WILLAMETTE BASIN PROJECT, OREGON

DEFINITE PROJECT REPORT

DETROIT DAM & RESERVOIR

NORTH SANTIAM RIVER

AUGUST 9, 1941

WAR DEPARTMENT
U. S. ENGINEER OFFICE
PORTLAND DISTRICT
PORTLAND, OREGON

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FINAL DEFINITE PROJECT REPORT ON

DETROIT DAM AND RESERVOIR

NORTH SANTIAM RIVER

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Final Definite Project Report on Detroit Dam and Reservoir, North Santiam River, Willamette River Basin, Oregon.

I Introduction

1. Scope: A preliminary definite project report on the Detroit site, in compliance with Circular Letter R. & H. No. 19, 1939, was prepared under date of February 1, 1940 and forwarded to the Chief of Engineers on February 17, 1940. The general plan of development outlined in that report was approved by the office of the Chief of Engineers subject to certain modifications and additional studies. (See letter file No. 7402 (Willamette River - Detroit Res.)-21).

2. Since the submission of the preliminary report, additional subsurface exploration has been done on the site, further field investigations have been undertaken and additional office studies, including those requested by the office of the Chief of Engineers, have been completed. This report summarizes the results of all investigations to date and is made with a view to obtaining final approval of the proposed plan of development so that utility negotiations may be completed and contract drawings and specifications prepared, in order that land acquisition and construction can commence if and when funds are made available.

3. Project Authorization: The Willamette River Basin Project was authorized by the Flood Control Act approved June 28, 1938, which reads as follows:

> "Sec. 4. That the following works of improvement for the benefit of navigation and control of destructive floodwaters and other purposes are hereby adopted and authorized to be

prosecuted under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with plans in the respective reports hereinafter designated: Provided, that penstocks or other similar facilities adapted to possible future use in the development of hydro-electric power shall be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers and of the Federal Power Commission.

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The general comprehensive plan for flood control, navigation and other purposes in the Willamette River Basin as set forth in House Document Numbered 544, Seventy-Fifth Congress, Third Session, is approved, and for the initiation and partial accomplishment of the plan recommended for initial development in said document there is hereby authorized $11,300,000; the reservoirs and related works to be selected and approved by the Chief of Engineers."

4. The funds authorized in the foregoing act are to be expended, in general, on other units of the "initial development". Additional funds must be authorized and further appropriations must be made before construction of the Detroit Dam proper can commence.

5. Authorized Plan: The approved plan of initial development for the Willamette River Basin is set forth in detail in House Document No. 544, Seventy-Fifth Congress, Third Session. The Detroit Dam and reservoir, controlling an area of four hundred thirty eight square miles and having a gross storage capacity of 172,000 acre-feet, is one of the storage units proposed in the initial development. The initial storage at this site is to be operated in the joint interest of flood control and increased low water flow on the North Santiam and Willamette Rivers. The supplemental new water made available from storage can be utilized for navigation and stream purification on the main stream and/or irrigation purposes.

6. The operation schedule contemplates drawdown of the reservoir to

minimum stage during the months of December and January to give full flood control reservation during this period. With such flood control reservation a flood with a peak discharge of 51,600 cfs, expected frequency once in twenty-five years, would be regulated to a maximum outflow of 10,000 cfs. The reservoir would be filled gradually during the high runoff spring months, after the flood hazard had passed, so that it would normally be full by May 1 of each year. The stored water would be released during periods of low runoff, or high irrigation demand, usually from July to September, inclusive, and the reservoir again drawn down to provide flood storage during the winter months. The discharge from the outlets under this method of operation will vary from about 500 to 10,000 cfs over a range of head varying from the minimum to the maximum.

7. Consideration of future power possibilities necessitate that the "initial low dam" development be so designed and constructed that it can be feasibly raised some 85 feet in the future to provide an additional 260,000 acre feet of supplemental power storage when such storage becomes warranted. Such raising would provide a total gross storage capacity of 432,000 acre feet and would, in addition to satisfying the same requirements as are satisfied by the initial low dam, make possible the development of 125,500 kw. of firm capacity, on a 60 percent annual load factor, at four proposed plants on the North Santiam River at and below the Detroit Dam site.

8. General information pertaining to both the "low" and "high" dam developments is given in Table I.

9. Location of Dam and Reservoir: The Detroit Dam site is located at mile 47 on the North Fork of the Santiam River, a tributary of the Willamette River, and is about six miles west and downstream of the community of Detroit. The river at the damsite flows in a northwesterly direction, through a deep and rugged gorge, with a fall of approximately 35 feet in a mile. The river at this point forms the boundary line between Marion and Linn Counties. Approximately two miles above the damsite the river valley widens forming a reservoir, for the low dam, with a length of about seven miles and a maximum width of approximately 3,500 feet.

10. Related Flood Control Improvements: The Detroit reservoir is one of seven multiple use storage units to be constructed on the major tributaries of the Willamette River. The Detroit reservoir will operate in parallel with these other storage units to reduce flood heights on the main stream and to provide increased low water flow for navigation and/or supplemental water for irrigation use.

II Initial Low Dam Project

11. General: The location, layout and design of the initial "low" dam must be such as to fit into the contemplated future "high" dam development. Based upon the foregoing criteria, utilities which require revision for the "low" dam are to be raised to an elevation to clear the future "high" dam. The "low" dam is so laid out and designed that it can be feasibly raised on either the upstream or downstream face. With the exception of providing an opening through the right abutment for future penstocks, no special provisions are to be incorporated in

the "low" dam to facilitate or reduce the cost of future raising.

12. Geology: The damsite has been extensively explored by means of fifty-four diamond drill holes, aggregating 5,175 feet in length; five 36 inch calyx holes, with a combined length of 283 feet; five tunnels, with a total length of 460 feet; and 250 linear feet of trenches. The extent of the exploration work undertaken and pertinent geologic data are shown on Plate VI and on Plates I to V, inclusive, of Appendix B. Appendix B also contains geologic reports by Mr. E. B. Burwell, Jr., Consulting Geologist.

13. Foundation rocks at the site consist of massive andesitic lavas and irregular intrusive masses of diorite, both of which are intruded by andesite porphyry dikes. Terrace gravels, stream alluvium, talus and slope wash make up the unconsolidated deposits. Some material suggesting glacial origin is found on the right abutment. Extensive jointing and fracturing accompanied by hydrothermal alteration has changed the original texture and composition of the andesite. Although this alteration has been extensive the quality of the rock is still adequate for a foundation for the type and heigth of dam proposed. Except for local weathering, the diorite and andesite porphyry are usually fresh and unaltered; the contacts between the andesite and the later intrusive rocks being sound and relatively tight.

14. The trends of the principal joints and fractures are, (a) north-south and (b) N. 30 to 40° West. Dip of the fractures is predominantly steep (74° to 85°) and to the west or southwest. Movement

along the fracture planes, so far as has been observed, has been slight with only local or discontinuous areas of clay gouge or secondary minerals developed. No major faults have been discovered in the foundation area.

15. Rocks of the right abutment above elevation 1350, because of their exposed position and extensively jointed condition, have been susceptible to deeper weathering than those exposed elsewhere in the foundation area.

16. The depth of overburden varies throughout the foundation area. Large areas have practically no overburden while the maximum depth explored was over 90 feet in DH-54 near the head of the buried V-shaped gully in the right abutment. Talus and slope wash, probably overlying some gravel and boulders, is from 30 to 50 feet deep on the bench above elevation 1370 on the left abutment. Deep holes in the river are filled with gravel and boulders to a depth of over 60 feet.

17. Laboratory tests indicate that the unjointed foundation rock has a compressive strength varying from 6,000 to 48,000 pounds per square inch; a shear-friction resistance of at least 1,000 pounds per square inch plus 1.15 times the normal load; a modulus of elasticity varying from 4,770,000 to 12,120,000 pounds per square inch; and a Poisson's ratio of from 0.18 to 0.28. Stress-strain field tests on undisturbed foundation rock in the tunnels indicate a modulus of elasticity varying from approximately 150,000 to 1,100,000 pounds per square inch and a Poisson's ratio of from 0.20 to 0.48.

18. <u>Adopted Low Dam Plan</u>: Reconnaisance surveys were made and study was given to various alternate damsites before the Detroit site

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was chosen for intensive investigations. Foundation conditions, comparative estimates of cost and consideration of the possibility of future raising resulted in the adoption of a concrete gravity dam structure for the Detroit site.

19. The proposed dam alignment is straight in plan with a crest length of 1,185 feet and a maximum height, from lowest point of foundation to top of dam of 359 feet. A free overflow, ogee type, river section spillway, with a net clear length of 332 feet, is proposed. The foundation rock at the downstream toe below the spillway will be protected by a concrete apron. Flood control outlets through the dam will consist of five 90 inch diameter and two 108 inch diameter controlled sluice outlets. Bulkheaded openings will be left through the right abutment for the installation of penstocks in the future if and when the dam is raised for power purposes. Pertinent descriptive data relating to the low dam are given in Table I and the layout is shown on Plate II, Sheets 1 and 2.

20. Spillway Design: The spillway is designed to pass a peak reservoir inflow of 176,000 cfs, with the flood control outlet closed, without overtopping of the abutments. The derivation of this spillway design flood is contained in a report, submitted to the Office of the Chief of Engineers, under date of June 19, 1940, entitled "Hydrology Report as Related to Spillway Requirements for Detroit Dam and Reservoir, North Santiam River," date June, 1940. Due to the height of the dam and the jointed condition of the foundation rock, a 210 foot concrete apron, contained between parallel concrete training walls, is proposed below the spillway section to provide balanced hydraulic donditions;

to cause the formation of the hydraulic jump; reduce velocities; prevent erosion of the river bed adjacent to the structure and to smooth out flow conditions at the site of the proposed power plant. Pertinent hydraulic data relating to the spillway design is shown on Plate III.

21. Outlet Design: The outlets are designed to pass a discharge of 10,000 cfs at minimum pool elevation. For this purpose five 90" diameter circular outlets, controlled by Paradox gates, are proposed through the overflow section and two 108" diameter circular outlets, controlled by 60" diameter needle valves are proposed through the left abutment. All outlets will be protected by trash racks and emergency ring follower slide gates. The Paradox gate controlled outlets, which discharge into the spillway apron, will be operated either fully open or fully closed. Fine regulation will be accomplished by the needle valve outlets, which discharge into a separate get-away channel adjacent to the left abutment. Pertinent hydraulic data relating to the design of the outlets is shown on Plate III.

22. Provisions for Future Raising: With the exception of bulkheaded openings in the right abutment section no special provisions are proposed to be incorporated in the low dam proper to facilitate future raising. Preliminary stress studies indicate that no unprecedented or unsolvable problems will arise in connection with such future raising insofar as internal stresses in the enlarged structure are concerned. However, practical considerations necessitate the provision of some means of unwatering the low dam and/or passing the flow during the period that future work on raising of the dam may be underway. Preliminary studies indicate that

the most practicable solution of this problem may be a large diameter diversion tunnel through the left abutment, to be driven during a low water period when the natural flow can be passed through the flood control outlets, at the time raising of the dam is undertaken. Further study is to be given this problem prior to construction of the low dam.

23. Stress Studies: Investigation of stresses in the low dam structure has to take into consideration the possibility of future raising. Preliminary stress studies, based upon unrestrained cantilever elements, have been made for both the high and low dams under various loading conditions. The details of these studies are contained in Appendix A and the results, for both reservoir empty and normal pool conditions, but excluding earthquake, are summarized in the following table. Stresses preceeded by a negative sign are tensions, all others are compressions.

Stresses	Reservoir Empty			Normal Pool		
	Low Dam	High Dam Raised: Up Stream	High Dam Raised: Down Stream	Low Dam	High Dam Raised: Up Stream	High Dam Raised: Down Stream
Maximum Foundation Pressure Lbs./sq. inch	319	361	412	186	259	245
Maximum Concrete Stress						
Compression (a) — lbs./sq. inch	319	364	412	290	404	402
Tension (a)-lbs./sq. in.	-11	-3	-7	0	0	0
Shear - lbs./sq. inch	0	0	0	139	194	196
Minimum Uplift Ratio (b)	-	-	-	1.05	0.97	1.00
Maximum Sliding Factor(c)	-	-	-	0.60	0.59	0.62
Minimum Shear-Friction Factor (d)	-	-	-	8.99	7.20	6.96

(a) Principal Stresses.

(b) Ratio of vertical pressure at upstream face to horizontal water pressure at upstream face.

(c) Ratio of horizontal force to vertical less uplift force.

(d) Based upon 600 pounds per square inch plus 0.65 times vertical load.

24. After the low dam is raised additional load will be transmitted to the original low dam concrete resulting, in general, in an increase in internal compression and shear stresses in the original low dam section. Stresses on the contact plane between the original low dam structure and the future high dam addition will vary with loading conditions and upon whether future raising takes place on the upstream or downstream face. These contact stresses will be of low magnitude in any case, the maximum shear stress varying from 12 to 62 pounds per square inch with corresponding normal stresses of 49 to 163 pounds per square inch. It may be noted that the normal stress on this contact plane will be two to four times greater than the corresponding shear stress.

25. Flowage and Utilities: A map of the proposed reservoir, with the normal pool elevations of both the high and low dams indicated, is shown on Plate I. Although the reservoir lies within the Willamette National Forest, approximately 70 percent of the reservoir lands are in private ownership. The greater part of the reservoir area, about 85 percent, consists of cut and burned over land supporting brush and a young stand of second-growth fir. Certain forest service facilities, the small community of Detroit and a number of small lumber mills will have to be moved out of the reservoir area. Flowage for the "low" dam is proposed to be acquired to the 1,000 year flow line which corresponds

to elevation 1,484.0. When the dam is raised, it is proposed to acquire additional flowage to an elevation five feet above the high dam normal pool level, or to elevation 1,574.0, which elevation lies above the "high" dam 1,000 year flow line. Flooding above this elevation will be so infrequent that it is considered more economical to pay for such damage, if and when it occurs, that to acquire flowage rights over the lands affected. A resume of the lands to be acquired and the area to be cleared for both the "low" and "high" dams is summarized as follows:

	Low Dam Acres	Increment High Dam Acres	Total High Dam Acres
Land to be acquired and cleared to elevation:--	1,484.0	-	1,574.0
Land to be acquired:			
Privately owned	1,505	970	2,475
Government owned	495	570	1,065
Total	2,000	1,540	3,540
Land to be cleared:--			
Open and cultivated	200	0	200
Cut over and brush	1,760	1,510	3,270
Timber lands	105	330	435
Total	2,065	1,840	3,905

26. The North Santiam River Highway, known as State Highway No. 222, is a contemplated major transportation route between the Willamette Valley and eastern Oregon and passes through the dam and reservoir site. This route has been improved and surfaced to a 22 foot width from State Route No. 99E eastwardly to Gates, approximately 9 miles below the damsite, and from the town of Detroit, at the upper end of the proposed reservoir, to its junction with the improved highway to Bend, Oregon. The unimproved portion of this road between Gates and the town of Detroit, which includes the section affected by the proposed Detroit Dam,

is narrow and crooked and at places only wide enough for one way traffic. The improvement of this portion has been held in abeyance for several years by the Oregon State Highway Commission and the Public Roads Administration pending development of the Detroit Project.

27. For the low dam development it is proposed to relocate the 8.4 miles of existing road affected to improved alignment, grade, width and surface at an elevation of 1,605, in general, which is sufficient to clear any probable future high dam development. No future work on this stretch of the highway will be required when the dam is raised, however, the high dam will require the relocation of an additional 3.5 miles of present improved highway east of Detroit. The proposed highway revision is shown on Plate V. The 8.4 miles of highway revision necessitated by the low dam is estimated to cost $1,982,000, or $1,182,000 more than the improved water grade highway originally contemplated by the Public Roads Administration. This increase in cost is chargeable to the Detroit Project. An additional cost of $350,000 is also chargeable to the high dam for revision of the 3.5 miles of existing improved highway above Detroit.

28. If the Mill City branch of the Southern Pacific Railway is abandoned, as discussed in paragraph 29, and the railroad subgrade is utilized as a grade for an improved highway over the 4.1 miles section lying downstream from and between the Detroit Dam site and Niagara, the cost of an improved road over this stretch is estimated at $374,000. If the Railway is abandoned as a private carrier but remains in place below the dam for construction purposes, the cost of highway improvement

over this 4.1 mile section will be increased to $589,000. Inasmuch as practical and economic considerations indicate that it would be advisable to leave the trackage in below the dam, at least during the dam construction period, the increased cost of highway revision due to such a procedure, estimated at $215,000, has been charged to the Detroit Project.

29. The Corvallis and Eastern Railway, a logging branch of the Southern Pacific System, passes through the damsite and reservoir and extends to a point about two miles east of the town of Detroit. The construction of the dam will require the purchase and abandonment or the reconstruction of approximately 15 miles of this railroad. It is estimated that, if approval can be obtained from the Interstate Commerce Commission, the cost of acquisition and abandonment will amount to about $750,000. If such approval cannot be obtained the railroad will have to be relocated to clear the high dam at an estimated cost of approximately $2,500,000. The cost estimates attached hereto are based upon purchase and abandonment of this line.

30. Only minor telephone and telegraphlines exist in the reservoir area. No problem would be raised by their revision or relocation.

31. Fish facilities: The North Santiam River supports migratory salmon and steelhead trout and resident game trout. Due to the height of the proposed storage dam and the great seasonal range in pool level, it is considered neither practical nor necessary to build fish lifts or ladders to pass fish around the structure. These fish are now being artificially propagated in hatcheries owned and operated by the State Fish and Game Commission and by the U. S. Bureau of Fisheries. Extensions of these hatchery facilities, to replace the natural spawning

grounds made inacessible by the construction of the seven authorized dams, are proposed at a cost of $1,000,000. Studies are now under way to determine the proper location of these new hatcheries and their egg taking stations.

32. Estimated Cost of Low Dam: The low dam development, outlined herein is estimated to cost $12,903,000 or approximately $75 per acre foot of gross storage capacity. A detailed estimate of cost for the low dam is contained in Table V, appended. The present estimated cost exceeds that contained in the project document by approximately 90 percent. This increase is primarily due to the fact that foundation conditions, after detailed investigation, necessitated the substitution of a concrete gravity structure for the concrete arch dam originally contemplated, thus increasing the amount of excavation by approximately 570,000 cubic yards and the concrete quantities by about 375,000 cubic yards; to the fact that construction costs have risen approximately 20 percent since the date of the original estimate; and to the further fact that an additional 5 percent has been added to the contingency items in the present estimate to allow for the present trend of rising construction costs.

33. Field reconnaisance and laboratory tests indicate that, with some processing, suitable concrete aggregates may be obtained for both the low and high dams from a terrace gravel deposit lying along the North Santiam River approximately 17 miles below the damsite. This deposit is ideally situated with respect to both rail and truck transportation. Another similar deposit which has possibilities for the initial low dam construction is located in the reservoir area, approximately 5 miles

upstream from the damsite. The estimates herein are based upon obtaining concrete aggregate from the downstream deposit and hauling to the damsite by rail.

34. Construction Program: Before large scale construction can be undertaken at the Detroit site, lands will need to be acquired and cleared, a major part of the highway relocation must be completed, the railroad must be abandoned or relocated and contract drawings and specifications for the dam prepared. The time required to complete this phase of the work is estimated to take at least twelve months.

35. The low water period at the damsite normally extends from about the first of June to the end of October. If construction was commenced during the beginning of a low water period it is estimated that the low dam could be completed within a period of from 24 to 30 months. During the first low water season it would be necessary to divert the river through a diversion channel along the left bank and to complete all concreting in the other portions of the dam up to at least elevation 1225. The river would continue to flow through the diversion channel during the winter months and concreting would continue on the dam, except when temporarily suspended due to high water. During the second low water season the river would be diverted through two temporary sluice openings left in the base of the completed portion of the dam; the original diversion channel section would be closed with concrete; and concreting would continue on the remainder of the structure, carrying all concrete work up to at least elevation 1280 as soon as possible. After all concrete has been completed up to an elevation of 1280, the temporary diversion sluices would be plugged and the river

diverted through the permanent outlets before the second high water season. From this point on, concreting could continue to completion without any further diversion problem.

36. Single step construction of the high dam and power plant would follow practically the same program indicated for the low dam except that closure of the temporary sluices, due to the greater volume of concrete involved, might run into a third low water season, thus increasing the period of construction to about 30 or 36 months.

37. If the low dam is raised in the future it would be advisable to draw the reservoir pool down to as low a stage as practicable in order to unload the low [illegible] structure before the second step concrete is placed thereon and tied into it. Such a procedure is recommended in order to obtain a better transfer of stresses across the contact plane, and to obtain better stress distribution between the component parts of the final structure after the water load comes on. Preliminary studies indicate that the most practicable method of diversion during the second step raising is to unplug the diversion sluice outlet left in the low dam during construction and, during a low water period, to drive a large diameter diversion tunnel under the low dam through the left abutment. The time required to raise the low dam to the ultimate height under this method of procedure is estimated at approximately 24 months. The power house construction could proceed in parallel with the work on the dam.

III Ultimate High Dam Plan

38. General: The ultimate high dam structure is so laid out and designed that full use is made of the original low dam concrete, pract-

ically all of which is contained in the high dam section. The high dam estimates, for the design utilizing the low dam as part of the structure, are based upon roughing, dowling, grouting and draining the contact plane between the two component parts of the finished structure. The type of facilities proposed for the high dam, and their relative location, are similar to those contemplated for the low dam with the exception that the spillway is to be controlled by three 100' x 18' automatic drum gates. The gate controlled spillway is economical for the high dam due to the fact that it reduces the maximum pool elevation by 17 feet, resulting in a reduction in concrete quantities and flowage costs, and permits dropping the elevation of the Breitenbush River crossing on the proposed relocated North Santiam River Highway to elevation 1,585.

39. Due to the fact that future construction costs, future power values, and incidental benefits cannot be accurately determined at this time, the ultimate height to which the dam can be economically raised in the future cannot be definitely fixed until the time such enlargement takes place. Comparative studies of cost of raising and incidental benefits, based upon present construction costs and power values and taking into consideration potential future downstream power developments, indicate that the height of the high dam as proposed, giving a gross storage capacity of 432,000 acre feet, is near the upper limit of economical storage development. This conclusion is also valid if single step construction is contemplated to full height at the present time.

40. Due to the uncertainty of future conditions and due to the fact that only a small initial cost is involved, the North Santiam River Highway and the railroad, if relocation of the latter is required, are

proposed to be raised, in general, to elevation 1,605 making it possible to utilize a high dam normal pool elevation of approximately 1,590 in the future if found economically advisable.

41. <u>Center Spillway Section Versus Right Abutment Spillway Sections</u>: Comparative layouts and estimates of cost have been made for (a) a high dam plan utilizing a center section spillway with the power house downstream from the dam on the right bank and (b) a high dam plan utilizing a right abutment spillway with the power house located at the downstream toe of the non-overflow river section. Comparative layouts for these two plans, based upon raising of the low dam in the future on the downstream face, are shown on Plates VII and IX, and detailed cost estimates are given in attached Tables VII and IX respectively. As may be noted from these tables the estimated cost of these two alternate plans of development, including power facilities, are approximately the same; $28,440,000 for the river section spillway and $28,338,000 for the right abutment spillway.

42. The right abutment spillway plan utilizes three 50' x 50' crane operated lift gates, in order to decrease the width and cost of the spillway get-away channel and also to make it possible to reduce the height of the dam by approximately 3.5 feet, while the river spillway section utilizes three 18' x 100' drum gates as mentioned hereinbefore. The river spillway plan is preferred and recommended by this office (although the right abutment plan may warrant further consideration if single step construction of the high dam is undertaken) for the following reasons; (a) the available location for the right abutment

spillway necessitates a horizontal curve, with high superelevation, in the get-away channel alignment complicating the hydraulics and raising an uncertainty as to the adequacy of the cost estimate for this feature, (b) the foundation rock in the right abutment is highly jointed and deeply weathered which condition, with the high velocities involved, necessitates lining of the get-away channel and raises an uncertainty as to the adequacy of the cost estimates for the training walls, and (c) the disgorgement of the spillway discharge into the river channel immediately below the power house location may set up extreme water surface fluctuations necessitating an increased length of training wall along the left side of the get-away channel and/or some provision for energy dissipation at the lower end of the spillway channel thus raising the cost over that estimated herein.

43. Rough comparative estimates of cost for an alternate left abutment spillway indicate that due to poor alignment and extensive excavation requirements no saving would be involved in such a layout over the plans discussed herein.

44. Two-Step High Dam Development: The alignment of the low dam is such that when additional storage at the site becomes warranted, raising of the dam can be undertaken on either the downstream or upstream face. Accordingly, layouts, designs and cost estimates have been prepared for both of these alternate methods of procedure. Plate VII shows the lay-out and Table VII gives a detailed cost estimate for raising of the low dam on the downstream face while Plate VIII and Table VIII contain similar data for raising on the upstream face. The foregoing designs

and cost estimates are based upon drum gate controlled river spillway sections as discussed hereinbefore.

45. Two-step construction of the high dam and reservoir can be accomplished at practically the same cost no matter whether the low dam is raised on the downstream or upstream side. A summary of the comparative cost estimates is shown in the following table. The increment cost of the 260,000 acre feet of additional storage provided by the high dam over that provided by the low dam is estimated at approximately $45 per acre foot, while the total cost of storage in the high dam is estimated at $66 per acre foot as compared with a cost of $75 per acre foot for the low dam.

Feature	Method of Raising	
	Downstream	Upstream
Cost of Initial Low Dam	$12,903,000	$12,903,000
Cost of Raising	11,566,000	11,523,000
Cost of High Dam and Reservoir	24,469,000	24,426,000
Cost of Power Facilities	3,971,000	3,971,000
Total Cost, High Dam, Reservoir and Power Plant	$28,440,000	$28,397,000

46. Geologic and foundation conditions are such as to slightly favor raising of the dam on the downstream face. Raising on the downstream side will also give better stress conditions on the contact plane between the two component parts of the two-step high dam under maximum pool conditions as the inclination of the contact plane practically corresponds with the direction of the first principal stresses under the foregoing conditions of load. Pertinent stability and internal stress studies for the various high dam plans are contained in Appendix "B".

47. Single Step High Dam Development: A study has been made to determine what savings in overall construction costs might result if the ultimate high dam were constructed to full height in a single step. This study was based upon the layout and cross sections shown on Plate VII, with the exception that only three Paradox gate controlled flood control outlets are required, in lieu of the five proposed for the low dam, due to the higher minimum pool elevation and correspondingly higher minimum head available on the outlets.

48. The cost of the Detroit Dam and reservoir, with single step construction to full height, is estimated at $22,115,000, as shown in Table VI, or $2,354,000 less than the two-step construction of a similar dam to the same height. The cost of power facilities at the Detroit site will be approximately the same whether the dam is built in one step or two. The savings in construction cost in favor of the single step program is due primarily to the fact that the diversion problem is not duplicated, the flood control outlets are reduced in number, the reservoir pool area below the high dam minimum pool elevation need not be cleared, no special treatment or provision has to be made for a contact plane between two concrete sections formed at different times, and to the fact that the total concrete volume required for the high dam can be placed more economically in one, steady, continuous operation than in two, separate and distinct operations.

49. Attention is invited to the fact that if the high dam design were to be based upon single step construction to full height, a somewhat more practical cross section and better alignment for the high dam

might be developed. However, no major change in quantities or costs would result from such adjustment.

50. Detroit Power Plant: The 371 feet of gross head made available by the Detroit high dam and the dependable annual power draft of 1,377 cubic feet per second made available by the reservoir will make possible the development of 26,800 kilowatts of continuous and dependable power through a power plant proposed to be located at the damsite. The location of the proposed power plant, with respect to the dam is shown on the various high dam layout drawings, its estimated cost is given in the various high dam estimates attached, and pertinent descriptive data relating to it is given in Table I. The proposed power plant, with an installed capacity of 90,000 kilowatts, is estimated to cost $3,971,000 or approximately $44 per kilowatt of installed capacity.

51. The Detroit power plant is designed for peak load operation on a low daily load factor, being capable of peaking at all times to the full capacity of its generators. Such output characteristics make it possible, and mutually beneficial, for the plant to be coordinated in operation with a high load factor run of river plant, such as the Bonneville Project on the Columbia River, or with a series of high load factor river plants similar to those projected for future construction on the stretch of the North Santiam River below the Detroit Dam site. If considered desirable at the time the Detroit power plant is constructed, the generator installation could be limited to one or two units, 30,000 or 60,000 kilowatts, until such time as the full peaking capability of the plant is needed.

52. Potential Downstream Power Developments: With regulation of the stream flow by the proposed Detroit high dam reservoir it would be physically possible, when warranted by power requirements, to economically develop an additional 650 feet of gross head through three potential power sites located on the North Santiam River below the proposed Detroit power plant. Preliminary cost estimates indicate that these potential downstream sites can most economically be developed by means of tunnels and side hill canals, utilizing relatively low dams primarily for diversion purposes. Accordingly, in order to keep the overall cost of power developments to a minimum, these plants should be designed to operate under practically a constant power draft, resulting in high load factor plants, all peak loads being carried by the Detroit power plant where storage is available and where peaking capacity can be obtained most economically. The foregoing method of operation would require the construction of a re-regulating dam below the Detroit power plant in order to smooth out the daily and hourly flow fluctuation from that plant. Pertinent data relating to the regulating dam and the potential downstream power sites is contained in the published project document, House Document No. 544, Seventy-Fifth Congress, Third Session. Co-ordinated operation of the Detroit power plant with the three potential downstream plants would make available the following new power output on the North Santiam River:

Plant Characteristics	Detroit Power Plant	Downstream Power Plants	Combined North Santiam River System
Installed capacity - kw.	90,000	52,800	142,800
Firm Capacity - kw.	82,400	51,700	125,500 (a)
Firm Output - Kwy.	26,800	48,500	75,300
Annual Load Factor - %	33	94	60

(a) Coincident Peak

53. If desired, the output of the Detroit power plant, in lieu of being coordinated with the potential downstream developments on the North Santiam River, could be utilized to peak up any other similar block of 48,500 kilowatts of continuous power in the region.

IV Benefits.

54. Flood Control: The initial low dam and the ultimate high dam will have similar flood control benefits and will, with coordinated operation of the other six authorized reservoirs in the basin, eliminate approximately 95 percent of the potential flood damage on the North Santiam River and approximately 75 percent of such damage on the Lower Willamette.

55. Supplemental low water supply: The operation of the foregoing reservoirs will maintain a minimum flow of 6500 c.f.s. at Salem on the main stream in the interest of navigation and, in addition, will furnish some 565,000 acre-feet of supplemental water during the irrigation season. The allocation of this new water from individual reservoirs between navigation and irrigation will depend primarily upon the location of the new lands that are put under irrigation.

56. Power: When power developments on the North Santiam River become warranted, the addition of supplemental power storage in the proposed Detroit reservoir, to make possible its coordinated use in the interest of power as well as for flood control and increased low water flow, is feasible. The total storage thus provided will make possible the economic development of some 1,020 feet of gross head at and below the Detroit reservoir and will make available a firm dependable output

of 75,300 kilowatt years. Sufficient capacity can be installed at the Detroit power plant to operate the overall development on a 60 percent annual load factor.

57. Other Benefits: The increased low water flow on the main stream, part of which is creditable to the Detroit reservoir, will incidentally benefit stream purification, fish life and the existing private power installations at Oregon City.

V Local Cooperation.

58. Local Cooperation: The provisions of the Flood Control Act of 1938 eliminated all requirements of local cooperation with respect to the Detroit Dam and reservoir. Close cooperation is being maintained, however, with the Public Roads Administration, the U. S. Bureau of Fisheries and various State Agencies, such as the Fish and Game Commissions, whose interests will be affected by the proposed improvement.

