

# 1300-Ton-Capacity Prestressed Anchors Stabilize Dam

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## SYNOPSIS

The Wanapum hydro power station on the Columbia River provides for ten installed and six future generating units. To reduce initial outlay, construction of the monoliths for these six units was limited to the intakes. The intakes, however, lacked stability. Tying them down to the rock foundation with prestressed anchors was a practical and most economical solution.

Thirteen anchors, each of nearly 1300-ton capacity and 70 feet deep, were required for each monolith, using the conventional 45° wedge theory. Although contract drawings showed details of the anchors, bidders were allowed to submit their own designs and stressing methods. The contractor proposed the BBRV system. In view of the importance of the structures and the large capacity of the anchors, a full-scale field pull-out test was required and performed successfully. Each anchor consisted of 4 cables, each cable containing 90 ¼-inch-diameter parallel high-strength steel wires with button head ends and anchor heads of the BBRV type. The bottom 30 feet of the cable were grouted before stressing. The work was completed successfully and at a lower cost than estimated. The non-linear elastic effects observed during the pull-out test were identified as characteristics of the steel wire and not as a possible increase in the effective length of the cables due to rock deformation or anchor slippage.

## INTRODUCTION

The Wanapum Project is on the Columbia River, about 415 miles north from the mouth, in central Washington. It consists of earth embankments and concrete structures: a gated spillway and a powerhouse with an integral intake (Fig. 1).

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Two short concrete gravity structures, one at the left end of the powerhouse and another at the right end of the spillway, contain fish-passing facilities.

The powerhouse structure, 1540 feet long, contains 10 generating units, an erection bay, and the intakes for an additional six units to be installed in the future (visible between the powerhouse and the spillway in Fig. 1). Each generat-

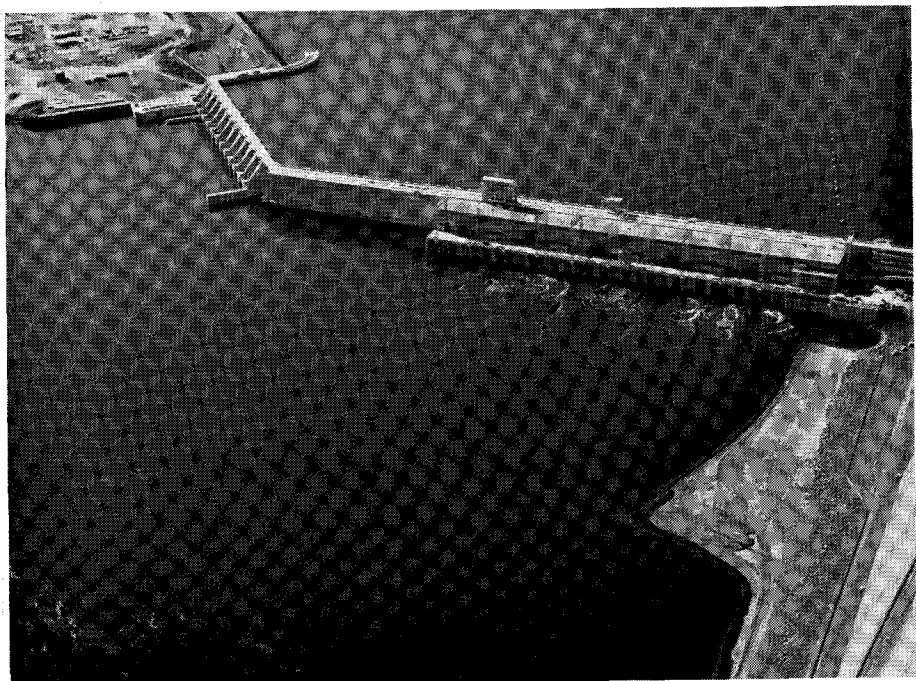


Fig. 1—Wanapum Hydroelectric Project

ing unit consists of an 87,000 kva generator coupled to a Kaplan turbine producing 120,000 horsepower under an 80-foot head. The 285-inch runner diameter gives an idea of the physical size of the equipment.

Each unit is housed within a reinforced concrete monolith 90 feet wide, separated from the adjoining monoliths by contraction joints. The length of the intake and draft tube water passages resulted in a monolith nearly 200 feet long. With a base this long, there was no difficulty in achieving the degree of stability required of dams.

The design of the six intakes for the future units, however, presented a problem because of their narrow 68-foot base and inherent lack of stability (Fig. 2).

The decision to build only the intakes of the future unit monoliths was mainly an economic one. Con-

structing just the intakes represented the least initial investment, even if some additional measures were necessary to make the intakes stable. Other factors considered were the requirements of diversion during the river closure, and the ease of future construction work when completing the six units.

The 18 openings, each 20 feet wide (Fig. 2), of the six intakes provided ample capacity for low flow season diversion when the construction of the left earth embankment was to block the river permanently. The actual headwater rise experienced during the diversion was only a little over two feet for a maximum flow of 120,000 cubic feet per second.

Plans for future completion of the six monoliths call for building a cofferdam downstream. Sheet piles were embedded in the draft tube

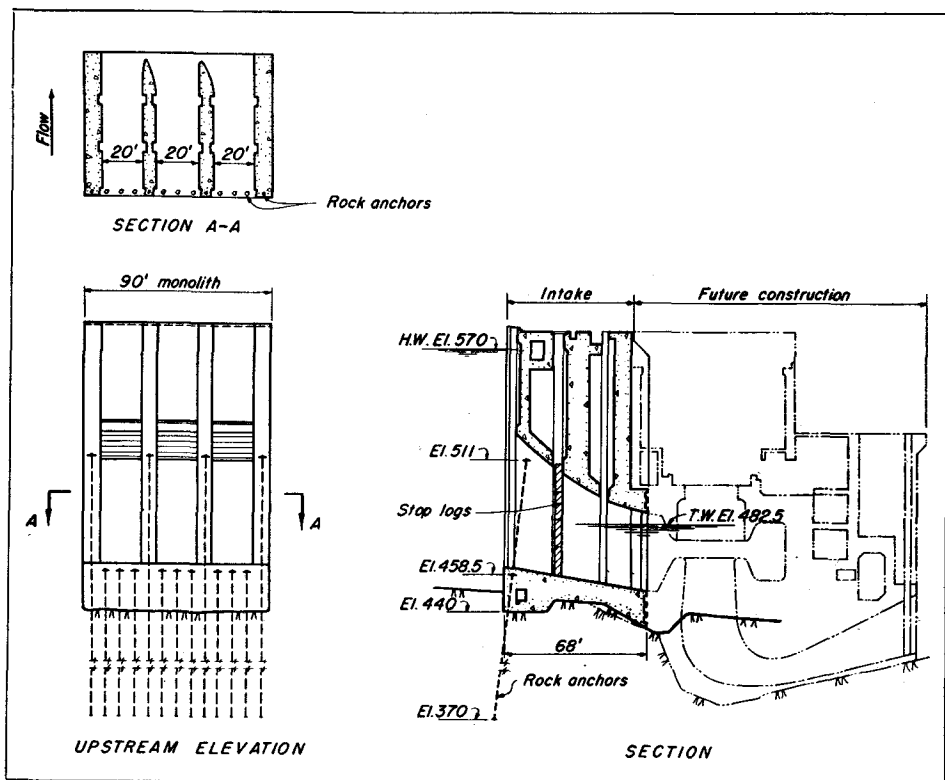


Fig. 2—Future Unit Intake

pier of monolith No. 10 on one side and in the spillway training wall on the other to provide connections for the two ends of the future cofferdam. The tailrace was excavated fully in the first stage of development, but rock excavation for the draft tubes of the future units was limited to a 15-foot-wide strip adjoining unit No. 10.

### DESIGN STUDIES

The study of ways to stabilize the narrow intake structure produced five different schemes. They were as follows (Fig. 3):

- a. Extending all piers and the floor slab upstream.
- b. "Buttress dam" scheme: piers extended upstream supporting an inclined deck composed of precast elements.
- c. Extending the two end piers of each intake monolith downstream.
- d. Weighing down the intakes with gravel fill.
- e. Anchoring down the intake with deep, prestressed rock anchors.

Cost estimates revealed that schemes *c* and *e* were the least expensive and about equal in price. Scheme *e*, however, represented the "cleanest" design, while various objections could be raised against other schemes.

