ASSESSMENT OF STRUCTURAL ADEQUACY

OF THE

CONCRETE FOUNDATION CELLS

FOR THE

EMERGENCY RAW COOLING WATER PUMPING STATION

AND

ACCESS ROADWAY

SEQUOYAH NUCLEAR PLANT

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EXECUTIVE SUMMARY

TVA contracted with Bechtel to perform an in-depth assessment of the structural adequacy of the Emergency Raw Cooling Water (ERCW) Pumping Station foundation and Access Roadway foundation cells. This assessment was required by the Nuclear Regulatory Commission (NRC) as a condition of the permit to restart commercial operation of the Sequoyah Nuclear Power Plant.

BACKGROUND

Concrete for these foundations was placed by the tremie concrete method in 1976-1977. At the time of the foundation construction, exploratory drilling confirmed that several "cavities" existed in the foundation concrete. Two of the roadway cells were removed and reconstructed. Subsequent analytical evaluation of the ERCW foundation, both in 1978-1979 and 1988, have consistently verified that the structural behavior of the as-built foundation would meet all design requirements with adequate safety margins.

However, the "cavities" (which are actually filled with segregated concrete constituents such as coarse aggregate, sand and cement paste) were not grouted after the exploratory drilling was completed.

SCOPE OF CURRENT REVIEW

Although the previous evaluations considered all of the factors mentioned above, an additional comprehensive assessment was performed to evaluate the records and address the specific topics listed below.

A. FOUNDATION CONSTRUCTION

The records indicated that the construction was performed using "state-of-the-art" techniques available at that time, although improvements could have been made in controlling the consistency of the concrete mix and techniques used in the restart of the tremie placement operation for the Pumping Station foundation.

B. ASSESSMENT OF TEST AND EXAMINATION DATA

Evaluation of the test and examination data indicated that the cells mostly consisted of sound concrete. The only questionable conditions having a potential impact on structural performance were the existence of lenses containing segregated concrete constituents in the Pumping Station foundation. The cell-rock interface was found to be consistent with assumed design conditions with the exception of two access roadway cells where the interface contains pockets of segregated aggregate.

C. STRUCTURAL EVALUATIONS

An additional structural evaluation was performed for the pumping station foundation using the following conservative assumptions: the worst case spacial distribution of anomolies at a single elevation is located at the same elevation with minimum cross-sectional area and maximum shear and moments, concrete carries no tension, and the gravel pockets contribute no strength to the structure.

Evaluation of the roadway cells also incorporated conservative assumptions by considering single cell response, bounding the possible range of concrete material properties, and justifying the load resistance capability of confined gravel at the cell-rock interface.

Even with these conservative assumptions, calculations for each of the structures indicate adequate structural performance in the current as-built conditions.

D. ASSESSMENT RESULTS AND ADDITIONAL EXPLORATION

Although the current analyses still verified adequate structural performance, a recommendation is made to perform a limited exploratory drilling program to augment the available data utilized in assessing the condition of two critical areas of the ERCW foundation located between the intake tunnel pairs in each cell. The drilled holes should be grouted to assure that the concrete in the areas investigated remains compatible with assumed conditions and/or is restored to the original design requirements.

Drilling and grouting is not recommended for the Access Roadway cells nor for the remainder of the ERCW foundation, since any improvement in the overall structural behavior from such a program would have negligible benefit. Additional drilling and grouting in the ERCW foundation would only be necessary if the information obtained from the limited program is found to adversely deviate from the presently available data.

CONCLUSION

The ERCW Pumping Station and access roadway cells are structurally adequate and will function as intended under the design loads. The recommended exploratory drilling and subsequent grouting for the Pumping Station cells will further confirm the predicted condition of the structure and will assure that the structure performs in a manner consistent with the originally envisioned design requirements. Since the existing condition of the structure does not affect the operability or safety of the plant, the proposed work can proceed during normal operation.

1. INTRODUCTION

As a condition to restart commercial operation of the Sequoyah Nuclear (SQN) Power Plant, TVA was requested to perform an in-depth assessment of the structural adequacy of the plant Emergency Raw Cooling Water (ERCW) Pumping Station foundation and Access Roadway cells. This requirement was prompted by the Nuclear Regulatory Commission's (NRC) concern that available records do not preclude the existence of open cavities in the concrete foundation cells, that TVA's calculations do not envelop the existing conditions, and that sufficient justification was not provided to obviate the need for grouting.

The ERCW Pumping Station is a reinforced concrete structure (Figure 1-1) 67 feet x 98 feet in plan and 47 feet in elevation (with 67 feet of mass concrete foundation under it) which houses eight ERCW pumps along with their traveling screens and other associated equipment. It is located just offshore in the Chicamauga Reservoir and is connected to the mainland by a rockfill dike and a series of concrete cells that form a continuous structure out to the ERCW Pumping Station (Figure 1-2).

The ERCW Pumping Station is supported on an unreinforced concrete foundation consisting of two 67 feet diameter cylindrical concrete cells founded on rock and formed with sheet piling that overlap sufficiently to produce an overall nominal dimension of 98 feet along a horizontal line running through the central axis common to both cells (Figure 1-3). The chord formed at the overlap of the two cells was connected with sheet piling to form a stable structural configuration prior to and during the filling of the cells with concrete. The cells are founded on bedrock to produce a foundation about 67 feet high. Under normal conditions the lower 55 feet of the foundation remains below the reservoir's water level.

Each foundation cell was designed to be solid concrete with the exception of four intake conduit liners, two of which were installed in each cell prior to concreting. These liners are fabricated of steel plate and pipe with appropriate stiffeners and function as blockouts to permit intake of the raw cooling water, installation and access for the traveling screens, and risers for the raw cooling water pumps (Figures 1-3, 1-4, 1-5, and 1-6).

The access roadway cells (Figures 1-2, 1-3, and 1-5) are also cylindrical structures about 32-2/3 feet in diameter formed by sheet piling and founded on rock. As seen in Figure 1-2, sheet pile-formed connector cells have been added between all the access roadway cells and the ERCW Pumping Station foundation to interlock the individual cells and to form the foundation for the roadway extending from the land out to the ERCW Pumping Station.

The ERCW Pumping Station piping and utility service is located beneath the roadway and connects directly to the Pumping Station at the termination of the road.

In response to the NRC's requirement, TVA requested that Bechtel perform an independent in-depth review of all available documentation and provide recommendations to address the structural adequacy of the ERCW Pumping Station foundation cells and access roadway cells. This work included a review of all available data such as: reports, calculations, drawings, memoranda, correspondence, specification, procedures, and construction records; an evaluation of the construction methodology utilized; a comparison to standard industry practice; and an independent assessment of all attributes which might affect the integrity of the installed structures. The resulting conclusions and recommendations were further reviewed by a recognized industry authority to obtain concurrence.

This report contains results of the review performed by Bechtel, conclusions drawn, and recommended resolution. The question raised by the NRC relates to the structural integrity of the foundation itself, not the superstructure it supports. Therefore, no review of the pumping station superstructure, piping, or equipment was performed.

The review was performed by engineers specialized in their areas of expertise with an overview provided by a consultant with extensive experience in all phases of the subject matter.

2. FOUNDATION CONSTRUCTION

2.1 Scope of Construction Activity

Construction of the ERCW Pumping Station foundation cells and the access roadway foundation cells were included as a part of a construction contract let to Gulf Foundation, Inc., St. Petersburg, Florida, in September 1975.

Primary work items required to construct the foundation cells were:

- Excavation and disposal of overburden and rockfill
- Excavation, disposal, and cleaning of bedrock
- Installation of steel intake conduit liners (fabricated by TVA)
- Placement of sheet piles which serve as formwork for the cell concrete
- Furnishing and placing unreinforced tremie concrete in cells
- Furnishing and placing grout in voids between steel liner and tremie concrete (responsibility for this task was subsequently assumed by TVA)

2.2 Initial Construction of Roadway Cells E and F

Gulf Foundation started work on the contract in November 1975, and placed the required tremie concrete in access roadway cells E and F in mid-March 1976. About a week after placement of cell E, recommendations were made to perform exploratory core borings in both cells for their full depth and at least 10 feet into bedrock "because of reported construction conditions" (2-1)* which questioned the integrity of both the concrete and the material on which the cells were founded.

Cores from ten holes were drilled and logged in mid-May. These borings identified pockets or seams of material with the following typical descriptions (2-2):

- o Clean aggregate
- o Weak, friable, sandy material
- o Weak, friable concrete
- o Cement paste, no aggregate
- o Poor concrete
- o Sand
- o Weak concrete with little or no coarse aggregate

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* Numbers in parenthesis refer to the references in Section 7.

The conclusion of the ensuing investigation was that cells E and F were placed on an unsuitable foundation interface and that the concrete as placed was of poor quality as a result of the Contractor not following his own placement procedure (2-3).

The situation was declared to be a reportable condition and was reported to the NRC in mid-June 1976. Subsequently, cells E and F were removed and reconstructed using modified procedures to assure an acceptable cell-rock interface and to produce cells with suitable concrete integrity. This activity was completed by July 1978 (2-4).

2.3 Construction of the Existing Cells

Both the Pumping Station and the roadway cells were constructed subsequent to the initial, unacceptable construction of the roadway cells E and F. Concrete was placed in all the foundation cells by use of tremie pipes equipped with hoppers to receive the output of concrete pumps (Figure 2-1). Tremie concrete for the Pumping Station cells was placed during the first half of June 1977 and the roadway cells were placed during August to October 1977. Prior to concrete placement for the ERCW Pumping Station foundation, Gulf Foundation's written procedure for placing tremie concrete by using concrete pumps was revised six times between its initial issue in March 1976 and Revision 6, issued in May 1977 (2-5). Additionally, after the unacceptable roadway cells E and F had been placed, TVA approved a contract revision to engage Suboceanic Consultants, Inc., Naples, Florida, an engineering firm specializing in underwater inspection work (2-6), to assist TVA personnel in examining and mapping the prepared foundation surface so that TVA could accept the work prior to placing the tremie concrete.

2.4 Placing Concrete by the Tremie Method

Placement of concrete by the tremie method is a technique of placing concrete under the surface of fresh concrete that has been previously placed. The primary purpose of the tremie method is to place concrete underwater with minimal disturbance or segregation of the constituents of the concrete mix by lowering a vertical pipe (called a tremie) into the form so that the pipe's discharge end is very close to the bottom of the placement at the start, and subsequently kept below the upper surface of the fresh concrete. At the start, concrete is then fed into the pipe behind a plug [sliding piston, Figure 2-2(a)] that is intended to prevent the concrete from mixing with the water until it exits from the pipe. When concrete exits from the pipe directly into water, some mixing and sedimentation (often called washout) occurs which reduces the quality of the concrete by increasing the water/cement ratio (thereby lowering strength) and/or separating and segregating the constituents of the concrete into nonhomogeneous segments, but once a seal is established between the fresh concrete and the bottom of the pipe, washout should not occur [Figure 2-2(b)]. The washout condition is worsened if the surrounding water is flowing as concrete drops through it. Accordingly, discharging the concrete directly into the water rather than the established mass of fresh concrete can produce:

(1) concrete of reduced strength due to mixing with additional water, and

(2) pockets of cement paste, sand, coarse aggregate or variable mixtures of these constituents.

Because tremie concrete is normally quite workable and placed at a slump in the range of 6 to 9 inches it is expected that it will flow into place as a homogeneous mass within the form. Placement in calm water while maintaining the tremie pipe buried below the surface of the fresh concrete will minimize mixing of the water with the concrete and would leave only a degraded layer of concrete at the top surface of the placement. This degraded layer should subsequently be removed. As the concrete is fed into the placement through the tremie pipe, no mixing will occur with any material other than the same homogeneous concrete provided a seal is maintained consisting of workable concrete above the discharge of the tremie pipe. Agitation and disturbance of the freshly placed concrete should be avoided, as this can result in washout of the cement.

If the seal is lost due to inadvertently raising the tremie pipe too high, there are three options for restarting concreting operations:

(1) continue placing concrete on top of concrete already placed by refilling the tremie [Figure 2-2(c)],

(2) continue placing concrete by inserting the tremie into the freshly placed concrete and then refilling it [Figure 2-2(d)], and

(3) stop placing concrete and prepare a construction joint on the underwater concrete surface prior to restarting tremie placement on the concrete in the normal manner [similar to Figure 2-2(b)].

Using options (1) or (2) will result in at least one seam of nonhomogeneous material remaining in the concrete placement. Option 1 is especially deleterious as it results not only in washout of cement from the initial batches of newly-placed concrete but also in wash-out of the previously placed concrete around the tip of the tremie pipe. It seems clear that even if a plug is inserted in the tremie pipe prior to reinserting the pipe in the fresh concrete, the water in the pipe would be expected to be forced into the fresh concrete, washing out the cement and, thus, creating localized lens of segregated material when concrete placement is resumed.

Apparently, this later condition was allowed to happen during the concrete placement, as noted in internal TVA meeting minutes (2-7) which state that the Contractor's placement procedure should "be corrected to read that should the tremie seal be lost during placement operations the tremie pipe would then be reinserted into the concrete as far as possible (instead of inserting the tremie 2 to 3 feet into the concrete) and tremie operations resumed."

No mention is made of inserting a plug (seal) in the tremie pipe prior to restarting although it is recommended in ACI 304. On the other hand, the TVA specification (2-8) requires: (1) tremie concrete to be brought up continuously to the required height, (2) introduction of a lift joint if equipment breakdown requires that placement of concrete be stopped, and (3) cleaning of lift surfaces sufficient to provide good bond with the concrete placed to continue the structure.

2.5 Previous Assessment of Foundation Construction

The implication of the TVA Specification is to produce a homogeneous, continuous concrete mass even though the permitted procedures for starting and reinsertion of the tremie pipe, if the seal is lost, would produce a seam of weak, segregated material. In fact, records from exploratory holes drilled downward through the ERCW Pumping Station foundation (2-9) indicate numerous anomalies in many of the boreholes of the otherwise continuous, homogeneous concrete. These interruptions are described as: cavity (gravel), and soft seam or soft material (concrete).

A clarification must be made, however, that "cavities" should not be construed to mean open voids. Such features would be an impossible result from the tremie concrete placement technique employed in this situation. Due to the fluidity of the concrete mass, it would be impossible for the plastic concrete to arch over and create a water filled "cavity." A more correct description would be pockets or lenses of sand or gravel created by the segregation process resulting from the apparent operational procedures employed by the Contractor.

The revised final report to the NRC concerning the ERCW Pumping Station and access roadway cells (2-4) correctly notes that the pockets of aggregate found in the ERCW Pumping Station foundation cells by the drilling program "were attributed to washout whenever the discharge end of one of the sixteen tremie pipes was inadvertently pulled out of the concrete." Thus, the means of creating the pockets were clearly recognized by TVA. It is reasonably concluded that the same basic scenario also apply to the creation of the other interruptions for both the Pumping Station and access roadway foundation cell concrete.

It should also be noted that since the unreinforced tremie concrete section is comprised of a relatively high cementitious content mix, it would be expected to undergo some thermal cracking with time due to stresses resulting from restrained thermal shrinkage. This is a common occurrence with tremie concrete. Thus, some inter-connection between the various sections of nonhomogeneous material and any discontinuities would be expected due to this thermal cracking.

Additional cores were also drilled in the roadway cells. Unlike cores taken from the ERCW Pumping Station foundation, these cores indicate continuous, homogeneous concrete throughout the full height of each roadway cell with the exception of some minor anomolies near the top surface of cell D and some pockets of segregated aggregate located at the cell-rock interface of cells D and F. These anomolies will be discussed further in sections 3.3.1 and 3.3.3 of this report.

In February 1978, TVA decided that it would not be necessary to grout the core holes in the roadway cells as well as the Pumping Station. Instead, all exploratory holes in these areas were to be plugged by backfilling with an appropriate concrete mix (2-10). The only exception noted was a requirement to grout the holes in roadway cell D, but this requirement was later rescinded in October 1978 (2-11). Specific actions which resulted from these various instructions were not noted in the available documentation.

In January 1980, holes were drilled in the walls and floor of the intake conduit liner L2 (Figure 2-1) to investigate the "dead" sounding spots found in the liner system. Field notes (2-12) made prior to drilling holes in the floor indicate the liner was found to be approximately a uniform 1/8 inch away from the concrete (this applies to holes drilled in the walls). Two holes drilled in the floor showed voids of 8 inches and 2-1/2 inches between the floor liner and the concrete under it. Other documentation (2-13) states that the void distances were 7-1/2 inches and 2-1/4 inches. These voids indicate that concrete of better flowability and/or improved placement procedures may have been necessary to produce a completely uniform, homogeneous concrete placement around such large obstructions which included numerous stiffeners and the supporting frames.

