from the Recharge Line to the sea must take place under the influence of this gradient. The solution for the rate of flow of fresh water oceanward may be obtained in a number of ways. The solution presented will follow the Dupuit-Forchheimer application of Darcy's law, as it is perhaps the simplest.

Darcy's law is expressed as $q = KA\ i$ in which q is the rate of flow for unit width of aquifer,

K is the permeability of the aquifer,

A is the area of a unit width of aquifer,

and i is the slope of the piezometric gradient.

Now, the aquifer area from the interface up to the impervious cap at any point is $M-z$ and the piezometric gradient at any point is $-\frac{dy}{dx}$. Substituting these values in the equation for Darcy's law

$$q = -K\ (M-z)\ \frac{dy}{dx} \qquad (2)$$

Differentiating the pressure balance equation previously given,

$$(H-z)\ (S-1) = y \qquad (3)$$

yields

$$dy = -(S-1)\ dz \qquad (4)$$

The equation for flow in the restricted aquifer then becomes

$$q = K\ (M-z)\ (S-1)\ \frac{dz}{dx} \qquad (5)$$

Integrating this equation, there results

$$\frac{qx}{K\ (S-1)} = -\frac{(M-z)^2}{2} + C \qquad (6)$$

At the point $x = O$, $z = O$, $C = \frac{M^2}{2}$

Then

$$\frac{qx}{K(S-1)} = -\frac{(M-z)^2}{2} + \frac{M^2}{2} \qquad (7)$$

Solving equation (7) for z, there results

$$z = M \pm \sqrt{M^2 - \frac{2\ qx}{K\ (S-1)}} \qquad (8)$$

Since at $x = O$, $z = O$, the negative sign must be used before the radical. This equation (8) gives the equation of the interface in terms of q.

Referring back to equation (7), at $x = L$, $z = M-h$ and the equation becomes

$$\frac{qL}{K\ (S-1)} = \frac{M^2-h^2}{2} \qquad (9)$$

or

$$q = \frac{K\ (M^2-h^2)\ (S-1)}{2L} \qquad (10)$$

The Dupuit-Forchheimer application of Darcy's law does not give any estimate of the value of the discharge face h, or its value relative to M. In order to obtain such a value, recourse must be made to equations of complex variables, particularly to the solution of the "basic parabola" of ground water flow determined by Kozeny and published in "Wasserkraft and Wasserwirtschaft" in 1931. Kozeny's problem, indicated on the sketch on Plate 2 attached, concerned the gravity flow of water through a porous soil above an impervious base, L, and its final discharge vertically through the section $\frac{q}{2K}$.

His solution resulted in the equation of the phreatic line bounding the flow on the top as being a simple parabola having its focus at $x = \frac{q}{2K}$ and passing through the points shown on the sketch.

If Kozeny's basic parabola is now inverted, it will be seen to be quite similar to the condition of the salt water wedge, except that his discharge face is horizontal, instead of vertical as has been assumed in the present problem. Actually, in the present problem a horizontal discharge face (with vertical velocities) is probably a better picture of actual conditions than a vertical discharge face, in view of the fact that ocean bottom gradients in the vicinity of the local coast line are much flatter than 1:1. Inserting the value of $\frac{q}{K}$ for h in equation (10) and solving for q the result is

$$q = \frac{K\,L}{(S-1)}\left[\left(1+\frac{M^2\,(S-1)^2}{L^2}\right)^{\frac{1}{2}}-1\right] \qquad (11)$$

Expanding the term in the parenthesis into a binomial series, the value of q is

$$q = \frac{K\,L}{(S-1)}\left[1+\frac{M^2\,(S-1)^2}{2L^2} - \frac{M^4\,(S-1)^4}{8L^4}+\ldots\ldots-1\right] \qquad (12)$$

or

$$q = \frac{K\,M^2\,(S-1)}{2L}\left[1 - \frac{M^2\,(S-1)^2}{4L^2}\ \ldots\ldots\right] \qquad (13)$$

This equation (13) shows that if $h = O$ in the Dupuit-Forchheimer equation (10), the value of q would be the same as the Kozeny value if only the first two terms of the binomial expansion were used in equation (12). However, the inclusion of an additional expansion term in the Kozeny solution (as in equation (13)) shows that setting $h = O$ results in too large a value of q. A correction factor n may be introduced into the Dupuit-Forchheimer approximate solution, giving the result

$$q = \frac{nK\,M^2\,(S-1)}{2L} \qquad (14)$$

in which $n < 1$ and varies with the ratio $\frac{M}{L}$.