Extending the piers upstream, as in schemes *a* and *b*, was undesirable for several reasons: (1) probable adverse effect on the approach flow to the spillway; (2) interference with the usual construction plant layout—whirley crane tracks

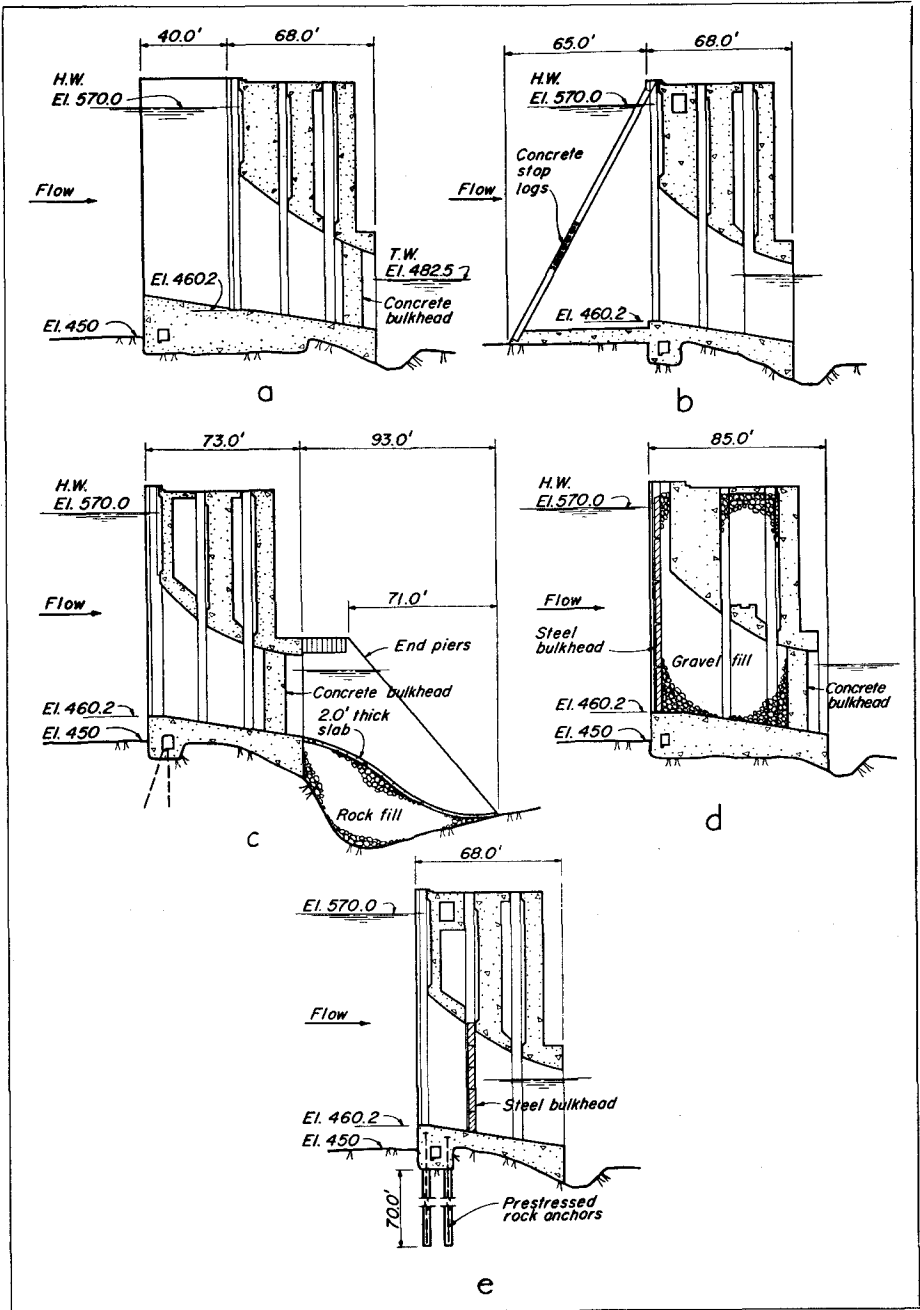


Fig. 3—Alternate Design Studies

laid close to the upstream face of the powerhouse structure; and (3) appearance inconsistent with the ten remaining intakes. Extending the end piers downstream in scheme *c* would create an undesirable slanting joint in the future spiral case walls and complications in construction. Gravel fill, in scheme *d*, represented a complication in both stages of construction.

The Board of Consultants for the Wanapum Development approved the selection of the prestressed rock anchor scheme, with a recommendation that some of the anchors be extended upward through the intake piers.

#### STABILITY OF THE INTAKES

The intake structure was designed for the following loading conditions:

1. Normal  
Max. controlled H.W. El. 570.0  
Min. operating T.W. El. 482.5
2. Maximum Design Flood  
H.W. El. 575.0-T.W. El. 527.4
3. Seismic  
H.W. El. 570.0-T.W. El. 482.5
4. Future Construction  
H.W. El. 570.0-No tailwater.

The allowable foundation pressure was 100 ksf. No tension was allowed on the base except for a small amount (50 psi) during earthquake.

The necessary anchorage for each intake monolith was provided by 13 anchors extending deep into the basalt foundation. Nine of them anchored the floor slab of the intake; the remaining four extended upward, anchoring the individual piers (Fig. 2). The design load per anchor was 2540 kips or nearly 1300 tons.

#### ANCHORAGE DEPTH

The length of the anchors was determined by assuming that: (1)

the foundation rock has no tensile strength and that (2) the pull of the anchors is resisted only by the submerged weight of rock. This conservative approach was deemed justified by the importance of the structure.

Assuming the rock breaking out at 45°, the pull of the anchor was resisted by the weight of a wedge of rock extending the full length of each intake monolith.

The depth of the anchors determined in this manner was then checked by investigating the stability of the structure and its foundation by the method described in a recent paper by O. C. Zienkiewicz and R. W. Gerstner.<sup>8</sup> Theoretical investigations which represent the main body of the paper deal with the stress distribution in an isotropic, homogeneous foundation. Especially noteworthy are horizontal tensile stresses near the surface when water load is not present, and tensile stresses in the foundation itself when all loads, including water, are acting.

Because of the great uncertainty existing on the subject of the tensile strength of rock, at present at least, the design method presented proposes to consider the ultimate behavior of the entire structure (foundation and dam) with reliance placed only on the weight and compressive strength of the rock. The factor of safety is defined as the ratio of the stabilizing moment over the overturning moment determined for the future intake structure and the mass of rock below it bound together with rock anchors (ABC on Fig. 4). Even though the factor of safety is a minimum value, as rock is assumed cracked along curve BC and its tensile strength is disregarded, the dimensions of the fu-



Considering, however, that rock anchors of the capacity required at Wanapum exceeded, by far, anything used in the United States and probably equaled the largest cables used elsewhere in the world, it was decided to show a specific acceptable design of the anchor cables on the contract drawings, but to clearly allow alternate designs.

The cable design as shown on the contract drawings was patterned after previous similar designs used successfully abroad. Each cable was to consist of a number of 7-wire, uncoated, stress-relieved strands conforming to ASTM Specification A-416, or of stress-relieved wires conforming to ASTM Specification A-421. The design allowable stress was 60% of the ultimate strength of 240,000 psi, resulting in a 2540 kip design load per anchor. Each anchor was to be assembled and lowered as a unit inside a 16-inch-diameter hole drilled in rock to the required depth. Near the bottom end the strands were spread inside the hole to form an elongated bulb in order to produce the effect of a mechanical anchorage. Near the top end, the strands or wires were spread gradually and looped over a temporary stressing head. The bottom 20 feet of the cable were to be grouted first. After the grout attained the necessary strength, the cable was to be tensioned and the remainder of it grouted permanently. At this stage, the loops at the top ends were to be cut off and the load transmitted to the concrete through bond.

The load to be placed on the cables was to consist of the design load plus an allowance for losses due to shrinkage, elastic shortening and creep of the concrete, and relaxation of the steel. The initial

load, however, was not to exceed 70% of the ultimate strength. After stressing of the cable, the specifications called for a one-hour wait. After this period, the tension in the cable was to be checked and, if any drop was observed, brought up to the original load. After a period of an additional 24 hours the tension in the cable was to be rechecked. If tension in the cable was found to remain in excess of the design load by at least 5% of the ultimate strength, the remainder of the cable was to be grouted. If the drop in tension was greater, the cable was to be retensioned to the original load and tension rechecked after an additional 24 hours.