No records were provided for this study which would confirm that any grouting or backfilling of the exploration holes in the ERCW Pumping Station foundation was ever performed. It is prudent to assume that no grouting was intentionally accomplished and if the exploration holes were backfilled it was not recorded.

2.6 Co[illegible] Current Standards

In 198[illegible] concrete by the tremie method was comp[illegible] 14). [illegible] findings of this res[illegible] sh[illegible] specificati[illegible] tremie concrete in use [illegible] significantly flawed and should be revised [illegible] the findings of [illegible] research. This suggests, of [illegible] at the specifications for and execution of cons[illegible] of the foundation for the ERCW Pumping Station a[illegible]dway cells could be substantially improved today with the benefit of the now-available knowledge.

The study confirmed the importance of flowability and cohesiveness in the concrete mix used in tremie placements. While the available information from construction records does not permit a direct evaluation of the cohesiveness and flowability of the tremie concrete mixes (Table 2-1), the record suggests that the basic mix designs were probably acceptable but could have been improved. The specific properties were as follows: 574 or 575 lb/cu yd of cementitious material (cement and fly ash, compared to a recommended content of 705 lb/cu yd), a sand/total aggregate ratio of 45-46% by weight (42-50% recommended), and a water/cementitious material ratio of 0.49 or 0.45 (recommended maximum = 0.45). The mixes properly included air-entraining and water-reducing agents, although their effectiveness in promoting cohesiveness and flowability cannot be evaluated from the available data.

The primary problem revealed by the concrete cylinder data sheets (2-15, 2-16) is that the consistency of the concrete produced for the ERCW Pumping Station foundation cell was poorly controlled. This is demonstrated by the variation in fresh concrete properties summarized in Table 2-2. In particular, the range of slump values for the ERCW Pumping Station foundation cell concrete spans 4 inches if the high and low values are discarded and 5 inches if all data is included. This is substantially greater than the 3 inch range for the roadway cell concrete when all values are considered. The ERCW Pumping Station values were all collected in a continuous placement over five days, while the roadway values were generated on ten different placement days occurring in a one and one-half month period, a situation that would lead to the expectation of a greater variation in the batches irregularly produced over a longer time span. Comparison of the air content and unit weight values similarly demonstrate the questionability of the ERCW Pumping Station foundation cell concrete control.

The demonstrated variability in the consistency of the concrete mix for the ERCW Pumping Station foundation cells indicates that during placement, the surface of the plastic concrete may not have risen uniformly over the entire plan area of the ERCW foundation cells. This greatly increases the probability of washout of the freshly placed concrete which would lead to pockets of segregated material within the finished product. It is clear that this was the end result in the ERCW Pumping Station foundation cells, while the more uniform concrete in the smaller roadway cells would have helped to produce a more uniform, homogeneous finished product. The overall effect in the end product will be discussed later in this report.

It was noted that the procedures approved for placing the tremie concrete in the ERCW Pumping Station foundation cells permitted lateral movement of the tremie pipes during the placement operation. Although this practice is discouraged in the current recommended standard of practice (2-14), recent verbal discussions with a representative of the contractor who was present during the placement operation indicated that the tremie pipes were not moved horizontally during concrete placement thereby eliminating this as a possible contributor to the problems encountered in the placement of the ERCW foundation cells.

3. ASSESSMENT OF TEST AND EXAMINATION DATA

3.1 Test Data for Insitu Concrete

The following items of data related to the insitu concrete were available for review:

ERCW Pumping Station foundation cells:

- Rock and Concrete Investigations, including drilling logs and geophysical surveys using sonic, gamma-gamma, caliper, and sonic cross-hole techniques (2-9)
- Concrete Cylinder Data Sheets (2-15)
- Concrete Pour Card for Seal Pour around Inside Edge of ERCWPS (3-1)
- Description of Concreting of Seal Pour (3-2)
- NCR for Seal Pour (3-3)
- Resolution of NCR for Seal Pour (3-4)
- Test Results on Concrete Cores from Hole No. 39 (3-5)

Access roadway foundation cells:

- Rock and Concrete Investigations, including drilling logs (2-9)
- Concrete Cylinder Data Sheets (2-16)

Test results on the compressive strength cylinders made at the time of the placement of the foundation cells show that at age 90 days the concrete averaged 5640 psi with a range in individual cylinder test results from 4140 psi to 6650 psi for the ERCW Pumping Station cells and averaged 5300 psi with a range in individual cylinder strength test results of 4209 psi to 6173 psi for the access roadway cells.

Compressive strength testing of concrete cores (drilling program discussed in sub-section 3.3) taken from Hole No. 39 in the ERCW cells (see Figure 3-1 for location) averaged 5660 psi for eight tests with a range in individual results of 4720 psi to 6744 psi. Tests on three of these cores to determine the modulus of elasticity gave an average of 5.09×10^6 psi (from values of 4.63, 5.25, and 5.39×10^6 psi). Two tests on the core samples to determine shear strength yielded test results of 124 psi and 966 psi. The former value bears a notation of "paste" on the test report notes, and the latter "rock." These results are further evidence of segregation, as the former value probably indicates shear strength of a test involving the failure of laitence and the latter a test involving more failure of aggregate rather than concrete. Sonic testing in Hole No. 44 gave two average dynamic modulii of elasticity (2-9):

(1) 6.27×10^6 psi, an average of four values takes in sound concrete, and

(2) 2.30×10^6 psi, an average of seven values taken in soft zones in the concrete.

The available data on concrete properties are shown in Table 3-1.

These strength and modulus of elasticity test results all demonstrate that the concrete as initially batched had the ability to far exceed the specified compressive strength requirements (3000 psi for portions of the the ERCW Pumping Station cells, 2000 psi for the remainder of the ERCW Pumping Station cells and the roadway cells), and did exceed the strength requirements in areas of continuous, uniformly placed concrete. The shear strength samples did not provide detailed enough information regarding the nature of the sample to permit any conclusions to be drawn from the test results.

3.2 Suboceanic Consultants Survey

Specifications for the ERCW Pumping Station foundation cells required a foundation surface which was 90% clean for the first five feet within the cell perimeter and 75% clean within the remainder of the cell. These requirements were recognized in Reference 3-6. In addition to verifying the general geologic condition of the foundation, Reference 3-6 notes the requirement for 90% cleaning on the perimeter and requests reinspection by TVA upon completion of additional cleaning. The 75% clean zone was acceptable with the provision that it remain clean until the time of concrete placement.

Foundation mapping was performed by Suboceanic Consultants and summarized in their report of October 18, 1976 (3-7). A report and map of the submerged bedrock surface were produced following several stages of cleaning. The map included the type of rock surface, notes on rock quality, degree of cleaning, and indicated the bottom elevation to the nearest foot. This phase of cleanup was completed on October 14, 1976.

A review of the bedrock map indicates the following breakdown:

45% Type I (clean rock surface)
25% Type II (clean rock surface or sparsely concealed with platy rock fragments)
30% Type III, IV, and V (clay, rock fragments, concrete grout)

Further documentation is contained in a TVA memorandum (3-8), which states that a final inspection of the bedrock immediately prior to concreting showed that both the 90% and 75% criteria had been met. Additional cleaning had been performed by removing zones of slumped concrete which had covered zones of unacceptable material. Reference 3-8 documents the final inspection and approval of the ERCW Pumping Station foundation interface.

It is concluded that the bottom surface of the ERCW Pumping Station cells were adequately cleaned in accordance with specifications and that TVA inspectors evaluated and accepted the condition of the foundation immediately prior to concrete placement.

Subsequent to placement of tremie concrete in the ERCW Pumping Station cells, eight bore holes penetrated the concrete to rock interface and continued a minimum of 15 feet into bedrock. The results of these investigations are discussed in Section 3.3.

Suboceanic Consultants also assisted in performing similar mapping and inspection for the roadway access cells (3-9). The report does not contain any information which would suggest that preparation of the foundation interface surfaces were in any way inadequate or not conforming to the governing specifications.

3.3 Exploratory Drilling and Borehole Geophysics

Exploratory drilling was conducted after placement of tremie concrete in the ERCW Pumping Station and roadway access cells. These investigations are summarized in the report: ERCW Pumping Station Rock and Concrete Investigations, January 1978 (2-9), except for No. 39A (3-5).

Documents indicate plans were made for at least 10 holes to be drilled in the Pumping Station cells. Records were available for nine of them. Six holes were drilled with percussion equipment: Nos. 37, 38, 39, 40, 42, and 43. Four holes were planned to be cored, Nos. 39A, 41, 42A, and 44. Drill logs are available for Nos. 39A, 41, and 44. Locations of holes are shown on Figure 3-1, except that Nos. 39A is adjacent to No. 39.

The pumping station borehole geophysics included caliper, gamma-gamma, and sonic (Figure 3-2) logs on Nos. 37, 38, 39, 40, 41, 42, 43, and 44. Cross-hole sonics were conducted on two pairs of holes, Nos. 43 and 44, and Nos. 37 and 38 (Figures 3-3 and 3-4).

Investigation of the roadway access cells placed after cells E and F were removed was limited to drilling cores in the six roadway access cells: one core each in cells A, B, C, and E, nine cores in cell D and six cores in cell F.

3.3.1 Zones of Weakness in Concrete

The drill logs and the geophysical data establish that there are weak zones in the ERCW foundation cell concrete. The weak zones are generally filled cavities or weak concrete within a large mass of sound concrete. These zones are expected to be discontinuous, near-horizontal lenses in nearly all cases. In areas where the calipers could extend into empty areas, it is believed that this could be done because loosely adhering material had been removed in the process of drilling the borehole, whether due to the core or percussion drill or the cleaning activity that followed the drilling of the hole. As discussed previously, it would not be possible for open "cavities" to exist in the tremie concrete.

After reviewing both the borehole data and the geophysical data for the ERCW foundation cells, it can be concluded that the data indicate that horizontal stratification of communicating zones generally do not occur in boreholes at the same elevation, even though some data indicates communication between adjacent zones in a few cases.

These borehole investigations establish that about 75% of the concrete consists of sound material at all elevations within the ERCW cells and is interspersed with near horizontal lenses of filled cavities or weak concrete. Reference 3-10 identifies zones of reduced modulus at elevations 659, 644.5, and 624 feet which are consistent with this description. However, borehole geophysics did establish an additional zone of weakness at elevation 635± feet, which suggests that approximately 40% of the concrete is sound at this elevation.

As shown on notes attached to borehole 37, 38, 43, and 44 data in Reference 2-9, Zone V and Zone IX are both badly attenuated and located at roughly elevation 635± feet. These notes indicate that both pairs of cross-hole surveys (37 and 38, and 43 and 44, Reference 2-9) did pick up zones of weakness at roughly the same elevation.

The sonic log of hole 37 in Reference 2-9 shows a consistent offset from the transmitter hole, No. 38, and the cross-hole sonic log.

It is concluded that weaknesses exist at elevation 659 feet, 644.5 feet, 624 feet, as well as at 635± feet. Of these, the latter discontinuity is the worst case and should be included in the structural analysis. It is concluded that the other three elevations are consistent with the statement quoted above.

The location of the subject discontinuity is at opposite ends of the long dimension of the cell, visible on geophysical data sonic logs at holes 39, 37, 38, and 40 (2-9) on the north end and holes 43 and 44 on the south end. There is no apparent indication of the discontinuity in sonic logs of boreholes 41 and 42 nor in the geologic log of hole 41 (2-9).

The core drill logs for the nineteen cores in the roadway access cells mentions only one zone of weak concrete with gravel pocketing near the top of the center of cell D. All eight other cores in cell D did not find any irregularity at this elevation so it is judged to be inconsequential.

3.3.2 Weathered Zones in Bedrock

Eight boreholes were drilled a minimum of 15 feet into bedrock in the ERCW Pumping Station. Borehole data confirm that the concrete to rock contact is generally sound with the exception that some isolated weak zones exist.

Graphic logs of these boreholes indicate that weathered rock exists in the foundation in isolated zones at varying depths beneath the top of rock within the ERCW Pumping Station cells. A distinction is made between "weathered rock" and "badly decomposed rock." Badly decomposed rock was noted in two of eight borings. Geophysical data confirm that badly decomposed rock zones are a zone of weakness but that weathered rock, although not uniform, generally have only moderately less density than sound rock. None of the borehole geophysical data indicates anomalous conditions beneath the Pumping Station cells. The bedrock zone beneath the tremie concrete did not require treatment prior to utilization as a structure foundation, as discussed in SNP FSAR, Section 2.5, Geology and Seismology, 2.5.4, Stability of Surface Materials, and 2.5.4.2, Zone of Deformed or Weak Material (3-11).

In addition, a program of settlement monitoring was conducted from June 1979 to September 1984, in order to monitor the Pumping Station (3-11, p. 2.5-43). Review of the raw survey data (3-11, Table Q2.78 A-1) indicates that the Pumping Station undergoes an expansion and contraction, apparently in accordance with seasonal temperature conditions, but there is no evidence that the Pumping Station has settled over the five year monitoring period.

Weathered zones in the bedrock foundation beneath the roadway access cells are more prevalent. Core logs (2-9) for cells A, D, E, and F indicate significant core loss and weathered zones in the rock underlying the concrete. Weathered rock that was recovered is described as soft, crumbly shale and highly weathered shale. Mud and clay are also present. Based on this core data, it is apparent that this foundation is more weathered than the foundation beneath the Pumping Station. This is to be expected, since excavation for the roadway cells was not as extensive as for the ERCW Pumping Station. However, as noted in Figures 2.5.1-104 through

2.5.1-119 of Reference 3-11, steps were taken to remove the overburden soil and to excavate into the weathered rock for at least 8 to 15 feet in order to improve the foundation condition. No unusual or adverse settlement can be observed in the data collected during the 5 year settlement monitoring program (3-11, Table Q2.78 A-1).

3.3.3 Cell-Rock Interface

As stated earlier, 90% of the perimeter area and 75% of the remaining cell area of the foundation rock surface was verified to be clear prior to concrete placement. Thus, it is concluded that at the bottom of the cell the concrete is expected to be sound and any weaknesses are limited to local areas where the rock was not completely cleaned. Additional weak zones might have occurred as a result of defects in placement of tremie concrete. It is expected that any defects directly under the tremie pipe are minimal due to greater assurance of placement of the tips of the tremie pipes in favorable locations at the start of concreting operations, however, some minor defects (washout) may occur as the concrete flows along the rock at the start of the placement. Additionally, since the concrete cell-rock interface was prepared with a conical surface (Figure 1-4), any laitance separating from the concrete mass would tend to spread out and run downhill to the center of the excavated area rather than maintain a mass in one location. It is, therefore, reasonable to assume that few defects at the interface are due to unsound concrete. Additional confirmation is obtained from borehole data which indicate the concrete to rock interface is reasonably sound.

For the roadway access cell concrete, the core borings show contact between concrete and rock for cells A, B, C, and E. In cell F, six cores through the concrete-rock interface typically found core loss once the rock was reached due to softness of the material. In cell D, nine cores indicated the limits of soft concrete and gravel lenses as shown in Figure 3-5 at the cell-rock interface. The structural implication of these anomolies will be discussed in section 4.3.2 of this report.

4. STRUCTURAL EVALUATIONS

4.1 Physical Data Versus Assumed Concrete Properties

A review of all data provided by TVA pertaining to the quality of the concrete for the Pumping Station foundation and roadway cells as produced and installed shows that the concrete is strong, sound and of adequate quality when it has been properly placed in the structure by the tre method.

The compressive strength test results (2-15, 2-16) obtained at the time the concrete was placed shows the concrete is capable of developing strength far in excess of the strength required by the original design requirement. The strength of the homogeneous concrete in the structure has been confirmed by compressive tests of core sections taken from Hole No. 39 (3-5).

Results obtained from the physical testing are summarized in Table 3-1 and demonstrate that the sound concrete in place is much stronger than the design strength of 4000 psi assumed in the evaluations. It should be noted that in both the cylinder and core compressive strength testing, no cylinder broke below 4140 psi and both the range and average of test results are unusually similar. Additionally, the concrete has had the benefit of ten years to continue curing under favorable conditions. Considering the expected strength gain from the additional curing, the assumption of a design strength of only 4000 psi for the analysis is very conservative since justification for using higher design strengths could be made based on the ACI 318 Code.

The modulus of elasticity values obtained from the core testing has demonstrated that the actual modulus of elasticity is at least 25% greater than the value used in the design calculations produced by TVA (Table 3-1).

4.2 ERCW Foundation Cell Evaluation

The original seismic analysis was performed with the concrete properties discussed above (4-1). After the borehole data were evaluated, the effects of local weak areas on seismic response were studied by locally varying the concrete modulus (4-2). The effects on stresses were evaluated by utilizing conservative assumptions regarding the condition of concrete in different areas of the structures. These are discussed in the following subsections.