TABLE I

Descriptive Data

Detroit Dam and Reservoir, North Santiam River, Oregon

Project Plan

1. General Data:

Name	Detroit
Stream	North Santiam River
Stream Mile	47
Drainage Area, Square Miles	438
Tailwater Elevations, M.S.L.	
T.W. for Mean Daily Discharge, 2,026 c.f.s.	1197.6
T. W. for Q = 3,000 c.f.s.	1198.3
T.W. for Q = 10,000 c.f.s.	1201.8
T.W. for Q = 111,000 c.f.s. (1000 year)	1226.9
T.W. for Q = 149,500 c.f.s. (spillway design—High Dam)	1231.7
T.W. for Q = 161,500 c.f.s. (spillway design—Low Dam)	1233.1

2. Data for Low Dam (Initial Development):

Reservoir Data:

Minimum Pool Elevation, M.S.L.	1286.0
Normal Pool Elevation, M.S.L.	1464.0
Pool Elevation for 1000-year Flood (120,000 c.f.s.) M.S.L.	1483.8
Pool Elevation for Spillway Design Flood (176,000 c.f.s.) M.S.L.	1489.0
Area at Elevation 1286.0, Acres	220
Area at Elevation 1464.0, Acres	1780
Area at Elevation 1489.0, Acres	2055
Storage Minimum Pool, Acre-feet	7,000
Storage Normal Pool, Acre-feet	172,000
Storage Maximum Pool, Acre-feet	219,000
Usable Storage, and Maximum Flood Control Reservation, Acre-feet	165,000
Usable Storage, and Maximum Flood Control Reservation, Inches per Sq. Mile	7.06
Highway Revision, Miles	12.5
Railroad Revision, Miles	None
Telephone Line Revision, Miles	12.5

Outlet Data:

Sluice Gates:	
Type	Paradox Slide Gates
Number and Size	5 - 90" diameter
Emergency Gates	90" Ring Follower
Elevation Centerline Intake, M.S.L.	1225
Maximum Cross Head at Outlets, Feet	262

Regulating Valves:

Type	Needle Valves
Number and Size	2 - 60" Diameter
Emergency Gates	108" ring follower
Elevation Centerline Intake, M.S.L.	1255
Gross Head, Feet	234
Capacity at Minimum Pool, C.F.S.	10,000
Capacity at Normal Pool, C.F.S.	18,100

Spillway Data:

Type	Free Overflow
Crest Length, Feet	332
Elevation Crest, M.S.L.	1464.0
Elevation Maximum W.S., M.S.L.	1489.0
Capacity with Pool Elevation 1489.0, C.F.S.	161,500

Dam Section:

Crest Length, Feet	1110
Crest Elevation, M.S.L.	1489.0
Freeboard above Maximum W.S., Feet	0
Maximum Height, Feet	360
Crest Width, Feet	20

3. Data for High Dam (Coordinated Plan):

Reservoir Data:

Minimum Pool Elevation for Power, M.S.L.	1425.0
Minimum Pool Elevation for Flood Control, M.S.L.	1503
Normal Pool Elevation, M.S.L.	1569.0
Pool Elevation for 1000-year Flood, M.S.L.	1572.2
Pool Elevation for Spillway Design Flood, M.S.L.	1577.3
Area at Elevation 1425.0, Acres	1360
Area at Elevation 1569.0, Acres	3450
Area at Elevation 1577.3, Acres	3610
Storage, Minimum Pool, Acre-feet	110,000
Storage, Normal Pool, Acre-feet	432,000
Storage, Maximum Pool, Acre-feet	460,000
Usable Storage, Acre-feet	322,000
Maximum Flood Control Reservation, Acre-feet	165,000
Maximum Flood Control Reservation, Inches per square mile	7.06
Highway Revision, Miles	16.0
Railroad Revision, Miles	None
Telephone Line Revision, Miles	12.5

Outlet Data:

Sluice Gates:	
Type	Paradox Slide Gates
Number and Size	3 - 90" Diameter
Emergency Gates	90" Ring Follower
Elevation Centerline Intake, M.S.L.	1225
Gross Head at Outlet, Feet	367
Regulating Valves:	
Type	Needle Valves
Number and Size	2 - 60" Diameter
Emergency Gates	108" Ring Follower
Elevation Centerline Intake, M.S.L.	1255
Gross Head, Feet	339
Capacity at Minimum Pool, C.F.S.	11,000
Capacity at Normal Pool, C.F.S.	14,250

Spillway Data:

Type	Drum Gates
Number and Size of Gates	3 - 18' x 100'
Elevation Crest, M.S.L.	1,551
Elevation Maximum W.S., M.S.L.	1,577.3
Capacity with Pool Elevation 1577.3, C.F.S.	149,500

Dam Section:

Crest Length, Feet	1,500
Crest Elevation, M.S.L.	1,577.5
Freeboard above Maximum W.S., Feet	0.2
Maximum Height, Feet	448
Crest Width, Feet	20

Power Plant:

Number of Units	3
Total Installed Capacity, kw.	90,000
Maximum Gross Head, Feet	371
Rated Gross Head, Feet	275
Minimum Gross Head, Feet	227
Number of Penstocks	3
Penstock Diameter, Feet	10
Penstock Length, Feet	600
Power Draft through Plant at Minimum Head and Maximum Output, c.f.s.	5205
Velocity through Penstocks at Minimum Head and Maximum Output, feet per second	22.10
Head Loss through Penstocks at Minimum Head and Maximum Output, Feet	7.59
Rated Power Draft, C.F.S.	1,377
Firm Capacity, kw.	82,400
Firm Output, 1000 K.W.H./Annum	235,000
Annual Load Factor, %	33

TABLE II

Precipitation and Run-off Records, North Santiam

Drainage Basin

Precipitation Stations

Station	Elevation	Type	Records Available From	Records Available To	Years	Character of Record
Detroit	1450	Manual	1894	1939	32	Broken
Detroit	1450	Recorder	1939	1941	2	Continuous
Hoover	1642	Manual	1909	1916	7	"
Mehama	628	Manual	1923	1941	18	"
Breitenbush	2225	Recorder	1940	1941	1/2	"
Marion Forks	2432	Recorder	1940	1941	1/2	"
Santiam Junction	3750	Recorder	1940	1941	1/2	"
Santiam Pass	4800	Recorder	1940	1941	1/2	"

Stream Gaging Stations

Station	Stream	Drainage Area Sq. Mi.	Type of Gage	Records Available From	Records Available To	Records Available Years	Character of Record
Detroit (near)	Breitenbush River	106	Staff	1910	1913	3	Continuous
Detroit (near)	Breitenbush River	108	Rec.	1932	1941	9	"
Detroit (Near)	North Santiam	224	Staff	1907	1909	2	"
			Staff	1910	1913	3	"
			Staff	1928	1932	4	"
			Rec.	1932	1941	9	"
Detroit Damsite	North Santiam	438	Staff	1938	1939	1	"
			Rec.	1939	1941	1	"
Niagara	North Santiam	470	Staff	1908	1910	2	"
			Staff	1911	1919	8	"
Mehama	North Santiam	655	Staff	1905	1907	2	"
			"	1910	1914	4	"
			"	1921	1933	12	"
			Rec.	1933	1941	8	"
Mehama	Little N.Santiam	110	Staff	1931	1941	10	"

TABLE NO. III

Climatological Data for

The North Santiam Drainage Basin, Oregon

Mehama, Oregon
Period from 1923 to 1941 incl.

	J	F	M	A	M	J	J	A	S	O	N	D	Annual
Precip.													
Mean	8.64	7.87	7.39	4.47	3.70	2.45	0.49	0.82	3.22	5.12	8.19	9.72	62.08
Max.	15.58	14.64	14.39	12.16	7.63	6.55	1.95	4.27	8.01	9.91	15.58	19.57	84.98
Min.	4.01	1.43	1.25	1.24	0.80	0.37	0.00	0.00	0.35	0.45	0.66	4.31	46.88
Snowfall													
Mean	5.5	1.7	0.5	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.4	8.3

Detroit, Oregon
Period from 1894 to 1941 incl.

	J	F	M	A	M	J	J	A	S	O	N	D	Annual
Precip.													
Mean	11.14	8.36	7.24	4.55	3.76	2.10	0.62	0.82	2.85	5.03	11.72	9.94	68.13
Max.	21.47	16.83	16.10	8.05	10.48	4.96	3.87	5.33	8.49	11.34	23.23	24.70	92.95
Min.	5.50	0.26	1.09	0.98	0.56	0.07	0.0	0.0	0.0	0.10	0.56	3.14	43.55
Snowfall													
Mean	23.9	15.6	11.8	2.9	0.4	0.0	0.0	0.0	0.0	0.1	2.5	13.7	70.9
Temp.													
Mean	34.2	37.6	41.7	47.5	54.1	59.1	65.0	65.0	57.8	51.3	41.3	36.5	49.3
Av.Max.	41.0	45.7	52.6	59.9	68.6	74.4	83.3	84.9	73.2	64.6	51.2	43.5	61.9
Av.Min.	27.4	29.5	31.1	35.1	39.6	44.4	46.8	45.0	43.1	38.1	31.3	29.5	36.7
Max.	64	69	79	90	95	105	104	106	99	90	70	60	106
Min.	-2	15	12	26	26	30	32	31	28	25	16	13	-2
Average frost-free days	6	6	9	18	28	30	31	31	30	24	11	9	233

Table IV

Runoff Data for the Detroit Dam Site

Period: 1905-1907, 1909-1919, 1921-1939, inclusive.

Basis: Mehama runoff x factors varying from .46 to .90	= Detroit runoff.
Niagara runoff x .97	= Detroit runoff.
Mean annual runoff - acre-feet	1,475,000
Maximum annual runoff - acre-feet (1912-13)	1,910,800
Minimum annual runoff - acre-feet (1930-31)	990,000
Mean daily discharge - c.f.s.	2,026
Minimum daily discharge - c.f.s. (Dec. 4-7, 1929)	320
Maximum recorded flood peak - c.f.s. (Nov. 22, 1909)	61,300

Seasonal Variation in Runoff - c.f.s.

Month	Mean Day	Minimum Day	Maximum Day
January	2,860	546	30,200
February	2,740	722	23,200
March	2,550	725	22,800
April	2,830	941	21,350
May	2,855	846	7,820
June	2,150	616	15,500
July	1,005	455	5,230
August	643	407	6,190
September	720	378	3,300
October	953	395	8,460
November	2,483	327	48,500
December	2,520	320	33,200

Table V

Detailed Estimate of Cost

Detroit Low Dam and Reservoir
(See Plate II)

Item NO.	Item	Unit	Quantity	Unit Cost	Amount	Total
A.	Reservoir and Flowage					$ 3,158,000
	Privately owned land	Ac.	1,750	50.00	87,500	
	Clearing and Grubbing	Ac.	2,060	100.00	206,000	
	Forest Service Lands and Facilities	L.S.			169,200	
	Detroit Community	L.S.			43,000	
	Rural Homes	L.S.			30,000	
	Sawmill and Logging Facilities	L.S.			55,000	
	Purchase Railroad (a)	L.S.			750,000	
	Highway Relocation - Station 125 to 630 (b)	Mi.	8.4	141,000.00	1,184,400	
	Highway Reconstruction - Station 630 to Niagra	Mi.	4.1	52,500.00	215,250	
	Telephone Facilities	Mi.	12.5	500.00	6,250	
	Contingencies - 15%				411,400	
B.	Concrete Dam and Apron					$7,602,000
	Preliminary	L.S.			180,000	
	River Diversion and Control	L.S.			75,000	
	Excavation, Common	C.Y.	133,000	1.00	133,000	
	Excavation, Rock	C.Y.	262,000	2.00	524,000	
	Foundation Cleanup	S.Y.	28,000	1.00	28,000	
	Backfill, Dumped	C.Y.	33,000	0.50	16,500	
	Concrete, Mass-dam	C.Y.	692,000	7.00	4,844,000	
	Concrete, Mass-walls	C.Y.	8,200	11.00	90,200	
	Concrete, Apron	C.Y.	19,000	10.00	190,000	
	Concrete, Heavy Structural	C.Y.	2,500	16.00	40,000	
	Concrete, Medium Structural	C.Y.	2,100	20.00	42,000	
	Concrete, Architectural	C.Y.	770	35.00	26,950	
	Grouting Water Stops, cooling, etc.	C.Y.	723,000	0.10	72,300	
	Reinforcing Steel	Lb.	2,016,000	0.05	100,800	
	Structural Steel	Lb.	184,000	0.10	18,400	
	Anchor bars - apron	L.F.	10,500	1.25	13,125	
	Porous Concrete Drain Tile	L.F.	20,000	1.25	25,000	
	Sewer Drain Tile	L.F.	10,000	2.50	25,000	
	Consolidation Grouting	L.F.	6,000	3.00	18,000	
	Cut-off Grout Curtain	L.F.	36,400	3.50	127,400	
	Foundation Drainage	L.F.	7,000	3.00	21,000	
	Contingencies - 15%				991,325	

Table V (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
C.	Flood Control Outlets					$ 706,000
	Concrete, Reinforced	C.Y.	370	25.00	9,250	
	Reinforcing Steel	Lb.	56,000	0.05	2,800	
	Structural Steel — Trash Racks	Lb.	160,000	0.10	16,000	
	90" Diameter Paradox Service Gate	Ea.	5	33,000.00	165,000	
	90" Diameter Ring Follower Emergency Gate	Ea.	5	30,000.00	150,000	
	60" Needle Valve	Ea.	2	21,000.00	42,000	
	108" Diameter Ring Follower Emergency Gate	Ea.	2	36,000.00	72,000	
	Steel Liner Plates	Lb.	770,000	0.10	77,000	
	Needle Valve House	L.S.			5,000	
	Operating Cranes	L.S.	2	30,000.00	60,000	
	Miscellaneous Equipment	L.S.			10,000	
	Operator's House	L.S.			5,000	
	Contingencies - 15%				91,950	
D.	Penstock Openings:					$ 3,000
	Concrete Plug	C.Y.	120	10.00	1,200	
	Reinforcing Steel	Lb.	18,000	0.05	900	
	Contingencies - 15%				900	
E.	Field Cost					$11,469,000
	Engineering & Overhead-12½%					1,434,000
F.	GRAND TOTAL-LOW DAM AND RESERVOIR					$12,903,000

(a) Based upon acquisition and abandonment of Mill City Branch of Southern Pacific Railway. If railway has to be relocated, cost will be increased by approximately $1,750,000.

(b) Based upon Public Roads Administration carrying $800,000 of Highway Revision Cost.

Table VI

Detailed Estimate of Cost

Single Step Detroit High Dam and Reservoir

River Spillway Layout

(See Plate VII)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
A.	Single Step High Dam					
1.	Reservoir and Flowage					$ 3,776,000
	Privately owned land	Ac.	2,720	50.00	136,000	
	Clearing below Minimum Pool	Ac.	1,350	20.00	27,000	
	Clearing above Minimum Pool	Ac.	2,550	120.00	306,000	
	Forest Service Lands and Facilities	L.S.			180,200	
	Detroit Community	L.S.			43,000	
	Rural Homes	L.S.			30,000	
	Sawmills and Logging Facilities	L.S.			55,000	
	Purchase Railroad (a)	L.S.			750,000	
	Highway Relocation - Station 125 to 630	Mi.	8.4	141,000.00	1,184,400	
	Highway Reconstruction- Station 630 to Niagra	Mi.	4.1	52,500.00	215,250	
	Highway Relocation - Station 163 to - 15	Mi.	3.5	100,000.00	350,000	
	Telephone Facilities	Mi.	12.5	500.00	6,250	
	Contingencies - 15%				492,900	
2.	Concrete Dam and Apron					$14,994,000
	Preliminary	L.S.			180,000	
	River Diversion and Control	L.S.			85,000	
	Excavation, Common	C.Y.	217,000	1.00	217,000	
	Excavation, Rock	C.Y.	395,000	2.00	790,000	
	Foundation Cleanup	S.Y.	44,000	1.00	44,000	
	Backfill, Dumped	C.Y.	73,000	0.50	36,500	
	Concrete, Mass-dam	C.Y.	1,499,000	6.75	10,118,250	
	Concrete, Mass-walls	C.Y.	11,300	11.00	124,300	
	Concrete, Apron	C.Y.	19,000	10.00	190,000	
	Concrete, Heavy Structural	C.Y.	19,700	13.00	256,100	
	Concrete, Medium Structural	C.Y.	7,000	17.50	122,500	
	Concrete, Architectural	C.Y.	1,170	32.50	38,025	
	Grouting Water Stops, Cooling, etc.	C.Y.	1,560,000	0.20	312,000	

Table VI (Continued)

No.	Item	Unit	Quantity	Unit Cost	Amount	Total
	Concrete Dam and Apron (Cont.)					
	Reinforcing Steel	Lb.	3,500,000	0.05	175,000	
	Structural Steel	Lb.	268,000	0.10	26,800	
	Anchor Bars - Apron	L.F.	10,500	1.25	13,125	
	Porous Concrete Drain Tile	L.F.	34,000	1.25	42,500	
	Sewer Drain Tile	L.F.	10,000	2.50	25,000	
	Consolidation Grouting	L.F.	13,000	3.00	39,000	
	Cut-off Grout Curtain	L.F.	50,000	3.50	175,000	
	Foundation Drainage	L.F.	9,500	3.00	28,500	
	Contingencies - 15%				1,955,400	
3.	Flood Control Outlets					$ 557,000
	Concrete, Reinforced	C.Y.	250	25.00	6,250	
	Reinforcing Steel	Lb.	37,000	0.05	1,850	
	Structural Steel-Trash Racks	Lb.	112,000	0.10	11,200	
	90" diameter Paradox Service Gate	Ea.	3	33,000.00	99,000	
	90" diameter Ring Follower Emergency Gate	Ea.	3	30,000.00	90,000	
	60" diameter Needle Valves	Ea.	2	21,000.00	42,000	
	108" diameter Ring Follower Emergency Gate	Ea.	2	36,000.00	72,000	
	Steel Liner Plates	Lb.	818,000	0.10	81,800	
	Needle Valve House	L.S.			5,000	
	Operating Cranes				60,000	
	Miscellaneous Equipment	L.S.			10,000	
	Operator's House	L.S.			5,000	
	Contingencies - 15%				72,900	
4.	Spillway Gates					$ 331,000
	Drum Gates - 100' x 18'	Ea.	3	81,000.00	243,000	
	Gate Frames & Miscellaneous	Lb.	291,000	0.10	29,100	
	Control Mechanism	Lb.	102,000	0.15	15,300	
	Contingencies - 15%				43,600	
5.	Field Cost					$19,658,000
6.	Engineering & Overhead - 12½%					$ 2,457,000
7.	Total—Single Step High Dam and Reservoir					$22,115,000

Table VI (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
B.	Power Plant and Equipment					$ 3,967,000
	Common Excavation	C.Y.	78,000	1.00	78,000	
	Rock Excavation	C.Y.	50,000	2.00	100,000	
	Foundation Preparation	S.Y.	1,700	2.00	3,400	
	Light Structural Concrete	C.Y.	700	25.00	17,500	
	Heavy Structural Concrete	C.Y.	9,600	15.00	144,000	
	Reinforcing Steel	Lb.	460,000	0.05	23,000	
	Structural Steel	Lb.	191,000	0.10	19,100	
	Penstock Steel	Lb.	2,280,000	0.10	228,000	
	13' x 13' Tractor Gate	Ea.	3	28,000.00	84,000	
	120" Butterfly Valve	Ea.	3	35,000.00	105,000	
	Stog Logs and Guides	Lb.	30,000	0.10	3,000	
	Superstructure	C.F.	650,000	0.75	487,500	
	Turbines and Governors	H.P.	135,000	5.25	708,750	
	Generators	K.W.	90,000	7.00	630,000	
	Switching Facilities and Transformers	K.W.	90,000	4.50	405,000	
	Crane and Auxiliary Equip.	L.S.			20,000	
	Miscellaneous	L.S.			10,000	
	Contingencies - 15%				459,750	
	Engineering & Overhead-12½%				441,000	
C.	Grand Total - Single Step High Dam, Reservoir, and Power Facilities					$26,082,000

(a) Based upon acquistion and abandonment of Mill City Branch of Southern Pacific Railway. If railway has to be relocated, cost will be increased by approximately $1,750,000.

(b) Based upon Public Roads Administration carrying $800,000 of Highway Revision cost.

Table VII

Detailed Estimate of Cost

Two-Step Detroit High Dam and Reservoir

Raised Downstream

River Spillway Layout

(See Plate VII)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
A.	Second Step - High Dam					
1.	Reservoir and Flowage					$ 779,000
	Privately Owned Land	Ac.	970	50.00	48,500	
	Clearing and Grubbing	Ac.	1,850	145.00	268,250	
	Forest Service Land and Facilities	L.S.			10,800	
	Highway Relocation Station 163 to -15	Mi.	3.5	100,000.00	350,000	
	Contingencies - 15%				101,450	
2.	Concrete Dam and Apron					$ 9,104,000
	Preliminary	L.S.			25,000	
	River Diversion and Control	L.S.			170,000	
	Excavation, Common	C.Y.	85,000	1.00	85,000	
	Excavation, Rock	C.Y.	140,000	3.00	420,000	
	Excavation, Concrete	C.Y.	12,500	10.00	125,000	
	Foundation Cleanup	S.Y.	20,000	1.00	20,000	
	Backfill, Dumped	C.Y.	58,000	0.50	29,000	
	Backfill, Grouted	C.Y.	5,500	5.00	27,500	
	Concrete, Mass-dam	C.Y.	807,000	7.25	5,850,750	
	Concrete, Mass-walls	C.Y.	3,500	12.00	42,000	
	Concrete, Apron	C.Y.	9,000	11.00	99,000	
	Concrete, Heavy Structural	C.Y.	19,700	14.00	275,800	
	Concrete, Medium Structural	C.Y.	4,900	19.00	93,100	
	Concrete, Architectural	C.Y.	1,170	32.50	38,025	
	Grouting Water Stops, Cooling, Etc.	C.Y.	837,000	0.25	209,250	
	Preparation Concrete Surface	S.Y.	28,000	2.00	56,000	
	Reinforcing Steel	Lb.	2,556,000	0.05	127,800	
	Structural Steel	Lb.	84,000	0.10	8,400	
	Anchor Bars - Face of Dam	L.F.	99,000	1.00	99,000	
	Anchor Bars - Stilling Basin	L.F.	4,600	1.25	5,750	
	Porous Concrete Drain Tile	L.F.	14,000	1.25	17,500	
	Sewer Drain Tile	L.F.	2,100	2.50	5,250	
	Consolidation Grouting	L.F.	7,100	3.00	21,300	
	Cut-off Grout Curtain	L.F.	13,300	3.50	46,550	
	Foundation Drainage	L.F.	6,500	3.00	19,500	
	Contingencies - 15%				1,187,525	

Table VII (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
3.	Flood Control Outlets					$ 67,000
	Remove Concrete	L.S.			2,000	
	Steel Liner Plates	Lb.	253,000	0.10	25,300	
	Move Needle Valve	L.S.			8,000	
	Move Auxiliary Equipment	L.S.			10,000	
	Operator's House				13,000	
	Contingencies - 15%				8,700	
4.	Spillway Gates					$ 331,000
	Drum Gates 100' x 18'	Ea.	3	81,000.00	243,000	
	Gate Frames & Miscellaneous	Lb.	291,000	0.10	29,100	
	Control Mechanism	Lb.	102,000	0.15	15,300	
	Contingencies - 15%				43,600	
5.	Field Cost					$10,281,000
6.	Engineering & Overhead-12½%					$ 1,285,000
7.	Total Cost of Raising Low Dam					$11,566,000
B.	Estimated Cost of Low Dam (See Table V)					$12,903,000
C.	Grand Total - High Dam and Reservoir					$24,469,000
D.	Power Plant and Equipment					$ 3,971,000
	Common Excavation	C.Y.	78,000	1.00	78,000	
	Rock Excavation	C.Y.	50,000	2.00	100,000	
	Foundation Preparation	S.Y.	1,700	2.00	3,400	
	Light Structural Concrete	C.Y.	700	25.00	17,500	
	Heavy Structural Concrete	C.Y.	9,600	15.00	144,000	
	Apron Concrete	C.Y.	340	10.00	3,400	
	Reinforcing Steel	Lb.	460,000	0.05	23,000	
	Structural Steel	Lb.	191,000	0.10	19,100	
	Penstock Steel	Lb.	2,280,000	0.10	228,000	
	13' x 13' Tractor Gates	Ea.	3	28,000.00	84,000	
	120" Butterfly Valves	Ea.	3	35,000.00	105,000	
	Stop Logs and Guides	Lb.	30,000	0.10	3,000	
	Superstructure	C.F.	650,000	0.75	487,500	
	Turbines and Governors	H.P.	135,000	5.25	708,750	
	Generators	K.W.	90,000	7.00	630,000	
	Low Tension Switching Facilities	K.W.	90,000	4.50	405,000	
	Crane and Auxiliary Equip.	Ea.	1	20,000.00	20,000	
	Miscellaneous	L.S.			10,000	
	Contingencies - 15%				460,350	

Table VII (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
	Power Plant & Equip. (Cont.)					
	Engineering & Overhead-12½%				441,000	
E.	Grand Total Cost - High Dam, Reservoir & Power Facilities					$ 28,440,000

Table VIII

Detailed Estimate of Cost

Two-Step Detroit High Dam and Reservoir

Raised Upstream

River Spillway Layout

(See Plate VIII)

Item No.	Item	Unit	Quantity Amount	Unit Cost	Amount	Total
A.	Second Step-High Dam					
1.	Reservoir and Flowage					$ 779,000
	Privately Owned Land	Ac.	970	50.00	48,500	
	Clearing and Grubbing	Ac.	1,850	145.00	268,250	
	Forest Service Land					
	and Facilities	L.S.			10,800	
	Highway Relocation					
	Station 163 to -15	Mi.	3.5	100,000.00	350,000	
	Contingencies-15%				101,450	
2.	Concrete Dam					$ 9,045,000
	Preliminary	L.S.			25,000	
	River Diversion and Control	L.S.			125,000	
	Excavation, Common	C.Y.	98,000	1.00	98,000	
	Excavation, Rock	C.Y.	138,000	3.00	414,000	
	Excavation, Concrete	C.Y.	300	15.00	4,500	
	Foundation Cleanup	S.Y.	15,000	1.00	15,000	
	Backfill, Dumped	C.Y.	58,000	0.50	29,000	
	Concrete, Mass-dam	C.Y.	840,000	7.25	6,090,000	
	Concrete, Heavy Structural	C.Y.	17,000	14.00	238,000	
	Concrete, Medium Structural	C.Y.	3,400	20.00	68,000	
	Concrete, Architectural	C.Y.	1,180	32.50	38,350	
	Grouting Water Stops,					
	Cooling, etc.	C.Y.	875,000	0.25	218,750	
	Preparation Concrete Surface	S.Y.	25,000	2.00	50,000	
	Reinforcing Steel	Lb.	1,840,000	0.05	92,000	
	Structural Steel	Lb.	87,500	0.10	8,750	
	Anchor Bars - Face of Dam	L.F.	88,000	1.00	88,000	
	Porous Concrete Drain Tile	L.F.	34,000	1.25	42,500	
	Consolidation Grouting	L.F.	7,400	3.00	22,200	
	Cut-off Grout Curtain	L.F.	50,000	3.50	175,000	
	Foundation Drainage	L.F.	7,600	3.00	22,800	
	Contingencies - 15%				1,180,150	

Table VIII (Continued)

em No.	Item	Unit	Quantity	Unit Cost	Amount	Total
3.	Flood Control Outlets					$ 88,000
	Remove Concrete	L.S.			5,000	
	Concrete Reinforced	C.Y.	370	25.00	9,250	
	Reinforcing Steel	Lb.	55,000	0.05	2,750	
	Structural Steel - Trash Racks (Salvage)	Lb.	160,000	0.05	8,000	
	Steel Liner Plates	Lb.	528,000	0.10	52,800	
	Contingencies - 15%				10,200	
4.	Spillway Gates					$ 331,000
	Drum Gate - 100' x 18'	Ea.	3	81,000.00	243,000	
	Gate Frames & Miscellaneous	Lb.	291,000	0.10	29,100	
	Control Mechanism	Lb.	102,000	0.15	15,300	
	Contingencies - 15%				43,600	
5.	Field Cost-Second Step High Dam					$10,243,000
6.	Engineering & Overhead-12½%					$ 1,280,000
7.	Total Cost of Raising Low Dam					$11,523,000
B.	Estimated Cost of Low Dam (See Table V)					$12,903,000
C.	Grand Total - High Dam and Reservoir					$24,426,000
D.	Power Facilities					$ 3,971,000
	Common Excavation	C.Y.	78,000	1.00	78,000	
	Rock Excavation	C.Y.	50,000	2.00	100,000	
	Foundation Preparation	S.Y.	1,700	2.00	3,400	
	Light Structural Concrete	C.Y.	700	25.00	17,500	
	Heavy Structural Concrete	C.Y.	9,600	15.00	144,000	
	Apron Concrete	C.Y.	340	10.00	3,400	
	Reinforcing Steel	Lb.	460,000	0.05	23,000	
	Structural Steel	Lb.	191,000	0.10	19,100	
	Penstock Steel	Lb.	2,280,000	0.10	228,000	
	13' x 13' Tractor Gate	Ea.	3	28,000.00	84,000	
	120" Butterfly Valve	Ea.	3	35,000.00	105,000	
	Stop Logs and Guides	Lb.	30,000	0.10	3,000	
	Superstructure	C.F.	650,000	0.75	487,500	
	Turbines and Governors	H.P.	135,000	5.25	708,750	
	Generators	K.W.	90,000	7.00	630,000	
	Low Tension Switching Facilities	K.W.	90,000	4.50	405,000	

Table VIII (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
	Power Facilities (Continued)					
	Crane and Auxiliary Equipment	L.S.			20,000	
	Miscellaneous	L.S.			10,000	
	Contingencies - 15%				460,350	
	Engineering & Overhead-12½%				441,000	
E.	Grand Total Cost High Dam, Reservoir and Power Facilities					$28,397,000

Table IX

Detailed Estimate of Cost

Two-Step Detroit High Dam and Reservoir

Raised Downstream

Right Abutment Spillway Layout

(See Plate IX)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
A.	Estimated Cost of Raising Low Dam					
1.	Reservoir and Flowage					$ 779,000
	Privately owned lands	Ac.	970	50.00	48,500	
	Clearing and Grubbing	Ac.	1,850	145.00	268,250	
	Forest Service Land and Facilities	L.S.			10,800	
	Highway Relocation - Station 163 to -15	Mi.	3.5	100,000.00	350,000	
	Contingencies - 15%				101,450	
2.	Concrete Dam and Apron					$9,239,000
	Preliminary	L.S.			25,000	
	River Diversion and Control	L.S.			120,000	
	Excavation, Common	C.Y.	195,000	0.75	146,250	
	Excavation, Rock	C.Y.	354,000	1.50	531,000	
	Excavation, Concrete	C.Y.	7,100	11.00	78,100	
	Foundation Cleanup	S.Y.	27,000	1.00	27,000	
	Backfill, Dumped	C.Y.	58,000	0.50	29,000	
	Backfill, Grouted	C.Y.	4,000	5.00	20,000	
	Concrete, Mass-dam	C.Y.	849,000	7.25	6,155,250	
	Concrete, Mass-walls	C.Y.	3,400	12.00	40,800	
	Concrete Apron	C.Y.	14,000	10.00	140,000	
	Concrete Wall Paving	C.Y.	1,300	13.00	16,900	
	Concrete, Heavy Structural	C.Y.	1,000	17.00	17,000	
	Concrete, Medium Structural	C.Y.	2,000	20.00	40,000	
	Concrete, Architectural	C.Y.	1,330	32.50	43,225	
	Grouting Water Stops, Cooling, etc.	C.Y.	879,000	0.25	219,750	
	Preparation Concrete Surface	S.Y.	29,000	2.00	58,000	
	Reinforcing Steel	Lb.	1,684,000	0.05	84,200	
	Structural Steel	Lb.	85,000	0.10	8,500	
	Anchor Bars-Face of Dam	L.F.	99,000	1.00	99,000	
	Anchor Bars-Apron and Walls	L.F.	10,600	1.25	13,250	
	Porous Concrete Drain Tile	L.F.	11,500	1.25	14,375	
	Sewer Drain Tile	L.F.	2,200	2.50	5,500	
	Consolidation Grouting	L.F.	7,100	3.00	21,300	
	Cut-off Grout Curtain	L.F.	13,300	3.50	46,550	

Table IX (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
	Concrete Dam & Apron (Cont.)					
	Foundation Drainage	L.F.	4,800	3.00	14,400	
	Bridge over Spillway	L.S.			20,000	
	Contingencies - 15%				1,205,650	
3.	Flood Control Outlets					$ 67,000
	Remove Concrete	L.S.			2,000	
	Steel Liner Plates	Lb.	253,000	0.10	25,300	
	Move Needle Valves	L.S.			8,000	
	Move Auxiliary Equipment	L.S.			10,000	
	Operator's House	L.S.			13,000	
	Contingencies - 15%				8,700	
4.	Spillway Gates					$ 374,000
	Fixed Wheel Gates-50' x 50'	Ea.	3	50,000.00	150,000	
	Operating Crane	Ea.	1		124,000	
	Stop Log	Ea.	1		50,000	
	Miscellaneous Equipment	L.S.			10,000	
	Contingencies - 15%				40,000	
5.	Field Cost					$10,459,000
6.	Engineering & Overhead-12½%					$ 1,307,000
7.	Total Cost of Raising Low Dam					$11,766,000
B.	Estimated Cost of Low Dam (See Table V)					$12,903,000
C.	Grand Total - High Dam and Reservoir					$24,669,000
D.	Power Plant and Equipment					$ 3,669,000
	Concrete Excavation	C.Y.	3,500	10.00	35,000	
	Foundation Cleanup	S.Y.	1,320	2.00	2,640	
	Mass Concrete-Wall	C.Y.	800	13.00	10,400	
	Light Structural Concrete	C.Y.	1,200	25.00	30,000	
	Heavy Structural Concrete	C.Y.	9,500	15.00	142,500	
	Concrete Apron	C.Y.	350	10.00	3,500	
	Reinforcing Steel	Lb.	500,000	0.05	25,000	
	Structural Steel	Lb.	161,000	0.10	16,100	
	Penstock Steel	Lb.	1,170,000	0.10	117,000	
	13' x 13' Tractor Gates	Ea.	3	28,000.00	84,000	
	120" Butterfly Valve	Ea.	3	35,000.00	105,000	

Table IX (Continued)

Item No.	Item	Unit	Quantity	Unit Cost	Amount	Total
	Power Plant & Equip. (Cont.)					
	Stop Logs and Guides	Lb.	30,000	0.10	3,000	
	Superstructure	C.F.	650,000	0.75	487,500	
	Turbines and Governors	H.P.	135,000	5.25	708,750	
	Generators	K.W.	90,000	7.00	630,000	
	Switching Facilities and Transformers	K.W.	90,000	4.50	405,000	
	Crane and Auxiliary Equip.	L.S.			20,000	
	Miscellaneous Equipment	L.S.			10,000	
	Contingencies - 15%				425,610	
	Engineering & Overhead-12½%				408,000	
E.	Grand Total Cost High Dam, Reservoir and Power Facilities					$28,338,000

WAR DEPARTMENT
U. S. ENGINEER OFFICE
PORTLAND DISTRICT
PORTLAND, OREGON

APPENDIX A TO ACCOMPANY DEFINITE PROJECT REPORT ON DETROIT DAM AND RESERVOIR NORTH SANTIAM RIVER, OREGON

STRESS AND STABILITY STUDIES

LLOYD L. RUFF

AUGUST 9, 1941

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Appendix A

Final Definite Project Report on Detroit Dam and Reservoir,
North Santiam River, Willamette River Basin, Oregon.

Plates Accompanying Appendix A

Final Definite Project Report on Detroit Dam and Reservoir

North Santiam River, Willamette River Basin, Oregon

Plate I.	Cantilever Stability Analysis, Low Dam.
Plate II.	Cantilever Stability Analysis, High Dam Raised Downstream.
Plate III.	Cantilever Stability Analysis, High Dam Raised Upstream.
Plate IV.	Normal and Shear Stresses, High Dam Raised Downstream.
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Plate VI.	Stresses on Contact Plane, High Dam Raised Downstream.
Plate VII.	Stresses on Contact Plane, High Dam Raised Upstream.
Plate VIII.	Cantilever Deflections, High Dam Raised Downstream.

Tables Accompanying Appendix A

Final Definite Project Report on Detroit Dam and Reservoir

North Santiam River, Willamette River Basin, Oregon.

Table I. Resume of Cantilever Stress Conditions, showing Maximum Stresses and Minimum Safety Factors - Detroit Low Dam.

Table II. Resume of Cantilever Stress Conditions showing Maximum Stresses and Minimum Safety Factors - Detroit High Dam Raised Downstream.

Table III. Resume of Cantilever Stress Conditions showing Maximum Stresses and Minimum Safety Factors - Detroit High Dam Raised Upstream.

Table IV. Stresses in Low Dam Concrete Before and After Raising to Ultimate Height, without earthquake.

Table V. Summary of Cantilever Deflections, Detroit High Dam Raised Downstream, Normal Pool Elevation 1569, no earthquake.

Part 1 - Deflections at Elevation 1450.
Part 2 - Deflections at Elevation 1550.

I INTRODUCTION

1. General: The Detroit Dam, a contemplated concrete gravity structure, is proposed to be built in a comparatively narrow rocky canyon of the North Santiam River. The reservoir to be formed by the dam will regulate the stream flow in the joint interests of flood control, navigation, irrigation, stream purification and eventually power use. The stream control functions may be served initially by building a low dam, and later, when justified by power demand, by raising to the full height required for power development; or the dam may be built in one step to its full height.