A possible failure of an anchor during tensioning was also considered. This could most probably occur through flaws in grouting. The specifications, therefore, called for removal of a slipping cable, and a special pay item for such an operation was listed. There were some fears, however, that such an operation might prove troublesome.

In view of unusually large loads and the importance of structures, it was decided to proceed cautiously, and require a full-scale field test before freezing the design and placing any anchors in the permanent structures. The purpose of the test was to establish adequacy of the various components of the anchor assembly (steel cables, bearing plates, etc.), to test the stressing equipment and to verify the design assumptions regarding the extent of the initial grout. The test was also expected to provide a check on grout composition and its placing methods. The over-all stability assumptions, based on the "wedge" theory, could not be verified in a full-scale test of this magnitude, as

both ends of the test anchor were necessarily pulling and pushing on the same mass of rock. A pull-out test of a full size 1300-ton capacity rock anchor cannot be performed economically to verify the accuracy or safety of the 45° wedge theory. This justified further the conservative assumptions made regarding rock resistance.

### FINAL ANCHOR DESIGN

As it turned out, the contractor took advantage of the clause permitting him to submit his own design of the anchor cables. His proposal differed in certain respects from the scheme shown on the contract drawings. It was based on the BBRV system developed originally in Switzerland and introduced in

the United States some time ago. The BBRV system uses parallel wires with cold formed button heads at the ends bearing on a perforated steel anchor head. Each anchor proposed for Wanapum consisted of four parallel cables, each cable composed, in turn, of 90 ¼-inch-diameter wires (Figs. 5 and 6). At the top end, the four cables, each of 635 kip design capacity, flared out toward individual anchor heads 7.875 inches in diameter. Each anchor head rested, in turn, on an 18.5-inch-diameter bearing plate 2.5 inches thick.

At the bottom end, the wires were threaded through four anchor heads, each 5.75 inches in diameter. A thinner, single, holding plate supported the bottom ends of wires and

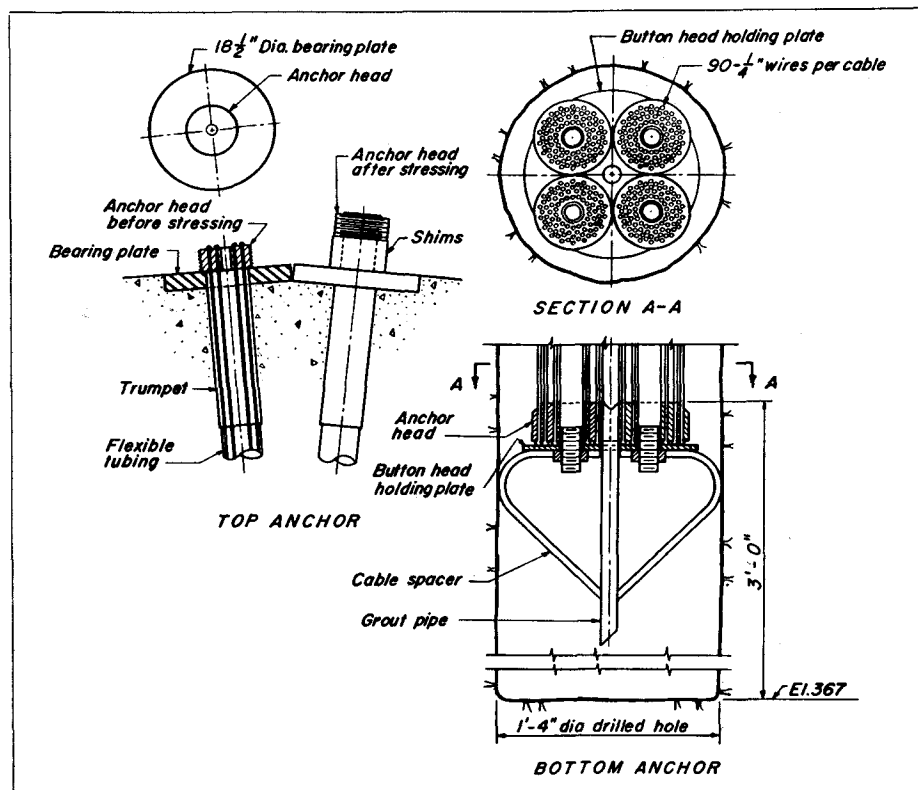


Fig. 5—Rock Anchor Details

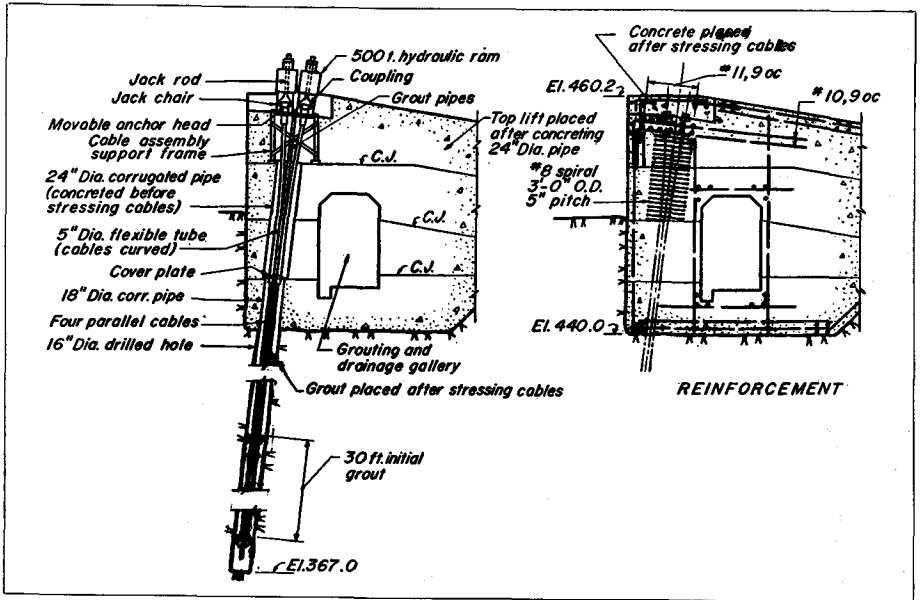


Fig. 6—Rock Anchor and Intake Floor

provided a common platform for all four bottom anchor heads and the grout pipes during lowering and grouting of the assembled anchor.

Since the design proposed by the contractor met the requirements of the specifications, its conditional acceptance followed pending the field test.

### TEST SETUP

The site for the field test was selected a short distance upstream of the intakes for the future units. The 16-inch-diameter hole extended through the same basaltic formation underlying all structures, and was drilled with a Robbins rotary drill rig. The rock wall surfaces of the hole were slightly rough, suggesting a good seat for the grout plug.

The test anchor was identical to the anchors proposed for the intake structure. It consisted of four cables, each built up to 90 parallel ¼-inch-diameter solid wires of high strength steel with cold formed button ends. At the top end, two square and

two circular bearing plates were used under anchor heads, as an experiment.

The over-all test setup is shown in Figure 7a. From top to bottom it consisted of the following features: a circular, heavily reinforced, movable, concrete head block resting, via four steel shims or four jacks, on a square, moderately-reinforced concrete cushion which transmitted the anchor force to the supporting rock. The jacks could be placed either on top of the concrete block (Fig. 7b), or below it (Fig. 7c). The 16-inch-diameter hole extended to a depth of 72.5 feet below the rock surface. The 84-foot-long cable passed through the head block and cushion into the hole to a depth of 68 feet below the top of the rock surface. The grouting of the lower end of the cable extended to a height of 37.66 feet above the base plate.

The grout mix consisted of one sack of cement, 25 lbs. of pozzolan, 1½ lbs. of intrusion aid, and water;

0.9 cu. ft. of water per sack of cement was required to make the grout pumpable. No sand was used. The grout was forced to the bottom of the hole through a 1¼-inch pipe mounted at the center of the cable assembly. The grout was vibrated by attaching an electric vibrator to the grout pipe. Ground water filled the hole before the start of grouting to Elevation 410, or 31 feet above the bottom.

Cables, bearing plates, concrete block, concrete cushion, reinforcing steel, shims, loading jacks, pull rods, coupling nuts and end nuts, and testing equipment, including pumps and supply lines, were designed and supplied by the Ryerson Steel Company, the subcontractor for the anchorages.

The test equipment consisted of stressing and measuring devices. The stressing devices were two

pumps with pressure lines, four specially designed 500-ton jacks, and four pull rods with nuts. The measuring devices consisted of two pressure gages on each of the pump supply lines, nine theodolite measuring points on the concrete block and cushion, and the necessary surveyor's rods.