4.2.1 Seismic Response

In the evaluation, it was assumed that the modulus of elasticity is reduced by 25%. Following procedures of the ACI 318 Code for determining the concrete modulus, this reduction would correspond to a decrease in concrete compressive strength of about 44%. Since the lenses are local and intermittent and the strength of sound concrete is greater than used in analysis, the assumed decrease in strength is considered to be a conservative approximation. Furthermore, the original modulus of elasticity used in seismic analysis was about 20% lower than that obtained from the cores (Table 3-1).

The assumed reduction in the concrete modulus of elasticity indicated that the fundamental frequency of the structure will decrease by about 14%, from 9.2 to 7.9 Hz. Accordingly, there will be some increase in the seismic response of the structure (10 to 20 percent) based on the average free field spectra of the four design time-history records. However, in reality the frequency of the structure is greater than calculated since the calculations did not consider the actual concrete strength, actual modulus of elasticity and contribution of the liner system and the sheet piling, all of which are factors that will tend to increase the stiffness and, thus, the frequency of the structure. Therefore, it is concluded that overall seismic response will remain about the same and use of the original seismic analysis results in stress evaluations is adequate.

4.2.2 Concrete Stresses

In determining the local effects, parts of the concrete at any elevation were considered to be either at full strength or weak and other parts simply filled with gravel (4-3, 4-4). The weak concrete areas were assumed to have 37.5% reduction in modulus of elasticity in calculating the modular ratio which implies the same reduction in the stress levels. The gravel areas were disregarded in stress calculations. Classification of particular zones was based on information obtained from the drilling and logging program as shown on Figure 3-1 and is summarized on Table 4-1. The cross-section used for analysis is conservatively based on the minimum net area, and assumes that the worst-case spatial distribution of nonconformances occurring at a single elevation based on the Figure 3-1 information are coincident. Furthermore, any contribution of the liner plate and the sheet piling is ignored.

This worst-case section was first analyzed assuming concrete can resist tension. The stresses at the most critical elevation of the structure are shown in Table 4-2. These stresses are slightly different than those in Reference 4-3 since they were adjusted to exclude the assumed gravel zone in the calculation. As noted in Table 4-1, the sound concrete is assumed to occupy an area of 37% and the weak concrete an area of 30%, the remaining 33% being conservatively assumed to consist of gravel. The stresses were calculated at elevation 628 feet, using the distribution of discontinuities identified at the 635 foot elevation considered to be the worst case. As shown in the table, the compressive stresses are about 15-20% of the allowable and tensile stresses are about 80-85% of the allowable. Therefore, it may be concluded that uncracked sections will remain uncracked under seismic loads. The allowable stresses based on the ACI 318 Code are also shown in Table 4-2.

If the foundation is assumed to resist zero tension due to the presence of microcracks induced by the thermal stresses, the lateral loads will be resisted solely by a compression block. The worst condition will occur if the river side is under compression, since that side provides the minimum net area for resistance to lateral load due to the presence of the intake conduits.

Thus, a second analysis was performed assuming concrete carries zero tension and the compression block is developed on the river side, as shown in Figure 4-1. The resulting concrete stresses are indicated in Table 4-3. It is seen that, even with this conservative assumption, the maximum stresses are in the 50-75% range of the allowable values.

Table 4-4 shows the maximum shear stress assuming that only the sound concrete over 37% of the plan area resists shear. Since the calculated stresses are conservative (see

Section 4.4), the actual margins between the allowables and calculated values will be even greater than those indicated in Tables 4-2 through 4-4.

4.2.3 Cell-Rock Interface Stresses

Calculations show that the maximum bearing pressure at the cell-rock interface is about 450 psi, based on the buoyant weight of the structure. The calculations also considered the no tension condition at the interface. This stress is very small compared to a minimum allowable stress of 2380 psi (4-5, [illegible] fc'). The calculated stress may be factored upward to account for the weak zones at the interface. Based on sub-section 3.3.3, assuming 90% of the concrete at the edge of the interface to be sound would result in a bearing stress of about 500 psi which is still significantly lower than the allowable value.

The results given above are based on simple, static equivalent methods and are consistent with the detailed nonlinear time-history analysis that was performed for a typical roadway access cell, assuming that it behaves as an isolated structure (4-6). The access roadway cell analysis showed that the interface stresses were in the order of 600 to 800 psi, much less than the actual concrete strength, and that progressive concrete chipping at the toe was not a credible event. Since an access cell is about the same height as the pumping station foundation with a diameter about one-half the least dimension of the cell under consideration, it is concluded that a detailed time-history analysis is not warranted.

So far as the shear force is concerned, it will be resisted mainly by the friction between the cell and rock. The buoyant weight of the cell is approximately twice the total lateral load. Assuming that the friction coefficient between the concrete and rock is 1.0 (similar to concrete placed against hardened concrete, Reference 4-5, although the rough interface between the concrete and rock would probably offer greater shear resistance), there is obviously an adequate factor of safety against overcoming the friction resistance.

4.3 Access Roadway Cell Evaluation

4.3.1 Structural Adequacy of a Single Cell

The access roadway cells were originally analyzed as a single, continuous, J-shaped structure. In subsequent analytical work (4-6), a nonlinear time-history analysis of a single cell acting alone was performed. Adequacy of the single cell under the resulting lateral and vertical loads was evaluated and the structure was shown to be acceptable.

Since the lenses and weak zones in the access roadway cells are shown to be minimal, the only significant parameters to be evaluated are the stresses at the concrete-rock interface and deflections.

Reference 4-7 indicates that the interface bearing stresses will be about 600 psi. The calculated bearing stress is less than the sound concrete allowable of 2380 psi and the sound rock allowable of 1500 psi. Chipping of concrete at the cell perimeter due to extreme toe pressures was calculated to not exceed a maximum of 12 inches inward from the perimeter and was shown to not affect the integrity of the structure.

The calculated deflection at the top of the structure is only a small fraction of an inch (4-8). Such displacements are not considered to affect the performance of the structure or the equipment supported by it.

Regarding the shear force, the resistance to sliding will be provided by friction. If the resistance of interconnecting beam ties near the top are ignored, and assuming a coefficient of friction of 1.0 (the value of 1.0 is appropriate since resistance to sliding is provided by the mechanical interlocking of the irregular cell-rock interface), the factor of safety against sliding is about 1.33. This value is based on buoyant weight. Therefore, sufficient friction resistance exists to prevent sliding.

The above results demonstrate that even if each access roadway cell is assumed to act alone (an extremely conservative assumption), their structural integrity will be maintained.

4.3.2 Cell-Rock Interface Stresses

As noted earlier, the borehole data indicates a sound concrete-rock interface for cells A, B, C, and E. In the case of cell D, borehole records show that the bottom four foot segment of the cell contains large lenses of washed aggregate and sand. Also, cell F data shows loss of material at the interface, indicating potentially weak concrete zones.

Reference 4-6 performed an approximate analysis and showed that, with the assumption of reduced modulus of elasticity for the bottom segment (based on Reference 4-9), adequacy of the structure can still be demonstrated. The resulting, increased interface bearing stress of 800 psi was evaluated as discussed below.

Even if the lower segment of the cell is assumed to behave as gravel, the calculated maximum bearing stress of 800 psi will be resisted by the gravel so long as adequate confinement exists. Calculations (4-8) show that adequate confinement is provided by the sheet piling and the rockfill surrounding the cell.

4.4 Adequacy of the TVA Evaluations

4.4.1 ERCW Foundation Cells

The geophysical data indicated that the zones of weakness comprising the maximum percentage of cross-sectional area is at elevation 635±. However, for conservatism this cross-sectional distribution was assumed to occur at the location of the worst condition for stress calculations was in the structure at elevation 628 feet, where the moments and shears are greater as a result of both a longer vertical cantilever section and a reduction in the concrete cross-section due to intrusion of the intake tunnels.

Insofar as the structural evaluations are concerned, it is concluded that conservative assumptions and approximations were made in the evaluations. For stress analysis of the structure, the following factors may significantly reduce the calculated stresses and/or increase the safety margin:

a) Effect of the actual concrete strength: Testing has shown that the average insitu strength of the ERCW Pumping Station concrete was about 5600 psi (Table 3-1) which is approximately 40% greater than the strength used in the evaluations. It is reasonable to assume that the concrete has gained additional strength during the ten year period since the testing was performed. The actual strength will decrease the seismic response through increased fundamental frequency, as well as resulting in greater allowable stresses.

b) Effects of the liner and the sheet piling: In all the structural evaluations the contributing effects of the liner and the sheet piling have been neglected. The liner plate will significantly contribute to both stiffness and strength, reducing the calculated stresses. The sheet piling will have a similar but lesser contribution even if a gap is postulated between the sheet piling and the concrete. This contribution may become more significant at ultimate loads, since, at ultimate strength, large displacements will occur and, thus, partially composite action between the sheet piling and concrete will be achieved.

c) Effect of location assumption for worst condition: It was also concluded in Section 3 that the worst case spatial distribution of nonconforming material occurs at elevation 635 feet. In stress analysis, however, this worst material condition was assumed to occur at elevation 628 feet where moments and shears will be greater.

d) Effects of the assumed weak concrete and gravel zones: As noted above, only 37% of the plan area at any elevation was assumed to consist of sound concrete. Furthermore, compression and shear in the gravel was ignored. Since the postulated gravel pockets (lenses) are confined by surrounding sound concrete and/or by the sheet piling, these lenses will carry compression and shear, thereby reducing the calculated stresses in sound concrete.

4.4.2 Access Roadway Cells

It is generally accepted that analysis of a single cell as an isolated structure is an extremely conservative approach. However, such an analysis will certainly envelop the worst possible condition of the access roadway cells and serves to limit the scope of the work to demonstrate structural adequacy.

The nonlinear analyses performed indicate an access roadway cell acting as an independent structure will be stable and function as intended. Since the cells are interconnected, the margin of safety is even greater than those indicated by single cell analysis.

In the case of cells D and F, it was shown that these cells will also function as intended, even if the lower section of the cell is assumed to consist of gravel or crushed rock.

The analyses and evaluation of cells D and F are considered conservative in view of the following factors:

a) The slope under cell D, and thus the assumed weak concrete zone, is toward the adjacent cell. In that direction, the cell, due to geometric constraints, cannot act as an isolated structure. In the other direction (i.e., north-south) the foundation data shows sound concrete at the perimeter where highest stresses will occur (Figure 3-5).

b) These cells are surrounded by cells that were shown to have sound cell-rock interface. Therefore, any reduction in stiffness will result in redistribution of stresses.

c) Borehole data for the access roadway cells indicate that the concrete is sound and any lenses that may exist would be localized. Therefore, the indicated lenses in the bottom segment of the cells are likely to be localized and thus confined by the surrounding sound concrete and/or sheet piling.

d) The assumption that the bottom four foot segment of cell D behaves like gravel or crushed rock is extremely conservative. In reality, concrete within this segment would tend to bond the aggregate, reducing the required confinement pressure.

5. ASSESSMENT RESULTS AND ALTERNATIVE ACTION PLANS

5.1 Assessment Results

Interpretation of the technical data and structural evaluations presented in the previous sections of this report has demonstrated that the ERCW Pumping Station and access roadway cells have more than adequate structural integrity to successfully resist the postulated design basis loads. This conclusion is reinforced by the obvious massive nature of the structures. Particularly in the ERCW Pumping Station foundation cells, it is easy to envision that the structural integrity is not affected as a result of the inclusion of occasional, generally noncontinuous lenses of washed-out concrete materials, particularly when the lenses are well dispersed throughout the structure as indicated by the boring program (2-9, Figure 3-1). Since this type of structure is proportioned on the basis of layout of intake requirements and not on structural strength, the resulting factors of safety are significantly greater than those conventionally required. Therefore, although the lenses in the ERCW Pumping Station cells may reduce the original factors of safety, the reduction, based on available data, will not be structurally significant and the cells will function as intended.

For the access roadway cells, the exploration program did not reveal any significant irregularities in the concrete mass, so their integrity is assured with no further discussion required.

However, between the elevations of 625 and 634 of the ERCW pumping station, the cross-sectional area of the foundation cells is substantially reduced and dispersed among five elements separated by the four intake tunnels. As seen in Figure 4-1, the most substantial concrete sections available to resist flexural compression about the neutral axis on the river (intake) side of the structure are located between the pairs of intake tunnels in each cell: elements B and D. The concrete along the outer edges of the structure, elements A and E, would contribute a relatively small proportion of resistance to flexural compression, as its relatively smaller cross-sectional area has a center of gravity much closer to the neutral axis. The thinner band of concrete along the center line of the two cells, element C, also is a lesser contributor due to its smaller concrete area and closer proximity to the neutral axis. This discussion confirms that elements B and D are the major load-resisting elements for the flexural compression condition.

The analysis using the idealized as-built conditions justified by the information obtained from the exploration program required that any contribution from element D be neglected, as the Hole No. 39 boring showed a gravel lens at elevation 628. While this condition still produced a satisfactory structural analysis result, conditions involving asymmetrical loading, such as torsion, might produce stresses approaching the permitted maximums.

5.2 Alternative Action Plans

Because the analyses consistently indicate that the ERCW Pumping Station foundation structure will remain within its allowable stress limits under the postulated design loads and no safety problem exists, continued use of the structure in its present condition must be seriously considered as a viable option. Alternatively, further exploration of the structure's actual condition could be undertaken to verify and, if necessary, to restore structural continuity in accordance with the needs of the critical sections or throughout the structure. These alternatives will be discussed in the following sections.

5.2.1 Continue Use in Present Condition

As previously stated, all analyses and calculations (employing very conservative simplifying assumptions) demonstrate that the structure will withstand the design loadings with stresses remaining comfortably below allowables. These analyses rely, however, on reasonable and conservative assumptions regarding the condition of the concrete in the structure made from the relatively limited exploration program. There remains a small probability that a more extensive exploration program would provide information that would necessitate increasingly conservative assumptions in the structural calculations. Also, as noted in Section 5.1, because the present information necessitates an assumption of an asymmetrical condition in the structure (no structural contribution from element D, Figure 4-1), postulated loadings could produce stresses due to altered structural performance from decreased torsional resistance.

5.2.2 Limited Exploration Program

The considerations described in the preceding sub-section make it prudent to consider a limited exploration program to verify an adequate condition of the structure at the locations which could significantly affect the structural behavior.

For this alternative, it is proposed that a limited exploration program would take place in the critical structural sections below elevation 640 where changes in the presently assumed conditions would have the greatest potential impact on the resulting conclusions. The limited program would involve exploration by drilling a minimum of six holes in element D, evaluating the data obtained from drilling, and pressure grouting the element in a manner that will assure that all major zones of loose, segregated material have been filled with grout. Similarly, a minimum of two holes would be drilled in element B to provide compatible verification. All drilled holes should be grouted to assure that the concrete in the area investigated remains compatible with the assumed conditions. Refer to Appendices A and B for details of the recommended exploration and grouting programs. Detailed specifications and procedures for the exploration and grouting should be formulated with the assistance of personnel having extensive experience in this type of grouting work.

During the walkdown performed in the course of this study it was observed that drilling can be readily accomplished at the lowest floor in the Pumping Station, elevation 688±. Enough exploration and grouting must be performed to confirm the status of existing material in the critical sections of the structure (elements B & D) and to stabilize any zones of loose, segregated material which might be encountered with the intent of assuring that the critical sections meet the original design requirements. This may require the drilling and grouting of additional holes in these elements based upon the information developed from the initially authorized holes. Caliper readings, sonic measurements and physical testing of core specimens are not considered necessary due to the limited nature of exploration.

Since the intent is to assure existence of sound concrete below elevation 640, the necessity of drilling additional holes should be based upon the results within each element. The condition of concrete above elevation 640 should not be the basis for additional drilling, unless substantially different data is obtained which would require reconsideration. Furthermore, any additional holes drilled to facilitate grouting activities below elevation 640 will not require exploration, unless substantially different data is obtained which would require reconsideration.

The idealized conditions for structural analysis after performing the preceding exploration and grouting are shown in Figure 5-1, and the results of calculations based on these conditions are shown in Table 5-1. Compared to previous calculation results summarized in Tables 4-3 and 4-4 (based on the conditions shown in Figure 4-1) the new calculation results show significantly lower calculated/allowable stress ratios and because of the verification of structural condition aspect of the exploration and grouting programs, the new calculation results provide a greater degree of assurance that they accurately represent the structural condition.

5.2.3 Extensive Exploration and Grouting Program

The program for exploration and grouting presented in Section 5.2.2 could be extended to include either elements A, C, and E of Figure 5-1 or all of the remainder of the foundation cells. If exploration and grouting programs were performed to upgrade the condition of the remaining elements to equal elements B and D, it would result in only a nominal improvement in stress analysis results. Similar improvement in the area above the neutral axis on Figure 5-1 would provide no additional reduction in stresses calculated at the controlling section shown in Figure 5-1, with the assumption that concrete does not resist any tension.