2. Sections Adopted: In the Project Document (H.D. 544, 75th Congress, 3rd Session) an arch dam was proposed for this site, but design studies using more complete foundation data, and covering various alternate types of dams indicated that the gravity type was best adapted to the site. After this decision was reached preliminary layouts and estimates were made to determine the alignment and section best suited to foundation conditions. As a result of these studies the gravity sections were adopted which are herein made the subject of more detailed analysis. The sections shown for the "high dam" are adapted to the requirements imposed by two-step construction (raising from a "low dam"). If a high dam were to be constructed in a single step minor modifications of the sections might be necessary or desirable.

3. Dams Investigated: Governed by the above considerations, design and stress studies were made for the structures listed below. Design and stress studies were not made for the single step high dam, the analysis of which would be closely comparable to that of item "b" or "c".

a. A "low dam" for flood control purposes, height above foundation 360 feet, length about 1,110 feet, planned to form an integral part of a future high dam.

b. A "high dam" for power as well as flood control purposes, increased to a height of 448 feet and a length of 1,500 feet by raising the low dam on the downstream face.

c. A "high dam", similar to above, except raised on upstream face of low dam.

4. Purposes and Scope of Investigation: The principal purposes of the present investigations were:

a. To check the overall safety and stability of the adopted dam sections, under various loading conditions.

b. To determine the internal and foundation stresses, and internal distribution of stresses for various loading conditions; and to correlate these stresses with the indicated safe working stresses for foundation rock and concrete in the dam.

c. To investigate problems involved in raising the low dam to the future high dam, and to determine the most desirable method of raising insofar as stress conditions are concerned. This involves an analysis of the low dam before and after raising and stress studies along the contact plane between old and new concrete.

5. Included within the scope of this study were the determination of stresses, coefficients of resistance to sliding, and locations of

resultant forces at the bases and at various elevations of the sections analyzed. In addition, normal and shear stresses in both horizontal and vertical planes, principal stresses, and cantilever deflections were computed.

6. Drawings and Tables: Certain data derived from these analyses are shown on tables accompanying this text. In addition, drawings accompanying the main report show:

a. Layout and sections of each dam covered in this analysis, giving plans of construction for each of the above mentioned conditions.

b. Diagrams on Appendix A Plates, showing sections analyzed and indicating the results of various stress and stability analyses which are discussed herein.

c. Tabulations on Appendix A Plates of principal data and results of analyses.

II BASIC DESIGN DATA

7. Design Assumptions and Constants:

a. Total load on the dam is resisted by gravity action of a series of cantilevers only, with no lateral restraint.

b. Unit weight of water = 62.5 lb./cu. ft.

c. Unit weight of concrete = 150 lb./cu. ft.

d. Coefficients of internal friction for concrete and rock = 0.65. (See Paragraph 17, Main Report)

e. Earthquake shocks are considered only with reservoir water surface at or below normal pool elevation.

f. The horizontal component of an earthquake shock has an acceleration of 0.1 that of gravity, a period of vibration of 1 second, and a direction of vibration normal to the axis of the dam.

g. The vertical component of an earthquake shock has an acceleration of 0.1 that of gravity, and a period of vibration of 1 second.

h. Uplift pressure on the base or a horizontal section through a cantilever element varies from full reservoir head at the upstream face to zero at the downstream face, acting over one half of the area of the base or section. Uplift pressure is assumed to be unaffected by earthquake shock.

i. The inclined stresses parallel to the faces of the dam are equal to the vertical stresses at the face of the dam divided by the squares of the cosines of the angles between the faces and the vertical direction minus the stresses normal to the faces of the dam multiplied by the squares of the tangents of the angles between the faces and the vertical direction.

j. Tailwater pressures are neglected.

k. Normal deflections of a cantilever element are in a direction normal to the axis of the dam, and are caused by normal shear forces and normal bending moments acting on the cantilever and its foundation.

l. Modulii of elasticity assumptions:

(1) Modulus of elasticity (E_c) for concrete in direct stress = 4,000,000 lb. per square inch.

(2) Modulus of elasticity (E_r) for foundation rock in direct stress under overflow section, alternate assumptions:

(a) 4,000,000 lb. per square inch.

(b) 2,000,000 lb. per square inch.

(3) Modulus of elasticity (E_r) for foundation rock in direct stress under non-overflow sections, alternate assumptions:

(a) 4,000,000 lb. per square inch.

(b) 2,000,600 lb. per square inch.

(c) 1,000,000 lb. per square inch (right abutment only).

(4) Modulus of elasticity (G) of cantilever or foundation material in shear is equal to 1/3 of the modulus in direct stress.

m. Poisson's Ratio = 0.20 for concrete and foundation rock.

n. Minimum base elevation of overflow section is assumed to be elevation 1180. Concrete below that elevation is in the form of a plug in a hole in the foundation, and is assumed to act as a part of the foundation.

o. Deflection for each block is that for a representative cantilever element one foot thick, with base at the average elevation of the base of the block and sloping transversely at an angle with the vertical direction equal to the average angle for the block.

p. Deflections of blocks 1 to 10 and 29 to 31 inclusive are negligible and are not shown.

8. Allowable stresses and requirements:

a. Maximum allowable compression in concrete or foundation rock = 650 lb./square inch.

b. Maximum allowable tension = 0, except with earthquake shock, when a maximum of 100 lb./square inch will be allowed.

c. Shear:

(1) Maximum allowable unit stress in direct shear = 300 lb./square inch.

(2) Minimum allowable computed shear-friction factor = 4.0. The shear friction factor is defined as:

$$\frac{(\text{vert. load} - \text{uplift})(\text{coeff. of int. friction}) + (\text{base area})(\text{unit shear})}{\text{total horizontal force}}$$

and is based on a coefficient of internal friction of 0.65 and a unit shear stress of 600 lb./square inch. (See paragraph 17, Main Report)

d. The minimum computed uplift ratio with horizontal earthquake shall not be less than 0.50. The uplift ratio is defined as the ratio of the total vertical load at the upstream face at any elevation to the full water pressure at that elevation.

9. Basis for Design Assumptions and Requirements: In general the assumptions made for this stress analysis are in agreement with standard practice. Geologic studies and laboratory and field tests were the basis

for the specific assumptions as to uplift and unit stresses. Those studies, and the results of field and laboratory tests are given in Appendix **B**. Compression test results listed there indicate the adequacy of the foundation to carry all superimposed loads.

10. Sections Investigated: Analyses were made of three sections each of the low dam, high dam raised upstream and high dam raised downstream. The three sections included (1) spillway section with flood control outlets, (2) non-overflow section with needle valve outlets, and (3) non-overflow section with penstocks. For each dam and each section analyses were made for the following conditions described in paragraph 11 and paragraph 12.

III ASSUMED LOADING CONDITIONS

11. Loading Conditions, excluding earthquake:

a. Reservoir empty. The loads are dead loads or weights only. The weight of concrete displaced by outlet conduits and penstocks is distributed over a block width of 50 feet when computing the weight of a cantilever element. The weights of conduit lining, conduit control gates and trash racks were omitted, as was the weight of concrete displaced by the galleries. The weight of the spillway gates and piers for the high dam was estimated to be 36,000 lbs. per linear foot.

b. Reservoir at normal pool. The horizontal and vertical components of the water load are combined with the dead loads for this condition. The effect of hydrostatic uplift pressure on the coefficients of resistance to sliding is also included in the analysis.

c. Maximum flood conditions. This condition combines the dead loads and the components of the maximum water load on the face of the dam. The effect of hydrostatic uplift pressure is included in determining the coefficients of resistance to sliding.

12. Loading Conditions, including earthquake effects: Earthquake accelerations were assumed to act horizontally either up or downstream, and vertically either upward or downward. Loading conditions during the occurrence of the assumed maximum earthquake shock are as follows:

a. Reservoir empty. The inertia forces caused by the acceleration of the mass of the dam are combined with the loads for normal conditions of the empty reservoir.

b. Reservoir at normal pool. The inertia forces of the mass of the dam and the hydrodynamic effect of the inertia of the water against the face of the dam due to earthquake shock are combined with the dead loads and the components of the water load for normal conditions. The effect of hydrostatic uplift pressure is included in determining the coefficients of resistance to sliding. (It should be noted that, for an earthquake shock to have the maximum effect on a loaded structure, it must act to increase those loads which cause the most unfavorable stresses and safety factors. It is theoretically possible for horizontal and vertical earthquake accelerations to occur at the same time, with a resultant increase in unfavorable loadings greater than that due to acceleration in one direction only. However, in these analyses, the accelerations were applied singly and the effects of combined accelerations were not determined.)

IV. RESUME OF STRESS STUDIES

13. Low Dam, without earthquake: Analyses were made for reservoir empty, reservoir at normal pool and reservoir at maximum pool.

a. Reservoir empty. Compressive stresses exist at the upstream face of the dam and small tensile stresses exist generally at the downstream face. The maximum compression is 319 lb./square inch at the base of the spillway section and the maximum tension is 11 lb./square inch at elevation 1400 of the non-overflow section with penstock.

b. Reservoir at normal pool. The direct stresses are all compressive. The maximum stress is 290 lb./square inch parallel to the downstream face at the base of the spillway section. The maximum horizontal shear stress is 139 lb./square inch at the downstream face of the spillway section at the base. The maximum sliding factor of 0.599, minimum shear-friction factor of 8.99 and minimum uplift ratio of 1.049 all occur at the base of the spillway section.

c. Maximum flood conditions. The direct stresses are compressive for all sections. The maximum compressive stress is 375 lb./square inch and occurs at the base of the spillway section, as does the maximum horizontal shear of 180 lb./square inch. The maximum sliding factor is 0.765 at elevation 1400 of the spillway section. The minimum shear-friction factor of 7.53 occurs at the base of the spillway section. The minimum uplift ratio is 0.499 at elevation 1300 of the non-overflow section with penstock.

14. Low Dam, with earthquake: Analyses were made for reservoir empty and reservoir at normal pool.

a. Reservoir empty. The direct stresses at the upstream faces of the sections are compressive for all conditions. The maximum is 362 lb./square inch at the base of the spillway section. Tensile stresses exist at the downstream face of the dam for all earthquake conditions except when the acceleration is horizontally upstream. The maximum tensile stress is 73 lb./square inch at elevation 1200 of the non-overflow section with needle valve outlets.

b. Reservoir at normal pool. With the exception of the stresses at the downstream face of the non-overflow sections at elevations 1400 and 1450 under conditions of horizontal earthquake acceleration downstream, all direct stresses are compressive. The maximum compressive stress is 396 lb./square inch parallel to the downstream face of the spillway section at the base. The maximum tensile stress acts parallel to the downstream face of the non-overflow section with needle valve outlets at elevation 1400 and equals 4 lb./square inch. The maximum horizontal shear stress is 190 lb./square inch at the base of the spillway section. The maximum sliding factor is 0.791, the minimum shear-friction factor is 6.91, and the minimum uplift ratio is 0.500. All occur at the base of the spillway section.

15. Summary, Stress Studies of Low Dam: The maximum stresses and minimum safety factors, except for the minimum uplift ratio, occur when the effect of horizontal earthquake acceleration is combined with the normal loads on the dam (see paragraph 14 - "b"). Less severe stress conditions occur when earthquake effects are excluded. The maximum stresses and minimum safety factors under both conditions are tabulated below. For a complete tabulation of all stresses see Plate I. For tabulation of maximum stresses and minimum safety factors see Table I of this report.

Stress or Factor	Unit	With Earthquake	Without Earthquake
Foundation Pressure	Lb./sq. in.	362	319
Compression	Lb./sq. in.	396	375
Tension	Lb./sq. in.	73	11

Stress or Factor	Unit	With Earthquake	Without Earthquake
Horizontal Shear	Lb./sq. in	190	180
Max. Sliding Factor	Ratio	0.791	0.765
Min. Shear Friction Factor	Ratio	6.308	7.531
Minimum Uplift Ratio	Ratio	0.500	0.499

16. <u>High Dam Raised Downstream, without earthquake:</u> This dam is analyzed for reservoir empty, at normal pool and at maximum flood conditions.

<u>a</u>. Reservoir empty. Compressive stresses exist at the upstream face of all sections and the downstream face of the spillway section. Small tensile stresses exist generally at the downstream faces of the non-overflow sections. The maximum compressive stress of 412 lb./square inch occurs at the base of the spillway section. The maximum tensile stress is 7 lb/square inch parallel to the downstream face at elevation 1400 of the non-overflow section with penstock. Major principal stresses are equal in magnitude to those shown, and are in a vertical direction; minor principal stresses are very small, and are not shown.

<u>b</u>. Reservoir at normal pool. The direct stresses are all compressive. The maximum stress is 402 lb./square inch parallel to the downstream face of the spillway section at the base. The maximum horizontal shear stress is 196 lb./square inch at the downstream face of the spill-

way section at the base. The maximum sliding factor is 0.618 at the base of the non-overflow section with needle valve outlets. The minimum shear-friction factor is 6.96 and the minimum uplift ratio is 1.002. They both occur at the base of the spillway section. The maximum major principal stress is equal in magnitude and similar in direction to the maximum stress shown above; the maximum minor principal stress is 169 lb./square inch normal to the major principal stress at the upstream face of the dam at the base.

c. Maximum flood conditions. The direct stresses are compressive for all sections. The maximum compressive stress is 424 lb./square inch at the base of the spillway section, at which point also occurs the maximum horizontal shear stress of 207 lb./square inch. The maximum sliding factor is 0.648 at the base of the non-overflow section with needle valve outlets. The minimum shear-friction factor of 6.70 occurs at the base of the spillway section, and the minimum uplift ratio of 0.820 occurs at elevation 1250 of the spillway section.

17. High Dam Raised Downstream, with earthquake: This dam is analyzed for reservoir empty and at normal pool.

a. Reservoir empty. The direct stresses at the upstream faces of the sections are compressive for all conditions. The maximum of 464 lb./square inch is reached at the base of the spillway section. Stresses at the downstream face vary from tension to compression under the various loadings.

For the spillway section the downstream stresses are all compressive except under horizontal earthquake acceleration downstream, when tension exists. Below elevation 1500 of the non-overflow sections tension exists at the downstream face for all conditions except that of horizontal earthquake acceleration upstream. The maximum tension is 87 lb./square inch parallel to the downstream face of the non-overflow section with needle valve outlets, at the base.

b. Reservoir at normal pool. The direct stresses are all compressive, the maximum being 542 lb./square inch parallel to the downstream face at the base of the spillway section. The maximum horizontal shear stress is 264 lb./square inch at the base of the downstream face of the spillway section. The maximum sliding factor of 0.883 and the minimum shear-friction factor of 5.07 occur at the base of the non-overflow section with needle valve outlets. The minimum uplift ratio is 0.495 at the upstream face of the spillway section at the base.

18. Summary - Stress Studies of High Dam Raised Downstream: The maximum stresses and minimum safety factors occur when the effect of horizontal earthquake acceleration is combined with the normal loads on the dam. Less severe stress conditions occur when earthquake effects are excluded. The maximum stresses and minimum safety factors under both conditions are tabulated below:

STRESS OR FACTOR	UNIT	WITH EARTHQUAKE	WITHOUT EARTHQUAKE
Foundation Pressure	lb./sq. inch	464	412
Compression	"	542	412
Tension	"	87	7
Horizontal Shear	"	264	207
Max. Sliding Factor	Ratio	0.883	0.648
Min. Shear-Friction Factor	"	5.067	6.700
Min. Uplift Ratio	"	0.495	0.820

19. For a complete tabulation of all stresses see Plates II, IV and VI. For tabulation of maximum stresses and minimum safety factors see Table II of this appendix. For stress distribution within the dam see Paragraph 24; for stresses on contact plane between old and new concrete see Paragraph 25.

20. <u>High Dam Raised Upstream, without earthquake</u>: The analyses were made for reservoir empty, at normal pool and at maximum flood conditions.

<u>a</u>. Reservoir empty. The direct stresses are all compressive except at the downstream faces of the non-overflow sections where small tensile stresses occur at elevations 1450 and 1500. The maximum compressive stress of 364 lb./square inch acts parallel to the upstream face at the base of the spillway section. The maximum tensile stress of 3 lb./square inch acts parallel to the downstream face at elevation 1450 of the non-overflow sections. Major principal stresses

are equal in magnitude to the values already shown, and are in a vertical direction; minor principal stresses are very small, and are not shown.

b. Reservoir at normal pool. The direct stresses are all compressive. The maximum stress is 404 lb./square inch parallel to the downstream face of the spillway section at the base. The maximum horizontal shear stress is 196 lb./square inch at the downstream face of the spillway section at the base. The maximum sliding factor of 0.588 and the minimum uplift ratio of 0.967 occur at the base of the non-overflow section with needle valve outlets. The minimum shear-friction factor is 7.20 at the base of the spillway section. The maximum major principal stress is equal in magnitude and similar in direction to the maximum stress shown above; the maximum minor principal stress is 165 lb./square inch normal to the major principal stress at the upstream face of the dam at the base.

c. Maximum flood conditions. The direct stresses are compressive for all sections. The maximum compressive stress is 422 lb./square inch parallel to the downstream face of the spillway section at the base. The maximum horizontal shear stress is 203 lb./square inch at the downstream face of the spillway section at the base. The maximum sliding factor is 0.645 and the minimum uplift ratio is 0.829. Both occur at elevation 1500 of the spillway section. The minimum shear-friction factor is 6.93 at the base of the spillway section.

21. High Dam Raised Upstream, with earthquake: This analysis was made for reservoir empty and reservoir at normal pool.

a. Reservoir empty. The direct stresses at the upstream faces of the sections are compressive for all conditions. The maximum is 413 lb./square inch parallel to the upstream face of the spillway section at the base. Stresses at the downstream faces of the three sections vary from tension to compression under the various loading conditions. The maximum tension of 34 lb./square inch acts parallel to the downstream face at elevation 1450 of the non-overflow sections.

b. Reservoir at normal pool. The direct stresses are all compressive. The maximum compressive stress is 530 lb./square inch parallel to the downstream face of the spillway section at the base. The maximum horizontal shear stress is 254 lb./square inch at the downstream face of the spillway section at the base. The maximum sliding factor is 0.784 at elevation 1450 of the spillway section. The minimum uplift ratio of 0.481 occurs at the base of the non-overflow section with needle valve outlets. The minimum shear-friction factor of 5.45 occurs at the base of the spillway section.

22. Summary - Stress Studies High Dam Raised Upstream: The maximum stresses and minimum safety factors occur when the effect of horizontal earthquake acceleration is combined with normal loads on the dam. Less severe stress conditions occur when

earthquake effects are excluded. The maximum stresses and minimum safety factors under both conditions are tabulated below:

STRESS OR FACTOR	UNIT	WITH EARTHQUAKE	WITHOUT EARTHQUAKE
Foundation Pressure	lb./sq. inch	409	361
Compression	"	530	404
Tension	"	34	3
Horizontal Shear	"	254	203
Max. Sliding Factor	Ratio	0.784	0.645
Min. Shear-Friction Factor	"	5.450	6.030
Min. Uplift Ratio	"	0.483	0.829

23. For a complete tabulation of stresses see Plates III, V and VII; for tabulation of maximum stresses and minimum safety factors see Table III of this appendix. For distribution of stresses within the dam, and principal stresses, see Paragraph 24. For stresses along contact plane between old and new concrete, see Paragraph 25.

24. Stress Distribution and Principal Stresses for High Dam Raised Upstream or Downstream: Stresses are uniformly distributed at any elevation within the dam between the values shown for the upstream and downstream faces. Major principal stresses are in general normal to horizontal planes when the reservoir is empty; and for reservoir at normal pool are roughly parallel to the downstream slope, except adjacent to the upstream face, where they become parallel to the upstream slope. Minor principal stresses are normal in direction to the major principal

stresses, and vary from a maximum at the upstream face to a minimum at the downstream face for reservoir at normal pool; for reservoir empty these stresses are very small, and are not shown or discussed.

25. Stresses on Contact Plane: Stresses on the contact plane between old and new concrete were determined with reservoir empty and reservoir at normal pool, without earthquake, for the overflow and non-overflow sections of the following dams:

a. High dam raised downstream. Normal stresses on the contact plane are all compressive, and vary from a maximum of 49 lb./square inch at the base of the non-overflow section with reservoir at normal pool to a minimum of 14 lb./square inch at elevation 1450 of the overflow section with reservoir empty. Shear stresses on the contact plane vary from a maximum positive value of 57 lb./square inch at the base of both sections, with reservoir empty, to a maximum negative value of 6 lb/square inch at elevation 1450 of the non-overflow section, with reservoir at normal pool.

b. High dam raised upstream. Normal stresses on the contact plane are all compressive, and vary from a maximum of 163 lb./square inch at the base of the non-overflow section, with reservoir at normal pool, to a minimum of 1 lb./square inch at elevation 1450 of the overflow section, with reservoir empty. Shear stresses on the contact plane vary from a maximum positive value of 62 lb./square inch at the base of both sections, with reservoir at normal pool, to a maximum negative value of 13 lb./square inch at the base of the overflow section, with reservoir empty.

26. Comparison, stresses on contact planes, for High Dam Raised Downstream and High Dam Raised Upstream: Compressive stresses on the contact plane, if within safe limits, are favorable, since they tend toward monolithic action of the old and new concrete. In this respect it should be noted that for the high dam raised upstream compressive stresses on the contact plane are higher at normal pool elevation than for the high dam raised downstream; also they tend to increase more rapidly as the pool water surface rises. Shear stresses on the contact plane are increased (but well within safe limits) for the high dam raised upstream, by increase in pool elevation; for the high dam raised downstream similar changes cause decreased or negative shear stresses on the contact plane. For a tabulation of computed stresses on the contact plane see Plates VI and VII; for a tabulation of stresses in the low dam concrete before and after raising to the high dam see Table IV.

V CANTILEVER DEFLECTIONS

27. Assumed Loading Conditions:

a. Horizontal loading consists of horizontal water pressure only. Reservoir water surface is assumed to be at normal pool elevation 1569. Tailwater is neglected.

b. Vertical loading, including weight of concrete, gates, piers, etc. is not considered in this analysis.

c. Space occupied by conduits, galleries, etc. is not considered to reduce the area of the section at any elevation.

d. Earthquake acceleration in either the horizontal or vertical direction is not considered in this analysis.

e. No other loads were considered in this analysis.

28. Explanation of Analysis: Normal deflections of a cantilever element are caused by normal shear forces and normal bending moments, singly or in combination. These deflections may be in an upstream or downstream direction, varying in direction and amount with the applied loads. (For detailed discussion of method of analysis, see Bureau of Reclamation Technical Memoranda No. 436 and 441).

29. The normal deflection caused by normal bending moments at any horizontal section is equal to the integral, from the base to that section, of the slope of the centerline. The slope of the centerline at any horizontal section is equal to the angular rotation, or tilting, of the foundation plus the integral, from the base to that section, of the curvature of the centerline. The curvature of a centerline of differential height is $\frac{M}{E_c I}$, (see notation).

30. The normal deflection caused by normal shear forces at any horizontal section is equal to the foundation deformation plus the integral, from the base to that section, of the detrusion, (change in horizontal position) of the centerline. The detrusion of a centerline, of differential height is equal to $\frac{VK}{AG}$, (see notation).

31. Tangential deflections and angular movements of the cantilever elements were not investigated. The calculations were performed by summation and tabulation of the quantities noted above, at intervals from the base of the dam to the horizontal section under consideration.

32. Notation: The following notation pertains to the above text:

M = Bending Moment.

I = Moment of Inertia of section.

E_c = Modulus of elasticity of concrete in direct stress.

E_r = Modulus of elasticity of rock in direct stress.

G = Modulus of elasticity of rock or concrete in shear.

V = Normal shear force on section.

K = (Detrusion due to actual shear distribution) divided by (Detrusion due to equivalent shear distributed uniformly).

A = Area of section = base width T.

33. Resume of Deflection Studies: Deflections normal to the axis of the dam were determined for representative cantilever elements in all blocks from 11 to 28 inclusive, with assumed loading conditions as already noted. All deflections include the movement of the base of the cantilever due to the yielding of the foundation under the applied load. These deflections do not include the deflection in an upstream direction due to concrete weight or any other vertical load.

34. In view of the results of geologic studies, covered fully in Appendix B of this report, it was considered desirable to determine the probable deflections with various values of E_r for the foundation rock, especially in the right abutment section, where test results showed the largest variation of the modulii, and where the maximum computed differential deflections occur between the overflow and non-overflow sections when E_r is a constant for the whole foundation area.

35. The maximum computed deflection at elevation 1550, which is approximately the elevation of the spillway crest, is 0.1121 feet measured in a downstream direction from the axis, and occurs in Block 19.

36. The maximum computed differential deflection at elevation 1550 is 0.0329 feet and occurs between Blocks 22 and 23, which is the point of contact of the overflow section with the right abutment non-overflow section. When a modulus of elasticity of 1,000,000 lb./square inch is assumed for the right abutment foundation the maximum computed differential deflection is 0.0303 feet and occurs between Blocks 14 and 15, which are the two left abutment non-overflow blocks nearest to the overflow section.

37. A graphical representation of the computed cantilever deflections is shown on Plate VIII, Appendix A, and a tabulation of the same deflection is given in Table 5, Appendix A. Part 1 of Table 5 is a tabulation of deflection at elevation 1450 and Part 2 of Table 5 is a tabulation of deflection at elevation 1550 for all blocks considered in the analysis.

38. Results of the analysis indicate that no excessive or unusual total deflections or differential deflections occur in the dam under the loading conditions considered, and that the final design of the dam to allow for these deflections will present no serious problems. Loading conditions including earthquake may be expected to give higher computed values of deflection. However, such deflections would be of momentary duration, and the effect on the dam is not considered of practical importance.

VI DISCUSSION

39. Stress Studies: The analyses of the low dam, and the high dam raised either upstream or downstream indicate that the sections adopted are safe. With unimportant exceptions the stresses and safety factors are within design limits for all loading conditions considered, the exception being for loading conditions of short duration (earthquake) where design limits for uplift ratio are exceeded slightly. The loading conditions which produce the most critical stresses or stability factors are as follows:

a. Reservoir at normal pool, horizontal earthquake acceleration upstream. This condition results in maximum compressive stress, maximum horizontal shear stress, maximum sliding factor, minimum shear-friction factor, and minimum uplift ratio (except for the low dam, when minimum uplift ratio occurs under maximum flood conditions).

b. Reservoir empty, horizontal earthquake acceleration downstream. This condition results in maximum tensile stress for all sections under consideration.

40. The maximum compressive stress is 542 lb./square inch, compared to the design limit of 650 lb./square inch, and occurs parallel to the downstream face at the base of the overflow section for the high dam raised downstream. The maximum tensile stress is 87 lb./square inch, (design limit 100 lb./square inch), parallel to the downstream face at the base of the non-overflow section with needle valve outlets for the high dam raised downstream. The maximum horizontal shear stress is 264 lb./square inch, (design limit 300 lb./square inch), at the point

where the maximum compressive stress occurs. The maximum sliding factor is 0.883 at the base of the non-overflow section for the high dam raised downstream. This is considered of less importance than the relatively favorable shear-friction factor, in view of the nature of the foundation structure (see Appendix B). The minimum shear-friction factor is 5.067, (design limit 4.0), occuring in the same location as the maximum sliding factor. The minimum uplift ratio is 0.483, (design limit 0.50), at the upstream face of the non-overflow section for the high dam raised upstream. Maximum stresses and minimum safety factors for each of the dams are described under the corresponding heading in Section II of this report, and are summarized in Table I to III, inclusive, of this report.

41. The points of application of the resultant forces on the bases and horizontal sections are within the middle third for most conditions of loading, and are well within the base of the dam for all conditions of loading. All tensile stresses are in portions of the dam not subject to external water pressure.

42. Internal stress studies indicate that the most unfavorable stresses occur at either the upstream face or the downstream face of the cantilever sections. It is unnecessary, therefore, to discuss these stresses further, since they are thoroughly covered in the gravity analyses.

43. <u>Stresses on Contact Planes</u>: In the cases of a high dam built in two steps, the normal stresses on the contact plane between old and new concrete are compressive for all loading conditions considered. The maximum compressive stress is 163 lb./square inch; the minimum compressive stress is 1 lb./square inch. Similarly, shear stresses on the contact plane vary from a maximum positive value of 62 lb./square inch to a maximum negative value of 13 lb./square inch.

44. For either dam, direct stresses on the contact plane are compressive, and all stresses including shear are held within safe limits. From this standpoint, then, a high dam raised either upstream or downstream may be considered practicable.

45. Cantilever Deflections: The maximum computed deflection at elevation 1550, which is approximately the crest elevation of the overflow section of the high dam, occurs in the overflow section and is 0.1121 feet in a downstream direction. The maximum computed differential deflection at elevation 1550 is 0.0329 feet at the point of contact between the overflow section and the right abutment non-overflow section. It should be noted that these deflections do not include the deflection in an upstream direction due to the weight of the dam, nor do they include the effects of earthquake shock (see loading conditions, Chapter IV).

46. Conclusions: From these analyses it may be concluded that for the low dam, and for the high dam, raised either upstream or downstream, the sections shown in the main report on Plates II, VII, VIII, and IX are safe under all loading conditions considered.

47. Recommendations for further studies: Subsequent to the completion of these design studies there has been a great increase in power demand throughout the Pacific Northwest. If Detroit Dam should be constructed in the near future as a power project, it would involve the building of the dam in one step to its ultimate height. As previously stated, the studies described in this report do not specifically cover the case of a "high dam" built in one step, but the studies and data presented for the "high dam raised upstream" are generally applicable.

48. In this connection it would appear desirable to make the following additional studies:

a. Preliminary studies to determine the most economical section that will be safe for a high dam built to the full height in one step.

b. Final analyses of the section indicated by the above studies. Since time may be limited, and because it is believed that this new section will not differ greatly from the section studied for the "high dam raised upstream", a complete analysis, while desirable, may not be essential. It is therefore recommended that further studies be primarily of those features and stresses which the present analysis has shown to be critical.

c. Extension of previous studies to include investigation of stresses as affected by the interaction of the blocks and abutments of the dam. This would involve a beam analysis of the dam, including consideration of twist effects.

TABLE I

RESUME OF CANTILEVER STRESS CONDITIONS

SHOWING MAXIMUM STRESSES AND MINIMUM SAFETY FACTORS

DETROIT LOG DAM

Loading Conditions	Maximum Foundation Pressure lb./sq. in.	Maximum Concrete Stress: Comp. lb. sq. in.	Tension lb. sq. in.	Shear lb. sq. in.	Minimum Uplift Ratio (a)	Maximum Sliding Factor	Minimum Shear Friction Factor (b)
A. Without Earthquake							
1. Reservoir Empty	319	319	-11	9	∞	0	∞
2. Normal Pool Elevation 1569	186	290	None	139	1.049	0.599	8.991
3. Maximum Pool Elevation 1577	240	375	None	180	0.499	0.765	7.531
B. With Earthquake - Reservoir Empty							
1. Horizontal Acceleration Upstream	276	276	None	30	∞	-	-
2. Horizontal Acceleration Downstream	362	362	-73	-35	∞	-	-
3. Vertical Acc. Upward	351	351	-10	10	∞	-	-
4. Vertical Acc. Downward	287	287	-12	8	∞	-	-
C. With Earthquake - Normal Pool							
1. Horizontal Acc. Upstream	254	396	None	190	0.500	0.791	6.808
2. Horizontal Acc. Downstream	196	196	-4	88	1.595	0.407	13.234
3. Vertical Acc. Upward	205	319	None	153	1.153	0.585	8.270
4. Vertical Acc. Downward	167	261	None	125	0.943	0.616	9.790

(a) Ratio vertical pressure at upstream face to horizontal water pressure.

(b) Based upon 600 lb./sq. in. + 0.65 vertical load.

TABLE II

RESUME OF CANTILEVER STRESS CONDITIONS

SHOWING MAXIMUM STRESSES AND MINIMUM SAFETY FACTORS

DETROIT HIGH DAM RAISED DOWNSTREAM

Loading Conditions	Maximum Foundation Pressure lb./sq. in.	Maximum Concrete Stress: Comp. lb./sq. inch	Maximum Concrete Stress: Tension lb./sq. inch	Maximum Concrete Stress: Shear lb./sq. inch	Minimum Uplift Ratio (a)	Maximum Sliding Factor	Minimum Shear Friction Factor (b)
A. Without Earthquake							
1. Reservoir Empty	412	412	-7	11	∞	0	∞
2. Normal Pool Elevation 1569	245	402	None	196	1.002	0.618	6.960
3. Maximum Pool Elev. 1577	258	424	None	207	0.820	0.648	5.700
B. With Earthquake-Reservoir Empty							
1. Horizontal Acceleration - Upstream	360	360	None	42	∞	-	-
2. Horizontal Acceleration - Downstream	464	464	-87	-42	∞	-	-
3. Vertical Acc. - Upward	453	453	-6	-3	∞	-	-
4. Vertical Acc. - Downward	371	371	-8	-3	∞	-	-
C. With Earthquake-Normal Pool							
1. Horizontal Acc.-Upstream	330	542	None	264	0.495	0.883	5.067
2. Horizontal Acc.-Downstream	254	362	None	128	1.503	0.420	10.216
3. Vertical Acc.-Upward	270	442	None	216	1.100	0.604	6.400
4. Vertical Acc.-Downward	220	362	None	176	0.900	0.636	7.620

(a) Ratio of vertical pressure at upstream face to horizontal water pressure.

(b) Based upon 600 lb./sq. inch ≁ 0.65 vertical load.

TABLE III

RESUME OF CANTILEVER STRESS CONDITIONS

SHOWING MAXIMUM STRESSES AND MINIMUM SAFETY FACTORS

DETROIT HIGH DAM RAISED UPSTREAM

Loading Conditions	Maximum Foundation Pressure Lb./sq. in.	Maximum Concrete Stress Comp. lb./sq. inch	Maximum Concrete Stress Tension lb./sq. inch	Maximum Concrete Stress Shear lb./sq. inch	Minimum Uplift Ratio (a)	Maximum Sliding Factor	Minimum Shear Friction Factor (b)
A. Without Earthquake							
1. Reservoir Empty	361	364	-3	36	∞	0	∞
2. Normal Pool Elevation 1569	259	404	0	196	0.967	0.588	7.196
3. Maximum Pool Elev. 1577	270	422	0	203	0.829	0.645	6.930
B. With Earthquake - Reservoir Empty							
1. Horizontal Acceleration - Upstream	313	315	0	69	∞	-	-
2. Horizontal Acceleration - Downstream	409	413	-34	41	∞	-	-
3. Vertical Acc. Upward	397	400	-3	40	∞	-	-
4. Vertical Acc. Downward	325	328	-3	32	∞	-	-
C. With Earthquake - Normal Pool							
1. Horizontal Acc. Upstream	339	530	None	254	0.483	0.784	5.448
2. Horizontal Acc. Downstream	245	278	None	134	1.450	0.401	10.593
3. Vertical Acc. Upward	285	444	None	213	1.069	0.522	7.350
4. Vertical Acc. Downward	233	364	None	175	0.869	0.672	7.060

(a) Ratio vertical pressure at upstream face to horizontal water pressure.