The accuracy of the pressure gages was approximately 50 psi. Thus, for low pressures the accuracy was not too good, but at 5000 psi (maximum jack pressure) the accuracy was 1%, which was quite sufficient. The measuring rods could be read to 0.002 foot. For a length increase of 0.2 foot, this led to an accuracy of 1%, which was sufficient.

### TESTING

The test procedure was originally divided into three phases as follows:  
Phase I—Loading up to 70% of

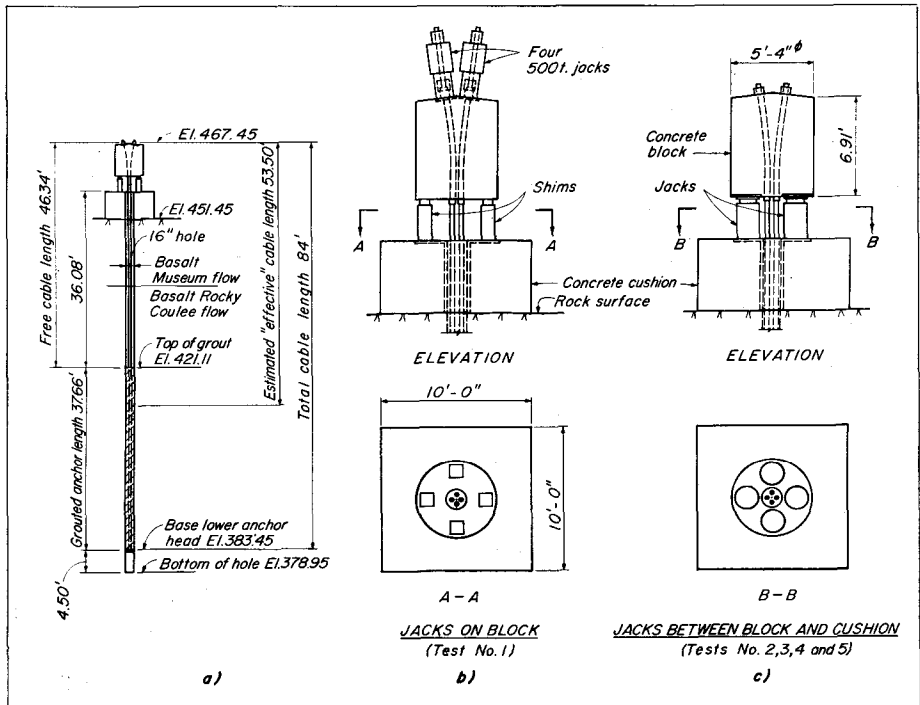


Fig. 7—Anchor Test Set-Up

ultimate strength of wires

Phase II—Load loss measurements for one week

Phase III—Loading to failure

Two tests were conducted on May 10, 1960. In Test 1, the jacks were placed on top of the head block as shown on Figures 7b and 8. This test was discontinued due to failure of one of the pull rods at about 35% of the ultimate strength. The results are shown on Fig. 9.

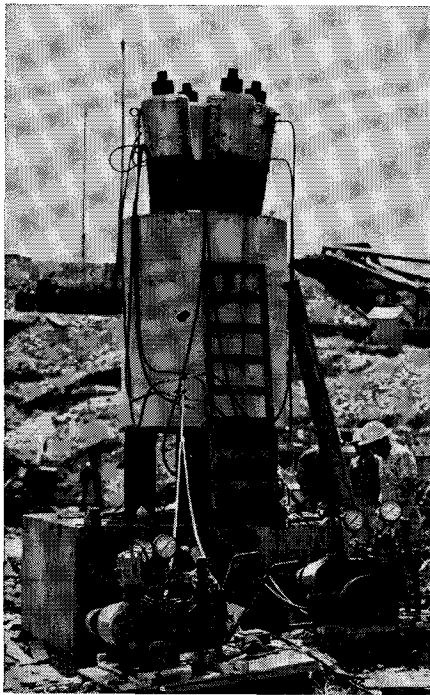


Fig. 8—Rock Anchor Pull-Out Test

For Test 2, the jacks were placed between the concrete head block and the concrete cushion as shown on Figure 7c. The change in setup from Test 1 to Test 2 has no effect on the loading of the test cable and grout anchor; it did, however, eliminate the possibility of further checking on pull rods and nuts. Though not essential for the pull-out test itself, these loading devices were to be used for stressing all cables

in the future intake and spillway piers.\* Following the initial failure, the pull rods were redesigned and laboratory tested to prove their adequacy before stressing of cables installed permanently in the structures.

The time-duration test, Test 2, was performed during the week of May 10-17, 1960. Throughout this period, elongation and load measurements were recorded. The range of elongations under constant load is shown on Figure 10. It amounted to only 0.011 foot.

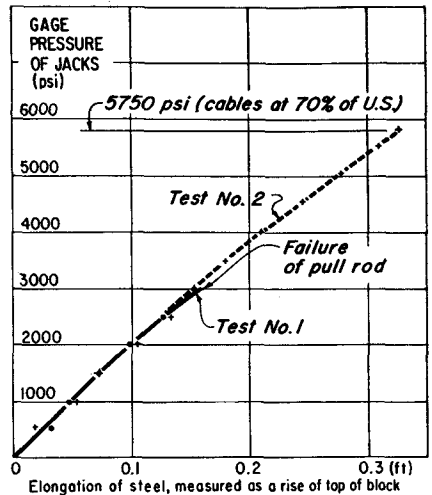


Fig. 9—Pull-Out Test Results No. 1 & 2, May 10, 1960

On May 17, 1960, after unloading the time test, two complete loading cycles were performed. In the first, called Test 3, the load was first brought up to 70% of the ultimate strength and then increased to 92% of the ultimate strength. At this point the pumps reached their capacity. This load was sustained for about two hours during which the elongation increased slightly, as shown on Figure 10. No other ef-

\*Similar cables were used in the spillway piers for anchoring the tainter gate trunnions (Ref. 12).

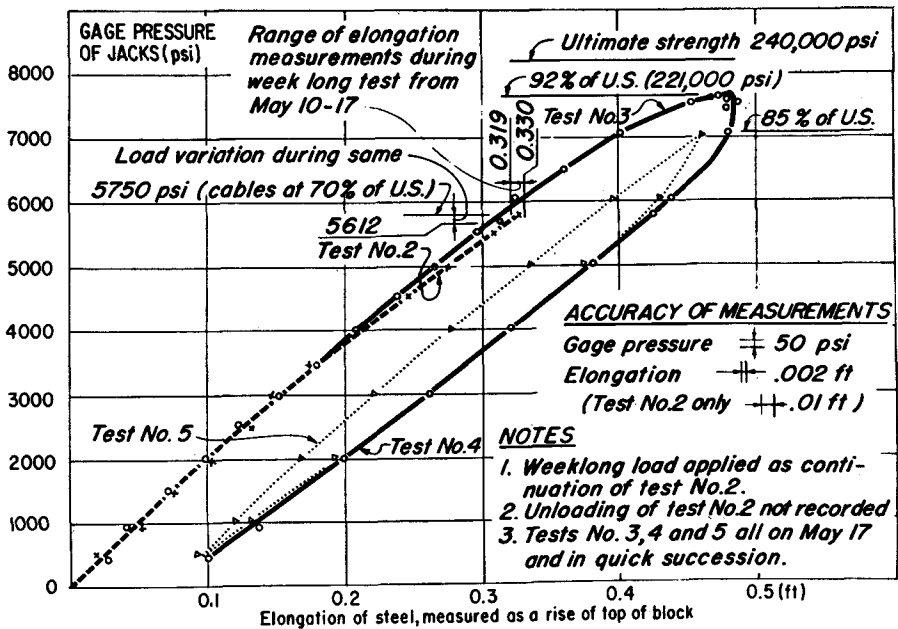


Fig. 10—Pull-Out Test Results No. 2, 3, 4 & 5, May 10-17, 1960

fects were noticeable. Thereafter the load was reduced to a nominal value, Test 4. Following this, a second loading cycle, called Test 5, was carried out, reaching a maximum load at 85% of the ultimate strength.

The behavior of the test apparatus was entirely satisfactory. Even at the minimum applied load of 92% of the ultimate strength only minor hair cracks in the reinforced concrete head were observed. Most of these originated at the corners of one of the square bearing plates. The other square bearing plate and the two circular plates did not show any cracking. Because of this, it was decided to use circular bearing plates for all cable installations.