Based on the above discussion, exploration and grouting beyond the limited program described in Section 5.2.2 should only be considered if unforeseen conditions are discovered during those programs.

5.2.4 Impact on Operation of the Plant

It is important to emphasize that safety of the ERCW Pumping Station foundation cells is not questioned. In addition, the integrity of the structure will not be affected during implementation of the exploration and grouting programs previously described. Therefore, these activities can be undertaken while the plant is in operation provided normal requirements for the availability of equipment are maintained during implementation of those programs.

6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary and Conclusions

The important aspects of this study and the conclusions reached regarding the ERCW Pumping Station cells can be summarized as follows:

1. There is sufficient data to clearly establish zones of sound concrete:

   There is reasonable evidence of zones of sound concrete at every elevation of the Pumping Station cells together with some weak concrete zones and lenses. The weak zones are discontinuous as established by borehole logs and geophysical data.

2. There are limited size weak zones filled with material, i.e., remnants of concrete that underwent washout during the tremie placement operations (gravel, sand, fly ash, cement, or any combination), with possible communication between the weak zones; no open voids exist:

   The drill logs and the geophysical data establish that these zones are generally filled cavities or weak concrete within a large mass of sound concrete. These zones are generally expected to be discontinuous, nearly horizontal lenses. In a few cases data indicate communication between adjacent zones. Both borehole and geophysical data indicate that horizontal stratification of communicating zones do not occur in a significant percentage of boreholes at the same elevation.

3. Cell to rock interface is adequate over a sufficient percentage of the contact surface:

   Based on additional review, we conclude that the bottom surface of the Pumping Station cells were adequately cleaned according to criteria and that site inspectors evaluated these criteria and gave acceptance of the foundation at the time of tremie concrete placement.

   Exploratory boring at the ERCW Pumping Station indicates that weathered rock exists in the foundation in isolated zones at varying depths beneath the top of rock within the ERCW cells. Weathered rock, although not uniform, have generally only moderately less density than sound rock. None of the borehole data indicates anomalous conditions beneath the Pumping Station cells. Therefore, weathered zones exist in bedrock but their existence will not be detrimental to the structural integrity of the cells.

4. Structural evaluations show that structural integrity of the ERCW Pumping Station will be maintained under seismic loads:

   The analysis results indicated that resulting stresses in the mass concrete were all below the code allowables. Therefore, it is concluded that an adequate margin of safety exists to assure the integrity of the structure.

Regarding the access roadway cell-rock interface, the following conclusions are drawn:

1. Analysis of the cells as isolated structures is an extremely conservative approach. These cells interlock and, although some relative movement between the adjacent cells is possible, they will primarily act as a composite structure.

2. The conservatively calculated concrete cell-rock interface stresses of 600 to 800 psi are acceptable. These stresses are well below the allowable bearing stresses for sound concrete and sound rock.

3. According to the boring logs, cell D and to a lesser extent, cell F appear to have a washed-out aggregate layer extending over some of the interface area at the bottom of the cell. Under seismic loads, the gravel will resist the flexural compression. The sheet piling and

the rockfill will provide adequate confinement for the gravel. The compression provided by the concrete mass above will generate sufficient shear resistance to sliding. It is expected that sheet piling will not deteriorate in fresh water.

6.2 Recommendations

1. It is recommended that the status of the material present in elements B and D, as shown in Figure 4-1, located along the river (east) side of the ERCW Pumping Station cells, be investigated by an exploration program to verify the existing data used in the evaluation of these elements.

   This exploration should be coupled with a grouting program structured to stabilize any zones of loose, segregated material which might be encountered with the intent of assuring that the critical elements meet the original design requirements. As a minimum, six holes should be drilled for exploration and possible remedial grouting in element D and two holes in element B. The assurance of structural integrity should be limited to the segments of the elements below elevation 640. After the initial holes have been explored, the results would be evaluated to determine whether further drilling prior to grouting was appropriate. Continued drilling and grouting would be required if needed to achieve the objective of assuring conformance to the original design requirements.

2. In all grouting operations, the holes should be extended into the rock foundation so that the grout would have the opportunity to penetrate weathered rock as well as the rock-concrete interface.

3. Detailed programs should be prepared before exploration and grouting are implemented. The programs should address requirements that will assure that the objectives of the exploration and grouting activities are achieved. Appendices A and B provide initial guidance for incorporation into these programs.

4. It is recommended that no exploration and grouting be performed on elements A, C, and E or any other portion of the ERCW Pumping Station foundation cells shown as being above the neutral axis in Figure 5-1, as any such effort would result in only a nominal increase in the structure's capacity.

5. No further exploration is recommended on the access roadway cells as the previous investigations and current evaluation demonstrate that the roadway cells are structurally satisfactory in their present condition.

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8. TABLES AND FIGURES

8.1 List of Tables

TABLE 2-1 - TREMIE CONCRETE MIX DESIGNS

CONCRETE CONSTITUENT	CONCRETE MIX DESIGN WEIGHTS, LB/CU YD	
	ERCW PUMPING STATION FOUNDATION CELLS	ACCESS ROADWAY FOUNDATION CELLS
Cement	350	300
Fly Ash	214	275
Water (1)	275	260
Sand	1385	1417
Coarse aggregate, 1 in. maximum size	1685	1688
	CONCRETE MIX DESIGN ADMIXTURE DOSAGES OZ/CU YD	
Air entraining admixture	5	15
Water reducing admixture (WRDA with Hycol)	17	--
Water reducing admixture (Daratard)	--	15

(1) Amount of ice used to replace water varies, 25 - 90 lb/cu yd

TABLE 2-2 - RANGE OF VARIATION IN FRESH CONCRETE PROPERTIES

PROPERTY	RANGE OF ERCW PUMPING STATION FOUNDATION CELL CONCRETE		RANGE OF ACCESS ROADWAY CELL CONCRETE	
	BASED ON ALL VALUES (14)	AFTER DISCARDING HIGH AND LOW (BASED ON 12 VALUES)	BASED ON ALL VALUES (17)	AFTER DISCARDING HIGH AND LOW (BASED ON 15 VALUES)
SLUMP, IN.	3 - 8	3-1/2 - 7-1/2	4-1/2 - 7-1/2	4-1/2 - 6-3/4
AIR CONTENT, %	1.0 - 8.0	2.0 - 4.0	0.5 - 5.4	0.6 - 1.9
UNIT WEIGHT, LB/CU FT	142 - 154	147 - 153	147 - 155	149 - 154

TABLE 3-1 - CONCRETE MECHANICAL PROPERTIES

PROPERTY	VALUE, psi
Compressive strength (ERCW Pumping Station foundation cell cylinder tests @ 90 days)	5640
Compressive strength (Access roadway cell cylinder tests @ 90 days)	5300
Compressive strength (ERCW Pumping Station foundation cell core tests @ about 75 days)	5660
Modulus of elasticity (ERCW Pumping Station foundation cell core tests)	5.09×10^6
Modulus of elasticity (design value used by TVA)	4.1×10^6
Dynamic modulus of elasticity (ERCW Pumping Station foundation cell sonic tests in sound concrete)	6.27×10^6
Dynamic modulus of elasticity (ERCW Pumping Station foundation cell sonic tests in soft zones in the concrete)	2.30×10^6

TABLE 4-1 - DISTRIBUTION OF CONCRETE AND GRAVEL ZONES ASSUMED IN ANALYSIS

Material	Compressive Strength, psi	Area, %
Sound Concrete	4000	37
Soft Concrete	1700	30
Gravel	0	33

TABLE 4-2 - FLEXURAL STRESSES IN AS-BUILT STRUCTURE CONSIDERING CONCRETE TENSION

Concrete		Stress, psi		Stress Ratio
Type	Stress Component	Calculated	Allowable	Calculated / Allowable
Sound	Compression	378	2600	0.15
	Tension	173	206	0.84
Soft	Compression	237	1105	0.21
	Tension	108	134	0.81

TABLE 4-3 - FLEXURAL COMPRESSION IN AS-BUILT STRUCTURE NEGLECTING CONCRETE TENSION

Concrete Type	Stress, psi		Stress Ratio
	Calculated	Allowable	Calculated / Allowable
Sound	1320	2600	0.51
Soft	806	1105	0.73

TABLE 4-4 - SHEAR STRESSES FOR THE AS-BUILT STRUCTURE ASSUMING ONLY SOUND CONCRETE CARRIES SHEAR

Stress, psi		Stress Ratio
Calculated	Allowable	$\frac{\text{Calculated}}{\text{Allowable}}$
76	126	0.60

Notes for Tables 4-2 through 4-4

(1) Allowable stresses are obtained from the ACI 318-71 code:
Compression = $0.65\ f_c'$
Tension = $3.25 \sqrt{f_c'}$
Shear = $2 \sqrt{f_c'}$ (compressive normal stresses are conservatively ignored)

(2) Sound Concrete: $f_c' = 4000$ psi, 37% of plan area
Soft Concrete: $f_c' = 1700$ psi, 30% of plan area
Gravel: $f_c' = 0$ psi, 33% of plan area

TABLE 5-1 - STRESSES IN AS-RESTORED STRUCTURE

Stress Type	Stress, psi		Stress Ratio
	Calculated	Allowable	Calculated / Allowable
Concrete Compression	918	2600	0.35
Concrete Shear	57	126	0.45

Notes:

(1) All concrete considered in the analysis has been verified to be sound.

(2) Concrete is assumed to carry zero tension.

(3) Concrete compression is based on the assumption that only elements B and D (Figure 5-1) resist lateral loads.

(4) Shear is based on sound concrete areas only.

(5) Allowable stresses are obtained from the ACI 318-71 Code

Compression: $0.65\ f_c'$

Shear: $2\sqrt{f_c'}$ (compressive normal stresses are conservatively ignored)