(b) Based upon 600 lb./sq. inch ≠ 0.65 vertical load.

TABLE IV

STRESSES IN LOW DAM CONCRETE BEFORE AND AFTER RAISING TO ULTIMATE HEIGHT
NO EARTHQUAKE

	Overflow Section Base at Elev. 1180						Non-Overflow Section Base at Elev. 1200						Non-Overflow Section Base at Elev. 1450					
	Vertical Stress		1st Prin. Stress		Shear Stress		Vertical Stress		1st Prin. Stress		Shear Stress		Vertical Stress		1st Prin. Stress		Shear Stress	
	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.	U.S.	D.S
Reservoir Empty																		
Low Dam	319	-3	319	-4	0	-2	302	-6	302	-9	0	-4	56	12	56	18	0	8
Raised Down Stream	412	118	412	118	0	57	397	118	397	118	0	0	143	103	143	103	0	0
Raised Up Stream	264	45	265	70	-13	33	247	39	250	60	-10	29	108	-2	108	-3	0	-1
Reservoir Full																		
Low Dam	129	186	129	290	0	139	124	172	124	268	0	129	54	13	54	21	0	10
Raised Down Stream	169	223	169	341	0	144	162	215	162	324	0	135	77	73	77	84	0	18
Raised Up Stream	194	259	242	404	62	194	185	247	233	386	62	185	71	71	82	111	18	53

TABLE V

PART 1

SUMMARY OF CANTILEVER DEFLECTIONS AT ELEVATION 1450

DETROIT HIGH DAM RAISED DOWNSTREAM

NORMAL POOL ELEVATION 1569, NO EARTHQUAKE

Block Number	Angle of Found. with Vert.	Elevation Base of Block	DEFLECTION IN FEET FOR VARIOUS ASSUMED MODULII: NON-OVERFLOW SECTION E_c=4,000,000 E_r=4,000,000	NON-OVERFLOW SECTION E_c=4,000,000 E_r=2,000,000	NON-OVERFLOW SECTION E_c=4,000,000 E_r= *	OVERFLOW SECTION E_c=4,000,000 E_r=4,000,000	OVERFLOW SECTION E_c=4,000,000 E_r=2,000,000
11	60°	1350	0.01269	0.01964	0.01964		
12	62°	1335	0.01568	0.02404	0.02404		
13	62°	1300	0.02265	0.03430	0.03430		
14	33°	1250	0.02912	0.04039	0.04039		
15	90°	1230	0.04318	0.06612	0.06612		
16	68°	1215				0.04590	0.06877
17	90°	1200				0.05212	0.07839
18	53°	1180				0.05237	0.07510
19	90°	1180				0.05956	0.08946
20	53°	1180				0.05237	0.07510
21	70°	1200				0.05112	0.07639
22	68°	1215				0.04590	0.06877
23	33°	1250	0.02912	0.04039	0.06289		
24	33°	1300	0.01852	0.02605	0.04110		
25	60°	1350	0.01269	0.01964	0.03352		
26	67°	1377	0.00883	0.01407	0.02456		
27	50°	1400	0.00526	0.00844	0.01478		
28	67°	1437	0.00226	0.00412	0.00780		

* Blocks 11 to 15 inclusive, E_r=2,000,000

Blocks 23 to 28 inclusive, E_r=1,000,000

TABLE V

PART 2

SUMMARY OF CANTILEVER DEFLECTIONS AT ELEVATION 1550.0

DETROIT HIGH DAM RAISED DOWNSTREAM

NORMAL POOL ELEVATION 1569, NO EARTHQUAKE

Block Number	Angle of Found. With Vert.	Elevation Base of Block	DEFLECTION IN FEET FOR VARIOUS ASSUMED MODULII				
			NON-OVERFLOW SECTION			OVERFLOW SECTION	
			E_c=4,000,000 E_r=4,000,000	E_c=4,000,000 E_r=2,000,000	E_c=4,000,000 E_r= *	E_c=4,000,000 E_r=4,000,000	E_c=4,000,000 E_r=2,000,000
11	60°	1350	0.02158	0.03119	0.03119		
12	62°	1335	0.02533	0.03657	0.03657		
13	62°	1300	0.03409	0.04911	0.04911		
14	33°	1250	0.04182	0.05587	0.05587		
15	90°	1230	0.05853	0.08613	0.08613		
16	68°	1215				0.06130	0.08874
17	90°	1200				0.06829	0.09952
18	53°	1180				0.06853	0.09476
19	90°	1180				0.07688	0.11214
20	53°	1180				0.06853	0.09476
21	70°	1200				0.06722	0.09737
22	68°	1215				0.06130	0.08874
23	33°	1250	0.04182	0.05587	0.08391		
24	33°	1300	0.02880	0.03872	0.05840		
25	60°	1350	0.02158	0.03119	0.05040		
26	67°	1377	0.01649	0.02412	0.03940		
27	50°	1400	0.01211	0.01655	0.02655		
28	67°	1437	0.00702	0.01046	0.01703		

* Blocks 11 to 15 inclusive, E_r= 2,000,000

Blocks 23 to 28 inclusive, E_r= 1,000,000

WAR DEPARTMENT
U. S. ENGINEER OFFICE
PORTLAND DISTRICT
PORTLAND, OREGON

LLOYD L. RUFF

APPENDIX B TO ACCOMPANY DEFINITE PROJECT REPORT ON
DETROIT DAM AND RESERVOIR
NORTH SANTIAM RIVER, OREGON

GEOLOGY AND FOUNDATION DATA

AUGUST 9, 1941

APPENDIX B

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APPENDIX B - GEOLOGY AND FOUNDATION DATA

DETROIT DAM SITE - NORTH SANTIAM RIVER, OREGON

CHAPTER I - INTRODUCTION

I - INTRODUCTION

1. <u>General</u>. A brief outline of the geology of the Detroit Dam Site has been presented in Chapter II of the Definite Project Report. Additional geologic and foundation data, including reports of the consulting geologist, are included in this appendix. A discussion of concrete aggregate is also presented.

2. <u>Exploration Work Completed</u>. First studies of the area by the Corps of Engineers were begun in 1935, and the geology was covered by W. E. McKitrick and A. M. Piper in preparation of the project report of 1937. At this time the preliminary exploration consisted of two EX size diamond drill holes, one tunnel, and several test pits and trenches. Exploration of the site was resumed in September, 1938 and shortly thereafter was placed under the supervision of the Planning Section of the Willamette Basin Project. A total of 4070 feet of drilling was done under two successive contracts between September, 1938, and March, 1939. Slightly over 1000 feet of the total footage were in overburden. The diamond drilling was almost entirely BX size (1-5/8" core) for each of the two contracts.

3. The latest phase of exploration was carried out between May 15 and October 1, 1940, and included: five 36" calyx holes totalling 283 feet, 460 feet of tunnels and drifts, 250 linear feet of trenches, and 850 feet of diamond drill holes. The drill holes were NX in size (2-1/8" core) except for 137 feet of six inch holes, drilled for the purpose of obtaining large diameter (4-3/4" cores) for laboratory testing. For the logs of the diamond drill and calyx holes, trenches, and tunnels, see Plate V, Sheets 1 to 5, of this appendix. Plan of the ex-

ploration is shown on Plate VI of the project report.

4. Additional Exploration Work Recommended. The foundation area for the low dam has been well explored, as has most of the area for the high dam. Since DH - 54 further outlined the gravel filled gully on the right abutment but did not reach bedrock, it might be advisable to drill one or two additional holes in that vicinity. The gravels, however, are highly impervious, and the gully is not believed to continue across the abutment at an elevation low enough to affect the low dam. Additional exploration for the penstocks and powerhouse foundations will be necessary before design and construction.

APPENDIX B - GEOLOGY AND FOUNDATION DATA

DETROIT DAM SITE - NORTH SANTIAM RIVER, OREGON

CHAPTER II - GEOLOGY

II - GEOLOGY

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Chart I. Trend of Surface Joints and Fractures.

TABLE TO ACCOMPANY APPENDIX B - CHAPTER II

Table I. Mineralogical Composition of Foundation Rocks

GEOLOGY

I - GENERAL

1. This report has been compiled for the purpose of bringing together the existent geologic data on the dam site and reservoir and making interpretations and recommendations to assist in the plans, estimates and design of Detroit Dam. A complete list of reports, memoranda, files, and communications from which these data are drawn is contained in Part XII of this chapter.

2. Location: Detroit Dam Site is located on the North Santiam River in the S. 1/2 of the NW 1/4 of Sec. 7, T. 10S., R. 5 E., Willamette Meridian, about .6 of a mile inside the boundary of the Santiam National Forest. It is 13.6 miles by road east of Mill City and 6.2 miles west of Detroit, on the county line between Marion and Linn Counties, Oregon.

3. Name: The site was formerly known as the Boundary site and is so listed in some of the earlier reports (c, d, i). (Letters refer to bibliography.) Helland (c) describes a site 1.3 miles east of the town of Detroit under the heading of Detroit Dam Site.. In the 1937 project report the name Boundary was changed to Detroit Dam Site and has since been known as such.

4. Surveys and Exploration: Profile surveys of the North Santiam were made in 1913 by the U.S.G.S. and were published in Water Supply Paper 349 (m). No reports on dam sites were included. In 1928 Mr. R. O. Helland (c), Assistant Engineer of the U.S.G.S., surveyed nine sites on the North Santiam. The Boundary site is included in the

geologic reconnaissance.

5. First studies of the area by the U. S. Engineer Department were begun in 1935, the geology being covered by McKitrick (d) and Piper (i). At this time the preliminary exploration consisted of two EX size diamond drill holes, one tunnel, and several test pits and trenches.

6. Exploration of the site was resumed in September, 1938 and soon after placed under the direction of the Planning Section of Willamette Valley Project. A total of 4070 feet of drilling was done under two successive contracts between September, 1938 and March, 1939. Slightly over 1000 feet of the total footage was in overburden; this necessitated the driving of casing. The diamond drilling was almost entirely BX size (1-5/8" core) for each of the two contracts.

7. The latest phase of exploration was carried out between May 15 and October 1, 1940, and includes: five 36-inch calyx holes totalling 283 feet, 460 feet of tunnels and drifts, 250 feet (approx.) of trenches, and 850 feet of diamond drill holes, 137 feet of which were 6 inches in diameter. This program was carried out under the direct supervision of the Planning Section with Mr. C. C. Mongold, Assistant Geologist, in charge of field operations. Details of the complete exploration are found on Plates I to VI of this Appendix.

8. General Features: Detroit Dam was first planned as a concrete arch type of structure. The first extensive drilling program (1938-39) was laid out on that assumption. Because of adverse bedrock topography, increased cost estimates, and various engineering difficulties, it became evident that only a concrete gravity or rock fill structure

could meet the problems encountered. Plans for a rock fill dam were immediately discarded. The structure now proposed is of concrete gravity type with two stage construction. A low dam to elevation 1489 (elevations are in feet above Mean Sea Level - U.S.C. & G.S. Datum - 1929) will be later embodied in a high dam to elevation of 1577.5 or 375 feet above the valley floor and 450 feet ± above the lowest point of the foundation. The Detroit site is the farthest east of the several possible sites, with favorable cross-section, in the N. Santiam Canyon. The Valley widens out about three miles upstream into the Detroit Basin and reservoir site.

II PHYSIOGRAPHY

9. The Cascade Mountains of Oregon are made up of two distinctive physiographic units. Thayer (1) has the following to say about the general relationship features of the Cascade Mountains. "I. C. Russell (k), in 1897, appears to have been the first to suspect that the Cascades were formed of two or more series of volcanic rocks, some of which were folded, and some of which were not folded. The accuracy of Russell's observations has been fully borne out by later geologists ---, and in 1933 Callaghan (a) suggested the terms Western Cascade and High Cascade for the older and younger volcanic series, respectively. These names have been accepted for the Oregon Cascades to differentiate the folded Miocene and older Tertiary volcanic rocks from the unfolded younger (Pliocene to Recent?) volcanic rocks, ---. The terms originate in the fact that in Oregon, between the North Santiam River west of Mt. Jefferson and the Rogue River southwest of Crater Lake, the Cascade

Range is made up of two distinct north-south ranges. In general, the Western Cascade Range is characterized by mature topography, with narrow valleys and sharp ridges which bear little obvious relation to bedrock structures. In the High Cascade Range the topography is controlled almost everywhere by the structure of the lavas."

10. The Detroit Dam Site is located in the Western Cascade Province, some 15 miles from its eastern border. While the North Santiam River Valley generally may be mature, the gorge section from Gates to Blowout Creek is little beyond the youthful stage at any point. A limited number of terraces and benches are to be seen, but are relatively narrow and indistinct.

III GEOLOGIC HISTORY

11. The geologic history of the Willamette Basin and adjacent areas of northwest Oregon dates from Tertiary to Recent. Marine formations predominate in the Coast Range and beneath the fill of the Willamette Valley, while lavas and fragmental volcanic rocks comprise the greater part of the Western Cascades. The more recent High Cascades are mostly andesitic and basaltic lavas.

12. Oligocene: The oldest rocks in the North Santiam Area, according to Thayer (1) and others, are Oligocene or later in age. Shallow Oligocene seas washed a shore line which coincided closely with the present eastern margin of the Willamette Valley. Volcanic activity is believed to have been extensive, because of the large amount of tuffaceous material deposited in the Eugene, Illahee, Brownsville and other marine formations. Beyond the margin of the sea, thick deposits

of tuff, agglomerate, and breccia were deposited. Today these rocks are included in the Mehama Volcanics (1) and Breitenbush Tuffs, and may even be related to the famous John Day beds of Central Oregon.

13. Toward the close of the Oligocene these formations were gently folded (at least in the lower Santiam), and subsequently eroded before the outpouring of the miocene Stayton basalts (h, 1).

14. Miocene: The Stayton Basalts are correlated with the more extensive Miocene Columbia River Basalts of the Pacific Northwest. Contemporaneous with or possibly in part following the Stayton Lavas, the extensive Sardine Lavas were extruded. Thayer (1) has the following to say about the Miocene history of the Santiam Area; "After an erosion interval the Stayton Lavas were erupted in Miocene time along the western margin of the range; within the range volcanism obscured the record and no break is apparent between the Breitenbush Tuffs and the Sardine Lavas. The Miocene volcanic activity was less violent than the earlier phases, and the Stayton and Sardine Lavas were extruded, probably from fissures, at the same time the region to the east was flooded by the Columbia River lavas. The Sardine Lavas accumulated to a depth of at least 6,000 feet before eruptions ceased. The Sardine Lavas thinned rapidly eastward and westward and their erosional products were deposited to the west ---". "Toward the end of Miocene time the volcanic activity died down, and the region was again folded. The thick massive lavas bent only slightly, but the weaker tuffs were buckled into the Breitenbush anticline. Intrusion of diorite porphyry probably closely followed the folding, and the range was subjected to a long erosion period which

probably was marked by a series of uplifts. By the time the first High Cascade lavas were erupted, the Western Cascade Range had attained essentially its present elevation and was maturely eroded."

15. Pliocene: Subsequent history of the Santiam area in the vicinity of the damsite was probably more or less continuous erosion down to the present time. O'Neill (h) tentatively assigns tuff deposits overlying the Stayton basalts to the Pliocene. Undoubtedly there were some sporadic outbursts of volcanic activity in the Western Cascades after the main activity during the Miocene.

16. Pleistocene: The dam area and gorge below were further modified by one and possibly two stages of glaciation. Thayer (l) finds definite evidence of glaciation as far west as Mill City, and concludes that at least one other stage reached the Detroit Basin. Cross sections at several of the damsites investigated in connection with the Detroit Dam are very suggestive of an old glacial valley from 150 to 200 feet above the present stream. If this is due to glacial action it must be younger than the Mill City stage of glaciation.

17. The more rapid erosion of the fragmental lower Sardine and Breitenbush formations gives rise to the Detroit Basin, much of which is to be occupied by the Detroit reservoir. Glacial till, glacial outwash, and Pleistocene and Recent stream gravels are abundant in the reservoir area. With these sedimentary deposits in the Detroit Basin and a sharp youthful valley below the damsite it is not improbable that there have been earth movements in the gorge area as recently as late Pleistocene.

18. Recent: Recent history of the dam site area has been chiefly one of continued downcutting in the gorge area. The valley sides have remained steep and many of the terrace and outwash gravels have been wholly or partially removed.

IV AREAL GEOLOGY

19 Units Mapped: Formations at the damsite, as shown on the maps (Foundation Exploration-Plan, Areal Geology), are divided first into overburden and rock outcrops. On the Areal Geology sheet the rock outcrops are further divided into the three major rock formations represented at the site. Further differentiation of the individual formations is made in the logs of drill holes, etc.

20 Rock Formations: The oldest and by far the most extensive rocks in the damsite area are the andesitic lavas of the Sardine Series. These are somewhat varied in texture and appearance, but are mapped in the field as the one unit andesite (an).

21. A 170 foot wide diorite dike (di) trends N. 4° W. (bearings, etc., are all given in reference to true N.) diagonally across the stream below the dam axis. It is no doubt related to the Halls diorite porphyry which Thayer (1) has mapped a mile or so upstream from the damsite. Additional diorite has been noted in the drill holes but either does not show or is too small to be mapped on the surface. Other outcrops of diorite occur in the side streams above the damsite and outside the area of detailed mapping.

22. The youngest unit of the foundation rocks consists of a series of andesite porphyry (anp) dikes cutting both andesite and diorite, and running in a more westerly direction than the diorite dike.

23. Overburden: Unconsolidated deposits are somewhat varied in origin and occurence, and date back well into the Pleistocene. Gravel and boulder deposits have been recognized up to elevation 1525 in DH 54. The high elevation gravel is believed to be continuous on the right abutment from road level (Elev. 1350) through Tunnel C up to the bottom of DH 54 and probably not only fill the buried ravine (see Rock Contour Map), but also cover the small bench above elevation 1500. A few rounded gravels and cobbles have also been noted on the left abutment at about elevation 1340, above the top of the rock cliff. These are quite possibly the same age as the high elevation gravels on the right abutment and may be quite extensive beneath the heavy slope wash and talus which obscures much of the high rock bench on the left bank.

24. Where the older gravels have been extensively explored they are found to be somewhat weathered and so well cemented that, during tunnel driving operations, individual boulders could be drilled without dislodging them. In Tunnel C the gravels were relatively impervious, and most of the water was encountered in the talus immediately overlying the gravel.

25. Helland (c) called attention to gravel deposits on the small bench at elevation 1325, upstream from the present axis on the left bank. No relative age is assigned to these.

26. Gravels of intermediate age form the right bank terrace below the dam area and are exposed in the railroad cut above CH 2. They were penetrated to a depth of 55 feet in DH 32. Small patches of this material are still plastered on the steep slopes above the tracks in the vicinity of the dam axis. The finer material in these deposits is somewhat

weathered and the material is probably unsuitable for use as aggregate. Helland (c) states that it represents one of the two stages of valley filling in the area. The age has not been determined, but may be at least as old as Illinoian. No deposits of Wisconsin age have so far been recognized this far down the river.

27. Recent gravels are confined to the stream channel and low elevation bars. Nearly 60 feet of gravel were found in the stream bed near the axis of the dam. Deep holes filled with gravel may also be expected downstream from the rapids in the river, which will also be downstream of the spillway apron.

28. Various explanations are offered for the silt-like deposits at highway level several hundred feet downstream from the dam axis. McKitrick (d) concluded this material was glacial till, while Piper (i) favored the glacial outwash interpretation. Mongold (e) believes the material to be associated with faulting and weathering of gouge and crushed rock zones. Since the material is outside the foundation area, the geologic explanation becomes an academic question and is left as such.

29. Alluvial fan deposits are found at the mouth of the small creek below the dam area and partially cover the gravel terrace in the same area.

30. Numerous talus slopes are present on each abutment while many of the discontinuous rock outcrops are separated only by thin soil cover.

V STRUCTURAL GEOLOGY

31. Examination of the damsite area and much of the river for

several miles below discloses very little information on the regional trend of the Sardine lavas. According to Thayer's geologic map (1) the dip of the formation should be ESE. Locally, however, the trend may deviate considerably because of the close proximity to the Halls diorite intrusive.

32. The andesites in their original form had at least the texture of a flow rock, although other evidence of flow is scarce. From an engineering standpoint the andesite in the foundation area may be considered as a single lithologic unit, and when unaltered, is satisfactory as a foundation for the dam.

33. No major faults or lines of weakness have been discovered in the foundation area or immediate vicinity. The dry gully in the right abutment, as far as it has been explored, is in an area of minor joints and faults, which is not to be considered a zone of serious weakness.

34. <u>Description of Fracture and Joint Systems</u>: Careful surveys were made of all fracture and joint systems, both on the surface and in the tunnels, trenches, and calyx holes. Descriptions of the fractures and joints found in the calyx holes accompany the detailed logs of Plate V, Sheet 5 of this appendix. The remaining ones are listed and briefly described in this section.

35. After approximately 100 joints and fractures had been mapped, it was found that they could be resolved into four or five distinct sets, two of which were predominant throughout the area.

36. The most important and least variable set of fractures in the foundation area strikes from N. 6° E. to N. 6° W, and dips from 74° to 78° westward. These fractures are not very evident on the low rock floor of the left abutment. In this area they cross fractures

which, because they are sub-parallel to the stream, have been deeply scoured out and enlarged; therefore the N.-S. set of fractures is relatively inconspicuous. Fractures of this set are seen to best advantage in the road cut on the right abutment, where they are predominant.

37. All fractures belonging to this N.-S. set are to some extent hydrothermally altered. Those fractures near river level are centered in a bleached and softened zone from 0.2 to 0.6 or 0.8 feet in width. The altered andesite in this zone is in most cases firm and strong. In the right abutment hydrothermal alteration appears to be much more extensive along these fractures. Soft, white, punky zones from 2.0 to 4.5 feet in width may be seen in the road cut in several places. Adjacent to Calyx Hole CH-3 on the right abutment a zone 26 feet wide has been bleached and softened by hydrothermal solutions rising along fractures of this set.

38. Undoubtedly this set of hydrothermally altered fractures on the right abutment has been unduly accentuated because of the position above the water table, and because of the increase in volume of the rock due to loss of load. Both of these causes allow the altered zone to slake and crumble, thus producing soft zones. This altered material below the zone of weathered joints is undoubtedly strong and firm.

39. It may be further noted that the N.-S. set of fractures is parallel to the diorite dike which crosses the river below the toe of the proposed dam (Plate I, Appendix B). The hydrothermal alteration and secondary mineralization undoubtedly accompanied the intrusion of this dike.

40. The second major set of fractures strikes N. 18°-42° W. and dips 76°-88° S.W., most commonly 81°-85° S.W. The fractures of this set are, by comparison with the previous set, only very slightly altered. They are much stronger fractures, however, and often are badly brecciated. The larger of these fractures are commonly paralleled by many weaker fractures, and several "fractures" mapped in this set are actually zones of very closely spaced parallel fractures.

41. This set of fractures apparently is later than the N.-S. series previously described. Intrusion of the andesite porphyry dikes either accompanied or followed the development of these fractures. Variability in the strike of these fractures is graphically shown by the orientation of the andesite porphyry dikes on Plate I of Appendix B.

42. Chart I of this appendix shows the orientation of all surface joints and fractures, plotted in relation to true north and to the axis of the dam.

43. Following is the field descriptions of the fractures shown on the field map. The exact dip of the fractures on the low rock plain of the left abutment could not generally be measured. The strike of the fractures was not measured unless it could not readily be determined by inspection of the field map.

F-1 Dip vertical to 85° S.W. Generally tight and unbrecciated. Locally open 1/8". Hydrothermally altered for 0.1'.

F-2 Dip variable, 80°-85° S.W. A clean, tight fracture. Slightly hydrothermally altered locally for 0.1'.

F-3 Dip variable, 85° N.E. to 85° S.W. Tight, unaltered fracture.

F-4 Dip 85° S.W. Badly altered locally in a zone 0.3' to 1.2' wide in spots unaltered. Tight at the surface.

F-5 Dip 50°-55° S.W. Tight. Locally brecciated and very irregular. Unaltered or nearly so.

F-6 Dip 55° S.W. Tight. Badly altered from 0.2' to 1.5', especially along southern portion.

F-7 Dip 57° S.W. Tight. Locally altered for 0.1' to 0.2'.

F-8 Dip vertical to 88° N.E. along northern portion, 80° S.W. at southern extent. Hydrothermally altered for 0.1' to 0.2' throughout.

F-9 Dip 70° S.W. Tight. Slightly hydrothermally altered for 0.1' to 0.2'.

F-10 Dip 53° S.W. Tight. Slightly hydrothermally altered for 0.1' to 0.2'.

F-11 Badly shattered and hydrothermally altered zone associated with F-9 and F-10.

F-12 Dip 51° S. Very badly altered and locally very badly brecciated up to 2.0' or 2.5' wide near western extent. Alteration gradually dies out eastward.

F-13 Dip 70°-80° S. Altered for 0.2' to 0.4'. Tight. Badly stream fluted.

F-14 Dip vertical? Badly eroded and filled with debris. Badly altered for most of its length.

F-15 Dip vertical to 70° S.W. Altered for 0.2' to 0.4'. Very irregular. Irregularly brecciated.

F-16 Dip vertical to 80° S.W. Generally tight and unaltered. The eastern portion of this fracture passes out into a badly jointed area.

F-17 Dip 67° S.E. Tight. Unaltered.

F-18 Dip 70°-80° S.W. Tight. Practically unaltered. Badly scoured and gauged out by the stream.

F-19 Dip 80°-85° S. Tight. Badly altered for 0.2' to 0.5'.

F-20 Dip 80°-85° S.W. Clean, tight, unaltered. Disappears gradually to the northwest.

F-21 Dip 85° S.W. to vertical. Clean, tight, unaltered.

F-22 Dip 80° to 85° S.W. Brecciated and slightly altered for 0.2' to 0.4'. Tight. Disappears gradually westward.

F-23 Dip 58° S. Badly hydrothermally altered from 0.4' to 2.0' near western end, alteration decreases eastward and the fracture gradually disappears. Tight.

F-24 Dip 60°-65° S.W. Badly altered, 0.2' to 1.2', and locally brecciated. Tight.

F-25 Dip vertical to 85° N.E. or S.W. Very badly altered, 0.5' to 2.0', and brecciated in central part. Tight.

F-26 Dip vertical to 70° S.E. Tight, unaltered, unbrecciated.

F-27 Dip 53°-55° S. Altered for 0.1' to 0.2' for entire length and for 0.6' to 1.0' locally in central part. Tight.

F-28 Dip 57° S.W. Altered for 0.8' to 0.2'. Tight.

F-29 Dip 61° S.W. Generally tight and unaltered, locally altered 0.1' to 0.2'.

F-30 Dip 60° S.W. Generally tight and unaltered, locally altered 0.1' to 0.3'.

F-31 Dip 80° to 85° S.W. Very badly brecciated and hydrothermally altered, 0.5' to 1.5', in southeastern half.

F-32 Dip vertical to 85° S.W. Generally unaltered, locally 0.1' to 0.2'.

F-33 Dip vertical to 80° S.W. Braided. Locally badly sheared and altered in southeastern half, tight and unaltered in northwestern half.

F-34 Dip 80° to 85° S. Very irregular, locally brecciated and altered for 0.3' to 0.8'. Tight.

F-35 Dip 80° to 85° S.W. Very irregular. Altered for 0.2' to 0.6'. Alteration decreases to the northwest. Tight.

F-36 Dip vertical to 85° S. Altered for 0.2' to 0.4'. Tight. Very irregular.

F-37 Dip 85° S.W. Altered for 0.2' to 0.4'. Tight.

F-38 Dip vertical to 85° S.W. Very irregular. Locally altered for 0.4' to 0.8'. Tight.

F-39 Dip vertical to 85° S.W. Slightly altered. Tight.

F-40 Dip vertical to 80° S.W. Deeply scoured by the stream. Slightly altered.

F-41 Dip 80° to 85° N.W. Clean, unaltered, very slightly open.

F-42 Dip vertical to 85° N.E. Clean, unaltered, locally brecciated. Many short clean fractures parallel to F-42.

F-43 Dip 85° S.W. Clean, tight, unaltered.

F-44 Dip 85° S.W. Clean, tight, unaltered.

F-45 Dip vertical to 85° S.W. Tight. Altered for 0.3' to 0.5'.

F-46 Dip 80° to 85° N.E. Tight, unaltered.

F-47 Dip 85° S.W. Locally badly brecciated. Unaltered. Tight.

F-48 Dip irregular, 70° to 85° S.W. Badly brecciated, practically unaltered.

F-49 Dip vertical. A brecciated zone 0.4' to 0.8' wide. Unaltered.

F-50 Dip 77° S.W. Unaltered. Open 0.1' to 0.2'.

F-51 Dip 75° S.W. Unaltered. Open 0.1'.

F-52 Dip 71° S.W. Unaltered. Tight.

F-53 Dip 68° S.W. Badly altered and bleached for 0.6' to 1.5'. Soft, punky.

F-54 Hydrothermally altered zone 26.0' wide. Thoroughly bleached and softened throughout and locally very badly altered.

F-55 Dip 86° S.W. Tight. Hydrothermally altered for 0.1' to 0.2'. Altered material is very soft and crumbly. Tight.

F-56 Dip 88° S.W. Tight. Practically unaltered.

F-57 Dip 84° S.W. Locally badly brecciated and braided. Contains hydrothermally altered material 0.1' to 0.2' wide.

F-58 Strike N. 3° E., dip 76° N.W. Hydrothermally altered for 1.5'. Soft and punky.

F-59 Strike N. 4° W., dip 76° S.W. Very badly hydrothermally altered and brecciated for 1.0' to 1.5'. Tight.

F-60 Strike N. 2° E., dip 72° N.W. Two parallel fractures, 1.4' apart, cross jointed between. Hydrothermally altered for 0.3' to 0.5'. Altered andesite is soft and punky.

F-61 Strike N. 4° E., dip 79° N.W. Badly hydrothermally altered for 2.0' to 3.5'. Altered andesite is soft and punky.

F-62 Strike N.-S., dip 75° W. Two fractures 3.0' apart very badly hydrothermally altered in zones 0.6' to 1.2' wide.

F-63 Strike N.-S., dip 75° W. Badly hydrothermally altered for 0.8' to 1.3'. Tight.

F-64 Strike N.-S., dip 77° W. Badly altered zone 3.5' to 4.2' wide. Altered andesite is soft and punky.

F-65 Strike N. 34° W., dip 86° S.W. Badly hydrothermally altered for 0.4' to 0.7'. Altered andesite is soft and punky.

F-66 Strike N. 11° W., dip 72° S.W. Hydrothermally altered for 0.2' to 0.8'. Tight.

F-67 Strike N. 6° W., dip 74° S.W. Altered for 0.1'. Tight.

F-68 Strike N. 4° W., dip 72° S.W. Altered for 0.1' to 0.3'.

F-69 Strike N. 2° W., dip 81° S.W. Badly altered for 1.0' near river. Practically unaltered above bridge (not shown on field map).

F-70 Dip 87° S.W. Altered 0.1' to 0.2'. Tight.

F-71 Dip 76° S.W. Zone of parallel fractures, 0.5' to 2.0' wide, braided. Practically unaltered.

F-72 Strike N. 20° W., dip 77° S.W. Closely fractured zone, practically unaltered.

F-73 Strike N. 23° W., dip 78° S.W. Parallel fractures, 0.5' to 1.5' zone. Practically unaltered.

F-74 Dip 86°-88° S.W. Many tight parallel fractures, 0.8' to 0.3' apart. Practically unaltered.

F-75 Strike N. 31° W., dip 88° S.W. In places parallel fractures are present. Tight. Practically unaltered.

F-76 Strike N. 8° E., dip 74° N.W. Unaltered. Tight.

F-77 Strike N.-S., dip 77° W. Unaltered. Tight.

F-78 Strike N.-S., dip 78° W. Unaltered. Tight.

F-79 Strike N. 19° W., dip 82° S.W. Altered for 0.1' to 0.2'. Irregular. Tight.

F-80 Strike N. 34° W., dip 81° S.W. Locally slightly altered for 0.2' to 0.4'. Tight.

F-81 Strike N. 34° W., dip 80° S.W. Practically unaltered. Tight.

F-82 Strike N. 39° W., dip 80° S.W. A platy zone 1.0' to 3.0' wide is locally present. Tight.

F-83 Strike N. 39° W., dip 83° S.W. Only slightly altered. Very prominent platiness parallel to F-83 in a zone 10.0' to 15.0' wide. Tight.

F-84 Strike N. 16° W., dip 87° S.W. Unaltered, tight.

F-85 Strike N. 38° W., dip 85°-88° S.W. Unaltered. Prominent platiness parallel to F-85 in a zone 6.0' to 8.0' wide. Tight.

F-86 Strike N. 42° W., dip 86°-88° S.W. Slightly platy parallel to fracture. Tight.

F-87 N. 60° W., dip 50° S.W. Filled with gouge or hydrothermally altered material for 0.3-1.0. Dip steepens in upper half to 68°.

F-88 N. 1° W., dip 81° S.W. Open 0.1-0.3. Practically Unaltered.

F-89 Strike N. 5° E., dip 65° N.W. Shattered 0.1 foot wide, unaltered, thrown several feet down on West side. Forms hanging wall of closely jointed zone.

F-90 Strike N. 5° E., dip 65° N.W. Shattered 1.0 foot wide, altered 0.4 foot wide, forms footwall of closely jointed zone.

F-91 Strike N. 20° W., dip 60° S.W. No alteration except weathering.

F-92 Reopened fractures once followed by diorite.

F-93 Strike N. 35° E., dip 80° S.W. Continuous crack open .03 foot.

F-94 Strike N. 5° W., dip 80° S.W. Shattered .1 foot, open and continuous.

F-95 Strike N. 60° W., dip is vertical, continuous closed crack.

F-96 Strike N. 3° W., dip vertical, .1 foot shattered zone is "dioritized".

F-97 Several small parallel fractures strike N. 50° E., dip vertically.

F-98 Strike N. 50° E., dip vertical, continuous crack open .03 feet.

44. Description of Joints and Fractures in Tunnels: (For location of these joints and fractures see Plate V, Sheet 5.)

Tunnel A.

No. 1.	Strike	N. 9° W.,	Dip	70° S.W.	Altered 0.2 ft.
2.	"	N.78° W.,	"	68° S.W.	Offsets #1 0.7 ft.
3.	"	N.74° W.,	"	62° S.W.	Altered 0.1 ft.
4.	"	N.81° W.,	"	71° S.W.	Altered 0.1 to 0.2 ft.
5.	Altered area, no fracture.				
6.	Strike	N. 79° E.,	Dip	81° N.W.	Altered 0.1 ft.
7.	"	N. 08° W.,	"	75° S.W.	Altered 0.1 ft.
8.	"	N. 11° W.,	"	68° S.W.	Very slightly altered.
9.	"	N. 78° W.,	"	73° S.W.	Altered 0.1 ft.
10.	"	N. 28° W.,	"	63° S.W.	Altered 0.2 ft.
11.	"	N. 01° W.,	"	74° S.W.	Altered 0.0 to 0.1 ft.