#### DISCUSSION OF TEST RESULTS

During testing, the steel cable is not the only elastic body subjected to load. Deformations of rock, concrete

cushion, and grouted end of the anchor, however, were shown to be negligible in comparison with elongation of the steel cable. For a jack load of 5000 psi, the combined above deformations were estimated at 0.002 foot, which is equal to the accuracy of the elongation measurements, while the corresponding cable elongation was 0.26 foot (see Appendix).

A detailed analysis was necessary to explain the non-linear load-deflection relationships in various parts of the curves shown on Figs. 9 and 10. In the Appendix, it is concluded that these non-linear elongation increments cannot be explained satisfactorily by an increase of the effective length of the steel cable due to deformations within the grouted end, effective length being defined as the sum of the free length as shown on Figure 7, and a certain, unknown portion of the grouted

end.\*

Instead, all non-linear effects can be accounted for by the load deformation characteristics of steel. These steel properties and their effects are as follows:

1. Deviations from Hooke's law, even at small stresses—This causes the curvatures observed in Tests 1, 2, and 3 up to about 60-70% of the ultimate strength. The material behaves elastically throughout the entire range.
2. Plastic deformations, starting for this high-strength steel at 70-80% of ultimate strength—There is no definite yield point. The large elongation of Test 3 at the higher loads, as shown on Fig. 10, is explained this way.
3. Hysteresis phenomena—That this effect further contributes to non-linear behavior of the steel can be seen from Test 4, but even better from the closed loop of Test 5. Note that plastic deformations did not take place during this test, as the cable returned to its initial length at the end of Test 5.
4. Raising of yield point—After unloading the cable during Test 4, it will be found that during Test 5 yield does not take place because the maximum value of 92% of ultimate strength, previously attained, has not been reached.
5. Time effects—Reloading the cable during Test 5 immediately after the unloading of Test 4 causes additional deviations from Hooke's law, over and above those mentioned before

\*"Equivalent" effective length could be a better term because this length was computed assuming constant stress also along its embedded portion.

under 1 above. This phenomenon was not observed during Tests 2 and 3, since apparently enough time elapsed to permit recovery of the material.

The actual characteristics of the high-strength steel wires used for the pull-out test were determined separately in the Cleveland Research Laboratories of United States Steel. The modulus of elasticity is  $29.1 \times 10^6$  psi. Individual readings as recorded on the chart (Fig. 11) indicate the small curvatures mentioned in Point 1 above. The hysteresis phenomenon, plastic deformation, and raising of the yield point are demonstrated on Figure 12. It is of interest to note that such a correlation between field and laboratory tests was also established for the pull-out test performed for Steenbras Dam in South Africa (see Ref. 9).

The so-called effective length of the cable was determined through a trial-and-error procedure by matching pull-out test results with labo-

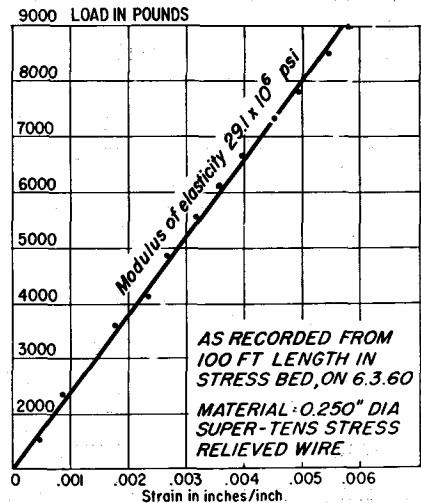


Fig. 11—Data from American Steel & Wire Div., U.S. Steel Corp. Laboratory Test

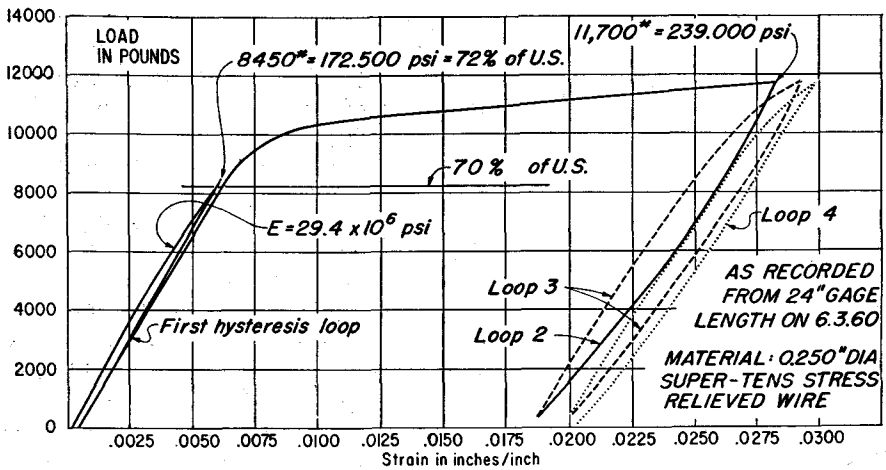


Fig. 12—Data from American Steel & Wire Div., U.S. Steel Corp. Laboratory Test

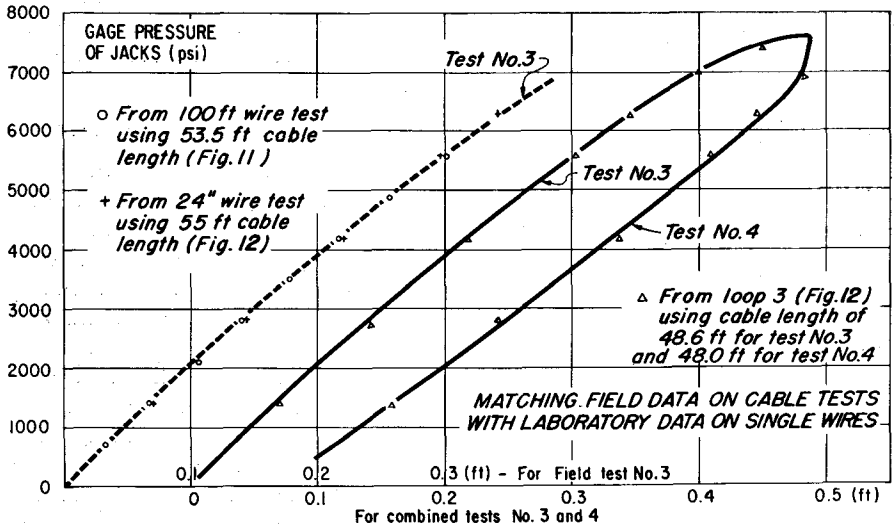


Fig. 13—Pull-Out Test, May 17, 1960

ratory data. Several such procedures are discussed in the Appendix and, as shown on Figure 13, result in an effective length of approximately 53.5 feet. With a free cable length of 46.34 feet, this leads to an effective embedded length of 7.16 feet. If it is assumed that strains in the cable decrease linearly from a maximum at the top of the grouted end equal to the strain in the free cable,

to zero at the fictitious end, the active embedded length would be twice as long, or 14.3 feet.

#### SIGNIFICANCE OF TEST RESULTS

The results of the pull-out test were also analyzed insofar as they could affect the anchor design. The following points were considered: length of grout anchor, depth of anchor, point of load application,

and the factors of safety.

### **Length of Grout Anchor**

The initial grout length of 20 feet shown on the contract drawings was increased in the test anchor to 30 feet at the suggestion of the contractor, based on BBRV practice<sup>5</sup> allowing a maximum shear stress of 170 psi at the "mortar column", i.e., along the outer periphery of the steel cable.

During the construction of the test anchor, however, a length of 37.66 feet was grouted accidentally. In the previous section an active embedded length of 14 feet was indicated. The question was now: What is the significance of these 14 feet with respect to the total length of 37.66 feet; would a length of 30 feet be sufficient or could the grouted length be reduced even further? The question of the factor of safety was intimately tied up with these.

First of all, it was realized that present knowledge is not sufficient to indicate a definite length of grout anchor which is acting. The complicated stress distribution within the anchor area will, as does all such stress phenomena, undoubtedly show a gradual change in stresses, but not linear, from maximum values to minimum or zero values. Professor Ross<sup>6</sup>, as well as others<sup>2,3</sup>, have indicated such a gradual change. In other words, instead of a length of 14 feet acting, it could well be that 20 or 25 or all 37.66 feet were active. This made it difficult to determine from deductive reasoning what anchor length should be used.\* Embedded length of only

14 feet would also result in average stress in grout-to-rock contact of about 300 psi. This stress was considered excessive in view of a straight, cylindrical hole, not underreamed, and uncertainties of underwater grouting.