8.2 LIST OF FIGURES

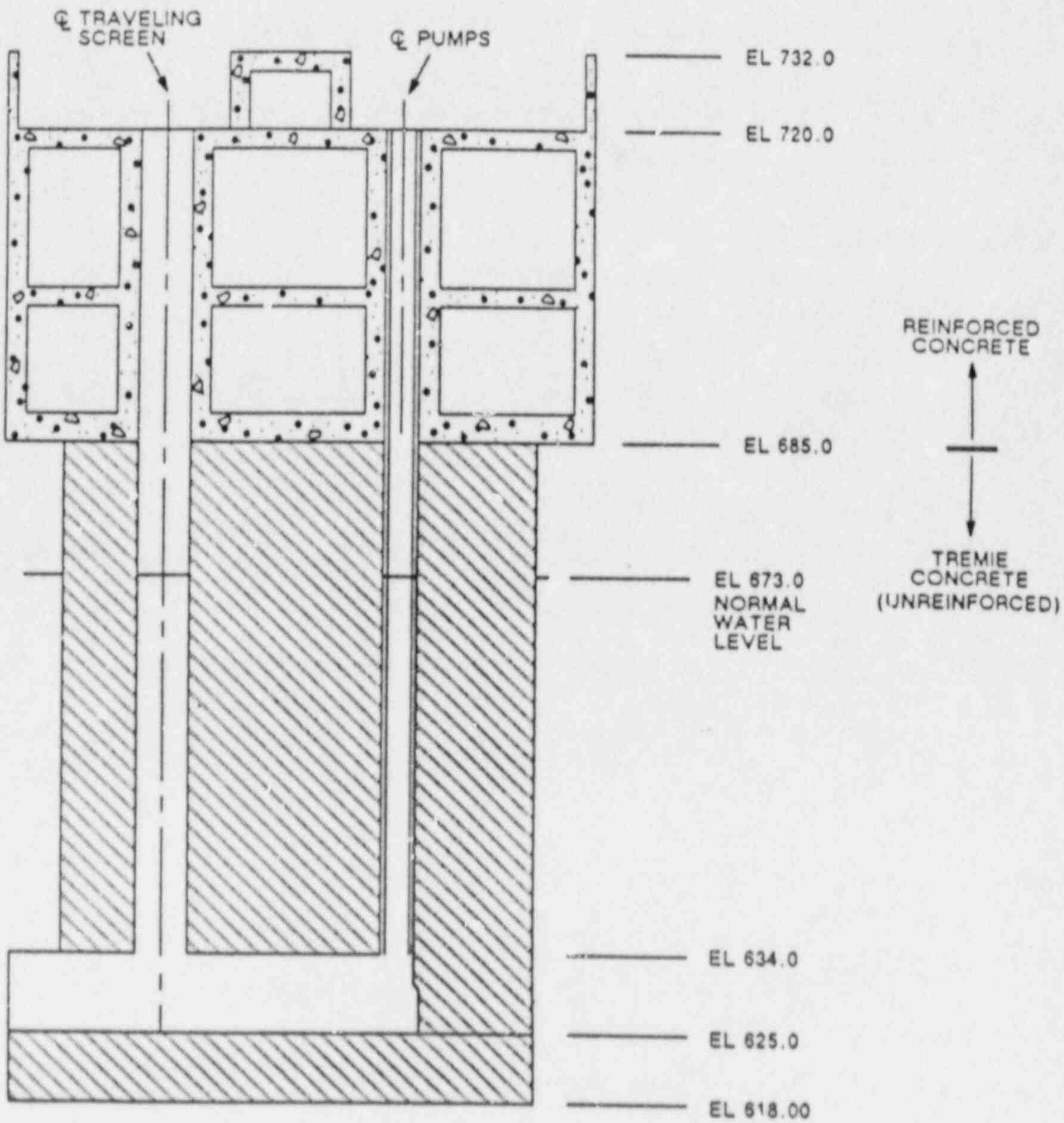


Figure 1-1

ERCW PUMPING STATION
SECTIONAL ELEVATION

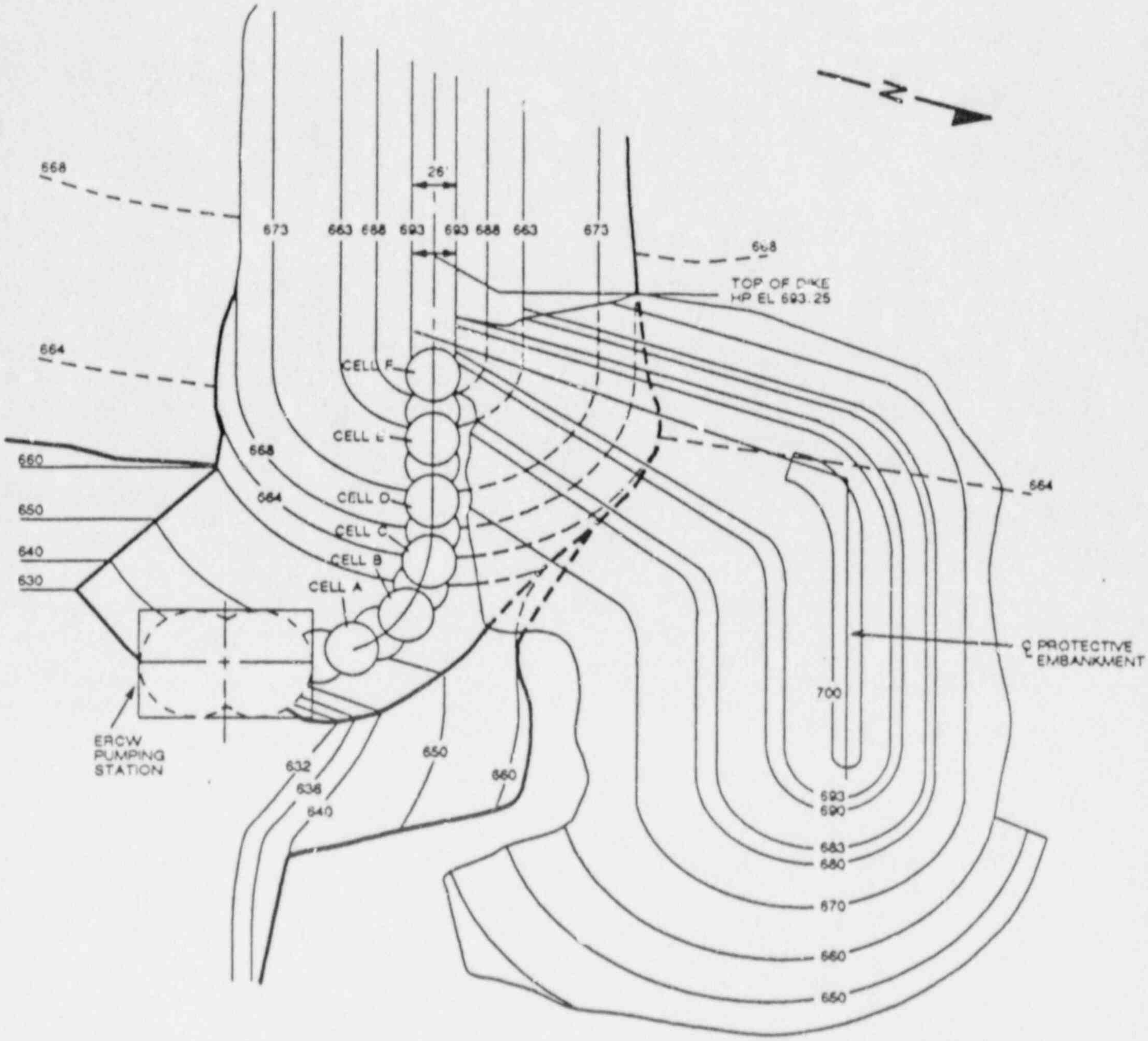


Figure 1-2

LAYOUT OF ERCW PUMPING STATION, ROADWAY CELLS, AND ASSOCIATED DIKES

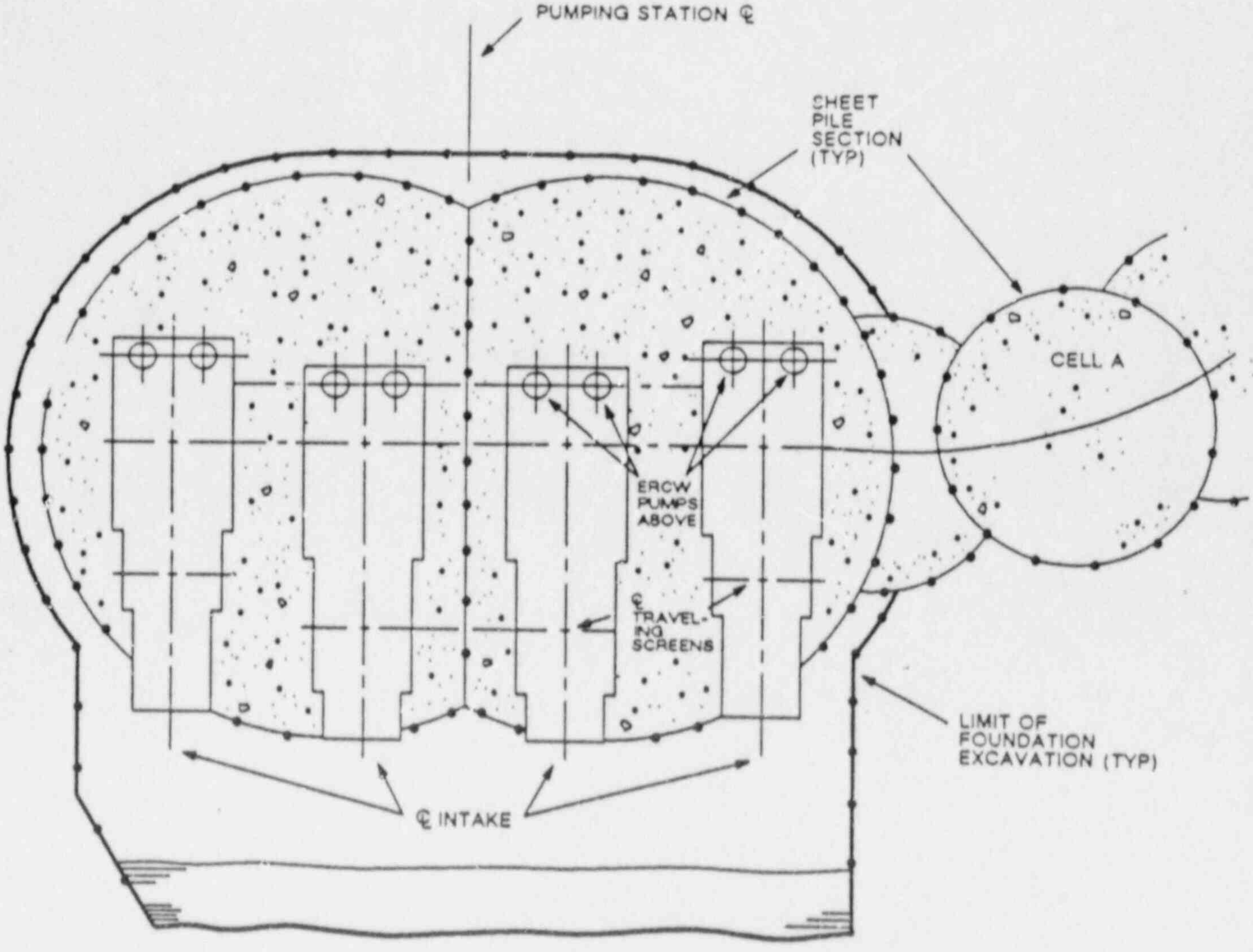


Figure 1-3
PLAN OF ERCW PUMPING STATION FOUNDATION CELLS
AT ELEVATION 633.5 ±

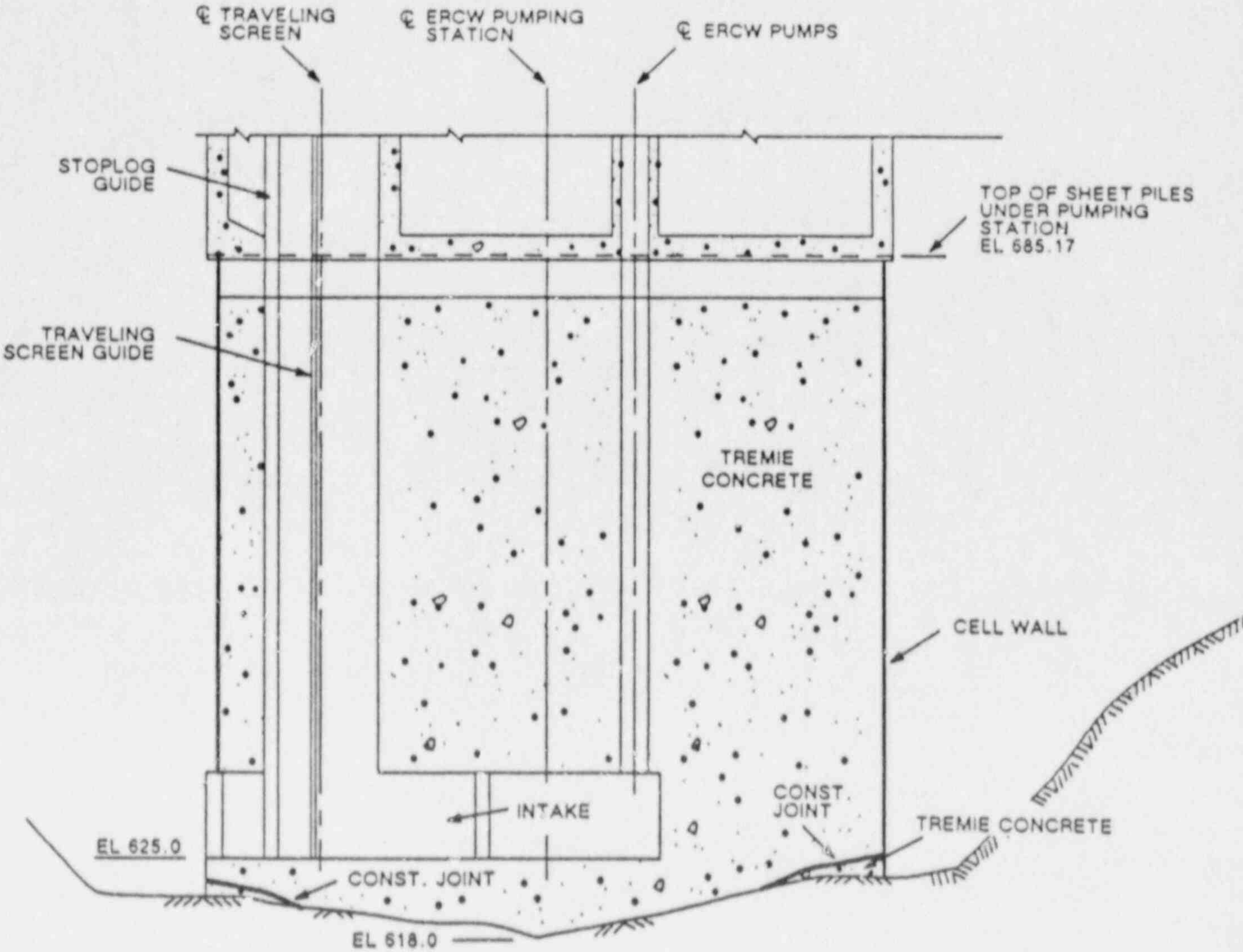


Figure 1–4

ELEVATION OF PUMPING STATION FOUNDATION

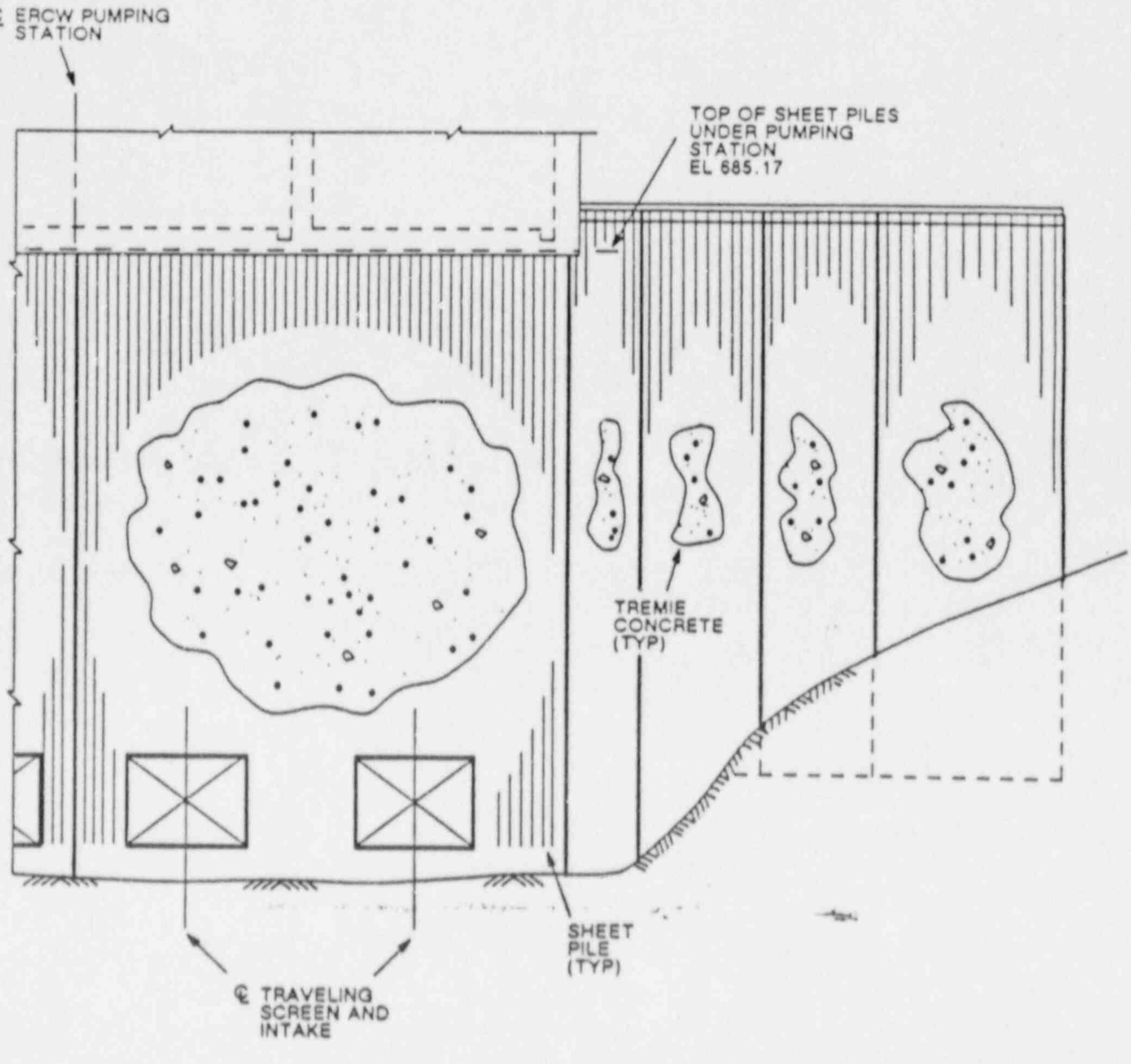


Figure 1-5

NORTH FOUNDATION CELL AND ROADWAY CELLS SHOWING EAST EXTERIOR ELEVATION

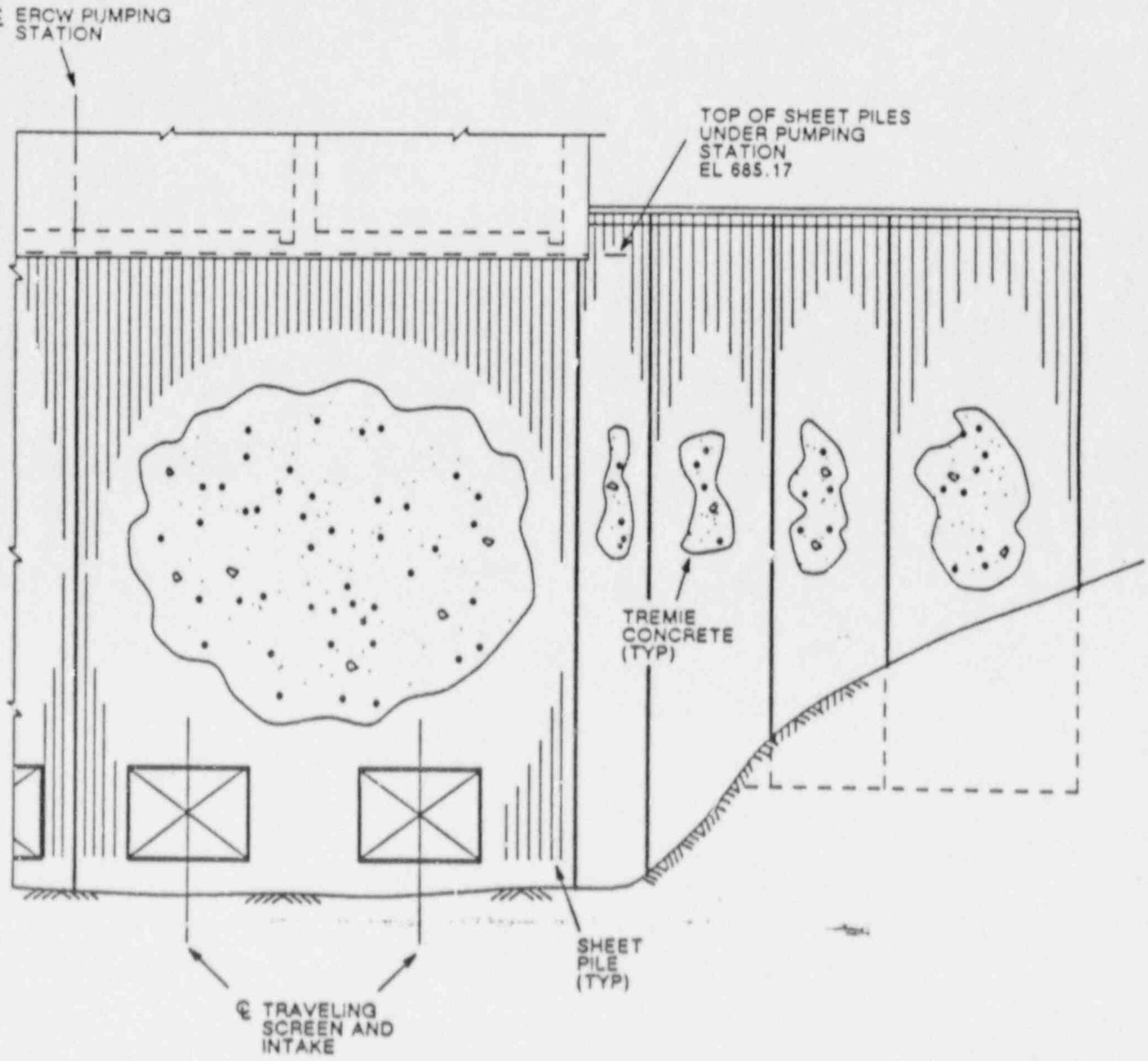


Figure 1-5

NORTH FOUNDATION CELL AND ROADWAY CELLS SHOWING EAST EXTERIOR ELEVATION

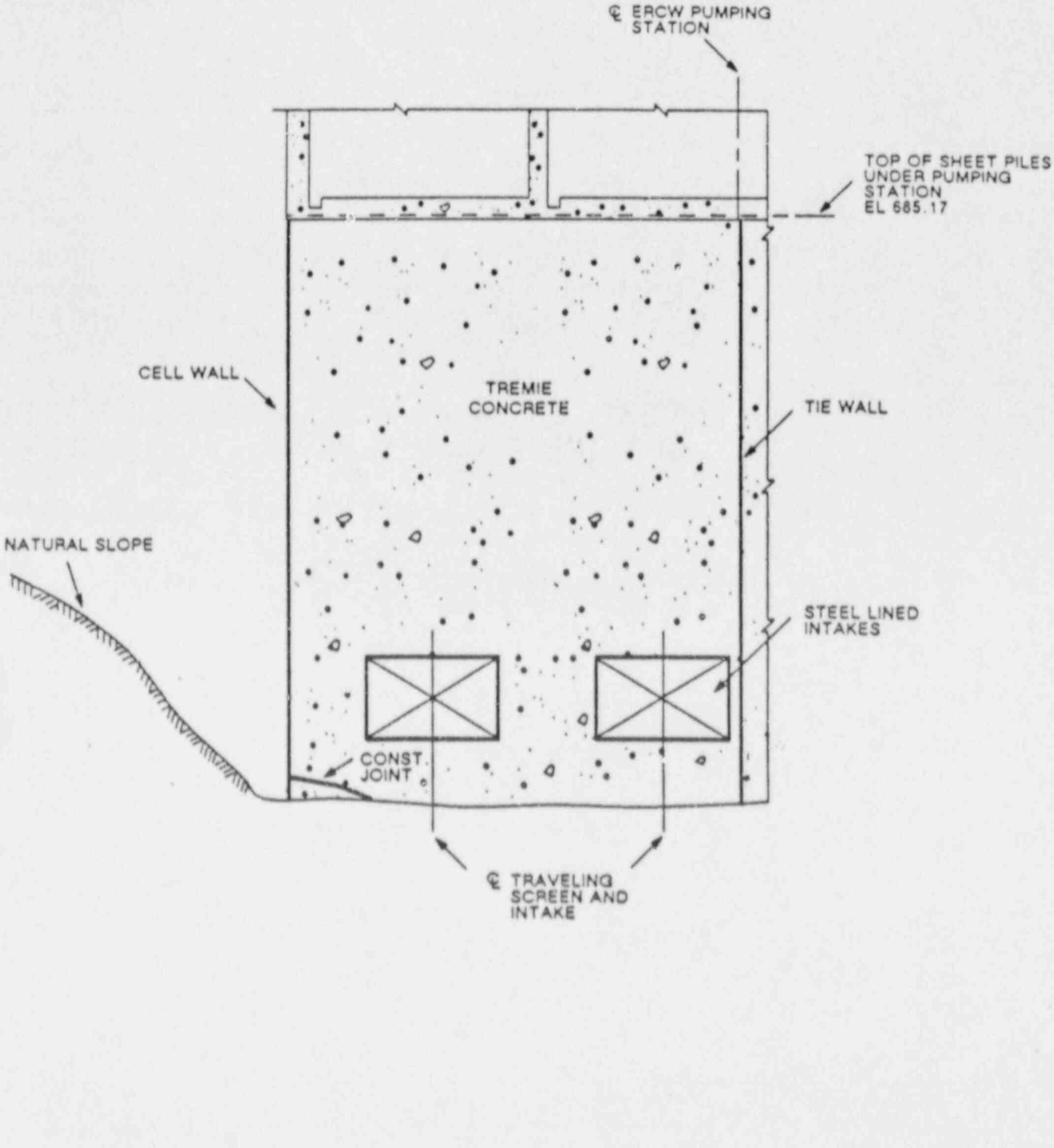
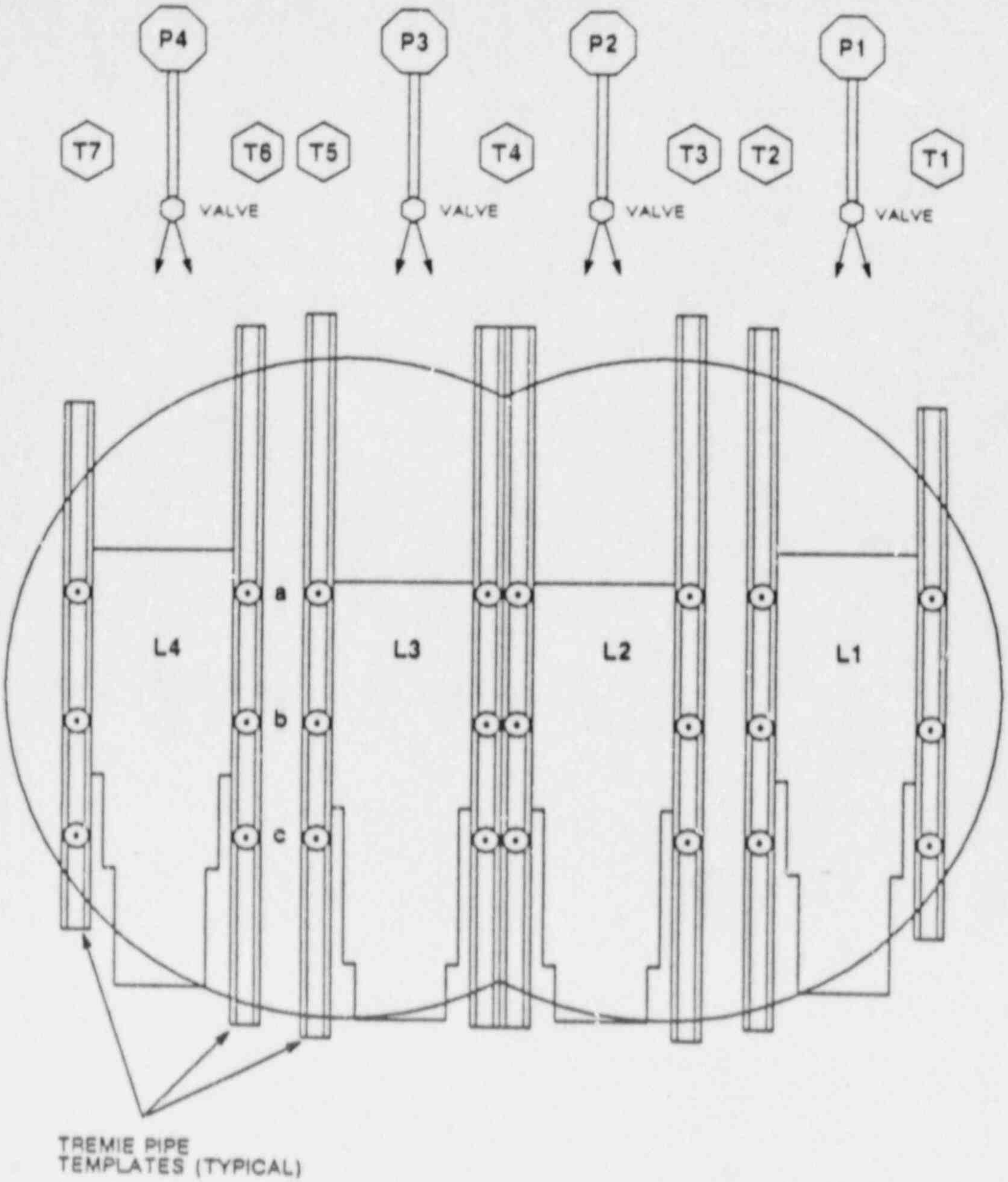


Figure 1-6

ELEVATION OF SOUTH FOUNDATION CELL AT PUMPING STATION ℄



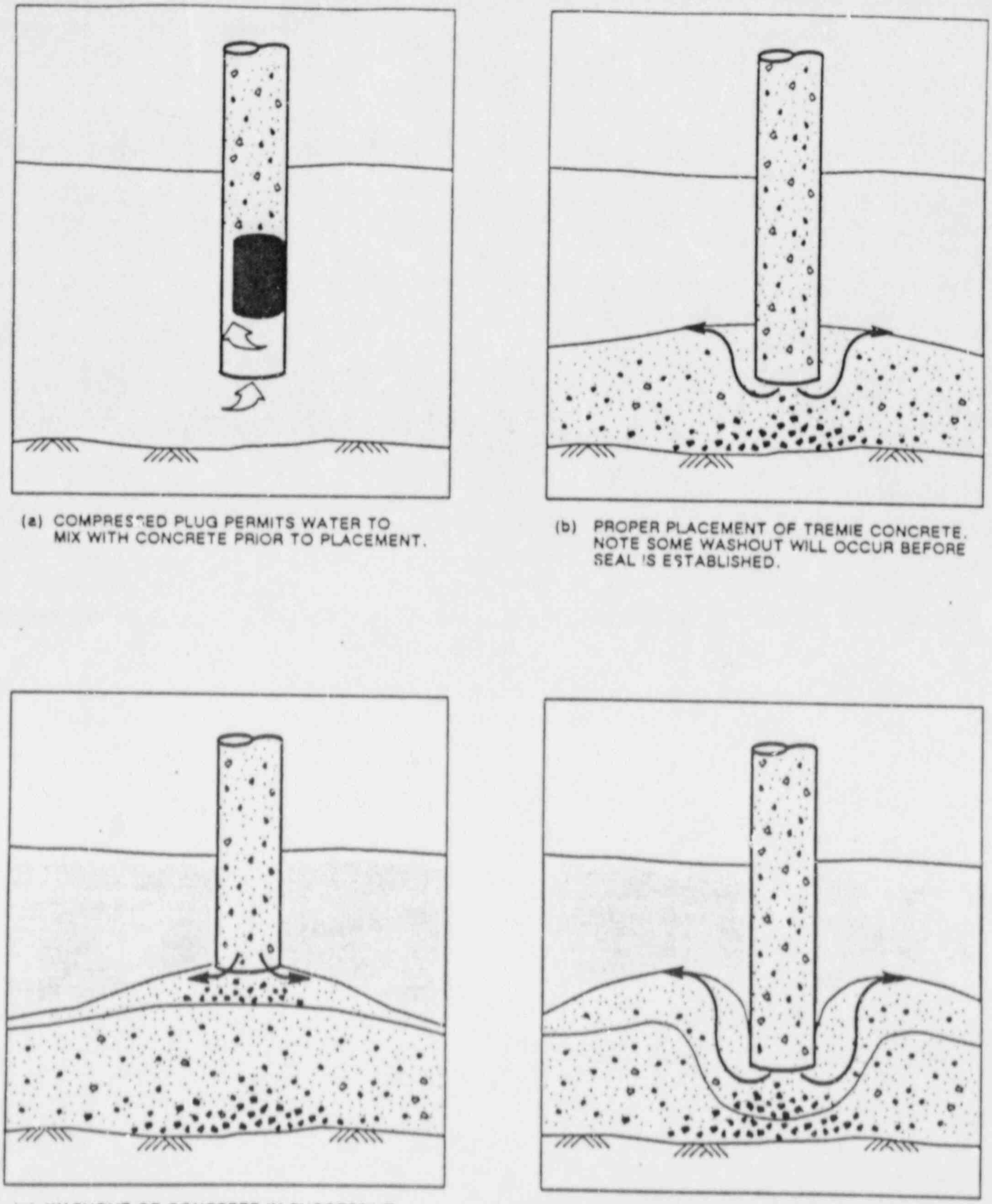
PLACEMENT SEQUENCE

1. START PUMPING CONCRETE IN TREMIE PIPES WITH PUMPS P1 THRU P4.
2. ALTERNATE PUMP LINES BETWEEN TREMIE PIPES a THRU c, UNTIL CONCRETE REACHES BOTTOM OF LINER.