Tunnel B.

No. 1. Strike N. 40° W., Dip 87°-90° S.W. Altered 0.1 to 0.5 ft. Braided at low angles, probable movement several in.

2. Strike N. 40° W., Dip 87°-90° S.W. Altered 0.1 ft.
3. Strike N. 22° W., " 70° S.W. Altered 0.2 ft.
4. Strike N. 85° E., " 75° S.E. Altered 0.2 ft.
5. Strike N. 70° E., " 75° S.E. Altered 0.1 ft.
6. Strike N. 05° E., " 65 ° W. Filled with zeolites. Shattered and weathered 0.5 ft. to 3 ft. Shows movement. Parallel to #7.
7. Strike N. 5° E., Dip 65° W. Altered and shattered 0.1 ft. Forms hanging wall of broken zone. Lack of clay or gouge shows that selvage on #6 is due to weathering and ground water seepage.
8., 9., and 10. Strike N. 35° W., Dip 75° N.E. Weathered up to 0.1 ft. Slight movement on #9. Belong to same system as Nos. 1 and 2.
11. Strike N. 70° E., Dip 75° S.E. Minor fracture.

Tunnel C.

No. 1. Fracture Strike N. 22° W., Dip 83° S.W. Altered 0.2 to 0.3 ft.

2. Fracture Strike N. 25° W., Dip 42° S.W. Altered 0.1 ft. locally.
3. Zone of platy jointing.
4. Fracture Strike N.-S., Dip 25°-35° W. Slickensided and containing clay.
5. Fracture Strike N. 15° W., Dip 68° S.W. Tight, unaltered.
6. Fracture Strike N. 12° W., Dip 43° S.W. Tight, unaltered.
7. Two parallel fractures. Strike N. 15° W., Dip 68° S.W. Very slightly altered.
8. Two fractures. Strike N. 05° W., Dip 72° S.W. Altered and oxidized for 0.2 ft. Soft.
9. Very prominent fracture. Strike N. 63° E., Dip 86° S.E. Tight, unaltered.
10. Fracture. Strike N.-S., Dip 68° W., Altered 0.1 ft.
11. " " N. 40° E., Dip 37° N.W. Tight, unaltered.
12. Fracture. Strike N. 61° W., Dip 84° N.E. Tight, unaltered.
13. Fractures parallel to 12.
14. Fracture Strike N. 10° W., Dip 68° S.W. Altered 0.4 ft.
15. Fracture zone. Braided altered fractures, 1.0 ft. wide, Strike N. 20° W., Dip 85° S.W.

Tunnel C. (continued)

16. Fracture Strike N. 63° W., Dip 65° S.W. Badly altered and oxidized for 0.4 to 0.8 ft. Very soft.
17. Fracture Strike N. 26° W., Dip 37° S.W. Hydrothermally altered for 0.1 to 0.2 ft. No soft material present.
18. Fracture Strike N. 36° W., Dip 80° N.E. Platy and shattered or crumbled for 0.1 ft. on each wall. Slickensided, no gouge.

Tunnel D.

No fractures or faults occur--only continuous joints are present--orientation as follows: N. 65° E., Dip 80° N.W.--N. 65° W., Dip 70° N.E. and N.-S., Dip 15° W. (See Plate V, Sheet 5.)

VI - Petrography

45. A microscopic examination of the foundation rock at the damsite was made to determine more closely the several types of rock and to study the alteration of these rocks in certain areas. Twenty samples were selected and one or more slides made from each. Forty slides were prepared by grinding rock sections to 3/1000 of an inch and mounting them on glass slides with the aid of Canada balsam. Determination of minerals was made by the use of a polarizing microscope. Details of the study are given in Table I at the end of this chapter. Rock types studied in the laboratory closely agreed with the field determinations and classification of O'Neill (h).

46. The andesite porphyry from the small dikes clearly shows the least amount of alteration of any of the rocks of the area, although some secondary minerals, chlorite and epidote, are present. The ferromagnesium minerals from which they are derived are still identifiable, which is not the case in the andesites. The rock is easily distinguished in the field by its greyish-green appearance and by the large phenocrysts of feldspar and altered hornblende. The hardness and toughness

of this particular rock is shown by its physiographic expression near the toe of the proposed dam, where it stands as low ridges above the surrounding rock floor. No compression or shear tests were made on the andesite-porphyry, since the test samples had been selected before this material was recognized as a distinct rock type.

47. A rather wide variation in mineralogical composition is noted in the diorite rocks studied. The feldspars have a considerable range in composition and the percent of quartz ranges from 5% to 60%. This variability is possibly due to the influence of the wall rocks on the rising dioritic magma. In DH 27 at elevation 1000/ a very light colored material composed of 98% feldspar and quartz was encountered. It is a fine-grained granite. Two samples from the large diorite dike (Appendix B, Plate I) have only 5% quartz and fall in the diorite classification. The remainder are quartz diorite, but for discussion purposes they are all referred to as diorite.

48. Alteration of the type previously described (chlorite and epidote) is present in these rocks to an equal or greater extent than in the andesite porphyry. Megascopic examination of the samples studies shows a slight staining and weathering effect not seen in the specimens of andesite porphyry. Diorite from the drill holes is usually fresh and sound. Tests on the diorite show compressive strength of 20,000 lbs. per square inch or over. Hybrid varieties are more variable and run as low as 7,770 per sq. inch, while others run around 12,000 to 13,000 lbs. per sq. inch.

49. Most of the andesite specimens studied were from the jointed

and altered area on the right abutment above the road. (Drill holes 2, 8 and 28.) Composition of the feldspars is quite uniform but their amount is variable from 5% to 50%. This is due in part to alteration and in part to the finer or groundmass portion of the rock being unidentifiable. Quartz appears as a replacement mineral (up to 40%) in at least half of the samples. Alteration products account for most of the remainder of the work. Kaolin is present in surprisingly small amounts for a rock of such lusterless, unsound outward appearance. Condition of the drill cores made it impossible to secure any specimens for shear testing in this particular area. From calyx hole and tunnel data the sound rock line was placed below this material.

50. Many local areas exist where the andesite has become a hybrid, quartz-bearing rock by the addition of hydrothermal solutions. While its strength is probably not as great as that of the unaltered andesite, it is still adequate for a foundation rock. More normal appearing andesite was encountered in tunnels A and D, although petrographic examination of this rock may show some of the alteration and hybridization found in other specimens.

VII - Foundations

51. Overburden: Little more than a brief summary is necessary in discussion of the overburden in the foundation area. The rock contour map (Appendix B, Plate III) has been drawn from field data on drill holes, tunnels and trenches. The overburden material contains a few large boulders (Tunnel C, RR embankment, etc.) but due to the close jointing in much of the area, the talus and slope wash is smallin size.

52. Maximum depth of overburden encountered was in DH 54 near the head of the buried gully on the right abutment. Seventy feet of this was talus overlying the cemented gravel and boulders. Apparently little of this material would be removed for the low dam. Further exploration should be done during or prior to construction to verify the rock bench in the vicinity of DH 16. The present interpretation shows a continuous rock line above the crest elevation of the low dam, but extending diagonally downstream from DH 8.

53. On the left abutment the greatest depth of overburden in the foundation area lies between DH 52 and DH 34, where the material is talus and slope wash. As previously mentioned in Sec. IV, gravel deposits are known to be 60 feet deep in the river channel.

54. Rock excavation: Limit of rock excavation is estimated from apparent sound rock in the drill cores, and in the calyx holes and tunnels. The sound rock contour map (Plate IV this Appendix) is based on these interpretations. This map is expected to set the maximum rather than the minimum or average limit of the rock excavation. Many areas especially below elevation 1300 are believed to be perfectly sound only a few feet below the surface of the rock. Estimated average depths of rock excavation based on exploration data are as follows:

Location	Elevations	Average Depth
Left Abutment	1400-1550	20 ft.
Left Abutment	1300-1400	16 ft.
Left Abutment	1200-1300	7 ft.
River Bed	1130-1200	3 ft.
Right Abutment	1200-1350	18 ft.
Right Abutment	1350-1550	38 ft.

55. Selective mining out of seams and altered areas below the general excavation line may be necessary in places. This should eliminate the necessity of deeper excavation under whole blocks.

56. Grouting: A program for grouting the foundation area has been outlined by E. B. Burwell (b), consulting geologist, in his report of October 25, 1940. (See Chapter II of this appendix) The initial stage of this program probably will effectively seal the majority of the openings over most of the area. In the final stage of grouting the abutments at the higher elevations will likely receive most of the grout. This is especially true of the high dam.

57. Pressure testing has given some indication of the extent of open jointing in the foundation area. Five-foot segments were tested concurrent with the drilling of each hole in the 1940 exploration program. This method is recommended over the methods used in the earlier program, when an attempt was made to place the packer at successively lower elevations (usually 1 or 2) until the bottom portion of the hole showed favorable holding and pressure tests.

58. Least favorable tests during the 1940 program were obtained in the NX pilot hole at CH 3. Water losses were as much as 2.1 cu. ft. per minute for many of the five-foot segments in the upper 80 feet of the hole. A large amount of seepage was noted in tunnel A at the time CH 3 was being drilled. Upon completion of the calyx hole and the tunnel, flourescine dye was placed in the bottom of CH 3 and the time of its appearance was noted in the tunnel below. A few minutes over five hours were required for the dye to travel the inclined distance of 90 ft. by gravity alone.

59. Depths of oxidization and of heavy discoloration of the drill cores are shown in the drill logs by a heavy vertical line. (Appendix B, Plate V, Sheets 1-4)

60. Incidental to examining the cores for determination of sound rock depths, the maximum depth to which the cores were stained by iron oxide was also noted as an indicator of the extent of the circulating ground water. A line drawn on the profile on Plate II, Appendix B shows the lower limits of the iron stained joints as determined from the drill cores. This probably does not represent the maximum depth to which good grout penetration will be obtained, but is considered the most important area.

61. Stability: With such foundation treatment as has been outlined, no known factors remain which would affect the stability of the dam. The joints and fractures are favorably located with reference to the axis of the dam and dip downstream or toward the left abutment at high angles (70°-85°). A few sub-horizontal fractures dip toward the same abutment. Proper care in washing and grouting these to eliminate any seepage is necessary. Differential settlement under maximum loading is to be expected in a jointed rock of this type and is considered in the design. Compression stresses are most nearly parallel to the fracture lines while the shearing stresses are more nearly normal to these lines.

VIII - DIVERSION, SPILLWAY

62. Diversion: Plans for diversion of the river during the early stages of construction call for earth cofferdams above and below the

the foundation area. Deep gravel deposits are known to exist in the river channel at the axis of the dam and for 200 feet or more upstream. To effectively seal the area against seepage two locations are suggested for the upstream cofferdam.

<u>a</u>. In the vicinity of DH 6 where the bedrock rises to elevation 1170. Drill records indicate large boulders in the gravel which might prevent the use of sheet piling. Removal of the gravel and boulders to bedrock may be necessary before placing of the cofferdam.

<u>b</u>. Just upstream of DH 1 and DH 7 a ledge of rock extends part way across the river channel which might be utilized in an effective cofferdam.

63. Considerable depth of gravel should be expected in the stream bed below the spillway apron and should be considered in placing a cofferdam in that area.

64. <u>Spillway and Outlets</u>: The downstream edge of the spillway apron of the low dam is favorably located on the diorite dike, which has already retarded erosion of the stream channel in that area. Excessive erosion downstream from the apron seems very unlikely. There already exists a deep pool partially filled with debris, which may be effective in retarding the action of the water.

IX - RESERVOIR

65. <u>Leakage</u>: The Detroit reservoir presents no problems involving leakage or evaporation. No low divides or narrow ridges separate the reservoir from adjacent river valleys, and the volcanic formations are

relatively tight and impervious. Several large deposits of gravel may give some additional storage, but the amount is not expected to be very great.

66. Landslides: Some small landslides are found at maximum pool elevation, and may affect the road and highway relocation in these areas. One of particular note near Berry Forest Camp (Sec. 16-T10S-R5E) has already been considered in the relocation surveys. Land slips should be of no concern in the Sardine lava series of the upper gorge. The Breitenbush Tuff series above the mouth of Blowout Creek may be subject to local sliding in deep roadcuts.

67. Silting: Deposition of some heavy debris must be expected where the Breitenbush and the main North Santiam enter the reservoir. Fluctuation of the pool level would distribute the material, with a certain amount of reworking, between minimum and maximum pool levels. The silt content of the Cascade streams this far back in the mountains is unusually low, although high water stages must necessarily carry a small amount. Some glacial flour is carried in the summer melt water of Whitewater and Milk Creeks. Wave action, while probably not severe, may be noticeable in some areas.

X - CONCRETE AGGREGATE

68. Alluvial Deposits: Preliminary studies of concrete aggregate deposits were made in 1940 on two widely separated areas. The most extensive deposit of river gravel and sand near the damsite is the river bed and the adjacent low terraces between Blowout Creek and the Breitenbush River. (See Plate VI, this appendix). This area lies three to five miles upstream from the dam and offers several good sites for an aggre-

gate processing plant. Rail transportation to the damsite could be probably arranged over the existing railroad tracks.

69. More than 200 acres in this vicinity are underlain by sand and gravel deposits, none of which has been penetrated more than 10 feet by any of the eight test pits in the area. Apparently sufficient material exists to allow selective operations and to allow a high percentage waste if necessary.

70. Another natural aggregate deposit of sufficient size for consideration is in the south half of Section 25 T9S R2E. It extends for one to two miles along the right bank of the river below Mill City. (See Appendix B, Plate VI) Only one shallow test pit was dug in the bottom of the state highway borrow pit. Further reconnaissance of the area indicated a sufficient quantity of material available for Detroit Dam. Use of the deposit is dependent, however, on the quality of the material, which is expected to be as good as the deposit above the damsite. Further tests would be necessary to determine its suitability as concrete aggregate. Both deposits show considerable variation in the natural grading, and therefore may be deficient in certain screen sizes. The +6" size ranges from 2.3% to 53.2% and the sand sizes range 23.3% to 66.9% of the total. Since either area is sufficiently large, Selective borrowing might possibly improve the size gradation and eliminate some or all the crushing costs. In general the two areas may be compared as follows: Both deposits are derived from the lavas and volcanic rocks of the Cascade Mountains. The Mill City deposit has the additional admixture of the rocks and minerals derived from the Santiam

canyon. Much of the material in the lower deposit, having travelled further, should, by comparison, be smaller in size and have a smaller percentage of unsound material, especially in the gravel sizes. The ratio of mineral grains to rock fragments should be higher in the sand sizes of the Mill City deposit.

71. Most of the samples tested so far are above the level of the permanent water table. Material below this surface may be less weathered and consequently give better test results.

72. Crushed Rock: Use of crushed stone from the foundation of the dam or a nearby area for all or part of the aggregate has been suggested. The petrographic study of the foundation rocks showed conditions very unfavorable for their use as aggregate. Although the material is considered more than adequate as a foundation rock, its mineralogical composition is not desirable for good concrete aggregate. Abrasion tests are necessarily high on crushed rock and the shape of the fragments may not be suitable. Magnesium sulphate losses would be high because of the structure of the rock and because of certain alteration minerals such as chlorite.

73. It is a foregone conclusion that most of the rock to be excavated for the foundation of the dam is of poor quality and therefore of little or no use. Alteration of the andesite country rock has been pretty general in the damsite area, and while possible, it is not very probable that extensive areas of suitable andesite for aggregate can be found. A possibility exists about a mile upstream, where the quartz diorite intrusive rocks are well exposed. Specimens of material from these larger bodies have not been examined microscopically.

XI - CONCLUSIONS

74. The North Santiam River offers a number of favorable sites for power dams with 100 to 150 ft. head. The cross-section of the Santiam canyon is very distinctly V-shaped in the bottom and widens out into narrow terraces or benches above. High storage requirements for flood control dams present many difficulties in selecting sites of favorable cross-section. This is true not only of the North Santiam River but of other streams in the Willamette Basin. The "inner canyon" is rarely of sufficient depth and a flood control dam must extend up on to the adjacent bench or terrace. The Detroit Dam site, located near the head of the Santiam gorge is most favorable for storage requirements.

75. Rock conditions at the other North Santiam dam sites investigated for the 1936 report (McKitrick, (d), (Piper, (i)) are similar to those at the Detroit (Boundary) site. Jointing and fracturing, intrusions, and secondary mineralization are generally present, therefore the lower sites would have little if any advantages in foundation conditions. Small advantages would likely be offset by increased height of dam to provide sufficient storage.

76. At the present time the most serious problem confronting the construction of the Detroit Dam is the location of suitable concrete aggregate deposits. From present indications the natural aggregate in the reservoir area is not satisfactory. Further research and testing is necessary to determine whether the North Santiam sands and gravel can be brought up to specifications. It may be necessary to add Willamette or Columbia River sand to Detroit sand which has been washed and milled.

77. Exploration of the damsite has been adequate to outline structural conditions of the foundation and determine within reasonable limits the rock surface and the sound rock line. Further exploration during construction often becomes necessary to outline minor irregularities and this may well be the case in a dam of this size.

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No. 16 - RT (Detroit) - March 5, 1940
No. 19 - RT (Detroit) - April 23, 1940

"Tests on Sand & Coarse Aggregate, Vicinity Detroit Dam Site"
No. 24 - RT (Detroit) - July 11, 1940
No. 32 - RT (Detroit) - November 25, 1940
No. 43 - RT (Detroit) - January 20, 1941

APPENDIX B ~~CHAPTER II~~

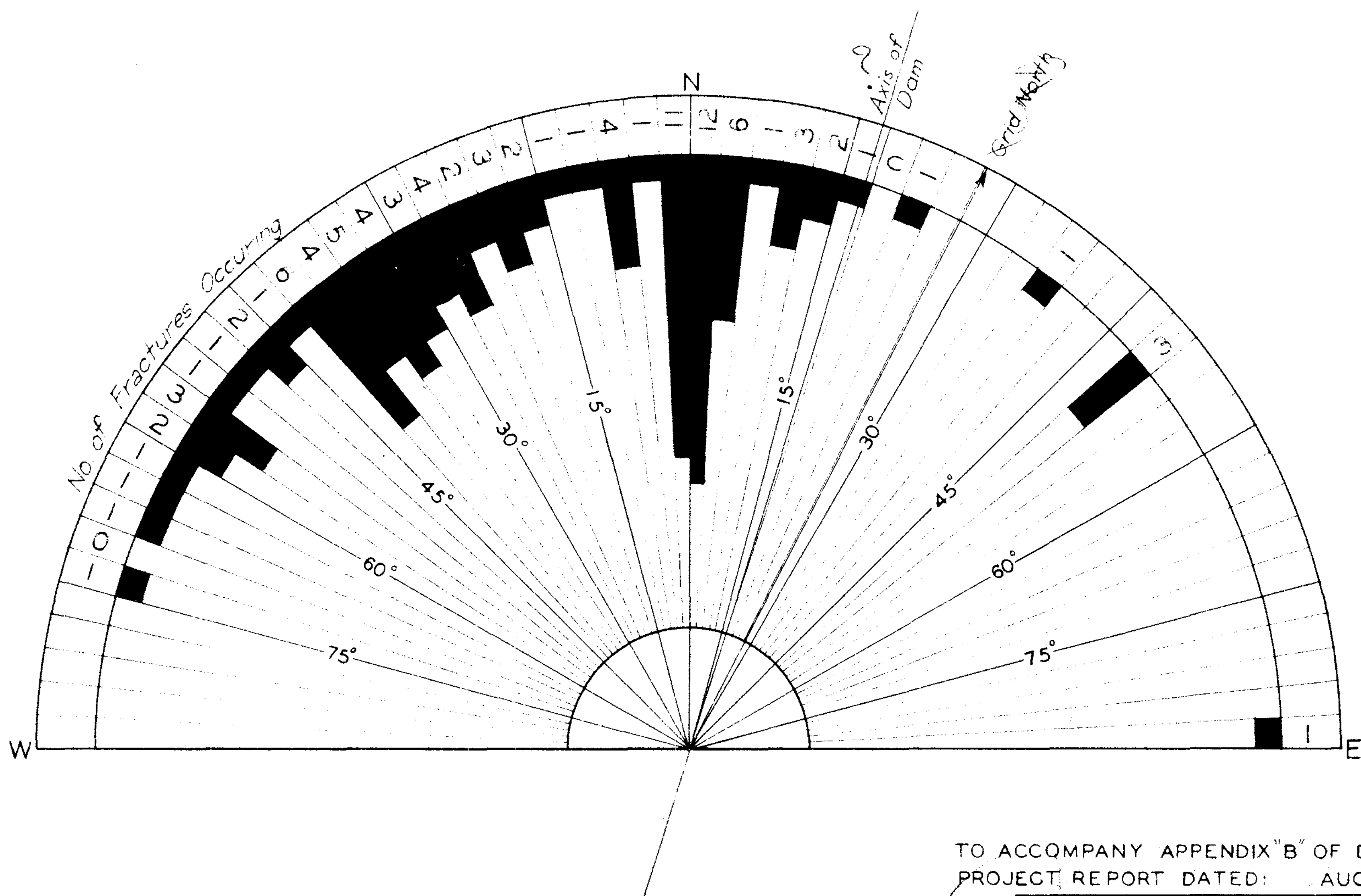
TABLE ~~V~~ 2

MINERALOGICAL COMPOSITION OF FOUNDATION ROCKS
DETROIT DAM SITE

Designation	Depth	Field Name	Microcline	Albite	Oligoclase-Andesine	Andesine	Andesine-Labradorite	Labradorite	Quartz	Magnetite	Apatite	Hornblende	Hypersthene	Pyroxene	Epidote	Chlorite	Sericite	Kaolinite	Limonite	Andesitic Mat.	Indeterminite	Petrographic Classification
			Minerals and Percentage																			
DH-28	60'	An				22			30	4						12	12			20		Andesite Micro-Porphyry (altered)
DH-28	41'	An				30				1				2		14	1	2	o	50		Andesite Micro-Porphyry
DH-28	17'	An				15			x	x						x		?				Andesite Micro-Porphyry
DH-8	52'	An				x				x					–	x						Andesite Micro-Porphyry
DH-8	32'	An				35				1					5	9					50	Andesite Micro-Porphyry
DH-8	24'	An				5			30	1					2	10	1	1			50	Andesite Micro-Porphyry (altered)
DH-2	50'	An				10			40	6						5	–			40		Andesite Micro-Porphyry (altered)
DH-2	19'	An				50				2					5	25		3			15	Andesite Micro-Porphyry
DH-2	41'	An				15			20	5					5	5				50		Andesite Micro-Porphyry
V	S	Anp				40			5	2		2				15'	½	½		35		Andesite Porphyry
On-1	S	Anp					20		5	4		o	5		10ø	5					50	Andesite Porphyry
On-2	S	Anp					14		5	3		o	5		10ø	10					50	Andesite Porphyry
On-3	S	Di			40				37	1	o					2	8	12				Quartz Diorite
On-4	S	Di				79			5	2		4				10		o				Diorite
On-5	S	Di			35				30			4				1	15	15	–			Quartz Diorite Porphyry
On-6	S	Di			20				46	2		1			8	7				16		Quartz Diorite Porphyry
On-7	S	An		x						x						x			x		50	Andesite Breccia
On-8	S	Di				50		20	5	3			5			5		5	–	7		Diorite
DH-27	332'	Gr	30	8					60	1		2						o				Granite
DH-4	55'	Di*				60			10	5		25						–				Quartz Diorite *

S = Surface samples
On = Samples by O'Neill
* = River Boulder
V = Randon Sample
x = Percent not estimated

o = Small % present
– = Shows trace
' = Includes some Magnetite
ø = Variety Clinozoisite



TO ACCOMPANY APPENDIX "B" OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

WILLAMETTE BASIN PROJECT

DETROIT DAM

TREND OF

SURFACE JOINTS & FRACTURES

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT

DRAWN: R.C.N. TRACED: G.E.G. DATE NOV 1940

Note: For additional data see Appendix "B" Plate I, also text.

APPENDIX "B" - ~~CHAPTER II~~ - CHART I - DE-40-25/1

APPENDIX B - GEOLOGY AND FOUNDATION DATA

DETOIT DAM SITE - NORTH SANTIAM RIVER, OREGON

CHAPTER III - REPORT OF CONSULTING GEOLOGIST

III - REPORT OF CONSULTING GEOLOGIST

Table of Contents

Report of E. B. Burwell, Jr., Consulting Geologist,

Detroit Dam and Reservoir, North Santiam

River, Oregon.

October 13, 1939.

Detroit Project on North Santiam River

1. The plan for the Detroit Project provides for the construction of a concrete dam to an initial crest elevation of 1475 and an ultimate crest elevation of 1580. The maximum height of dam for the initial stage would therefore be approximately 325 feet and for the final stage approximately 430 feet. As this dam is to be very high, careful consideration must be given to all the facts that relate to its foundation.

2. Geologic Setting. The narrow gorge in which the dam site is located is formed in and underlain by thick Tertiary lava flows of the andesitic type which appear to dip westerly at low angles. The base of the andesite was not reached in borings drilled to a depth 200 feet below the stream bed and it extends to an elevation of more than 1000 feet above the river in the valley walls. A few miles above the dam site, near the village of Detroit, the lavas rise above drainage and fragmental rocks of volcanic origin - tuffs and agglomerates - come to the surface. At a number of places the lavas are cut by intrusive masses of diorite, a rock similar in composition to andesite, but distinguishable from it by its granitoid texture.

3. Character and Condition of Foundation. The principal rock mass at the dam site is composed of andesite, a dense, fine grained rock of volcanic origin. It will form all of the foundation except in subordinate areas where intrusive bodies of diorite in the form of dikes and tongues

are present. The andesite is a dense, hard, durable rock, has a high strength, is free from porous layers, such as are commonly found in lava flows, and, where structurally sound and unweathered, is a suitable rock for a high dam of the gravity, arch, or buttress type. Preliminary tests indicate compressive strengths ranging from 14,000 to 30,000 pounds per square inch. No scoriaceous contacts, vesicular layers or cinder beds have been discovered in outcrops or in any of the 43 core borings made in the site area. While the andesite has undergone a considerable amount of hydrothermal alteration, and such alterations have in places lowered the quality of the rock somewhat, the changes are relatively unimportant from the engineering point of view; for where unweathered and confined, the poorest grade of andesite observed has a strength far in excess of the requirements.

4. The diorite masses which will form a relatively small portion of the foundation are composed of thoroughly competent rocks and no evidence was found that the nature of the contact zones is such as might affect the design or stability of the dam. Some non-metallic mineralization by materials weaker than the inclosing rock has taken place in the contact zones and in premineralization fractures in the andesite, but the strength of the foundation as a whole has not been appreciably affected and such mineralization has been advantageous to the extent that it has healed the older fracture systems.

5. The principal flaws in the foundation arise from the several rather conspicuous fracture systems which cut both the andesite and diorite masses. All of these have promoted weathering of the abutment rocks, in places to considerable depths, and along some of the fractures minor shear zones of shattered rock have developed. In general, however,

these planes and zones of weakness appear to have been greatly accentuated by weathering and it is probable that most of them will become tight and many of them barely perceptible below the zone of weathering.

6. There are two fracture systems that warrant special mention and consideration. One of these strikes approximately N 45° W and dips 80° or more to the southwest. These fractures cross the proposed axis of the dam at an angle of about 70°. If continuous and open, they would have an important bearing on leakage and might conceivably affect the shear strength of the foundation. They are conspicuous along the bedrock floor at the foot of the left abutment and have to a large extent controlled the course of the river through the site area and probably the buried ravine in the right abutment. The fracture spacing varies from a few feet to 20 feet or more. On the valley floor, where the outcropping rock is fresh and sound, they are tight, are arranged in an echeloned pattern and are not continuous over great distances, though some may extend through the dam site area. Generally they are single fractures, but a few show narrow fractured rock zones along them. While this system of fractures has promoted weathering in the abutments and bears on the grouting problems, particularly in the abutments, its bearing on the stability of a gravity-type dam is believed to be of little significance. Owing to their nearly vertical position, shearing of the foundation would take place before sliding along these fractures could occur.

7. The other set of important fractures strikes N 10° E and dips 70° or more to the northwest. They are therefore roughly parallel to the axis of the dam and dip in a downstream direction. The most conspicuous development of these fractures is seen along the highway cut in the right abutment a short distance upstream from the proposed location of the heel

of the gravity dam. In a zone approximately 100 feet wide, 8 of these fractures were observed. Along some of them shear zones as much as 2 feet wide occur, but nowhere was fault-gouge or finely crushed rock found. It is believed that the amount of movement along them has been small. Deep weathering has taken place along some of them, however, and the crushed rock zone at the outcrop consists of soft fragmental rock and clay. These fractures were not observed to extend downward through the abutment to the valley floor and are apparently, in part at least, cut out by intersecting fractures of other systems. It is believed that weathering has enormously accentuated these zones of weakness in the right abutment and that below the zone of weathering the conditions will not be such as to challenge the feasibility of the foundation for the high dam proposed. Further investigation of these fractures and minor shear zones by means of tunnels and large diameter drill holes is, however, warranted. No evidence was found that any faulting of major proportions occurs within or near the dam site area. The opinion that below the zone of weathering the major fracture systems are tight is substantiated by the satisfactory core recovery obtained from most of the borings drilled in the valley bottom where erosion has kept pace with weathering.

8. <u>Stability of the Right Abutment</u>. No evidence has been brought to light that the roughly pyramidal shaped rock mass that forms the right abutment is out of place or that it is parted from the main valley wall mass by any dangerous plane or zone of weakness. While it is probable that a prominent set of northwest fractures cuts the right abutment along the buried ravine, it is believed that, for all practical purposes, the right abutment mass is monolithic with the valley wall mass. Investigation of the buried ravine area by means of tunneling is desirable, however, and

has been included in the suggested plan of explorations.

9. Type and Location of Dam. In the writer's opinion no geologic conditions are now known that would preclude the construction of a safe and effective concrete dam of the gravity, arch, or buttress type. Both the foundation and the topography are believed, however, to favor the gravity type. While it appears that the choice will be governed primarily by engineering considerations and economics, it seems reasonably certain that a gravity-type structure would minimize the disadvantages of the foundations resulting from the fracture systems described above.

10. The location for a gravity dam as now proposed has been governed largely by economic considerations as determined from the preliminary bedrock contour map. Further investigation of the shear zones in the right abutment may bring to light conditions that will justify shifting the axis somewhat farther downstream, but no major change is indicated.

11. Stripping Requirement. In the examination of cores special attention was paid to the condition and depth of weathering in all critical areas in an effort to determine the depth of stripping necessary to secure acceptable foundation rock. All of the cores taken thus far are of small diameter (1-5/8 inches or less). Owing to rather high core losses in many of the abutment holes, some of which were undoubtedly due to causes other than weathering, a well defined strip line is not apparent along any given section through the abutments. It is certain, however, that the depth of stripping on the abutments will vary widely due to variable thicknesses of overburden and to irregularities of weathering along fractures. It is anticipated that the depth of rock stripping normal to the slope of the abutments will range from 20 to 50 feet. Complete

removal of the projecting rock masses below the highway bridge should be provided for as they are badly broken and unstable. In the valley bottom where the rock is, in general, sound at the outcrop, allowance of 5 feet of stripping below the bedrock surface appears adequate. All of these figures may be altered materially by the further more reliable explorations which have been recommended. Some indication of the probable depths to sound rock is given by the following data on selected borings:

Hole	Location	Elev.	Rock Elev.	Sound Rock Elev.
5	Valley bottom	1195	1130	1126
7	Valley bottom	1213	1213	1213
15	Left abutment	1375	1368	1357
16	Right abutment	1575	1517	1515
19	Right abutment	1643	1620	1578
22	Right abutment	1427	1396	1372
23	Left abutment	1471	1433	1433
24	Right abutment		1393	1369
25	Left abutment	1404	1395	1391
28	Right abutment	1449	1449	1387
30	Right abutment	1433	1431	1386
34	Left abutment	1505	1456	1456
38	Valley bottom	1207	1207	1204
41	Valley bottom	1198	1198	1194

12. Leakage Conditions. A cursory examination of the reservoir geology and a study of the topography of the North Santiam watershed indicate that there is no probability of important leakage if the dam abutments and foundations are satisfactorily sealed by means of a grout curtain constructed at the heel of the dam. A considerable quantity of

rather deep grouting is indicated for the abutments, for among the most prominent fractures are those that trend roughly normal to the axis of the dam. Below the valley bottom it is believed the grouting requirements will not be excessive.

13. Earthquakes. No great earthquakes have occurred in Oregon within historic time and only 2 of the 13 recorded since 1856 are listed as of intensity 8 as measured by the Rossi-Forel scale. One of these occurred in 1873 and had its epicenter near the Oregon-California border; the other in 1936 with its epicenter close to the Oregon-Washington border. Concerning these two most noteworthy earthquakes, the following statements are taken from Earthquake History of the United States, Part I, Coast and Geodetic Survey, U. S. Department of Commerce:

> 1873. November 22. A severe earthquake centering between Crescent City, California, and Port Orford, Oregon, and presumably near the State border, was felt from San Francisco, California, to Portland, Oregon, over an area 280 miles by 160 miles. Between Port Orford and Crescent City many chimneys were thrown down.
>
> 1936. July 15. North Oregon just south of the border. The earthquake was destructive at Freewater, State Line, and Umapine, Oregon. At Freewater, plaster and windows were broken, chimneys were shifted at the roof line, and there was various damage to schoolhouses and other buildings. At Walla Walla movable objects were shifted and a few chimneys were wrecked. In the central region there was much cracking of the ground in one case over an area 1500 by 75 feet. The shock was felt over a large area in Washington, Oregon, and Idaho. There were numerous aftershocks up to November 17. The two strongest attained intensity 5 on July 18 and August 4.

14. No seismic disturbances of sufficient intensity to cause damage to an engineering structure have been felt at Detroit within historic time and no active faults are known to exist in the area. However, as this dam is to be very high, and the possibility of seismic disturbances does exist, there appears to be justification for an earthquake assumption in the design of the dam.

15. Further Investigations. Further subsurface investigations by means of 30-inch drill holes and tunnels to determine the character of the fracture zones in depth and the amount of stripping necessary to provide sound foundation rock are shown on the accompanying map. (Plate I.) While the important areas have been rather extensively explored with small diameter drill holes, much additional information regarding the structural details and depth of weathering is to be desired and this can best be accomplished by explorations of sufficient size to permit an inspection of the foundations in place.

16. Conclusions. The present incomplete data indicate that there are no geological conditions at the Detroit site that would preclude the construction of a safe and effective concrete dam of the maximum height proposed.