On the basis of the results of the test and in the presence of a positive anchor head at the base of the cable, it was considered safe to reduce the grouted length to 30 feet, without conducting an additional field test to verify the behavior of such an anchor. At the same time, it was not considered prudent to further reduce the grout anchor length, and thereby go beyond existing experience without additional tests. Such tests were not conducted as the possible savings would be minor.

### **Load Point and Factor of Safety**

For reasons discussed above, the observed or, rather, the deduced active length of the grouted end was not a reliable figure. Similarly, little could be learned from the test about the location of the hypothetical point of load application.

The decision was made to follow the present practice<sup>5,8</sup> for rock anchor design and to simply assume that the anchor force acts at the midpoint of the grouted length. In the case of the Wanapum anchor with a 30-foot length, this led to a point 15 feet above the anchor plate. For the 70 feet of length beneath the rock surface, this left a vertical distance of 55 feet to the point of load application. This distance formed the height of the triangle in the 45° wedge theory, and it was also used in the application of Zienkiewicz's stability method.

From the discussions of the test results, it follows that the so-called

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\*The use of strain gages attached to the wires was considered at Wanapum, but the idea was abandoned under the pressure of the construction schedule.

effective anchor length is somewhat less (53.5 feet). However, it can be argued that any "give" of the foundation rock will result in a lowering of what might be called the effective point of application. Thereby, over-all stability should be improved as more rock is engaged in the grip of the anchor, or, in terms of actual stress distribution, tensile stresses are decreased in magnitude as they now extend to greater depth. In case of the positive anchorage of BBRV cables, such process could possibly continue until all anchor force is developed in the extreme bottom section. Obviously, there is a very large inherent "reserve" strength of the anchor available in case of extreme loading.

#### Loss of Prestress

The initial prestress will undergo only minor stress losses for the following reasons: Creep in the steel is expected to be less than 1% because most of the creep takes place soon after initial loading and will be eliminated by subsequent retensioning. The losses due to creep in the compression members can be only a fraction of those encountered in more conventional concrete structures because of the large masses of rock and concrete, and correspondingly low compressive stresses as indicated by the pull-out test. The losses due to shrinkage would be very limited in the shallow intake base slab, but would be somewhat higher in the piers. There would also be some uncertainty as to the amount of creep in the grout around the lower end of the anchor.

The question arose as to whether the initial prestress should be lowered in view of the low anticipated losses. The decision was to keep it at 70% of ultimate strength and thus

retain an extra margin of safety.

#### CONCLUSIONS FROM THE PULL-OUT TEST

The pull-out test of the proposed rock anchor was successful. It has demonstrated the soundness and practicability of the design and the excess strength available in the steel cable and the anchor grout. The construction methods and materials were found to be satisfactory, and were approved for use on the project. The circular bearing plates were selected.

The accuracy obtained in the measurements was unusually high because of the large gage length in the elongation measurements. This accuracy proved to be adequate for observing small non-linear steel deformations.

Part of the test data was shown to correspond with elastic, non-linear, hysteresis effects; other parts were explained by the normal yield characteristics of steel.

Effects of compression of rock, concrete cushion, concrete block and grout were shown to be negligible in Tests 2 through 5, and were included in the evaluation of Test 1 (see Appendix). The original tensioning load, however, equal to 70% of ultimate strength, was retained for use on all anchors.

From a review of all observations, it was concluded that the elongation measurements coincided with an effective cable length of 53.5 feet.

The grouted anchor length of 37.66 feet used in the test was reduced to 30 feet for use in the permanent structures. Considering the unusual capacity of the anchor, it would not have been advisable to decrease this length any further without additional testing.

It was shown that the proposed

anchor provides a factor of safety probably in excess of that computed on the basis of the conservative, conventional method of analysis, as well as on the basis of a recently proposed new method of stability analysis.

### CONSTRUCTION

In general, drilling of holes in rock preceded placement of concrete in the intake floor slab. The alignment of holes was checked carefully in order to make sure that they did not deviate excessively from the vertical center lines.

Maximum deviation found at the bottom of a hole was  $19\frac{1}{4}$  inches.

Where holes were kinked, they were reamed out to a larger diameter in order to assure at least  $\frac{1}{2}$  inch clearance between the rock walls and the cable wires when the latter were being tensioned.

Each four cables were pre-assembled into a single anchor, lowered into the hole as a unit (Figures 14 and 15), and the bottom 30 feet of the hole grouted. Grout was designed to develop a strength of 4000 psi in 14 days at which time tensioning would start. It was similar in composition to the grout used in the pull-out test. It was also required that the intake concrete at the bearing plates of the anchor heads would



Fig. 14—Crane Lifting an Anchor

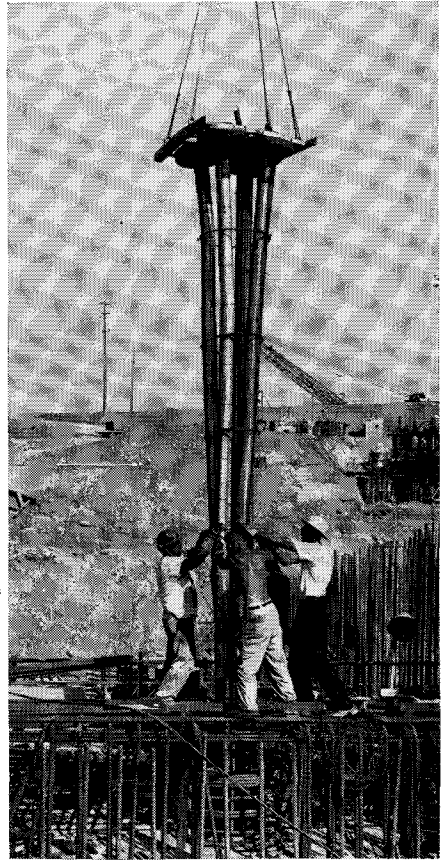


Fig. 15—Lowering an Anchor

reach 4000 psi strength before tensioning would be allowed.

Tensioning the cables (Fig. 16) and checking the tension after prescribed periods of time followed the requirements of the specifications. Before the upper portion of any anchor could be grouted, tensioning of adjacent anchors, one on each side, was required. This requirement applied only to the anchors within a single intake monolith.

Out of 78 rock anchors placed, only one showed signs of trouble during tensioning. When the load reached 67% of ultimate strength a permanent elongation of 7.5 inches developed. A core hole was drilled to intercept the bottom anchor grout and to recover samples for examination. Subsequent laboratory tests indicated poor compressive strength. It was concluded, therefore, that the grout was at fault. Possibly its lack of strength was due to a flow of ground water along the hole. After some consideration, the following corrective procedure was used in preference to the removal of the cable:

1. The core hole was pressure grouted.
2. After 15 days the cable was tensioned to 45% of ultimate strength. This was a "safe" limit based on a study of load-elongation curves recorded during the first attempt to load the cable.
3. The tension was checked after 24 hours and found practically unchanged. An additional 18 feet of the cable was grouted on top of the original 30 feet of grout.
4. After 14 days the cable tension was brought up to 70% of ultimate strength.
5. After another week had passed,



Fig. 16—Four 500-Ton Jacks for Stressing an Anchor

the tension was checked and the remainder of the cable grouted permanently.

During and after reservoir filling, the intake structures were surveyed frequently in order to check their structural behavior. The maximum measured deflection at the level of the intake deck was 0.047 foot. This corresponded to a deflection of 0.025 foot at the level of the downstream shelf at Elev. 500, and no movement was visible inside the gallery at Elev. 444.

Attempts to correlate the measurements with the computed deflections resulted in an effective foundation modulus of  $1.2 \times 10^6$  psi. This value appeared low, considering the general character of the rock. The deflection of the intake structures followed closely the rising water level, then stabilized with the reservoir full.

#### CONSTRUCTION COST

The actual cost of rock anchors was about \$750,000, which was con-

siderably below the estimated figure used in comparative studies of different ways to stabilize the intakes. The unit cost amounted to \$3.80 per kip of prestressing load. The above figures include drilling of holes in rock, furnishing and installing anchor cables and fittings, grouting, forms for formed holes in concrete, and the pull-out test. The prices of the two largest items, drilling 16"-diameter holes and furnishing and installing the cables, were respectively \$43.40 per foot of hole and \$0.59 per lb. of cable and fittings.