3. MOVE TREMIE PIPES T1 THRU T7 BACK AND FORTH ACROSS CELL, AS REQUIRED, TO MAINTAIN A UNIFORM FLOW AND HEIGHT OF PLACED CONCRETE.
4. AS REQUIRED, CONNECT PUMP LINES ALTERNATELY BETWEEN TREMIE PIPES T1 THRU T7 TO MAINTAIN UNIFORM FLOW.

NOTE: THIS IS A SCHEMATIC LAYOUT PREPARED AND SUBMITTED BY THE CONTRACTOR. ALL DOCUMENTATION SUGGESTS A MINIMUM OF TWELVE TREMIE PIPES WERE USED. HOWEVER, THE ACTUAL NUMBER CANNOT BE VERIFIED, ALTHOUGH IT IS THOUGHT TO BE SUBSTANTIALLY LESS THAN THE TWENTY-FOUR TREMIE PIPE LOCATIONS SHOWN ABOVE.

Figure 2-1

TREMIE PIPE LAYOUT



(a) COMPRESSED PLUG PERMITS WATER TO MIX WITH CONCRETE PRIOR TO PLACEMENT.

(b) PROPER PLACEMENT OF TREMIE CONCRETE. NOTE SOME WASHOUT WILL OCCUR BEFORE SEAL IS ESTABLISHED.

(c) WASHOUT OF CONCRETE IN SUCCESSIVE LAYERS WHEN THE SEAL IS BROKEN DUE TO LIFTING OF TREMIE PIPE ABOVE CONCRETE SURFACE.

(d) WASHED-OUT CONCRETE BEING PLACED AFTER REINSERTING TREMIE PIPE INTO CONCRETE CAUSED BY WATER BEING TRAPPED IN PIPE WHICH DILUTES THE CONCRETE.