Supplementary Report of E. B. Burwell, Jr., Consulting Geologist,

Detroit Dam and Reservoir, North Santiam

River, Oregon.

October 25, 1940.

17. A report on "Geologic Conditions Relating to the Location, Design and Construction of the Proposed Detroit, Cottage Grove, Fern Ridge and Dorena Dams" was prepared by me and submitted under the date of October 14, 1939. Subsequently, extensive subsurface investigations have been made at the Detroit and Dorena sites and the Cottage Grove and Fern Ridge projects have entered the construction stage. During the period October 7 to 11, inclusive, 1940, I made further studies of the geologic conditions at Detroit and Dorena and inspected the foundations for the spillway structure at Cottage Grove. On October 12, 1940, a conference was held in the District Office, at which time the results of these investigations and the various problems and questions presented for my consideration were discussed. This supplementary report confirms the findings and opinions given at that conference.

DETROIT

18. Under the subject "Supplementary Geologic Report on Detroit Dam Site, North Santiam River, Oregon" the District Engineer, in his letter of October 2, 1940, requested a supplementary geologic report on the Detroit dam site covering specifically the following features:

> "(a) A check of all geologic interpretations made by this office. It is especially desired that an accurate and complete check be made of our 'rock' and 'sound rock' contour maps. The latter map fixes, in general, the excavation lines for the proposed structure.

"(b) A study of the most suitable alignment for the 'low' and 'high' dam axis as determined by geologic considerations. The low dam axis must be located to fit economically into a future high dam plan. Future raising may take place either on the downstream or upstream face of the initial low dam.

"(c) A recommendation in regard to foundation grouting, drainage, and, if pertinent, special treatment of fracture areas. Your opinion is especially desired in regard to depth, grouting pressures and extent of grouting that might be required for (1) consolidation of critical foundation areas, and (2) the provision of a suitable cut-off grout curtain near the upstream face of the dam.

"(d) A reconsideration of the stability of the right abutment. Reference is made to paragraph 8 in your report of October 18, 1939. Your further considered opinion is desired in regard to the possibility that the whole right abutment area, downstream from Tunnel T-C, is a slide block which has become separated from the main valley wall mass. If such a condition is possible, a statement as to the seriousness of this feature from a structural standpoint is desired.

"(e) Reconsideration of any other geologic features brought to light by the latest exploration program which might change the conclusions arrived at in your original report or which might affect the design of the structure."

The above items are discussed below in the order listed.

19. A careful study was made of bedrock conditions revealed by the 36-inch drill holes, tunnels and trenches and the cores were examined from all small diameter drill holes located in the areas in which structures would be placed. My findings relative to bedrock configuration and sound rock elevations agree reasonably close with those made by the District office. In some locations, where the sound rock elevation was not clearly defined and it was necessary to make a choice between a maximum and minimum strip line, I have taken the conservative viewpoint and used the maximum. On a basis of all of these determinations, it appears that the average depth of rock stripping necessary to secure foundations suitable for the high dam

proposed will range from 40 to 60 feet in the right abutment, from 5 to 10 feet in the valley bottom, and from 20 to 25 feet in the left abutment. The data on sound rock depths are tabulated below:

Hole No.	Depth to rock in feet	Depth to sound rock in feet
13	18.3	58.9
9	61.3	75.7
19	23.2	64.6
16	58.3	60.0
14	5.0	48.0
8	4.5	Not reached at 54.8
28	0	66.3
29	1	42.3
30	2	40.1
47	0	10.0
53	0	13.5
35	31.7	46.2
37	19.6	46.7?
4	64.7	69.5
5	65.2	68.9
38	0	3.0
40	0	3.0
41	0	9.2
45	0	10 Vertical
31	0	5
43	13.0	20.2
46	0	2.1
15	6.7	32.4
49	13.0	24.8
25	9.4	34.8?
17	11.4	22.0
51	11.9	29.7
34	49.5	54.1
52	30.3	43.0
50	17.0	25.0
48	10.0	67.0

Special mention should be made of 36-inch hole CH 2 in the valley floor where local conditions apparently are responsible for the deep sound rock elevation. It seems probable that a few feet away from the intrusive contact, which this hole penetrates, the sound rock elevation lies within 5 to 10 feet of the surface. There will doubtless be other localized areas where, due to structural conditions, a deeper than average weathered rock zone will be found and have to be removed, but these should not materially affect the total quantities.

20. Structural conditions in the andesite mass which forms the Detroit dam site and the effects of structure on rock weathering are such as to make it desirable to locate the axis of the dam as far down stream as topography and engineering considerations permit. Deep weathering has penetrated the prominent shear fractures which cut the right abutment between tunnels T-B and T-C (strike N 5 to 10° W dip 65°$\pm$ to S.W.) and clay fillings from a fraction of an inch to 6 inches are present above the highway level. While the movement along these fractures appears to have been small and they are not regarded as flaws of a serious nature, they have materially affected the quality of the foundation in the area in which they are most conspicuously developed and should be avoided as far as possible in locating the upstream limits of the dam. It is to be noted that these fractures are nearly normal to the line of maximum thrust and it is conceivable that they might afford a considerable amount of slack in the right abutment foundation unless thoroughly treated. For these reasons it is recommended that the heel of the ultimate stage dam be located not farther upstream than that shown on the sheet No. 1 of the present project plan. It would seem desirable, therefore, if this recommendation is to be followed, to decide in advance of the initial stage construction on which face the dam would be raised when and if the ultimate stage construction is undertaken.

21. While the foundation rock at Detroit is a strong, durable andesite, it is cut at rather close intervals by fractures of both tension and shear types. Partial removal of restraining loads incident to the development of the North Santiam River valley has resulted in the opening of many of these fractures to considerable depths in the valley walls and deep weathering has taken place in masses that lie above the water table. The two most noticeable fracture systems are those which for all practicable

purposes may be considered as having trends parallel and normal respectively to the axis of the dam. The former, especially, might afford some slack in the abutment rocks in the direction of the maximum thrust and the latter are of special importance from the leakage point of view. As the 36-inch holes and the tunnels indicate the presence of a considerable number of open or clay-filled fractures below the probable foundation elevations in the abutments, it is believed that consolidation grouting should be provided for beneath the abutment monoliths of the dam. Reasonably tight bedrock conditions are indicated beneath the valley floor by the 36-inch holes and the pressure tests made on small diameter holes so that curtain grouting only seems necessary under this portion of the dam. The following general plan of grouting is suggested:

a. Prior to placing concrete in the dam, consolidate the upper 30 feet of the abutment foundation by means of drilling and grouting 30-foot wagon-drill holes on 20-foot centers over the entire area of the abutment monoliths using pressures up to but not exceeding 50 pounds per square inch. Careful washing of all holes to remove clay-fillings should be done.

b. Construct a shallow curtain along a line 5 feet upstream from the heel of the dam for the full length of the dam before placing concrete by drilling and grouting wagon-drill holes 30 feet deep on first 20-foot centers, then 10-foot centers, then 5-foot centers, if necessary, to secure a tight curtain using pressures not exceeding 50 pounds per square inch.

c. Construct a deep curtain along the full length of the dam and for a reasonable distance beyond the ends of the dam from the grout gallery after a sufficient weight of concrete has been placed to permit the use of high pressures. It is recommended that the grout gallery be made at least 10 feet high to facilitate drilling and grouting operations. It is further recommended that grout pipes be set on 2-1/2 foot centers and that the grouting be done in two stages, the first stage extending to a depth of 75 feet in the valley bottom and increasing to 100 feet in the abutments, and the second stage, if found necessary after exploratory drilling and pressure testing, extending to a depth 150 feet in the valley bottom and increasing to a depth of 200 feet in the abutments. All deep curtain drilling would be done with rotary drills. Provision should be made for drilling angle holes either in a plane parallel to the axis of the tunnel or inclined upstream at angles up to 60 degrees from the horizontal. Upheaval gages installed in the grouting gallery and anchored through pipes set in drill holes to the foundation at depths below that of the grout holes should be provided for to avoid excessive pressures. As this dam would develop a head of over 300 feet at ultimate stage, pressures of 150 pounds per square inch for the first stage grouting, and 300 pounds for second stage are recommended. The order of grouting for both stages would be to drill and grout holes first on 20-foot centers, then on 10-foot centers, etc., until an effective seal has been provided as determined by drilling and pressure testing.

22. Though extensively fractured throughout and cut by a deep, filled ravine in the vicinity of tunnel T.C., no evidence has been disclosed by the recent explorations that the right abutment rock mass is a slide block or that its stability is questionable because of some major structural weakness. The primary considerations here are deep stripping to secure sound foundation rock and careful grouting to provide uniformity in structural strength in the upper part of the foundation mass.

23. In the light of the geologic conditions disclosed by investigations made subsequent to my earlier report, it is concluded that no foundation conditions are present which would preclude the construction of a safe and effective concrete dam of the gravity type to the maximum height considered.

APPENDIX B - GEOLOGY AND FOUNDATION DATA

DETROIT DAM SITE - NORTH SANTIAM RIVER, OREGON

CHAPTER IV - LABORATORY TESTS OF FOUNDATION ROCK

CHART TO ACCOMPANY APPENDIX B - CHAPTER IV

Chart I. Shear Resistence of Foundation Rock.

TABLES TO ACCOMPANY APPENDIX B - CHAPTER IV

Table I. Physical Data - Foundation Rock.

Table II. Compression and Shear Test Data.

LABORATORY TESTS OF FOUNDATION ROCK

~~1.~~ ~~Summary of Test Results:~~ Results of laboratory tests show in Tables ~~I~~ 4 and ~~II~~ 5 and Chart ~~I~~ 2 ~~at the end of this chapter of the appendix~~ indicate that the unjointed foundation rock has a compression strength varying from 6,000 to 48,000 pounds per square inch; a shear-friction resistance of approximately 1,000 pounds per square inch plus 1.15 of the normal load; a modulus of elasticity varying from 4,770,000 to 9,440,000 pounds per square inch unrestrained; and a Poisson's ratio of from 0.18 to 0.28. Stress-strain field tests on undisturbed foundation rock in the tunnels indicate a modulus of elasticity varying from approximately 150,000 to 1,100,000 pounds per square inch and a Poisson's ratio of from 0.20 to 0.48.

APPENDIX B - ~~CHAPTER IV~~

TABLE ~~X~~ A

PHYSICAL DATA - FOUNDATION ROCK
DETROIT DAM SITE

Hole No.	Sample No.	Depth	Elevation	Description	Specific Gravity: Dry	Specific Gravity: Wet	Density: Dry	Density: Wet	Absorp. %	Modulus of Elasticity at Load 1000 lb./sq.in.: E	Modulus of Elasticity at Load 1000 lb./sq.in.: E_r	Poisson's Ratio
9	3	95.8	1496.6	Altered andesite	2.64		165.0		0.34			
16	4	99.8	1475.0	Altered andesite	2.71		169.4		0.63			
19	5	75.2	1567.9	Andesite diorite	2.59		162.1		0.75			
25	6	47.3	1357.3	Andesite diorite	2.61		163.2		0.56	4,770,000		
29	7	46.2	1394.0	Andesite	2.79	2.79	174.7	174.7	0.00			
32	8	73.0	1192.9	Andesite breccia	2.76		172.5		0.07			
39	9	45.7	1632.6	Andesite breccia	2.63		164.5		0.93			
42	10	44.4	1372.7	Diorite	2.57		160.7		0.90			
40	11	0.3	1201.2	Andesite	2.68		167.6		0.45			
40	12	2.1	1199.4	Andesite	2.69		168.2		0.23	6,630,000		
40	13	4.0	1197.5	Andesite breccia	2.70		168.9		0.05	6,290,000		
18	14	64.5	1457.3	Andesite diorite	2.70		168.9		0.84			
18	15	69.0	1452.8	Andesite diorite	2.63		164.5		0.65			
35	16	32.5	1195.9	Andesite (oxid.)	2.70		168.9		1.36			
35	17	34.0	1194.4	Andesite diorite	2.68		167.6		0.78			
35	18	37.5	1190.9	Altered andesite	2.67		166.9		0.78	9,890,000		
11	19	6.0	1270.4	Andesite	2.74		171.5		0.20			
11	20	8.5	1267.9	Andesite	2.82		176.2		0.42			
46	1	25.9	1183.2	Andesite	2.67	2.68	166.6	167.3	0.52	7,250,000	7,800,000	0.183
46	2	39.8	1169.3	Andesite diorite	2.63	2.66	164.1	166.1	1.09	5,320,000	5,815,000	0.186
51	3	25.1	1422.8	Andesite diorite	2.67	2.69	166.6	168.0	0.61	5,160,000	*	~~0.676~~
49	4	48.9	1322.4	Diorite	2.63	2.65	164.1	165.5	0.79	9,440,000	12,120,000	0.282

* Not computed — Specimen behaved abnormally.

average .22 1295 8/54,750,000 6,843,750 .217

5891

2.67

APPENDIX B - ~~CHAPTER IV~~

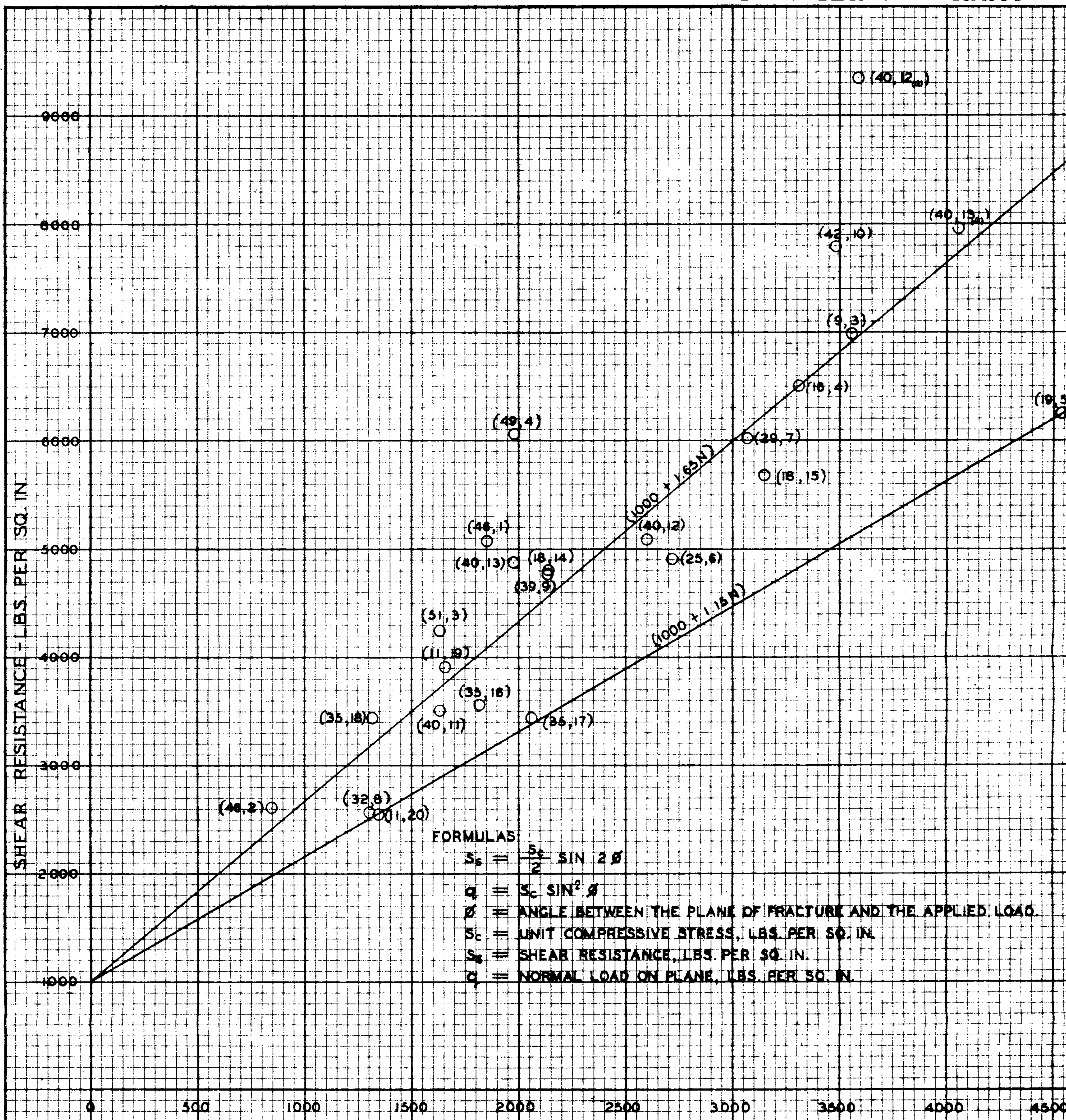
TABLE ~~II~~ 15

COMPRESSION TESTS
DETROIT DAM SITE

Hole Number	Sample Number	Depth	Elevation	Description	Compressive Strength Lbs./Sq. In.	Computed Shearing Strength Lbs./Sq. In.	Computed Normal Load Lbs./Sq. In.
9	3	95.8	1496.6	Altered andesite	17,300	6,990	3,560
16	4	99.8	1475.0	Altered andesite	16,100	6,500	3,310
19	5	75.2	1567.9	Andesite diorite	13,200	6,260	4,540
25	6	47.3	1357.3	Andesite diorite	11,600	4,910	2,720
29	7	46.2	1394.0	Andesite	14,900	6,020	3,070
32	8	73.0	1192.9	Andesite breccia	6,360	2,570	1,310
39	9	45.7	1632.6	Andesite breccia	12,870	4,770	2,140
42	10	44.4	1372.7	Diorite	21,000	7,800	3,480
40	11	0.3	1201.2	Andesite	9,190	3,510	1,635
40	12	2.1	1199.4	Andesite	12,610	5,090	2,600
40	13	4.0	1197.5	Andesite breccia	14,080	4,880	1,980
18	14	64.5	1457.3	Andesite diorite	12,900	4,800	2,140
18	15	69.0	1452.8	Andesite diorite	13,400	5,680	3,150
35	16	32.5	1195.9	Andesite oxidized	8,820	3,570	1,820
35	17	34.0	1194.4	Andesite diorite or altered oxid. andesite	7,770	3,440	2,060
35	18	37.5	1190.9	Altered andesite	10,320	3,440	1,320
11	19	6.0	1270.4	Andesite	10,850	3,910	1,660
11	20	8.5	1267.9	Andesite	6,130	2,540	1,350
46	1	25.9	1183.2	Andesite	15,830	5,089	1,852
46	2	39.8	1169.3	Diorite (andesite inc.)	8,920	2,614	852
51	3	25.1	1422.8	Andesite breccia	12,720	4,250	1,636
49	4	48.9	1322.4	Diorite	20,700	6,068	1,977
25	6a	47.3	1357.3	Andesite diorite	48,200	16,700	6,780
40	12a	2.1	1199.4	Andesite	28,000	9,300	3,580
40	13a	4.0	1197.5	Andesite breccia	19,700	7,880	4,060
35	18a	37.5	1190.9	Altered andesite	28,200	9,800	3,980

401,670 / 15,448

148,351 / 5,710



TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

NOTE:
NUMBERS INDICATE DRILL HOLE NO. & SAMPLE NO. RESPECTIVELY.
O-COMPRESION TEST, VALUES COMPUTED FROM ANGLE OF FRACTURE.

DH-25, SAMPLE 6(a)
NORMAL LOAD = 6,780
SHEAR RES. = 16,760

WILLAMETTE BASIN PROJECT. OREGON

DETROIT DAM AND RESERVOIR

SHEAR RESISTANCE OF FOUNDATION ROCK

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT
DRAWN BY: J.D.W.
TRACED BY: G.E.G.
DATE: 8-30-40

APPENDIX B

GEOLOGY AND FOUNDATION DATA

DETROIT DAM SITE
NORTH SANTIAM RIVER, OREGON

CHAPTER V

FIELD TESTS OF FOUNDATION ROCK

CHAPTER V

FIELD TESTS OF FOUNDATION ROCK

TABLE OF CONTENTS

TABLES TO ACCOMPANY

APPENDIX B, CHAPTER V

CHARTS TO ACCOMPANY

APPENDIX B, CHAPTER V

~~CHAPTER V~~

FIELD TESTS OF FOUNDATION ROCK

1. Tests and Purpose: Two foundation loading tests were completed on Detroit foundation rock, one in each of tunnels T-D and T-A. These tests were made to determine Poisson's ratio and the modulus of elasticity for the foundation rock in place.

~~2. Location of Tests:~~ The test in tunnel T-D was located at Station 0/37 about 30 feet from the face of the cliff at an approximate elevation of 1300 feet, and at local damsite coordinates N. 1387, W. 1089. The ~~grid~~ bearing of the tunnel is N 87° E ~~N. 58° 38' E.~~

~~3.~~ In tunnel T-A the test was set up at Station 0/97 approximately 60 feet from exposed rock face, elevation 1237 feet, and coordinates N. 1358, W. 899. The tunnel bears N 6° 33' W ~~S. 87° 00' W.~~

~~4.~~ The material in both tunnels is a dense fine grained andesite. In the area tested in tunnel T-D there is evidence of considerable jointing on three planes, with some joints spaced as close as one foot. In tunnel T-A the test was made in a section where the rock indicated large blocky jointing and no fracturing. Chart 1 shows the location of the ~~The predominant~~ fractures and joints in the test areas. ~~are shown on Appendix "B", Plate V, Sheet 5.~~

5. Equipment and Setup: Chart I of Appendix "B", Chapter V and figure 1 show the arrangement of the equipment. Load was applied in a horizontal direction normal to the tunnel center line with a hydraulic jack of 100-ton capacity, to which a pressure gage was attached. Pressure in the jack was varied by means of a hand operated pump. The bearing areas consisted of steel plates, 13-9/16" diameter, 3/4 inch thick transmitting the load from a 12 x 12 timber to a concrete bearing

block, and one 13-9/16" diameter and $2\frac{1}{2}$ inches thick transmitting and distributing the load from a $1\frac{1}{2}$ inch diameter steel ball to a concrete block. Each of these plates was imbedded in a concrete block mounted on the tunnel wall. The thickness of concrete was kept at a minimum, 4 to 6 inches being sufficient to cover projecting rock and produce a bearing surface.

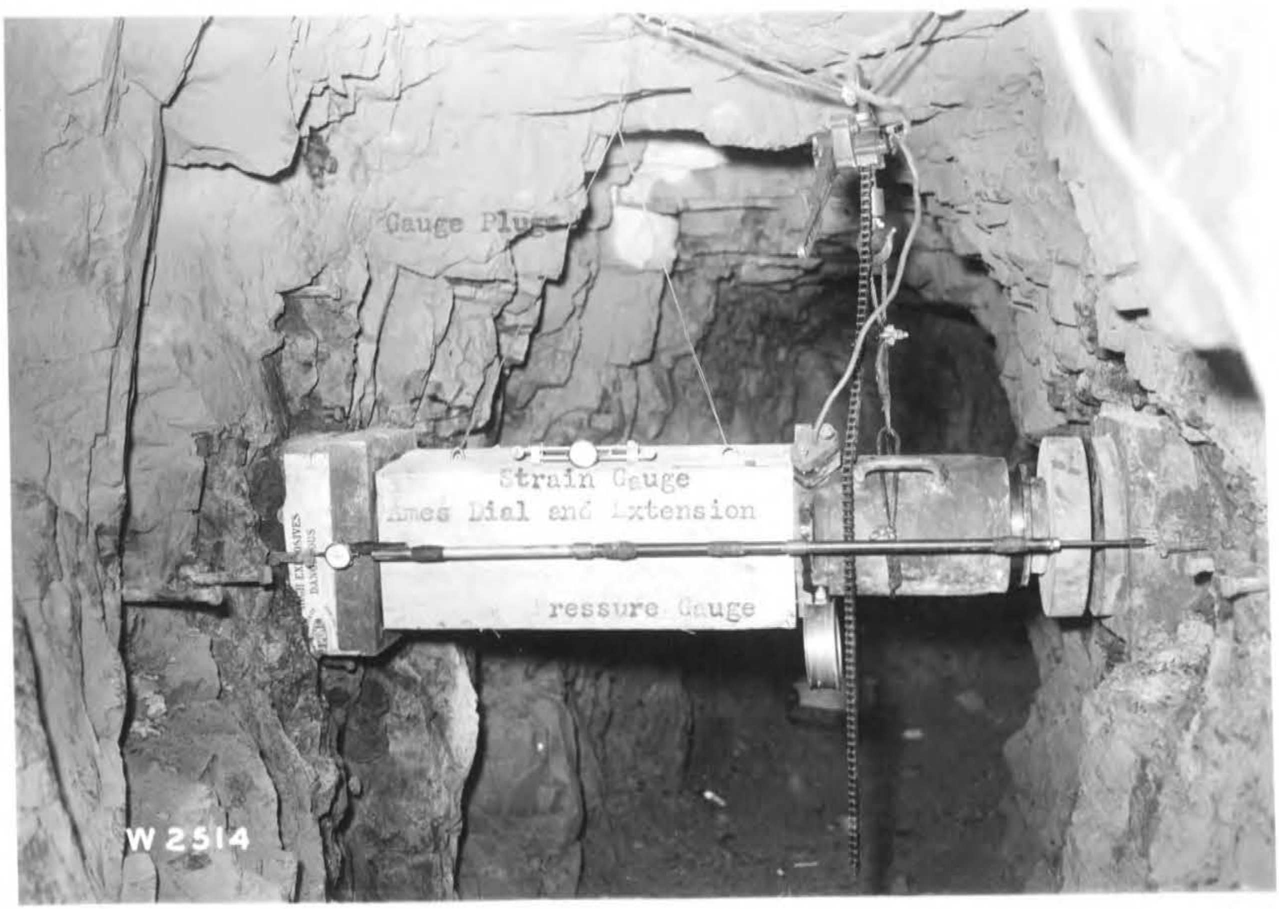


Fig. 1. Test Equipment Mounted in Tunnel T-D.

11

6. Gauge plugs, 3/4-inch machine bolts, were located and designated as shown by Chart ~~X~~ 3. At least 6-inches of each bolt was grouted into drill holes with high-early strength cement and sand grout. The bolt heads were aligned in a vertical plane parallel to the tunnel center line and as near the wall as possible. Rock between the gauge plugs, projecting beyond this plane, was trimmed to afford clearance for the strain gauge. A complete set of plugs was installed on each wall, one set directly opposite the other. Holes for strain gauge points were drilled in the gauge plugs at distances indicated. The distance between plugs 1, 2 and 3 was made equal to the gauge length of a 10-inch strain gauge. The distance between plugs 3 and 4 was made equal to the distance between the two parallel planes of bolt heads.

7. <u>Deformation Measurements</u>: Radial deformation, movement in the plane of the walls towards the loaded area, was measured with a 10-inch Whittemore Strain Gauge between points 1, 2 and 3. Radial deformation between points 3 and 4 and wall deformation, separation of the tunnel walls in the direction of the applied load, were measured with an Ames Dial attached to an extension. The extension was made up of 1/2-inch pipe and was provided with a hardened tapered steel point which could be adjusted for various tunnel widths. A thermometer was also attached to the pipe.

8. <u>Load</u>: The total load applied was determined by readings on a pressure gage attached to the hydraulic jack. This total load was uniformly distributed over a circular area of 144.5 sq. inches.

9. Tests Completed: Radial and wall deformations were observed for the following loadings:

Test No.	Loadings lbs./sq. in.	Time - Beginning to End of Test Hours
In Tunnel T-D.		
1	0 to 210 to 0	1
2	0 to 367 to 0	1
3	0 to 806 to 1012 to 1382 to 1012 to 806 to 592 to 367 to 210 to 0	5½
In Tunnel T-A.		
1	0 to 210 to 0	6
2	0 to 518 to 0	18
3	0 to 1382 to 0	5

10. Nomenclature:

W_1, W_2, W_3 = Wall deformation of points 1, 2 and 3 in the direction of applied load, in inches. Assumed equal to one-half the measured wall separation.

W_{1-2} = Wall deformation W_1 minus wall deformation W_2.

U_1, U_2, U_3 = Radial deformation of points 1, 2 and 3, in inches.

U_{1-2} = Radial deformation of point 1 minus radial deformation of point 2.

σ = Poisson's ratio.

E = Modulus of elasticity in pounds per sq. inch.

q = Intensity of load in pounds per square inch on loaded circle.

a = Radius of the loaded circle. For 13 9/16" diameter, a is 6.781 inches.

r_1, r_2, r_3 = Radius in inches to points 1, 2 and 3.

E_1, K_1 = Complete elliptical integrals for point 1.

k_1, k_2, k_3 = Constant $\frac{a}{r_1}, \frac{a}{r_2}, \frac{a}{r_3}$.

11. Equations: The observed deformations were substituted in the following equations. When solved simultaneously, these equations give values for Poisson's ratio and the modulus of elasticity.

a. For wall deformation (W) in the direction of an applied load.

$$W = \frac{4(1-\sigma^2)qr}{\pi E}\left[\int_0^{\frac{\pi}{2}}\sqrt{1-\frac{a^2\sin^2\theta}{r^2}}\,d\theta - \left(1-\frac{a^2}{r^2}\right)\int_0^{\frac{\pi}{2}}\frac{d\theta}{\sqrt{1-\frac{a^2\sin^2\theta}{r^2}}}\right]$$

See "Load distributed over a part of the boundary of a semi-infinite object." - "Theory of Elasticity" by Timoshenko, p. 334, 1st Edition.

b. For radial deformation (U).

$$U = -\frac{(1-2\sigma)(1+\sigma)\,q\,a^2}{2Er}$$

Equation derived by H. D. Eberhart from equation by A. E. H. Love. Equation is identical to that given by Timoshenko for a concentrated load (P) on a semi-infinite solid. Value of $P = q\,\pi\,a^2$ ("Theory of Elasticity", page 332, 1st Edition).

12. Constants combined:

a. $$W = \frac{4(1-\delta^2)qr}{\pi E}\left[\int_0^{\frac{\pi}{2}}\sqrt{1-\frac{a^2\sin^2\theta}{r^2}}\,d\theta - \left(1-\frac{a^2}{r^2}\right)\int_0^{\frac{\pi}{2}}\frac{d\theta}{\sqrt{1-\frac{a^2\sin^2\theta}{r^2}}}\right]$$

$$= \frac{4(1-\delta^2)qr}{\pi E}\left[E_1 - \left(1-\frac{a^2}{r^2}\right)K_1\right]$$

E_1 and K_1 being elliptical integrals, see "A Short Table of Integrals" Peirce p. 121.

(1) W_1, For all No. 1. Points. $r_1 = 10''$.

$$k_1 = \frac{a}{r_1} = \frac{6.781}{10} = .6781$$

$$\sin^{-k_1} = 42^\circ - 42'$$

$$(E_1)_1 = 1.3705$$

$$(K_1)_1 = 1.8216$$

$$W_1 = \frac{4(1-\delta^2)\,q\,10}{3.1416\ E}\left[1.3705 - 1.8216\left(1-\frac{6.781^2}{10^2}\right)\right]$$

$$= \frac{4.93(1-\delta^2)\,q}{E}$$ — — — — — — —

(2) W_2, For $r_2 = 20''$

$$k_2 = \frac{6.781}{20} = .339$$

$$\sin^{-k_2} = 19^\circ - 49'$$

$$(E_1)_2 = 1.5246$$

$$(K_1)_2 = 1.6191$$

$$W_2 = \frac{4(1-\delta^2)\,q\,20}{3.1416\ E}\left[1.5246 - 1.6191\left(1-\frac{6.781^2}{20^2}\right)\right]$$

$$= 2.33\,\frac{(1-\delta^2)\,q}{E}$$ — — — — — — —

(3) W_3, For $r_3 = 30''$

$$k_3 = \frac{6.781}{30} = .226$$

$$\sin^{-1} k_3 = 13^{\circ}\text{-}04'$$

$$(E_1)_3 = 1.5505$$

$$(K_1)_3 = 1.5915$$

$$W_3 = \frac{4(1-\delta^2)\, q\, 30}{3.1416\, E} \left[1.5505 - 1.5915 \left(1 - \frac{6.781^2}{30^2} \right) \right]$$

$$= \frac{1.54\,(1-\delta^2)\, q}{E}$$

(4) $$W_{1-3} = \frac{4.93(1-\delta^2)q}{E} - \frac{2.33\,(1-\delta^2)\, q}{E}$$

$$= \frac{2.60\,(1-\delta^2)\, q}{E}$$

b. $$U = -\frac{(1-2\delta)(1+\delta)\, q\, a^2}{2\, E\, r}$$

(1) U_1, for $r_1 = 10''$

$$U_1 = -\frac{(1-2\delta)(1+\delta)\, q\ 6.781^2}{2\, E\ 10}$$

$$= -\frac{2.299\,(1-2\delta)(1+\delta)\, q}{E}$$

(2) U_2, for $r_2 = 20''$

$$U_2 = -\frac{(1-2\delta)(1+\delta)\, q\, a^2}{2\ E\ r_2}$$

$$= -\frac{1.149\,(1-2\delta)(1+\delta)\, q}{E}$$

(3) U_3, for $r_3 = 30''$

$$U_3 = -\frac{(1-2\delta)(1+\delta)\, q\, a^2}{2\ E\ r_3}$$

$$= -\frac{0.766\,(1-2\delta)(1+\delta)\, q}{E}$$

(4) $$U_{1-2} = -\frac{(1-2\delta)(1+\delta)\, q\, a^2}{2\, E} \left(\frac{1}{r_1} - \frac{1}{r_2} \right)$$

$$= -\frac{1.149\,(1-2\delta)(1+\delta)\, q}{E}$$

(5) $$U_{2-3} = -\frac{(1-2\delta)(1+\delta)\, q\, a^2}{2\, E} \left(\frac{1}{r_2} - \frac{1}{r_3} \right)$$

$$= -\frac{0.383\,(1-2\delta)(1+\delta)\, q}{E}$$

13. Equations Combined.

a. Solve for δ using observed U_1 and W_1. Divide W_1 by $-U_1$.

$$\frac{W_1}{-U_1} = \left[\frac{4.93\,(1-\delta^2)\,q}{E}\right]\left[\frac{E}{2.299\,(1-2\delta)(1+\delta)\,q}\right]$$

$$\frac{W_1}{-U_1} = \frac{4.93\,(1-\delta)}{2.299\,(1-2\delta)}$$

$$W_1\,(1-2\delta) = -U_1(2.15)(1-\delta)$$

$$W_1 - 2W_1\delta = 2.15\,U_1\delta - 2.15\,U_1$$

$$2W_1\delta + 2.15\,U_1\delta = 2.15\,U_1 + W_1$$

$$\delta(2W_1 + 2.15\,U_1) = W_1 + 2.15\,U_1$$

$$\delta = \frac{W_1 + 2.15\,U_1}{2W_1 + 2.15\,U_1}$$

b. Solve for δ using observed U_{1-2} and W_1. Divide W_1 by $-U_{1-2}$.

$$\frac{W_1}{-U_{1-2}} = \left[\frac{4.93\,(1-\delta^2)\,q}{E}\right]\left[\frac{E}{1.149\,(1-2\delta)(1+\delta)\,q}\right]$$

$$W_1\,(1-2\delta) = -U_{1-2}(4.29)(1-\delta)$$

$$W_1 - 2W_1\delta = 4.29\,U_{1-2}\,\delta - 4.29\,U_{1-2}$$

$$2W_1\delta + 4.29\,U_{1-2}\,\delta = 4.29\,U_{1-2} + W_1$$

$$\delta = \frac{W_1 + 4.29\,U_{1-2}}{2W_1 + 4.29\,U_{1-2}}$$

c. Solve for δ using observed U_{1-2} and W_{1-2}. Divide W_{1-2} by $-U_{1-2}$.

$$\frac{W_{1-2}}{-U_{1-2}} = \left[\frac{2.60\,(1-\delta^2)\,q}{E}\right]\left[\frac{E}{1.149\,(1-2\delta)(1+\delta)\,q}\right]$$

$$\frac{W_{1-2}}{-U_{1-2}} = \frac{2.26\,(1-\delta)}{1-2\delta}$$

$$W_{1-2} - 2\delta W_{1-2} = 2.26\,\delta\,U_{1-2} - 2.26\,U_{1-2}$$

$$\delta = \frac{W_{1-2} + 2.26\,U_{1-2}}{2W_{1-2} + 2.26\,U_{1-2}}$$

14. <u>Observed Values</u>: Tables 6 ~~I~~ and 7 ~~IV~~, and charts ~~II~~ to ~~V~~ inclusive summarize all measurements made in tunnels T-D and T-A. Values which deviated excessively have not been included in these averages. Individual radial strain gauge measurements are believed to be accurate to ± .0002 inch. Wall separation measurements probably vary in a range of ± .0005 inch.