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Mr. D. P. Roberts was Project Manager in Chicago for Harza Engineering Company, and Mr. Robert B. Jackson, presently the Company's Chief Construction Engineer, was the Resident Engineer. He was succeeded in the latter post by Mr.

U. V. Engstrom at about the time the construction of the project reached its halfway mark.

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## APPENDIX

### DEFORMATIONS OF CONCRETE, ROCK, AND GROUT

The steel anchor is not the only elastic body subjected to load in this test. It is necessary to evaluate the deformations of the rock, concrete cushion, and grout anchor. On the other hand, concrete block and shim deformations do not affect elongation measurements except in Test 1.

As shown on Figure 7, the rock is loaded by a 10 x 10-foot-square concrete cushion. The average deflection at the top of the rock, assuming a uniform pressure distribution over the square area, was computed as 0.00049 feet at a load in the steel cables equal to 70% of ultimate strength (jack pressure of 5750 psi). The modulus of elasticity of rock was assumed at 4,000,000 psi.

Average compression of the cushion at the same cable load was computed as .00039 feet. The local deflection under the jacks was .00158.

The total deflection is, therefore, .00049 + .00039 + .00158, or .00246 feet. The accuracy of the deflection measurements is of the order of .002 feet. In view of this, and because of the very small ratio to the cable elongation, these deflections can be neglected. This is true also for the .0011-foot compression of the anchor block which entered into the calculations only for Test 1 at its maximum load of about 36% of ultimate strength.

The deformation of the bottom grout was obtained by superposition of the effect of the four separate cables. Each cable was assumed to represent a concentrated load ap-

plied at the top surface of a semi-infinite mass, and the resulting deformation was computed accordingly. The total elongation of .0076 feet is part of the behavior of the grout anchor and, therefore, is not to be used as a correction.

### DISCUSSION OF MEASUREMENTS AND COMPUTATIONS

#### Effective Length of Cables

In a pull-out test, the effective length of the cable is not readily available, since steel elongations inside the grouted cable end are not known. The wires of the cables, however, were tested in the laboratory to obtain the modulus of elasticity. The equivalent length of cable which leads to the measured elongation can then be computed from

$$L = \frac{\Delta L \cdot A \cdot E}{P}$$

The only remaining difficulty is to select from the measurements corresponding values of the load  $P$  and the elongation  $\Delta L$ . Five values for the effective length are derived as follows:

1. The first estimate of the effective cable length is based on the straight line portions of the curves formed by the data from the origin up to an elongation of 0.1 foot, as shown on Fig. 9. The tangent in the origin to the curves on both Figures 9 and 10 are about the same. The load corresponding to the elongation of 0.1 foot, however, cannot be determined very accurately. The gage pressure could be as small as 2000 psi or as high as 2100 psi. For the modulus of elasticity of steel

equal to  $29.1 \times 10^6$  psi (Fig. 11), the effective length is computed as 49.7 feet (for 2000 psi gage pressure). If the gage pressure is 2100 psi, this effective length reduces to 47.3 feet.

An average value for this method would be 48.5 feet. With a free cable length of 46.34 feet (see Fig. 7), the effective embedded length would be  $48.5 - 46.34$  or only 2.16 feet.

2. A second way of determining an effective cable length is to match the entire elongation curves shown in Fig. 10 with the original load-strain data of the steel, as given by the manufacturer for a 100-foot wire length and shown in Fig. 11. In order to compare these curves, the load-strain data must be changed to a load-elongation curve which requires the assumption of an effective length. The curve best suited is plotted on Fig. 13 and corresponds to an effective length of 53.5 feet. This leads to an effective embedded length of 7.16 feet.

3. The third method is identical to the second, except that now the manufacturer's results for a 24-inch gage length are used, as shown on Fig. 12. An effective length of 55 feet appears to give the best results now.

4. A fourth method is matching of the hysteresis loop of the material shown on Fig. 12, with the load curves of Tests 3, 4, and 5. The match, shown on Fig. 13, indicates an effective length of 48.6 feet for Test 3, and of 48 feet for Test 4. No further computations for Test 5 were carried out.

It is clear, from the above computations, that it is necessary to discuss in more detail the several phenomena of non-linearity which have affected the test results.

## Discussion of Measurements—Test 1

The results of this test are plotted on Fig. 9 together with those of Test 2. The data show a distinct curvature which could be due to gradual increase of effective length with increased load, or could be caused by non-linear behavior of the steel wires in the cable. The elongation may also be influenced by any one or a combination of the following factors: closing of fissures in the rock, deformations in the grout, and flattening of slight curvature in some shim plates.

The closure of rock fissures under the small rock stresses at the first few load increments would, if any took place, be an elastic phenomenon. Even at 3000 psi jack pressure, with a cable load of 1,551,000 pounds and assuming an effective rock support area equal to that of the cushion,  $10 \times 10$  feet, the uniform rock stress is only about 100 psi. If elastic, the same effects should have returned for Test 2. They did not. The character of this hard columnar basalt is such that permanent closure of fissures is highly unlikely because it is doubtful that such fissures exist at all except for vertical fissures which could have little effect here. In the third place, if a fissure was closed, it would result in a reduction of elongation measurements, resulting in an upward curve. This did not take place. Because of these arguments, closure of rock fissures as the cause of non-linearity should be eliminated from further consideration.

Deformations of the grout due to concentrated loads were computed above. Their effect is very small. Necking of the steel inside the upper portion of the grout anchor and accompanying slippage of the steel wire in the same area are expected

to be very small and do not explain the anchor behavior as shown later on.

The flattening of some of the shim plates which were originally slightly dished should have some effect. It was noticed during the test, however, that a gap of the order of  $\frac{1}{16}$  inch almost closed completely with the first application of 500 psi jack pressure. Again, this phenomenon, if important, would have steepened the curve, rather than flattened it.

A more likely explanation of the observed curvature would be the increase of the effective length of the

cable under increasing loads. Using the same approach as previously for the computation of the effective length, the data in Table 1 were prepared at each load increment for Test 1. With an original free length of 46.34 feet, the computed increase of effective length in Table 1 appears to be a satisfactory explanation of the curvature of the test data except for the lowest pressures. The possibility of explaining this curvature on the basis of non-linear behavior of steel will be discussed later on. Final conclusions will be drawn after discussion of similar

**TABLE 1—APPARENT EFFECTIVE LENGTH OF CABLES**

Actual Gage Pressure (psi)	Elongation (ft.)	Increase in Ft. per 1000 psi	Effective Length (ft.)	Remarks
<b>TEST 1</b>				
519	.028	.054		reading not accurate
995	.047	.040	39.7	
1538	.072	.046	45.7	
2020	.097	.052	51.7	
2505	.123	.054	53.6	
2900	.160	.094	93.1	unreliable—failure of pulling rod
<b>TEST 2</b>				
975	.050	.0513	51.0	
1950	.100	.0513	51.0	
2987	.148	.0464	46.0	
3994	.208	.0595	59.1	
5000	.278	.0696	69.2	
5781	.327	.0628	62.3	
<b>TEST 3</b>				
975	.046	.0472	47.0	
1987	.098	.0514	51.1	
3000	.151	.0523	52.0	
4000	.209	.0580	57.7	
5000	.267	.0580	57.7	
6000	.324	.0570	56.7	
7000	.400	.0760	75.5	Meaningless— yielding of steel
7600	.474	.123	122.0	
<b>TEST 4</b>				
7000	.480	.0100	9.4	Meaningless— hysteresis of steel
6000	.438	.052	51.8	
5000	.380	.058	57.7	
4000	.321	.059	58.7	
3000	.261	.060	59.6	
2000	.198	.063	62.6	
1000	.137	.061	60.6	

effective length computations for Tests 2, 3, and 4.

### Discussion of Measurements—Test 2

The results of this test are plotted already on Figures 9 and 10 as gage pressures of the jack versus elongation at the top of the block. The larger scatter of data for Test 2 is due to round off errors that occurred here in rod readings; the resulting accuracy is only 0.01 foot instead of 0.002 foot.

At gage pressures of 4000-5000 psi stresses in the cable reach 60-70% of the ultimate strength, and it is quite possible that initial yielding of the steel is sufficient to explain deviations from the straight line. If increased effective length alone accounts for the elongation, then the effective length at 70% of the ultimate strength is about 62-69 feet, as shown in Table 1. This will be further commented upon after presenting data for Tests 3 and 4.