Figure 2-2
DEPOSITION OF TREMIE CONCRETE

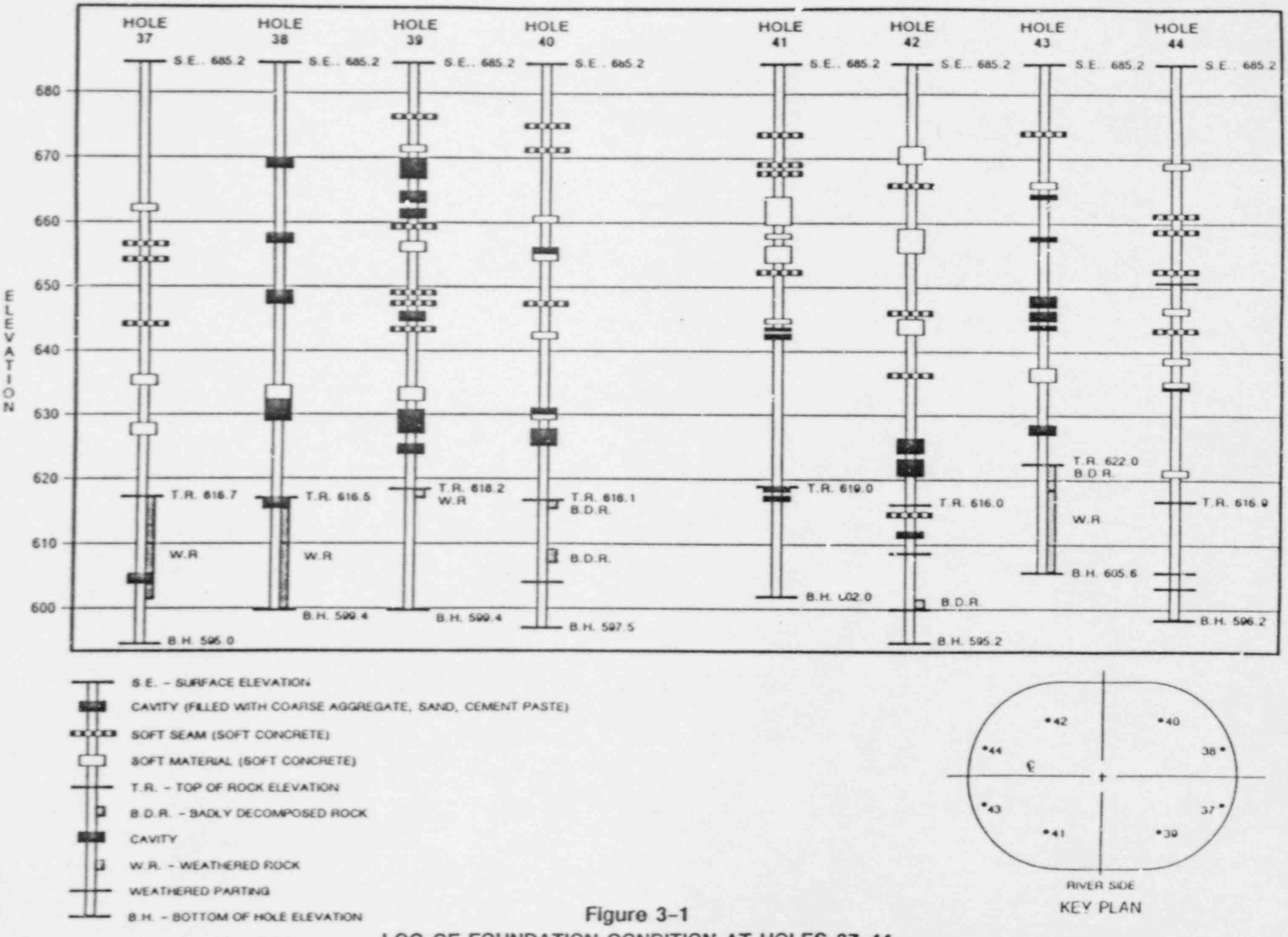


Figure 3-1
LOG OF FOUNDATION CONDITION AT HOLES 37-44

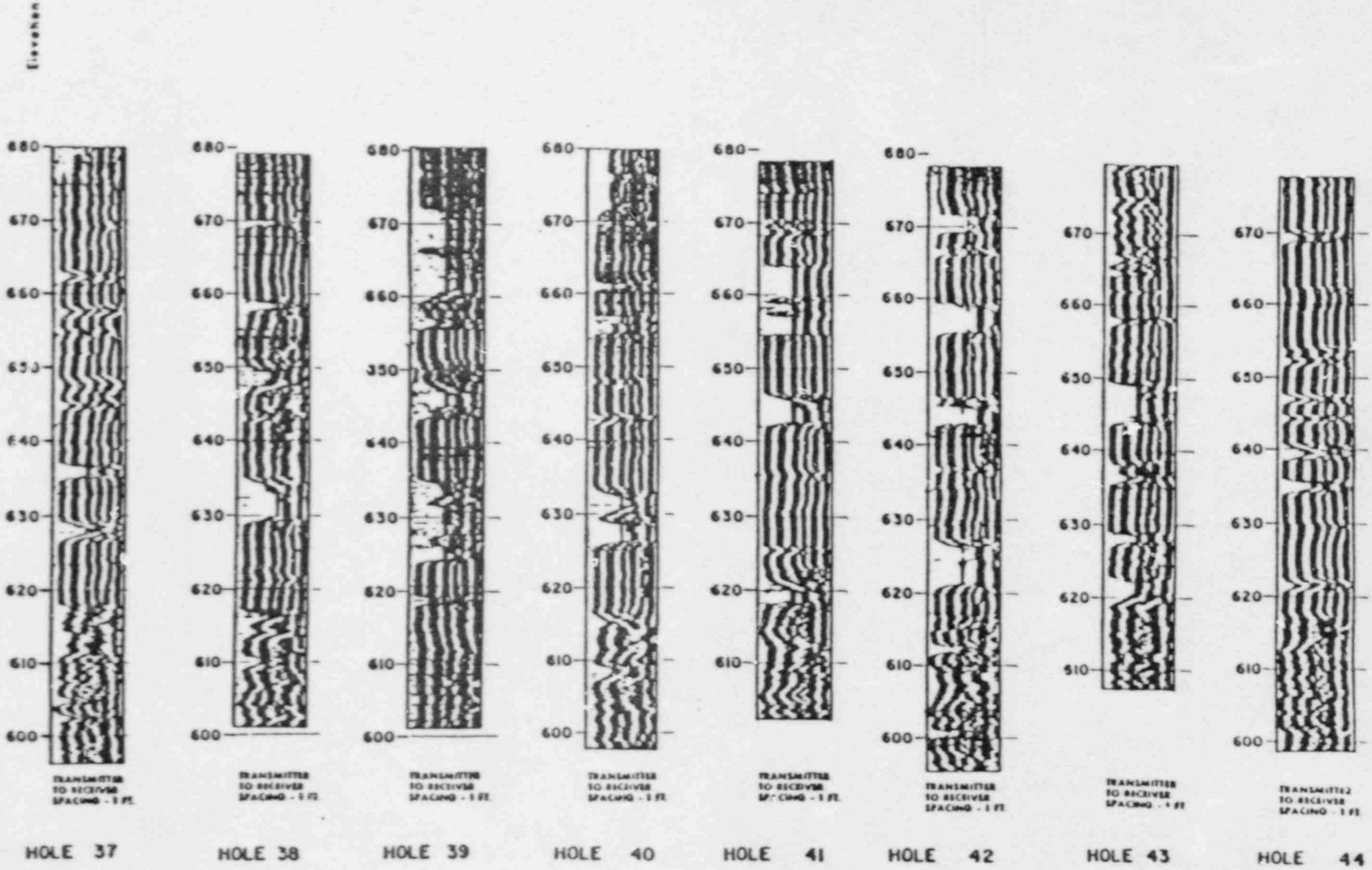
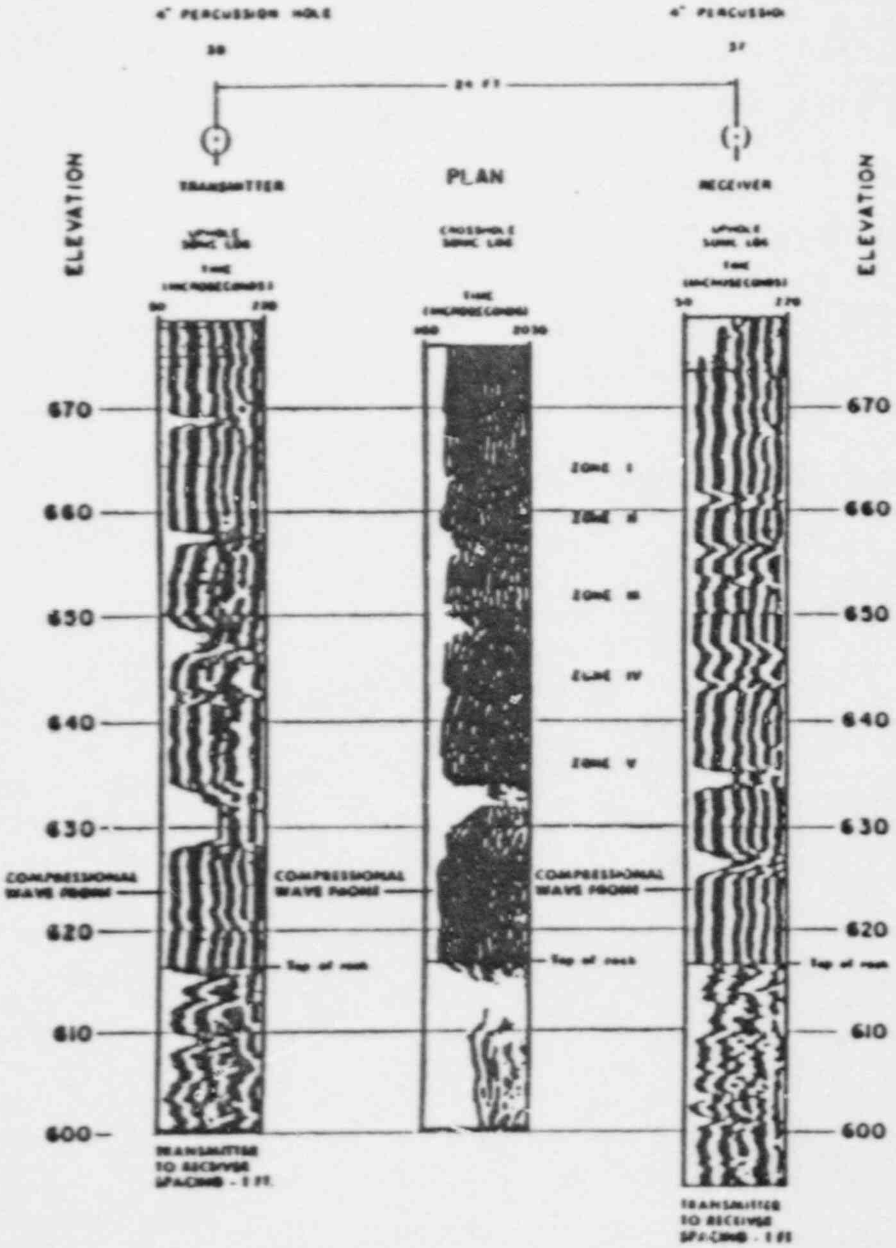


Figure 3-2

SONIC LOGS, HOLES 37-44



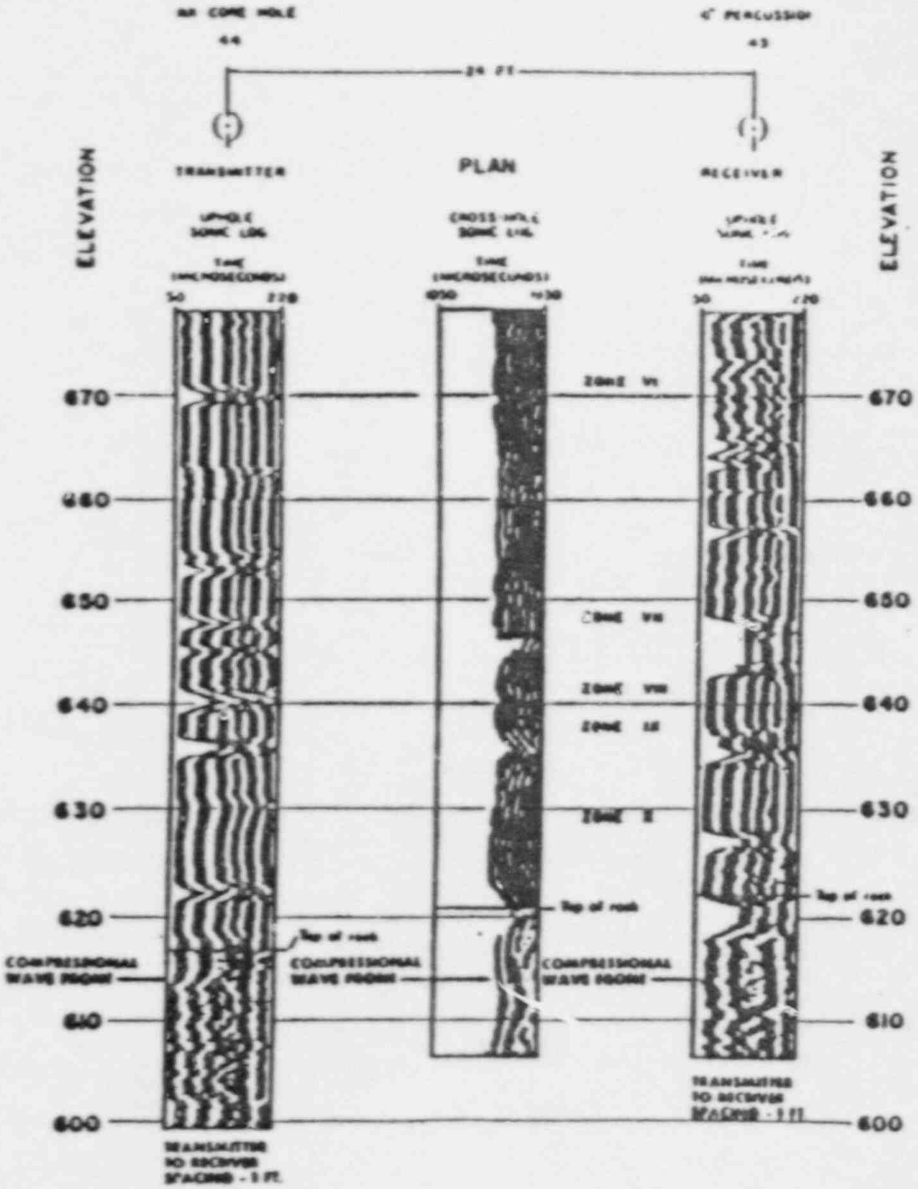
NOTE BY TVA:

A SUDDEN COMPRESSIONAL WAVE FRONT TIME INCREASE ON THE CROSS-HOLE SONIC LOG IS INTERPRETED AS A POSSIBLE ZONE OF WEAKNESS THAT MAY BE CONTINUOUS BETWEEN BOREHOLES. ZONES I AND III DO NOT APPEAR TO BE CONTINUOUS. THESE ZONES HAVE STRONG COMPRESSIONAL WAVE FRONT ARRIVALS AND NO CONTINUOUS HORIZONTAL LOW VELOCITY LAYERS. ZONES II AND IV HAVE CONTINUOUS LOW VELOCITY LAYERS BUT DO HAVE STRONG COMPRESSIONAL WAVE FRONTS.

THE SUDDEN INCREASE IN COMPRESSIONAL WAVE FRONT TIME ON CROSS-HOLE SONIC LOG FOR THESE ZONES IS PROBABLY DUE TO BOREHOLE INFLUENCE AND NOT WHAT IS BETWEEN THE BOREHOLES. ZONE V HAS A BADLY ATTENUATED COMPRESSIONAL WAVE FRONT AND A CONTINUOUS HORIZONTAL LOW VELOCITY LAYER BETWEEN BOREHOLES. THIS IS A POSSIBLE ZONE OF WEAKNESS.

Figure 3-3

UPHOLE AND CROSS-HOLE SONIC LOGS, HOLES 37 AND 38



NOTE BY TVA:

A SUDDEN COMPRESSIONAL WAVE FRONT TIME INCREASE ON THE CROSS-HOLE SONIC LOG IS INTERPRETED AS A POSSIBLE ZONE OF WEAKNESS THAT MAY BE CONTINUOUS BETWEEN BOREHOLES. ALL ZONES EXCEPT ZONE IX DO NOT APPEAR TO BE CONTINUOUS. THESE ZONES HAVE STRONG COMPRESSIONAL WAVE FRONT ARRIVALS AND NO CONTINUOUS HORIZONTAL LOW VELOCITY LAYERS BETWEEN BOREHOLES. ZONE IX HAS A BADLY ATTENUATED COMPRESSIONAL WAVE FRONT AND A CONTINUOUS HORIZONTAL LOW VELOCITY LAYER BETWEEN BOREHOLES.