15. <u>Tunnel T-D Results</u>: Referring to Table 6 ~~I~~ and charts 4 ~~II~~ and 5 ~~III~~ for tunnel T-D, it will be noted that the values for radial deformation on the upstream or right wall indicate considerable increase in distance between points 2 and 3 and a decrease between points 1 and 2. The greater part of this movement can probably be attributed to closing of the joints in the rock. The downstream or left wall produced radial deformations of much less magnitude and in the direction of the loaded area, indicating that the rock in this wall was not jointed as extensively as the opposite wall. Comparison of radial deformation on the right and left walls also indicates that the greater part of the wall separation probably occurred in the right wall. Wall separation was measured from wall to wall each reading being the sum of the deformation of both walls.

16. For evaluating Poisson's ratio and the modulus of elasticity of rock in Tunnel T-D, average values for radial movement observed on the downstream or left wall, and average values of the observed wall separation for one wall were applied. Since the deformation was greatest on the points close to the load, values, shown in tables 7 ~~II~~ and 8 ~~III~~, were determined either from the deformation at point I or the difference in deformation between points 1 and 2. By using the

differences, effect of permanent set is reduced to that occurring in a 10" gauge length. Values are computed from total deformation occurring on the application of each load, and for deformation recovered on release of the load. Deformation caused by an applied load may include sliding on joints and permanent set in addition to elastic deformation. When the load is removed, the recovered deformation may include sliding on joints in addition to elastic deformation.

17. Tunnel T-A Results: Table ~~IV~~ 9 and charts ~~IV~~ 6 and ~~V~~ 7 show the results obtained in tunnel T-A. Comparison of the radial deformation in the two walls shows that the movement is more uniform in direction and magnitude and that it is not as great for similar loadings as obtained on the left wall of Tunnel T-D. In tunnel T-A the radial deformation on the left, or river wall, shown on Chart ~~IV~~ 6, presents the best example of uniform elastic deformation observed. Very little permanent set is evident on release of load. The right or hill wall shows some evidence of sliding along a joint and a slight permanent shift in position due to load.

18. Average wall separation in tunnel T-A for a maximum load of 1,382 lbs. per square inch was less than one-quarter of that obtained in tunnel T-D. The cumulated permanent set or wall deformation at the completion of test No. 3 was less than one-eighth of that obtained in tunnel T-D.

19. Values for Poisson's ratio and the modulus of elasticity of the rock in tunnel T-A were determined by substitution in the equations using the difference of observed deformations of points 1 and 2, table 10 ~~V~~, sheet 2. In addition it was possible to measure the radial movement

between points I-3 and I-4. Assuming that point 4 remained fixed, the total radial movement of points 1, 2 and 3 could be determined. Results using the total radial movement and the wall deformation of point 1, excluding cumulative permanent set, are shown in Table ~~V~~ 10, sheet 1. Table ~~VI~~ 11 summarizes the results obtained in Tunnel T-A.

20. Permanent Set: Permanent set, deformation not recovered on release of load for wall separation in these two locations are plotted on Chart ~~VI~~ 8 for various distances from the center of the loaded area. It will be noted that the permanent set in Tunnel T-D is much greater than in Tunnel T-A. In Tunnel T-D the amount of permanent set increases with the load, but in Tunnel T-A the permanent set is approximately constant for loads from 500 to 1382 pounds per square inch.

21. Summary: Values for Poisson's ratio and the modulus of elasticity derived from these field tests are tabulated below.

Load per Square in.	Poisson's Ratio	Modulus of Elasticity	Cumulative Permanent Set, Wall Deformation, 10 inches from Center of Loaded Area in in.
Tunnel T-D.			
Load applied(*)			
210	0.34	147,000	.0022
367	0.48	301,000	.0036
806	0.46	228,000	
1,012	0.46	218,000	.0078
1,382	0.46	224,000	
Load released (**)			
210	0.35	200,000	
367	0.31	543,000	
806	0.46	195,000	
1,012	0.45	220,000	
1,382	0.43	286,000	
Tunnel T-A.			
Load applied(*)			
210	0.23	750,000	.0004
518	0.36	880,000	.0009
1,382	0.38	1,080,000	.0007

Load released (**)			
210	0.41	680,000	
518	0.44	1,020,000	
1,382	0.32	1,100,000	

(*) Values are derived from measured deformations for each load which include only a portion of the cumulative permanent set.

(**) Values are derived from deformation recovered on release of load which do not include permanent set.

21. It is believed that the most of the movement recorded was caused by sliding along and closing up of joints rather than elastic deformation. Laboratory compression tests on sound 4-1/8" cores from drill holes indicate a Poisson's ratio of about 0.18 to 0.25 and an unrestricted modulus of elasticity varying from about 4,000,000 to 9,000,000 pounds per square inch.

APPENDIX B - CHAPTER V

TABLE I
DETROIT DAM SITE
TUNNEL T-D

SUMMARY OF FOUNDATION LOADING TEST DATA

Test No.	Loading Lbs./Sq. In.		RADIAL DEFORMATION IN INCHES (*)				DEFORMATION OF WALL (*) In Direction of Load, In.			
			RIGHT WALL		LEFT WALL					
	Initial	Final	U_1 to 2	U_2 to 3	U_1 to 2	U_2 to 3	W_1	W_2	W_3	W_4
	Load Increase from 0 Lbs./Sq. In. These values include permanent set produced during each test but do not include cumulative permanent set resulting from previous loadings.									
1	0	210	+.0005	+.0008	+.0007	+.0004	.0063	.0030	.0020	.0007
2	0	367	-.0004	+.0007	+.0001	-.0003	.0052	.0030	.0017	.0002
3	0	806	-.0015	+.0027	+.0004	+.0001	.0136	.0067	.0035	.0003
	0	806	-.0017	+.0028	+.0004	+.0001	.0145	.0077	.0039	.0009
	0	1,012	-.0016	+.0033	+.0006	+.0003	.0177	.0085	.0040	.0006
	0	1,012	-.0018	+.0034	+.0006	+.0001	.0186	.0094	.0045	.0011
	0	1,382	-.0018	+.0041	+.0008	+.0002	.0246	.0121	.0061	.0011
	Load Decreased to 0 Lbs./sq. in. These values indicate deformation recovered on release of load and do not include permanent set.									
1	210	0	.0000	+.0003	+.0005	.0000	.0042	.0016	.0010	.0000
2	367	0	-.0008	+.0082	+.0004	.0001	.0039	.0025	.0015	.0002
3	1,382	0	-.0001	+.0031	+.0010	.0005	.0199	.0100	.0054	.0003
	1,012	0	.0000	+.0027	+.0008	.0001	.0182	.0087	.0045	.0002
	806	0	-.0004	+.0028	+.0005	.0002	.0162	.0077	.0041	.0001
	592	0	-.0003	+.0022	+.0007	.0002	.0140	.0065	.0035	.0003
	367	0	-.0002	+.0013	+.0006	.0001	.0096	.0045	.0024	.0002
	210	0	-.0004	+.0011	+.0008	.0001	.0080	.0037	.0016	.0001

Note: A plus sign (+) preceding values for U_{1-2} and U_{2-3} indicates spreading, due to the load, between these points.

(*) Each value is an average of one or more readings on four similar gage lengths.

APPENDIX B, CHAPTER V

TABLE II

DETROIT DAM SITE
TUNNEL T-D
COMPUTATIONS
SHEET: 1 of 2

Solve for σ and E $\qquad \sigma = \dfrac{W_{1-2} + 2.26\ U_{1-2}}{2W_{1-2} + 2.26\ U_{1-2}} \qquad E = \dfrac{2.60\ (1-\sigma^2)\ q}{W_{1-2}}$

Test No.	Load q Lbs./Sq.In.	W_{1-2}	U_{1-2} Left Wall	$2.26\ U_{1-2}$	$W_{1-2} + 2.26 U_{1-2}$	$2W_{1-2}$	$2W_{(1-2)} + 2.26 U_{1-2}$	Poissons Ratio σ	σ^2	$1-\sigma^2$	$E = \dfrac{2.60\ (1-\sigma^2) q}{W_{1-2}}$
		For Increasing Load. 0 Lbs./sq. in. to load indicated. Values include deformation and permanent set.									
1	210	.0033	-.0007	-.0016	.0017	.0066	.0050	.34	.11	.89	147,000
2	367	.0022	-.0001	-.0002	.0020	.0044	.0042	.48	.23	.77	334,000
3	806	.0068	-.0004	-.0009	.0059	.0136	.0127	.46	.21	.79	243,000
3	1012	.0092	-.0006	-.0014	.0078	.0184	.0170	.46	.21	.79	226,000
3	1382	.0125	-.0008	-.0018	.0107	.0250	.0232	.46	.21	.79	227,000
		For Decreasing Load. Values include deformation recovered on release of load.									
1	210	.0026	-.0005	-.0011	.0015	.0052	.0041	.37	.14	.86	180,000
2	367	.0013	-.0004	-.0009	.0004	.0026	.0015	.27	.07	.93	682,000
3	806	.0085	-.0005	-.0011	.0074	.0170	.0159	.46	.21	.79	195,000
3	1012	.0095	-.0008	-.0018	.0077	.0190	.0172	.45	.20	.80	228,000
3	1382	.0099	-.0010	-.0023	.0076	.0198	.0175	.43	.19	.81	294,000

APPENDIX B, CHAPTER V

TABLE 11
(CONT)
DETROIT DAM SITE
TUNNEL T-D
COMPUTATIONS
SHEET: 2 of 2

Solve for δ and E. $\delta = \frac{4.29\ U_{1-2} + W_1}{2W_1 + 4.29\ U_{1-2}}$ $E = \frac{4.93\ (1 - \delta^2)\ q}{W_1}$

Test No.	Load q Lbs./Sq.In.	W_1	U_{1-2} Left Wall	$4.29 \times U_{1-2}$	$W_1 + 4.29\ U_{1-2}$	$2W_1$	$2W_1 + 4.29\ U_{1-2}$	Poissons Ratio δ	δ^2	$1-\delta^2$	$E = \frac{4.93(1-\delta^2)q}{W_1}$
For Increasing Load. 0 Lbs./sq. in. to load indicated. Values include deformation and permanent set.											
1	210	.0063	-.0007	-.0030	.0033	.0126	.0096	.34	.11	.89	146,000
2	367	.0052	-.0001	-.0004	.0048	.0104	.0100	.48	.23	.77	268,000
3	806	.0145	-.0004	-.0017	.0128	.0290	.0273	.47	.22	.78	214,000
3	1012	.0186	-.0006	-.0026	.0160	.0372	.0346	.46	.21	.79	211,000
3	1382	.0246	-.0008	-.0034	.0212	.0492	.0458	.46	.21	.79	220,000
For Decreasing Load. Values include deformation recovered on release of load.											
1	210	.0042	-.0005	-.0021	.0021	.0084	.0063	.33	.11	.89	220,000
2	367	.0039	-.0004	-.0017	.0022	.0078	.0061	.36	.13	.87	405,000
3	806	.0162	-.0005	-.0021	.0141	.0324	.0303	.46	.21	.79	194,000
3	1012	.0182	-.0008	-.0034	.0148	.0364	.0330	.45	.20	.80	219,000
3	1382	.0199	-.0010	-.0043	.0156	.0398	.0355	.44	.19	.81	278,000

APPENDIX B, CHAPTER V

TABLE ~~III~~ 10

DETROIT DAM SITE
TUNNEL T-D
SUMMARY OF COMPUTED VALUES FOR POISSON'S RATIO
AND THE MODULUS OF ELASTICITY

Load	Measurements Used in Computations	Poisson's Ratio	Modulus of Elasticity
Load Applied			
210	W_{1-2} & U_{1-2}	.34	147,000
	W_1 & U_{1-2}	.34	146,000
	Ave.	.34	147,000
367	W_{1-2} & U_{1-2}	.48	334,000
	W_1 & U_{1-2}	.48	266,000
	Ave.	.48	301,000
806	W_{1-2} & U_{1-2}	.46	243,000
	W_1 & U_{1-2}	.47	214,000
	Ave.	.46	228,000
1012	W_{1-2} & U_{1-2}	.46	226,000
	W_1 & U_{1-2}	.46	211,000
	Ave.	.46	218,000
1382	W_{1-2} & U_{1-2}	.46	227,000
	W_1 & U_{1-2}	.46	220,000
	Ave.	.46	224,000
Load Released			
210	W_{1-2} & U_{1-2}	.37	180,000
	W_1 & U_{1-2}	.33	220,000
	Ave.	.35	200,000
367	W_{1-2} & U_{1-2}	.27	682,000
	W_1 & U_{1-2}	.36	405,000
	Ave.	.31	543,000
806	W_{1-2} & U_{1-2}	.46	195,000
	W_1 & U_{1-2}	.46	194,000
	Ave.	.46	195,000
1012	W_{1-2} & U_{1-2}	.45	222,000
	W_1 & U_{1-2}	.45	219,000
	Ave.	.45	220,000
1382	W_{1-2} & U_{1-2}	.43	294,000
	W_1 & U_{1-2}	.44	278,000
	Ave.	.43	286,000

TABLE IV

DETROIT DAM SITE
TUNNEL T-A

SUMMARY OF FOUNDATION LOADING TEST DATA

Test No.	Loading Lbs./Sq. In. Initial	Final	RADIAL DEFORMATION IN INCHES* River Wall, Left U_1 to 2	U_2 to 3	U_3 to 4	Hill Wall, Right U_1 to 2	U_2 to 3	U_3 to 4	DEFORMATION OF WALL* IN DIRECTION OF LOAD, INCHES W_1	W_2	W_3	W_4
	Load Increased from 0 Lbs./sq. in. These values include permanent set produced by load during each test, but do not include cumulative permanent set resulting from previous loadings.											
1	0	210	+.0002	.0000	+.0006	.0000	+.0002	+.0003	.0015	.0009	.0007	.0001
2	0	518	+.0002	+.0001	+.0003	-.0002	+.0002	+.0002	.0024	.0010	.0005	.0002
3	0	1,382	+.0003	+.0005	+.0005	-.0004	+.0006	+.0008	.0053	.0024	.0012	.0005
	Load decreased to 0 Lbs./sq. in. These values indicate deformation recovered on release of load and do not include permanent set.											
1	210	0	+.0001	-.0002	+.0003	.0000	+.0001	+.0003	.0011	.0003	.0002	.0001
2	518	0	+.0002	+.0002	-.0005	-.0002	+.0003	-.0001	.0019	.0007	.0003	.0001
3	1,382	0	+.0003	+.0005	+.0011	-.0004	+.0006	+.0006	.0055	.0026	.0013	.0004

Note: A plus sign (+) preceding values for U_1 to 2, U_w to 3, and U_3 to 4 indicates spreading, due to load, between these points.

*: Each value is an average of one or more readings on four similar gage lengths.

APPENDIX B, CHAPTER V

TABLE V
DETROIT DAM SITE
TUNNEL T-A

COMPUTATIONS
SHEET: 1 of 2

Solve for σ and E. $\sigma = \dfrac{2.15U_1 + W_1}{2W_1 + 2.15U_1}$, $E = \dfrac{4.93(1-\sigma^2)q}{W_1}$

Test No.	Load q Lbs./Sq. In.	W_1 + 2.15 U_1: W_1	U_1 Left Wall	$2.15U_1$	$W_1+2.15U_1$	$2W_1$ + 2.15U_1: $2W_1$	$2W_1+2.15U_1$	Poisson's Ratio σ	σ^2	$1-\sigma^2$	$E = \dfrac{4.93(1-\sigma^2)q}{W_1}$
For increasing load. 0 Lbs./sq. in. to load indicated. Values include deformation and permanent set.											
1	210	.0015	-.0005	-.0011	.0004	.0030	.0019	.21	.04	.96	660,000
2	518	.0024	-.0006	-.0013	.0011	.0048	.0035	.31	.09	.91	970,000
3	1,382	.0053	-.0013	-.0027	.0026	.0106	.0079	.33	.11	.89	1,150,000
For decreasing load. Values include deformation recovered on release of load.											
1	210	.0011	-.0002	-.0004	.0007	.0022	.0018	.39	.15	.85	800,000
2	518	.0019	-	-	.0019	.0038	.0038	.50	.25	.75	1,000,000
3	1,382	.0055	-.0019	-.0041	.0014	.0110	.0069	.20	.04	.96	1,190,000

APPENDIX B, CHAPTER V

TABLE V
(CONTINUED)
DETROIT DAM SITE
TUNNEL T-A

COMPUTATIONS
SHEET: 2 of 2

Solve for σ and E. $\sigma = \dfrac{W_{1-2} + 2.26\, U_{1-2}}{2W_{1-2} + 2.26\, U_{1-2}}$, $E = \dfrac{2.60\,(1-\sigma^2)\,q}{W_{1-2}}$

Test No.	Load q Lbs./Sq.In.	W_{1-2} + 2.26 U_{1-2}: W_{1-2}	U_{1-2} Left Wall	$2.26U_{1-2}$	W_{1-2} + $2.26U_{1-2}$	$2W_{1-2}$ + $2.26U_{1-2}$: $2W_{1-2}$	$2W_{(1-2)}$ + $2.26\,U_{1-2}$	Poisson's Ratio σ	σ^2	$1-\sigma^2$	$E = \dfrac{2.60(1-\sigma^2)\,q}{W_{1-2}}$
For Increasing Load. 0 Lbs./sq. in. to load indicated. Values include deformation and permanent set.											
1	210	.0006	-.0002	-.0004	.0002	.0012	.0008	.25	.06	.94	850,000
2	518	.0014	-.0002	-.0004	.0010	.0028	.0024	.42	.18	.82	790,000
3	1,382	.0029	-.0003	-.0007	.0022	.0058	.0051	.43	.18	.82	1,020,000
For Released Load. Load in Lbs./sq. in. to 0 lbs./sq. in. Values include deformation recovered on release of load.											
1	210	.0008	-.0001	-.0002	.0006	.0016	.0014	.43	.18	.82	560,000
2	518	.0011	-.0002	-.0004	.0007	.0022	.0018	.39	.15	.85	1,040,000
3	1,382	.0029	-.0003	-.0007	.0022	.0058	.0051	.43	.18	.82	1,020,000

APPENDIX B, CHAPTER V

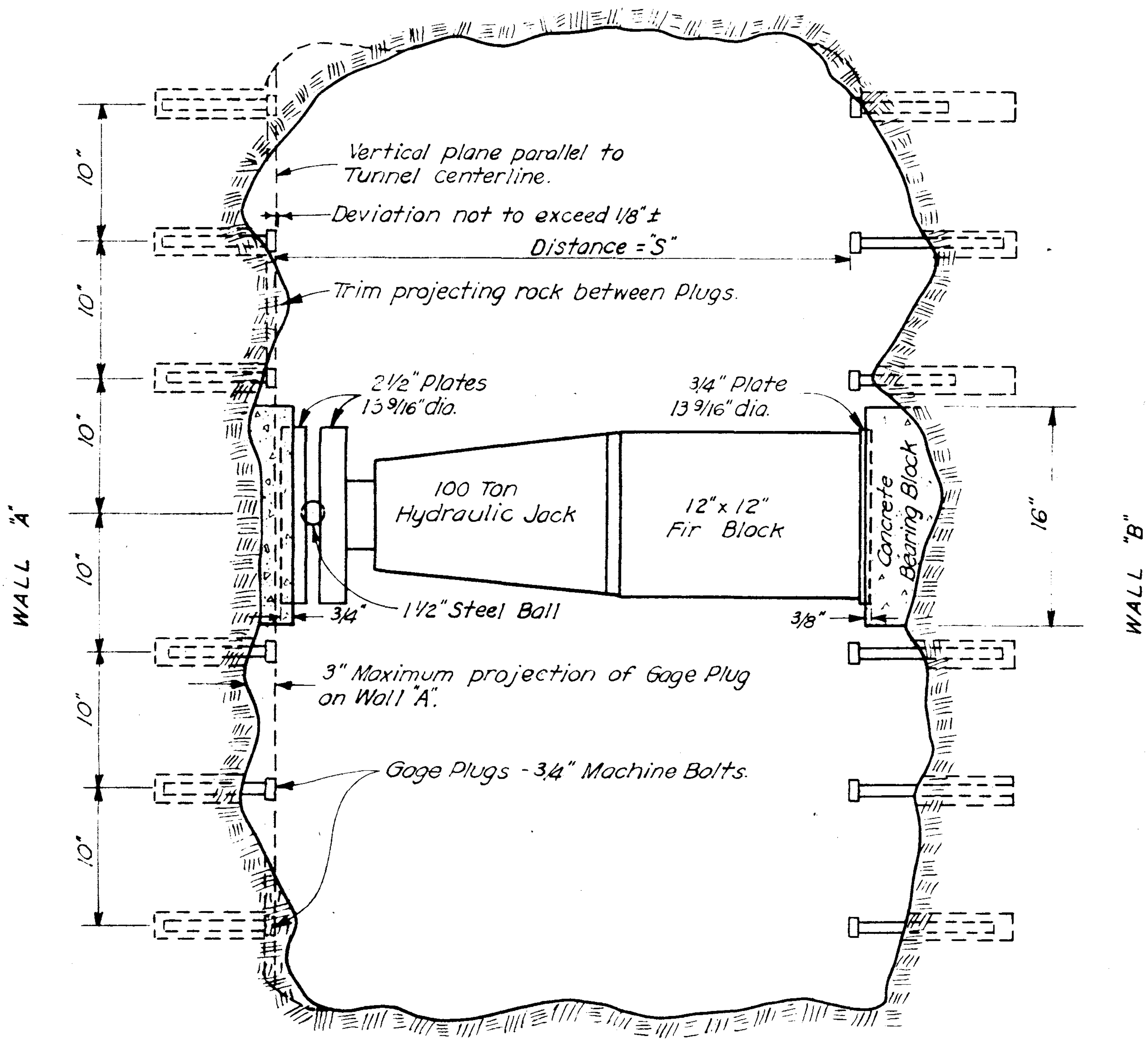
TABLE ~~VI~~

DETROIT DAM SITE
TUNNEL T-A

SUMMARY OF COMPUTED VALUES FOR POISSON'S RATIO

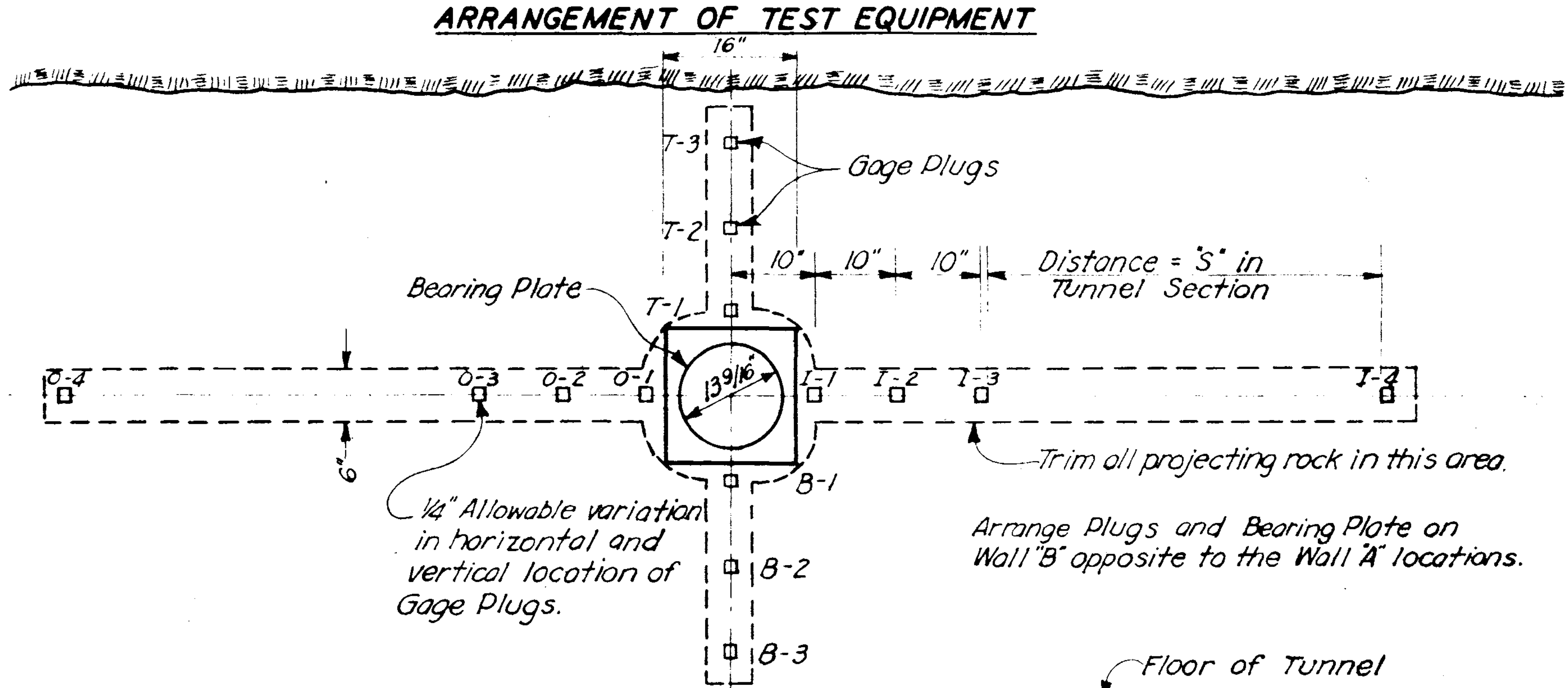
AND THE MODULUS OF ELASTICITY

Load - Lbs./ Sq. Inch	Measurements Used in Computations	Poisson's Ratio	Modulus of Elasticity
Load Applied.			
210	W_1 & U_1	.21	660,000
	W_{1-2} & U_{1-2}	.25	850,000
	Ave.	.23	750,000
518	W_1 & U_1	.31	970,000
	W_{1-2} & U_{1-2}	.42	790,000
	Ave.	.36	880,000
1382	W_1 & U_1	.33	1,150,000
	W_{1-2} & U_{1-2}	.43	1,020,000
	Ave.	.38	1,080,000
Load Released.			
210	W_1 & U_1	.39	800,000
	W_{1-2} & U_{1-2}	.43	560,000
	Ave.	.41	680,000
518	W_1 & U_1	.50	1,000,000
	W_{1-2} & U_{1-2}	.39	1,040,000
	Ave.	.44	1,020,000
1382	W_1 & U_1	.20	1,190,000
	W_{1-2} & U_{1-2}	.43	1,020,000
	Ave.	.32	1,100,000



SECTION THROUGH TUNNEL

ARRANGEMENT OF TEST EQUIPMENT



ARRANGEMENT OF GAGE PLUGS AND BEARING AREA

Note:

Plugs are numbered 1, 2, 3, & 4 radially from Bearing Plate.
T indicates location above Bearing Plate.
I " " ahead on Tunnel stationing.
B " " below Bearing Plate.
O " " back on Tunnel stationing from Bearing Plate.

TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

WILLAMETTE BASIN PROJECT, OREGON

DETROIT DAM AND RESERVOIR

FOUNDATION LOADING TEST

SET UP OF TEST EQUIPMENT

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT

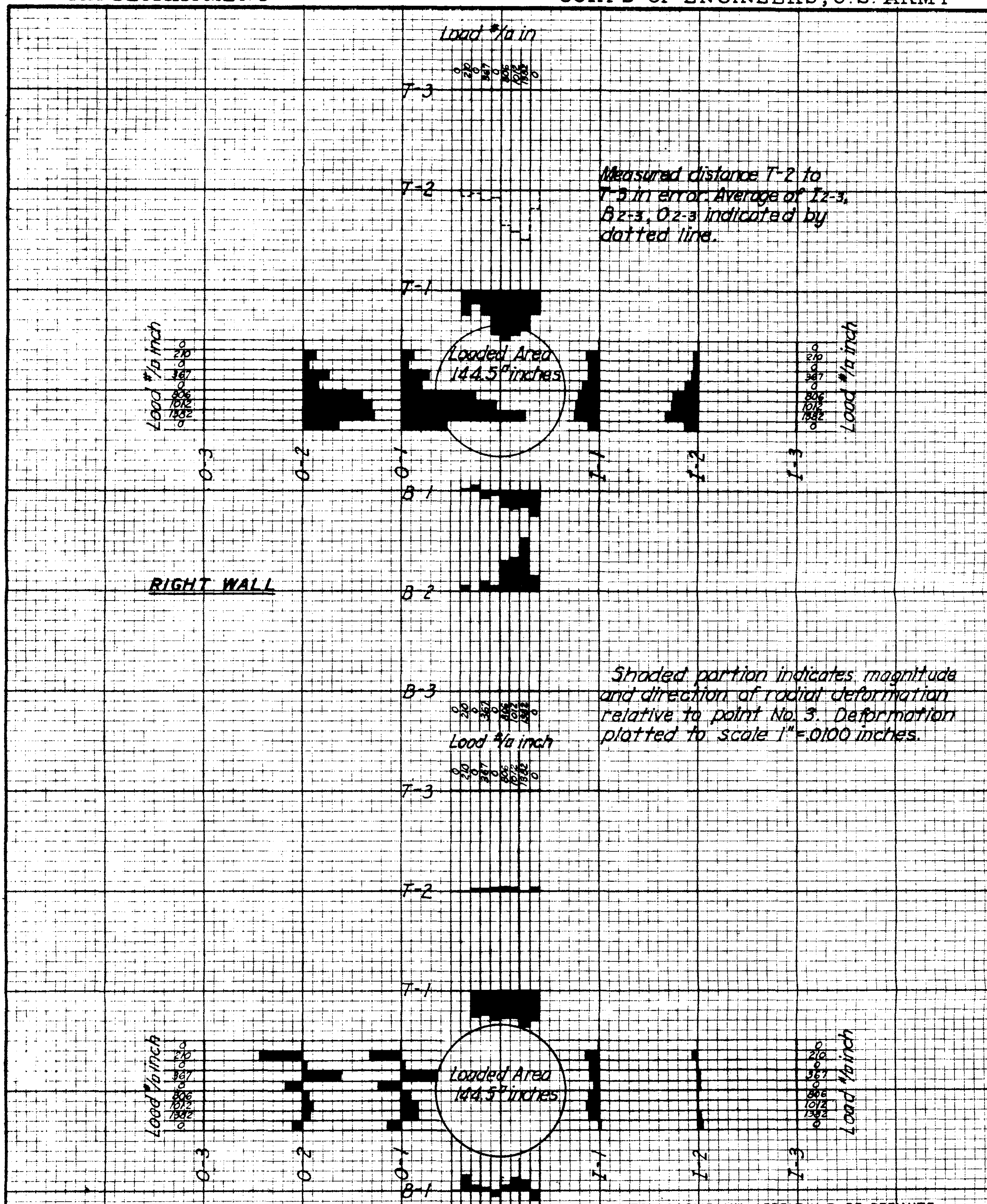
DRAWN BY: P.A.S.

TRACED BY: G.E.G.