### Discussion of Measurements—Test 3

Data for this test are plotted also on Fig. 10, and effective length increases are computed in Table 1. Elongations follow those of Test 2 rather closely up to about 3000 psi gage pressure, while above this pressure Test 3 shows much less curvature.

### Discussion of Measurements—Test 4

The decrease of the load from 92% of ultimate strength is shown on Fig. 10, and effective lengths are computed in Table 1. It is clear from the plot and these lengths that the hysteresis phenomenon of the steel is involved.

### Discussion of Measurements—Test 5

The loading and unloading cycle following Test 4 was called 5, and is also shown on Fig. 10. Effective

lengths were not computed as these data would not throw any new light on the explanation of the observed phenomena.

## EXPLANATION OF RESULTS

In order to decide whether changes in effective length should be assumed as an explanation of the test results, or whether steel wire properties account for these data, a separate discussion of these two will now be presented.

### Variable Effective Length Theory

At first the hypothesis of increasing effective cable length under increasing loads appears attractive. In the following, it will be shown that the hypothesis is not consistent with all available observations. Two arguments will be developed against the hypothesis, one based on simple comparison of data in Table 1 and one involving elastic properties of the materials involved.

The effective length computed from Tests 1 and 2 differ considerably, especially when it is known that the free length of the cable is 46.34 feet. The first observations in Table 1 result in effective lengths which are smaller than this free length, which is of course absurd. Disregarding the length of 93.1 feet for Test 1 because of failure, one can still explain the two highest values for Test 2 as an increase of effective length, perhaps due to slip of the wires near the top of the grout. Comparison of these latter data with the results of Test 3 at the same pressures, however, can only lead to the conclusion that effective length increase is not the explanation for the highest values attained in Test 2. This difference is also shown clearly on Fig. 10. If there was such an increase, the effective length in

Test 3 could not be smaller.

The highest values of elongations for Test 3 can be identified as yielding of the steel rather than effective length increase. This would seem to be true also for the first value of Test 4, where the sudden load drop is almost certain to be caused by the hysteresis phenomenon of the steel. The middle portion of Test 4 appears well in agreement with that of Test 3, but at the lower loads one would now expect a decrease in effective length rather than the increase shown in Table 1. From these several contradictions it can be concluded that the hypothesis of increased effective length is not tenable.

## Properties of Steel Wires

### *Accuracy of Measurements*

Small deviations from linear stress-strain relations can be measured with the equipment available for the field test. The following comparison with laboratory instruments shows that field data have an accuracy as high as 0.1 of laboratory data:

Huggenberger	
mechanical gage	0.000 006 in/in
Tuckerman optical gage	0.000 002 in/in
Electrical strain gage	0.000 004 in/in
Field rod and level with	
50-foot gage length	0.000 040 in/in

The high accuracy obtained for the field data is also indicated on Fig. 10.

### *Non-Linear Behavior of Steel*

From laboratory measurements on tensile specimens, it is known that even a single crystal specimen shows some deviation from Hooke's law in the elastic range<sup>7</sup>. This phenomenon can be explained by the thermal effects. When a specimen is loaded relatively quickly, there is no time for heat exchange with the surround-

ings. The "sudden" elongation causes the specimen to cool. When the specimen remains under load for some time, it will warm up again to the temperature of its environment and consequently it will elongate slightly. This elastic after-effect is due to the thermoelastic behavior of the material. It results in a higher value for the modulus of elasticity during the quick test and a reduction due to time.

In actual tests, some heat exchange will take place during the test itself, especially in the field test which lasted about half an hour. Thus the time elongation takes place, at least partly, during the test, causing the elongation to be larger than the straight line would imply and thus show a distinct curvature. For steel, the difference in modulus of elasticity is about one-third to one percent. This phenomenon is called hysteresis and since the specimen returns to its initial state the term elastic hysteresis is sometimes used. An example of such a hysteresis curve is Test 5, shown on Fig. 10.

The difference between the loop of Test 5 and the one of Tests 3 and 4 can be explained as follows: In Test 3, the hysteresis curvature is overshadowed by the plastic deformation of the steel, which constitutes a permanent elongation of the wires. With larger stresses the so-called internal friction becomes more complicated since not only the mechanical energy losses due to heat exchange have to be considered, but also the losses due to plastic deformation within the individual grains. This yielding of individual crystals contributes to the loss of energy and increases the area of the hysteresis loop, as shown by the larger area between Tests 3 and 4. An important point is now that after un-

loading of the specimen (as in Test 4) renewal of the tensile load (Test 5) will not cause further sliding in these crystals until the tensile load reaches the value previously applied. Therefore, the higher loads of Test 5 show hysteresis phenomenon, whereas in Test 3 additional deformation is due to plastic deformation. Another way of expressing the effect of Test 3 is to say that loading up to 92% of the ultimate strength eliminated the plastic behavior of the material up to that load. The cable now behaves elastically, though nonlinearly, up to that point.

#### *Time Effects on Steel Deformations*

Investigations<sup>7</sup> have also shown that the time which elapses between unloading and reloading is of great influence on the stress-strain curve during reloading of the specimen. This affects the curvature of Test 5 as compared to Tests 2 and 3. If reloading begins immediately after unloading, deviations from the straight line occur at very low stresses, and the proportional limit is greatly reduced. If, however, a considerable interval of time elapses between unloading and reloading, the material recovers its initial elastic properties completely. During the field tests, Test 2 occurred about six hours after unloading Test 1, and Test 3 did not commence until after removal of load for about four hours. Indeed, comparison of Tests 2 and 3 with Test 1 would indicate that these periods are sufficient for recovery of the material. Test 5, however, was executed almost immediately after concluding Tests 3 and 4.

#### **Explanation of Measurements**

The non-linear elastic effects observed in the various tests have been identified exclusively in the previous section as characteristics of the steel

wires only. It was also shown that the observations could not be explained as an increase in effective length with load increase and vice versa. Whether or not combination of these effects takes place cannot be determined directly. A small effect due to increased length could be present, but this is actually not too important. An idea of the effect of the steel wire properties on the tests can be obtained by comparing laboratory data for the same steel wires with the test results.

#### *Test Results of Steel Wire*

The load-strain relationship below the proportionality limit, as measured on a 100-foot-long specimen, is shown on Fig. 11. The straight line through the laboratory measurements indicates a value of  $29.1 \times 10^6$  psi for the modulus of elasticity.

The load-strain relationship at higher loads is shown on Fig. 12. The modulus of elasticity for the first loading is computed as  $29.4 \times 10^6$  psi. It is important to note that the gage length for this test is only 24 inches, as the modulus of elasticity reduces slightly for larger gage lengths.

The hysteresis phenomena are also shown on Fig. 12. The first cycle at low stresses, shown on the extreme left of the drawing, is of little significance. The second, third, and fourth cycles, however, clearly show the steel properties previously discussed, namely, a strong curvature even at small loads, and an increase of the yield point of the material.

#### *Comparison of Test Results With Laboratory Data on Steel Wires*

The hysteresis curve obtained from laboratory tests and identified as Loop 3 on Fig. 12 consists of a loading and unloading cycle com-

parable to the combination of Field Tests 3 and 4. The only difficulty in comparing the two is that the field test is based on total elongations for an unknown length and the laboratory tests are for strains. It was possible to virtually match these curves by assuming a certain effective cable length. The converted Loop 3 of Fig. 12 is shown together with Tests 3 and 4 on Fig. 13. The test results would indicate a lower limit for the effective length of about 48.6 feet.

### *Effective Cable Length Estimate*

In the preliminary discussion of the effective length, several contradictory results were obtained. These were:

For tangent to curves in origin	47.3-49.7 feet
Matching loading up part of curves	53.5 feet
Hysteresis curve of Test 3	48.6 feet
Hysteresis curve of Test 4	48.0 feet

### *Conclusions*

In view of the above discussions, the following can be concluded:

The hypothesis of changing effective length does not stand up in the light of the various comparisons made.

The hysteresis curves lead to too small values of effective length because of comparison of field test results with results of 24-inch gage.

The matching of the load curve on Fig. 11 for the 100-foot-long cable with loading Tests 2 and 3 should give the most reliable results. The length so determined is about 53.5 feet.

The effective cable length of 53.5 feet leads to an embedded section of  $53.5 - 46.34 = 7.16$  feet. Or assuming that the cable elongation decreases linearly from a maximum, equal to the strain in the free cable, to zero at the fictitious end, the embedded "active" length would be twice as long, or 14.3 feet.

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