For a value of $S = 1.025$ (a reasonable value of sea water density with respect to fresh water), a value of $\frac{M}{L} = 8$ would be required before n decreased to .99, or affected the approximate solution by as much as 1%. For maximum $\frac{M}{L}$ values up to ¼ pertaining to the experimental work on the coast of Los Angeles County, the corresponding value of n would be $1-.98\times10^{-5}$. Corrections of this order of magnitude could be neglected.

For those conditions where the correction is negligible, the equation can be revised to read

$$qL = \frac{K\,M^2\,(S-1)}{2}$$

Since all factors on the right hand side of this equation are constants for a given situation, the product qL is also a constant. This leads to the interesting conclusion that if q is increased, the length of the salt water wedge is decreased in the same proportion, and vice versa. If is it desired to decrease the length of sea water intrusion into a fresh water aquifer from 1,000 feet to 500 feet, the seaward flow of fresh water must be doubled. By the same reasoning, the sea water can never be completely excluded from the aquifer, since then the length L would be zero, requiring an infinite seaward flow of fresh water.

It may be noted that the discharge q of fresh water into the ocean required to stabilize the sea water wedge at a distance L from the assumed vertical discharge face is independent of the depth H to the bottom of the aquifer, and depends solely upon the values of M, K and $(S-1)$. On the other hand, the pressure head required at the toe of the sea water wedge is determined solely by H and $(S-1)$. As H varies, both y_o and y_L change so that the difference $y_o - y_L$ remains constant, consistent with a constant value of q.

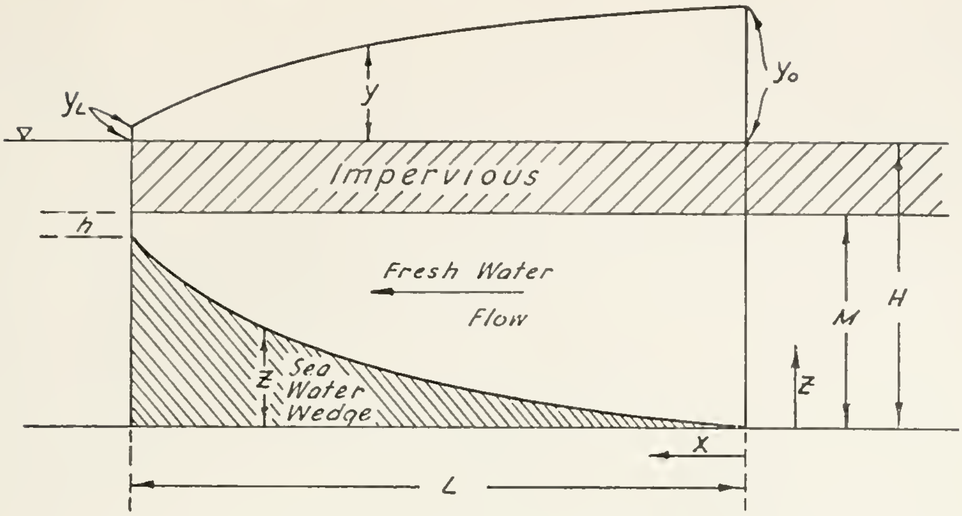


PLATE I

Section Through Intruded Pressure Aquifer

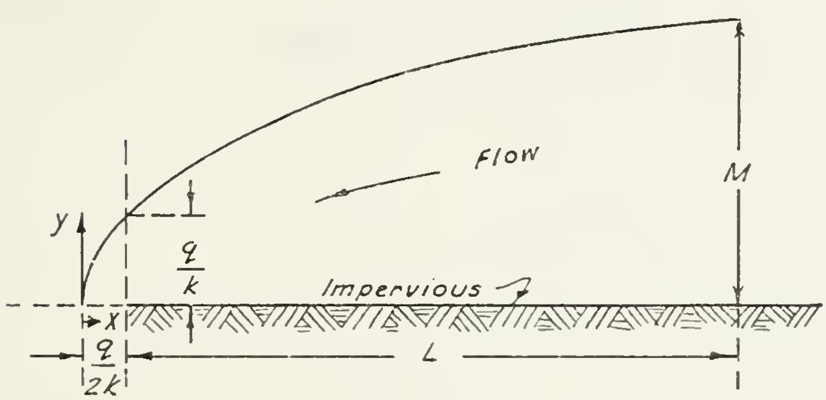


PLATE 2

Kozeny's Basic Parabola for Ground Water Flow

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT

WEST BASIN BARRIER TEST

LOS ANGELES COUNTY FLOOD CONTROL DISTRICT
HYDRAULIC DIVISION

APPENDIX F

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