DATE: DECEMBER, 1940

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



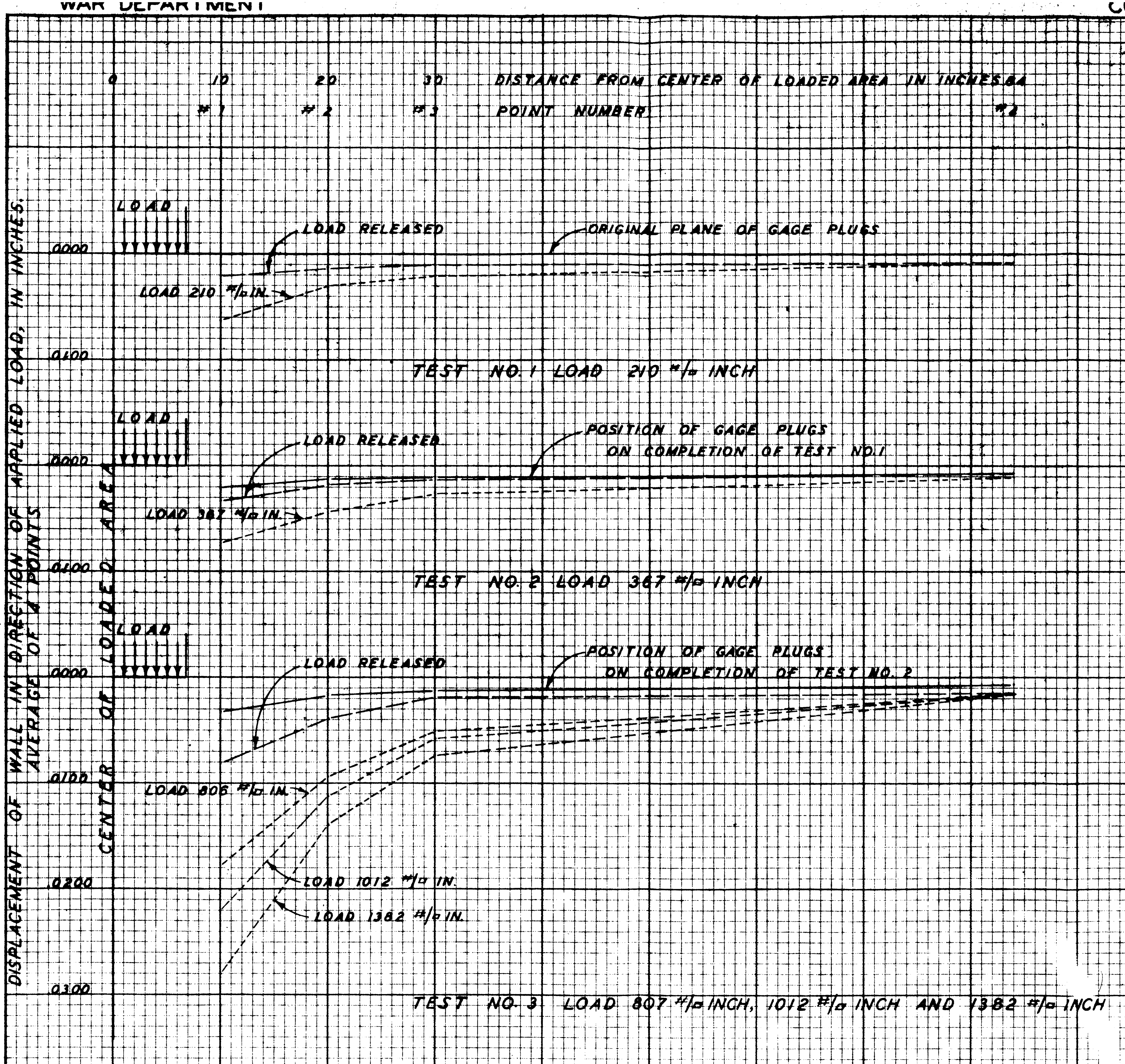
TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

WILLAMETTE BASIN PROJECT, OREGON
DETROIT DAM AND RESERVOIR
FOUNDATION LOADING TEST

RADIAL DEFORMATION
TUNNEL T-D

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT
DRAWN BY: P.A.S.
TRACED BY: G.E.G.
DATE: DECEMBER, 1940

CHAPTER IV - CHART II DE-40-28/1



NOTE:
EQUIPMENT SET UP IN TUNNEL T-D AT ELEV. 1300, AT STA. 0+37 COORDINATES N 1387, W 1089.
LOADED AREA 144.5 □ INCHES.
DISPLACEMENT IS SHOWN IN DIRECTION OF APPLIED LOAD.

TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

WILLAMETTE BASIN PROJECT, OREGON

DETROIT DAM AND RESERVOIR

FOUNDATION LOADING TEST

WALL DEFORMATION TUNNEL T-D

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT

DRAWN BY: P.A.S.

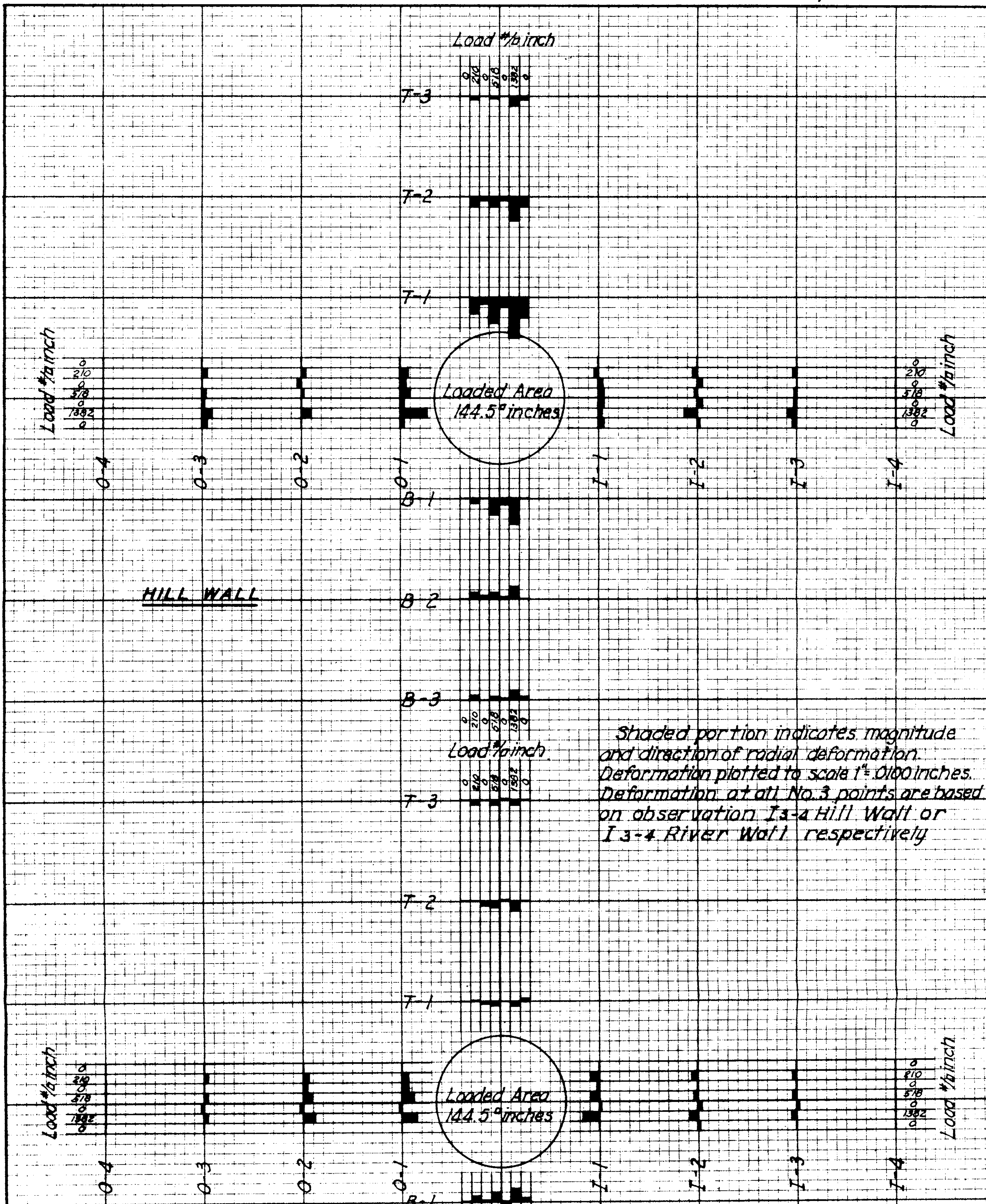
TRACED BY: C.L.S.

DATE: DECEMBER, 1940

32

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



Shaded portion indicates magnitude and direction of radial deformation. Deformation plotted to scale 1" = .0100 inches. Deformation at all No. 3 points are based on observation I3-4 Hill Wall or I3-4 River Wall respectively

TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

WILLAMETTE BASIN PROJECT, OREGON

DETROIT DAM AND RESERVOIR

FOUNDATION LOADING TEST

RADIAL DEFORMATION

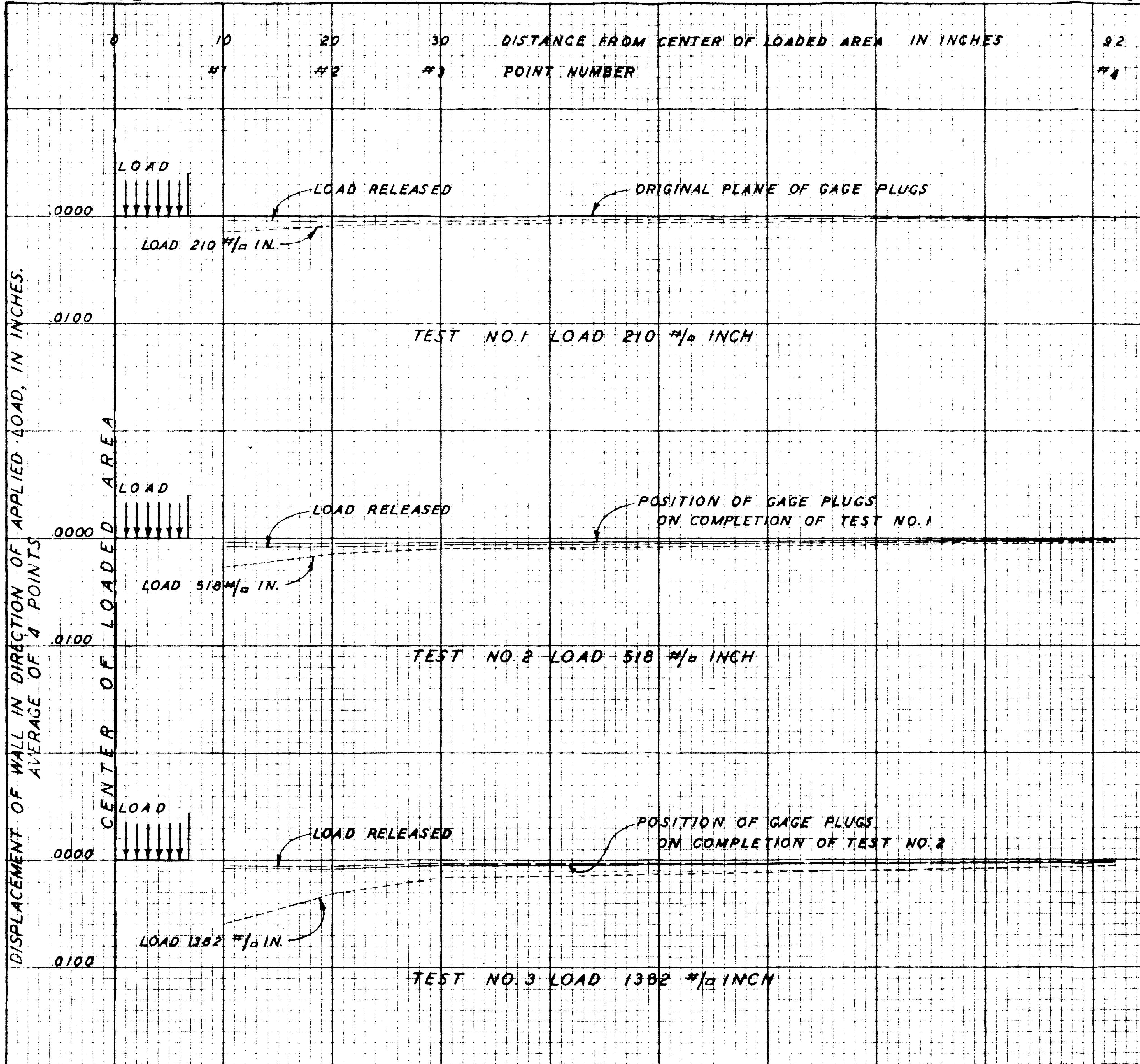
TUNNEL T-A

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT

DRAWN BY: P.A.S.

TRACED BY: G.E.G.

DATE: DECEMBER, 1940



NOTE:
EQUIPMENT SET UP IN TUNNEL T-A AT ELEV. 1237, AT STA. 0+97 COORDINATES N. 1358, W 899.
LOADED AREA 144.5 □ INCHES.
DISPLACEMENT IS SHOWN IN DIRECTION OF APPLIED LOAD.

TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

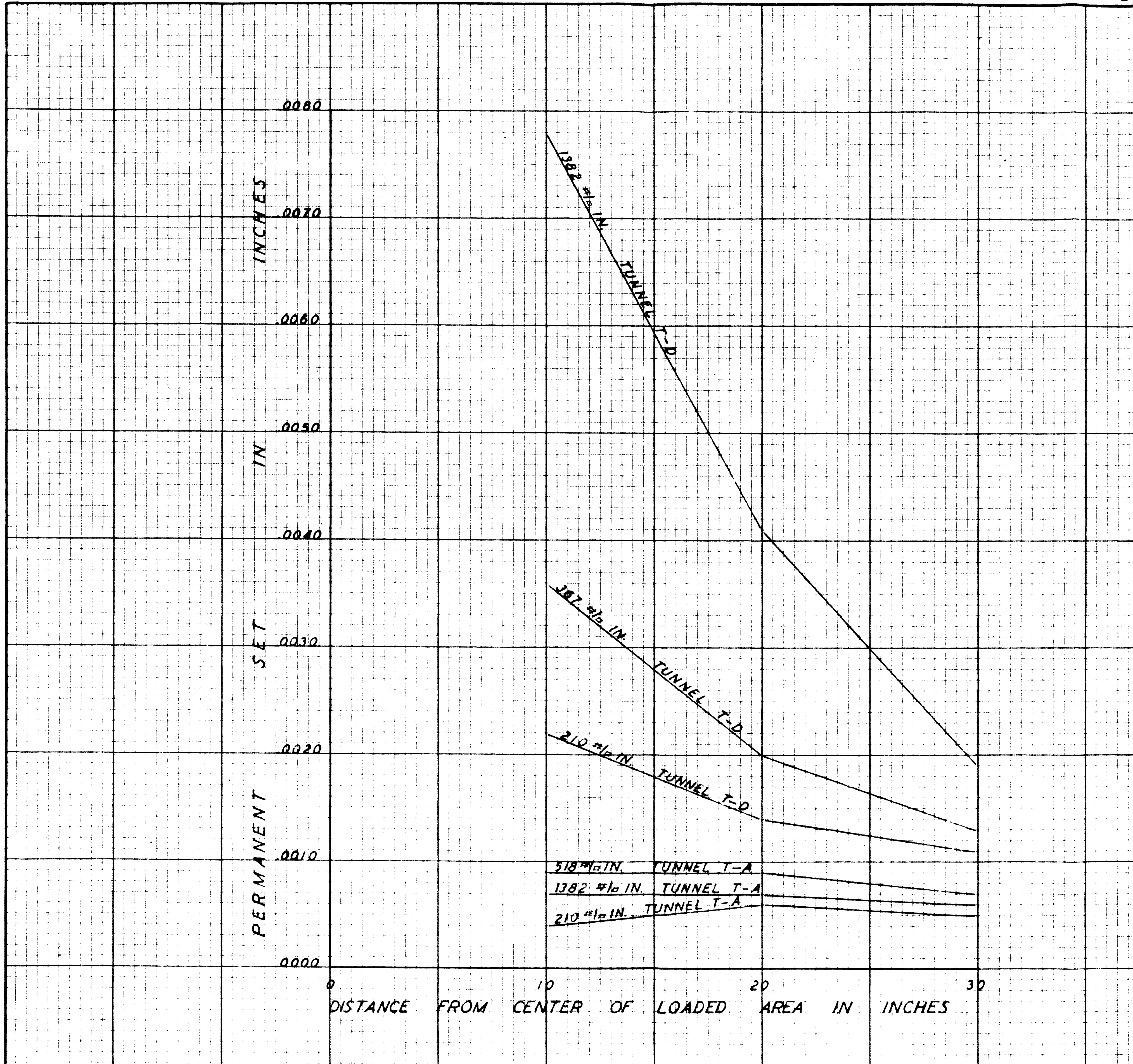
WILLAMETTE BASIN PROJECT, OREGON

DETROIT DAM AND RESERVOIR

FOUNDATION LOADING TEST

WALL DEFORMATION TUNNEL T-A

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT
DRAWN BY: P.A.S.
TRACED BY: C.L.S.
DATE: DECEMBER, 1940



TO ACCOMPANY APPENDIX B OF DEFINITE PROJECT REPORT DATED: AUG. 9, 1941

WILLAMETTE BASIN PROJECT, OREGON

DETROIT DAM AND RESERVOIR

FOUNDATION LOADING TEST

PERMANENT DEFORMATION OF TUNNEL WALL

U.S. ENGINEER OFFICE, PORTLAND, OREGON, DISTRICT

DRAWN BY: P.A.S.

TRACED BY: C.L.S.

DATE: DECEMBER, 1940

APPENDIX B - GEOLOGY AND FOUNDATION DATA

DETROIT DAM SITE - NORTH SANTIAM RIVER, OREGON

CHAPTER VI - AVAILABILITY AND SUITABILITY OF CONCRETE AGGREGATE

VI - AVAILABILITY AND SUITABILITY OF CONCRETE AGGREGATE

Table of Contents

PLATES TO ACCOMPANY APPENDIX B - CHAPTER VI

TABLES TO ACCOMPANY APPENDIX B - CHAPTER VI

1. Aggregate Deposits: Preliminary studies have been made on two possible sources of concrete aggregate. The deposit near Blowout Creek in the reservoir area, Sections 15 and 16, T.10S., R.5E., is the nearest deposit containing sufficient yardage for Detroit Dam (see Plate VI this appendix). The distance is approximately 3.5 miles above the dam site. The deposit covers more than 100 acres, and has been sampled to a depth of 10 feet, approximate summer ground water table, in several of the test pits. No data are available on the thickness of the gravels but an average of 20 feet or more is not unlikely.

2. The second area in Section 25, T.9S., R.2E., one and one-half miles below Mill City, 15 miles below the dam site, is comparable in size and thickness to the Blowout Creek deposit. Only one sample was taken in the area. Comparative preliminary tests indicate that the material may be slightly better than the Blowout Creek deposits.

3. Laboratory Analysis: Laboratory tests on aggregate from the above deposits were made by Professor S. H. Graf at the Engineering Laboratory, Corvallis, Oregon and included sieve analysis, abrasion, soundness (magnesium sulphate), decantation (silt), bulk specific gravity, absorption, organic impurities (color number), mortar strength tests and compressive strength tests on concrete cylinders. The test data is summarized on Tables I to III of this chapter of Appendix B.

4. Sieve analysis on the nine large samples shows considerable variation in the size gradation. Six inch plus material ranges from 2.3% to 53.2%. Material under six inches shows --

a. Average percent of gravel 59.50

b. Average percent of sand 40.50

c. Average fineness modulus gravel 8.19

d. Average fineness modulus sand 3.20

e. Average combined fineness modulus 6.32

f. Average gravel/sand ratio 1.74

5. Abrasion tests by the Los Angeles method show satisfactory results for all samples tested.

6. Magnesium sulphate soundness tests were made according to A.S.T.M. specifications and also by the modified method as developed by Charles E. Wuerpel at the Central Concrete Laboratory, West Point, New York. The average weight loss of material tested by the A.S.T.M. method is higher than the other method and specification limits must be in accordance. Mr. Wuerpel makes the following comment in a communication to the District Engineer dated August 1, 1940: "There is one important feature of the magnesium sulphate accelerated soundness test used by this laboratory which was not mentioned in recommending its use to the Portland District. The use of the centrifuge eliminates attack on the sand particles by the physical action of the salt on the outside of the particles. For this reason the percentage of loss is lower by a practically constant amount than would be obtained by a well conducted test by the existing A.S.T.M. procedure. In order to maintain the criterion of acceptance on a standard basis, it is the practice of this laboratory to multiply the net losses by the centrifuge method by a factor of 2.20 in order to make our results comparable with tests conducted properly by the A.S.T.M. procedure".

7. Comparative tests to date by Professor Graf do not show a constant conversion factor to be applicable. The variation may be from 0.85 to 2.43 as shown in the following table submitted by him:

	Sample Number	% Loss Wuerpel Method	% Loss A.S.T.M. Method	Ratio A.S.T.M. / Wuerpel
Fine Aggregates	BH-2	12.76	31.01	2.43
	BG-2	12.78	30.53	2.39
	BA-1	23.32	32.09	1.38
	BI-2	23.28	41.47	1.77
Coarse Aggregates	BG-1	11.27	11.73	1.04
	BI-2	13.06	11.12	0.85
	BG-2	4.63	11.04	2.38
	BF-2	6.36	9.21	1.45

8. On the fine aggregate specified losses were to be not over 30% by the A.S.T.M. method and not over 10% by the Wuerpel method. The only sample passing the A.S.T.M. test is one from pit B-D at the Mill City deposit. No samples pass the Wuerpel test with a 10% minimum loss allowance. Even with a 15% limit only four of the samples pass the test. (See Table I this section).

9. Sands from the five test pits — B-A, B-F, B-G, B-H and B-I were examined under a microscope by Mr. R. C. Newcomb, Junior Geologist, in an attempt to discover the cause of the unsoundness. His observations are as follows:

> a. "Composition of Grains. - A hasty examination of all samples showed the sand to have essentially the same composition throughout the five test pits. ——— Relation of type of grains in the various screen sizes is typically shown by sample BA-1."

Screen Size	Grains approximate per cent	
	Rock	Mineral
4 - 8	100	0
8 - 14	98	2
14 - 28	95	5
28 - 48	70	30
48 - 100	35	65

Composition of the rock fragments or grains is largely fine grained felsites 5 to 15% soft and weathered. Composition of the mineral fraction is quartz, feldspar, iron oxides - mostly magnetite, pyroxenes and amphiboles in the order named.

b. "Description of 'before' and 'after' $MgSO_4$ samples. -- The 'before' sand grains have a dull, chalky or clayey surface; the grain surfaces are fairly smooth and evenly rounded. The 'after' sands are clean and bright with the same general outline, but with a surface rough and irregular in detail. Sharply angular fragments mark the grains broken by the $MgSO_4$. The 'punky' coating, oxidized streaks, and weathered spots of the 'before' sands show as holes in the 'after' sands. Some badly oxidized and weathered grains occur in the 'before' sands but are present only as angular fragments, if at all, in the 'after' sands. Fresh unweathered fragments generally have suffered only a scouring effect. The basic (basaltic) grains are generally weathered more than the more siliceous (andesitic) grains.

c. "Cause of unsoundness. - The grains broken by the $MgSO_4$ are in practically all cases oxidized and weathered ones. Fresh unweathered grains are practically never broken. Since the rock grains are commonly an aggregation of fine mineral crystals, it is common for one mineral (especially feldspar or chlorite) to be pried out of the rock grain by the $MgSO_4$. From the correlation between unsoundness and alteration or weathering observed in individual grains it is concluded that the unsoundness is due primarily to weathering and to porosity produced by weathering. Minerals such as feldspar and chlorite, into which incipient weathering can easily pass along cleavage lines, cause most of the unsoundness."

9. While these findings are not conclusive they at least indicate some of the defects in the fine aggregate.

10. For the coarse aggregate the specified losses were not more than 15% by the A.S.T.M. method and not more than 10% by the Wuerpel method. All samples tested by the former method pass the test as satisfactory. By the latter method seven out of ten samples meet the required specifications. If weathering is an important factor in causing the unsoundness of the fine aggregate it possibly is also a contributing factor in causing unsoundness in the coarse material. It further holds that if surface weathering is mainly responsible for the unsoundness its volume decreases in proportion to the increase in size of the aggregate.

11. Bulk specific gravity determinations range from 2.49 to 2.78 with three of the four samples tested being greater than 2.60.

12. Absorption tests indicate satisfactory material in each of the two areas tested.

13. Results of decantation tests show a low content of silt but calorimetric tests show a high organic content for all the samples from the Blowout Creek Area. An average color number of 3.5 indicates the need for washing or other treatment to meet the specified color number of 2.

14. Mortar strength tests comparing the test sand from pits B-B and B-D with standard Ottawa sand were made in two ways. In the first series for each test sample the present A.S.T.M. method using a water-cement ratio of 0.6 with mixes adjusted to give 100% flow were used. This method requires the use of the graded test sand. As will be noted in Table II, the samples tested by this method failed to meet the specified strength of 90% of the standard sand which was designed for 3000 pounds per square inch at 28 days. In the second series tests were made by the old A.S.T.M. method of 1:3 (by weight) mixes for both the standard sand and the test sand. In this case the ordinary 20-30 mesh standard Ottawa sand was used. The test sand proved to be stronger in each case than the standard sand.

15. Compression tests on concrete cylinders using various combinations of coarse and fine aggregate were made. Tests using both coarse and fine aggregate from the Detroit Area failed to meet the specified 90% of design strength. Tests using Willamette River sand from the Corvallis Sand and Gravel Company, Corvallis, Oregon and coarse aggregate from the Detroit Area gave more satisfactory results. Indications are that the Mill City gravel may be of slightly better quality. See Table III this section of Appendix B.

16. Conclusions:

a. A sufficient quantity of aggregate is to be found in either of the two areas investigated near the dam.

b. Coarse aggregate should be satisfactory with a minimum of processing.

c. The suitability of the fine aggregate has not been definitely proven and may take extensive processing or an admixture of outside sand or crushed rock to bring it up to specifications.

17. Recommendations:

a. Additional samples of fresh sand from the river bed in the vicinity of Blowout Creek or the mouth of the Breitenbush River should be tested to determine the maximum quality of fine aggregate obtainable in the area and indicate to what extent weathering has affected the low terrace gravels.

b. Magnesium sulphate tests after mill tests or abrasion tests on test pit gravels to check the amount of $MgSO_4$ loss due to weathering on the surface of the individual fragments.

c. Additional magnesium sulphate tests and concrete strength tests on aggregate which has already undergone one standard magnesium sulphate test.

d. Exploration of additional natural deposits, and of quarry rock, should processing fail to bring the Detroit aggregate up to specifications.

e. Prior to construction a complete survey should be made to determine the aggregate deposit or deposits which may be used in Detroit Dam. Previous observations and tests on Willamette Basin aggregates indicate certain deficiencies which may become a serious problem in the larger dams and therefore materially affect the cost of aggregate.

SUMMARY OF AGGREGATE TESTS - DETROIT DAM SITE

TEST PIT NO. (g)		B-A	B-F		B-G		B-H		B-I		B-B		B-B	B-D	B-E	B-A
Date Sample Taken		8/15/40	8/25/40	8/26/40	8/28/40	8/29/40	8/20/40	8/20/40	8/30/40	9/4/40	1/4/40	1/4/40	5/22/40	1/4/40	5/22/40	5/22/40
Elevation Ground Surface	M.S.L.	1324	1307	1307	1303	1303	1325	1325	1333	1333	1334	1334	1334	770		1333
Depth to Water Table	Feet	5	9	9	8.5	8.5	10	10	?	?	(f)	(f)	(f)	(f)	(f)	3 (d)
Sample Number		1	1	2	1	2	1	2	1	2	1	2	3	1	1	1
Depth of Sample	Feet	3-5	3-5	7-9	3-5	8-9	3-5	7-9	3-5	8-10	1.1-2.1	2.1-3.6	1.1-3.6	5.3-6.4	2.1-3.1	0.6-1.8
Weight of Sample	Lbs.	6377	5442	8195	4197	3357	4448	3684	6792	5662	76	77	1500	148	50	2500
Over 6"	%	34	35.5	53.2	8.8	33.2	2.4	2.3	30.3	53.1	21.6	21.1		26.9	21.6	4.7
Gravel (a)	%	62.9	64.5	59.5	35.6	66.0	41.3	53.2	78.3	72.5	73.1	76.7		72.1		
Sand (a)	%	37.1	35.5	40.5	64.4	34.0	58.7	46.8	21.7	27.5	26.9	23.3		27.9		
G/S Ratio (a)		1.695	1.82	1.47	0.55	1.94	0.70	1.23	3.62	2.64	2.71	3.28		25.8		
F.M. Gravel (a)		8.52	8.54	8.88	8.07	8.47	8.30	7.58	8.95	8.41	8.45	8.57		8.28		
F.M. Sand		3.14	2.83	2.98	2.90	3.38	3.24	3.48	3.60	3.28	3.24	3.37		3.57	2.40	
F.M. Combined (a)		6.62	6.50	6.49	4.75	6.74	5.32	5.65	7.80	6.99	7.05	7.36		6.96		
Grading-Percent Retained																
(Material under 6")																
Coarse - Screen	6"	0	0	0	0	0	0	0	0	0	0	0	0	0		
	3"	24.1	35.0	43.9	10.7	25.4	18.9	7.3	47.2	27.0	24.6	20.9	25.9	18.1		+6"
	1½"	33.1	20.4	23.8	29.1	30.0	27.4	17.7	24.3	25.3	30.0	40.7	27.4	29.5		
	3/4"	21.5	19.4	16.1	30.0	21.7	28.5	25.1	13.1	21.0	21.4	20.7	21.7	26.1		
	3/8"	13.4	13.9	8.7	16.5	12.5	15.0	25.6	7.6	14.7	13.8	10.1	13.8	15.3		
	#4	7.9	11.3	7.5	13.7	10.4	10.2	24.3	7.8	12.0	10.2	7.6	11.2	11.0		
Fine - Screen	#4	0	0	0	0	0	0	0	0	0	0	0	0.0	0	0.8	
	#8	11.3	16.2	9.7	7.5	21.1	15.6	22.3	28.0	23.1	20.4	19.6	17.7	21.9	2.5	
	#14	27.5	20.7	22.6	19.2	29.2	28.7	30.7	35.0	26.3	26.0	30.3	26.9	35.0	15.4	
	#28	34.6	21.3	36.1	37.0	26.1	30.2	26.2	18.5	22.7	24.3	27.8	27.7	27.6	31.9	
	#48	19.2	21.0	21.7	29.5	16.2	17.5	15.4	9.9	16.2	18.3	14.4	19.4	10.7	25.1	
	#100	5.3	13.3	6.9	5.7	5.0	5.5	3.9	4.8	6.6	8.2	5.7	6.4	3.1	14.8	
	Pan	2.1	7.5	3.0	1.1	2.4	2.5	1.5	3.8	5.1	2.8	2.2	3.4	1.7	9.5	
Color No. - A.S.T.M.		3	4	3	3	3.5	4	4	4	3			3	2 (h)		
Abrasion-Coarse-L.A.Test																
100 Rev.		3.68 (k)	3.30 (k)	3.74 (k)	416 (k)	4.04 (k)	3.72(k)	3.94(k)	3.80(k)	3.48(k)	2.9	3.4		2.4		
500 Rev.		21.42 (k)	18.64 (k)	19.50 (k)	20.98 (k)	19.22(k)	20.24(k)	20.28(k)	19.50(k)	19.04(k)	17.7	19.2		14.0		
Soundness Test (b)	actual															
Coarse	%	8.72 (k)	7.37 (k)	6.36 (k)	11.27 (k)	4.63 (k)	9.90(k)	8.88(k)	8.66(k)	13.06(k)			10.60 (k)			
Fine	loss	23.32	15.68	22.56	13.38	12.78	14.67	12.76	17.61	23.28					33.9	
Soundness Test (c)	weighted															
Coarse	average			9.21 (k)	11.73 (k)	11.04(k)				11.12(k)						
Fine	% loss	32.09				30.53		31.01		41.47	30.8	31.7		23.9		
Specific Gravity																
Coarse													2.61(k)	2.78		
Fine													2.49	2.74		
Absorption																
Coarse	%												1.82(k)	1.41		
Fine	%												3.78	3.78		
Decantation (silt)											3.29(e)	1.56(e)		1.89 (e)		
Coarse	%												.43	.24		
Fine													1.66	.43		

(a) Based on Material under 6"
(b) Wuerpel Method - Central Concrete Laboratory, West Point, N.Y.
(c) A.S.T.M. - 5 cycles
(d) Water Table - January 4, 1940
(e) Washing loss
(f) Not reached
(g) For Location see DE-40-24/1
(h) After washing
(k) Does not include material retained on 3" screen

APPENDIX B - CHAPTER VI

TABLE II

MORTAR STRENGTH TESTS ON FINE AGGREGATE
DETROIT DAM SITE

	Cylinder: Size	Age: Days	Sand	Strength: Lb./sq. in.	Shear ∠ with vertical	Ratio-test sand: to standard	Remarks
A.S.T.M. (a)	2" x 4"	7	Ottawa	2890	32°	-	
A.S.T.M. (a)	2" x 4"	7	B-D #1	1495	33°	0.515	
A.S.T.M. (a)	2" x 4"	28	Ottawa	3790	28°	-	
A.S.T.M. (a)	2" x 4"	28	B-D #1	2600	30°	0.685	
A.S.T.M. (b)	2" x 4"	7	Ottawa	2880	29°	-	W/C = .365
A.S.T.M. (b)	2" x 4"	7	B-D #1	3560	28°	1.233	W/C = .333
A.S.T.M. (b)	2" x 4"	28	Ottawa	3210	31°	-	W/C = .365
A.S.T.M. (b)	2" x 4"	28	B-D #1	5240	27°	1.636	W/C = .333
A.S.T.M. (a)	2" x 4"	7	Ottawa	2740	27°	-	
A.S.T.M. (a)	2" x 4"	7	B-B	1800	24°	0.66	
A.S.T.M. (a)	2" x 4"	28	Ottawa	4360	25°	-	
A.S.T.M. (a)	2" x 4"	28	B-B	3410	23°	0.78	
A.S.T.M. (b)	2" x 4"	7	Ottawa	2300	25°	-	W/C = 0.428
A.S.T.M. (b)	2" x 4"	7	20-30 B-B	2740	27°	1.21	W/C = 0.443
A.S.T.M. (b)	2" x 4"	28	Ottawa	3540	18°	-	W/C = 0.428
A.S.T.M. (b)	2" x 4"	28	20-30 B-B	5030	24°	1.42	W/C = 0.443

(a) Present Standard A.S.T.M. Method
W/C = 0.6 - 100% flow.

(b) Old A.S.T.M. Method
(1:3 by weight)

APPENDIX B - CHAPTER VI

TABLE III

COMPRESSION TESTS ON CONCRETE CYLINDERS
DETROIT DAM SITE

Cylinder Size	Age Days	Coarse Aggregate	Sand	Stress Lb./sq. in.	Angle of Break	% of Design Strength (a)	Remarks
6" x 12"	7	B-D #1	Corvallis, Ore.	1620	29°		No aggregate was sheared
6" x 12"	28	B-D #1	Corvallis, Ore.	2920	27°	97.0	No aggregate was sheared
8" x 16"	7	B-A & B-B	B-A & B-B	1520	24° (c)		Very slight aggregate failure
8" x 16"	7	B-A & B-B	B-A & B-B	1580	24° (c)		Very slight aggregate failure
8" x 16"	7	B-A & B-B	B-A & B-B	1450	23° (c)		Very slight aggregate failure
8" x 16"	28	B-A & B-B	B-A & B-B	2200	23° (c)	73.0	Few aggregate particles sheared. Failure mainly in cement paste and of bond between cement & aggregate.
8" x 16"	28	B-A & B-B	B-A & B-B	2260	24° (c)	75.3	
8" x 16"	28	B-A & B-B	B-A & B-B	2340	24° (c)	78.0	
8" x 16"	7	B-A & B-B	Corvallis, Ore.	1520	27°		No aggregate failure
8" x 16"	7	B-A & B-B	Corvallis, Ore.	1420	23°		No aggregate failure
8" x 16"	7	B-A & B-B	Corvallis, Ore.	1490	28°		No aggregate failure
8" x 16"	28	B-A & B-B	Corvallis, Ore.	2740	25° (c)	91.4	Very slight aggregate failure. Failure mainly in cement paste and of bond between cement & aggregate.
8" x 16"	28	B-A & B-B	Corvallis, Ore.	2730	24° (c)	91.0	
8" x 16"	28	B-A & B-B	Corvallis, Ore.	2760	30° (d)	92.0	

(a) Designed Strength 3,000 lb./sq. inch at 28 days
(c) Signifies conical fracture
(d) Signifies diagonal fracture