Figure 3-4

UPHOLE AND CROSS-HOLE SONIC LOGS, HOLES 43 AND 44

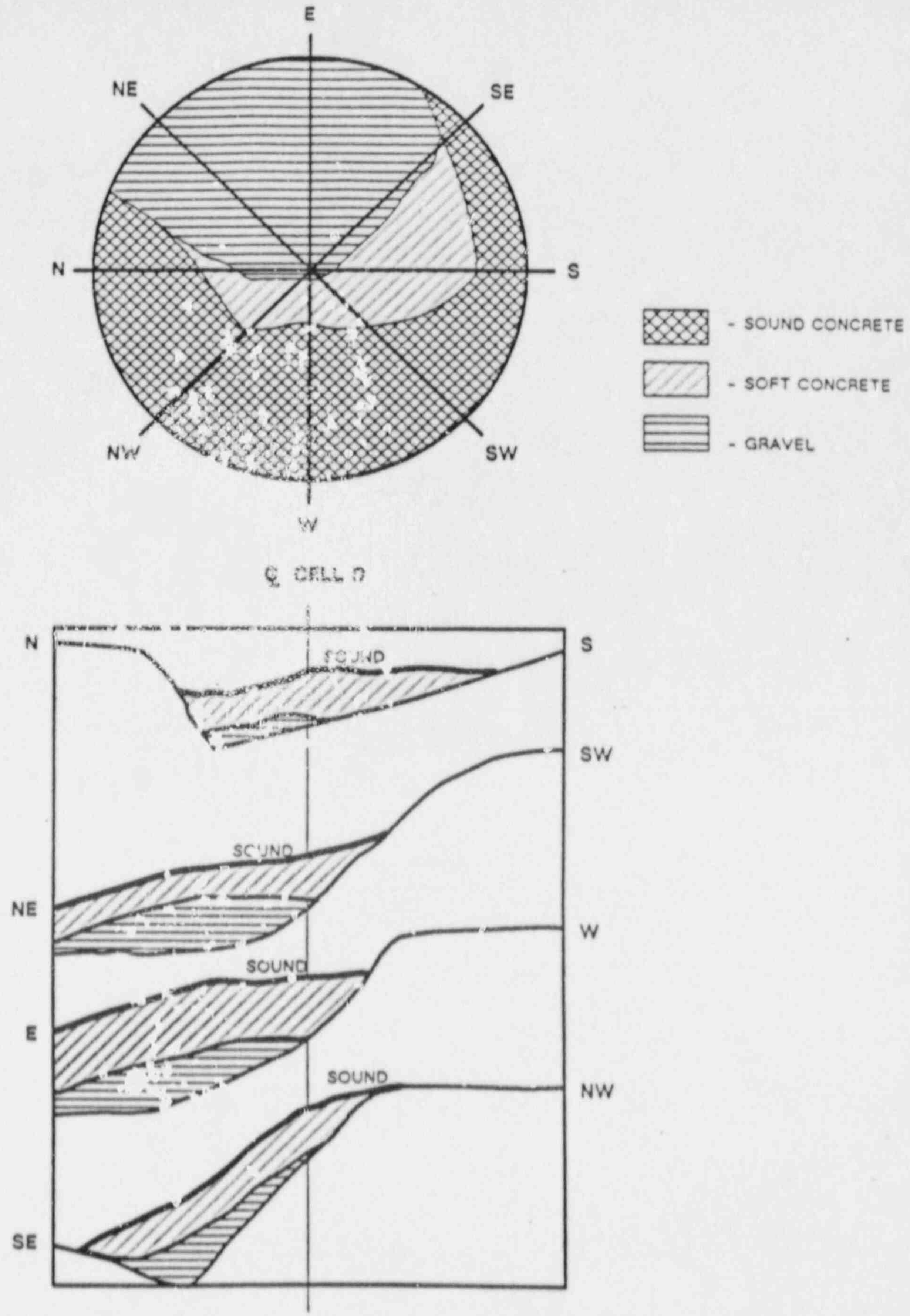


Figure 3-5
ZONES OF SOUND CONCRETE, SOFT CONCRETE, AND GRAVEL AT CONCRETE-ROCK INTERFACE OF CELL D

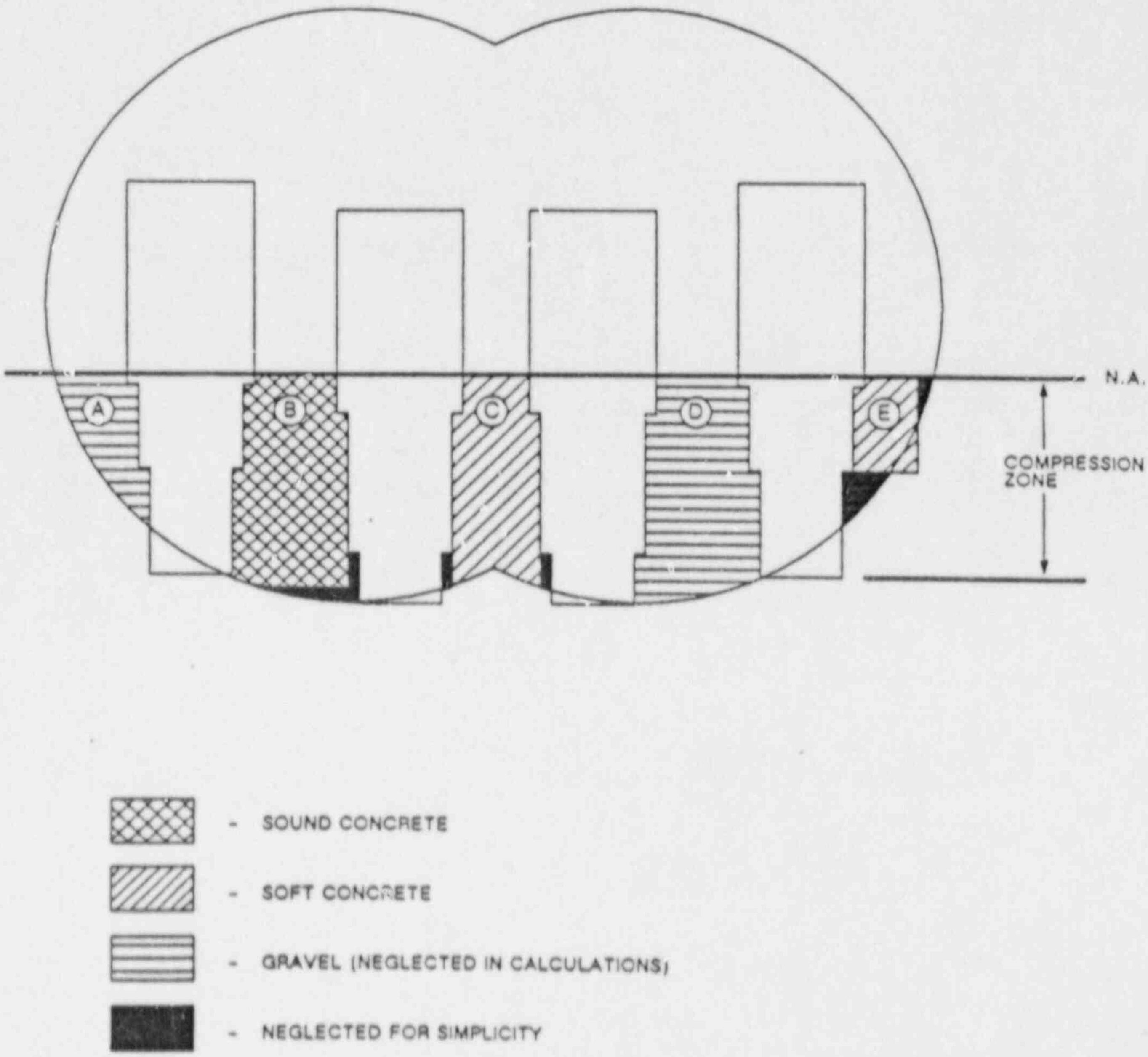


Figure 4-1

IDEALIZED AS-BUILT CONDITIONS
USED IN ANALYSIS

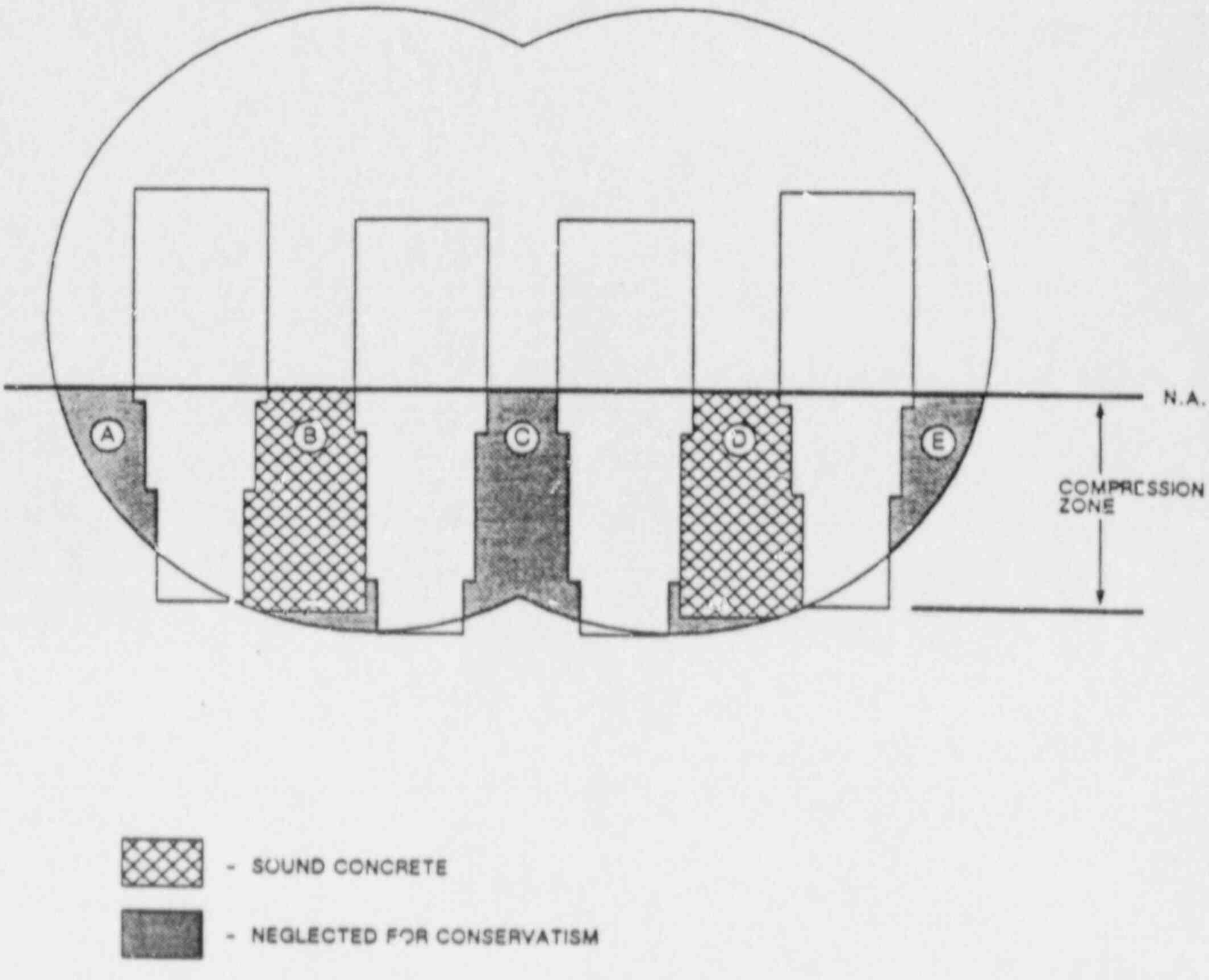


Figure 3-1

IDEALIZED CONDITIONS
USED IN ANALYSIS (AFTER LIMITED RESTORATION)

APPENDIX A - RECOMMENDED EXPLORATION PROGRAM

A.1 Introduction

This Appendix provides recommendations for accomplishing the exploration portion of the program proposed in Section 5.2 of the report. The focus of this program is element D and, to a lesser extent, element B (Figure 4-1) where changes in the presently evaluated condition would have the greatest effect on structural performance of the ERCW Pumping Station foundation. The zone of interest should be limited to structural elements below elevation 640.

Changes in elements A, C, and E, as well as the west portion of the structure (above the neutral axis in Figure 4-1), would be expected to only nominally affect the structural performance, so an exploration and grouting program for these areas is not recommended nor provided.

The exploration program involves core drilling the Pumping Station foundation. Previous drilling under similar conditions has involved drilling through sound concrete and rock; weathered rock; clean aggregate; weak, friable, sandy material; weak, friable concrete; cement paste with no aggregate; poor concrete; sand; weak concrete with little or no coarse aggregate; and similar materials in varying combinations.

A.2 Equipment

Core holes should be drilled with an NWM or equivalent diamond fan-discharge bit with a wireline core barrel. If available, a thin-walled coring bit, such as NWT or equivalent, should be used through the zone of interest and continuing for a minimum of 15 feet into bedrock. A thin-walled coring bit is preferred because a bit cutting a narrow annulus is less likely to destroy the core in a weakly cemented aggregate material such as very lean concrete.

The required exploration holes should be drilled with NX size coring equipment (3 inch nominal diameter) in order to obtain the optimum core retrieval. Any additional holes required for grouting purposes may be drilled with AW size core equipment (1.9 inch nominal diameter).

To be suitable, drilling equipment must have adequate power for the necessary drilling, but must also operate with minimal vibration to avoid disturbing or damaging the core samples. The equipment must be capable of being broken down to sizes with sufficient portability so that it can be transported by manual means and installed at potentially congested locations with limited access.

An hydraulic core extruder should be available for use in minimizing damage to the core while it is being extracted from the barrel.

Microcrystalline wax and a means to melt it should be furnished by the driller for preserving the moisture content of core samples as requested by the engineer or geologist. Sample jars should be provided for possible sampling of disaggregated material.

The use of percussion equipment should be prohibited.

A.3 Personnel

Drilling should be performed by operators experienced in drilling the types of materials to be encountered in the work. The drill operator should maintain, for each hole drilled, a detailed driller's log, noting bit pressure, significant changes in circulating water pressure, water losses and recovery, voids, and ease of drilling. The drill operator should also be alert to distinguish between drilling in soft concrete as compared to drilling in concrete aggregate without cementitious binder.

Direction of all drilling activities is to be provided by an experienced engineer or geologist. For each exploration hole, the engineer or geologist shall prepare detailed core logs of the materials removed, noting especially the zones of soft material and locations of core loss. The engineer or geologist should also maintain notes to appropriately record the drilling activities, noting such items as evidence of communication between adjacent holes, such as the circulation of drilling fluid. At the discretion of the engineer or geologist the solid material returned in any flow from drilling should be retained for possible analysis.

A.4 Drilling Program

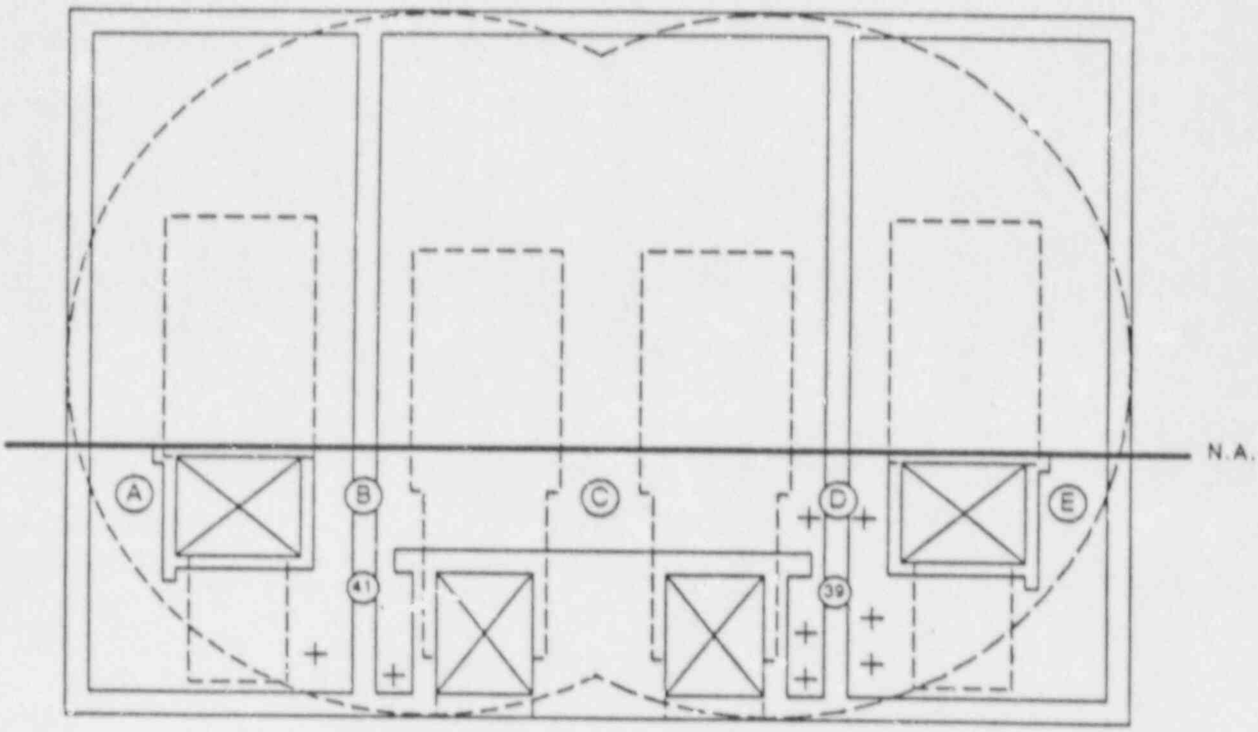
A minimum of six exploration holes should be drilled in element D. After grouting is completed in element D, a minimum of two exploration holes should be drilled in element B. All holes should be located as indicated schematically on Figure A-1, but may be locally adjusted due to space and/or equipment constraints. Sequential holes for grouting purposes should be nominally five feet apart. Holes should be drilled as close to vertical as possible, consistent with accessibility constraints and the desired location of the holes at depth. The location and alignment of all exploration holes shall be subject to the review and approval of the responsible engineer or geologist.

During the drilling operation, effort should be made to retrieve cores drilled in soft materials. If backing off or significant grinding occurs, the core should be retrieved and another run started. Cores shall be stored in core boxes and photographed as soon as possible after boring.

The engineer or geologist should evaluate the core logs to determine if any unacceptable conditions exist in the zone of concern below elevation 640. If no unacceptable condition exists in the zone of concern after drilling the required exploratory holes, the individual holes may be grouted and the program is considered completed. Unacceptable conditions shall include conditions differing from those assumed in the initial analysis, which is anything other than concrete or rock of sufficient soundness to meet the design strength requirements.

Additional holes may be drilled to improve communication for grouting as directed by the grouting engineering specialist (see Appendix B, B-3). Exploration is not required for such holes. This should be done at appropriate spacing when the core logs indicate there are unacceptable conditions in the zone of concern but there is only minimal communication in this zone. Any proposed additional coring for exploration purposes should receive engineering approval prior to drilling.

Efforts should be made to minimize the cutting of reinforcing steel using such techniques as initial exploratory chipping of concrete to identify reinforcing steel locations in the upper portion of the slab and relocating the drill if reinforcing steel is encountered at the lower level. Any cutting of reinforcing steel should be dealt with under normal procedures.



ERCW PUMPING STATION AT ELEVATION 688

(41) - PREVIOUS INVESTIGATION HOLE LOCATION-TYPICAL

+ - RECOMMENDED EXPLORATION HOLE LOCATION AT ELEVATION 620' - 630'

NOTE: HOLES MAY BE DRILLED AT AN ANGLE IN ORDER TO ACCOMMODATE ACCESSIBILITY AT ELEV. 688'

Figure A-1

RECOMMENDED EXPLORATION LOCATIONS

APPENDIX B - RECOMMENDED GROUTING PROGRAM

B.1 Introduction

This Appendix provides recommendations for accomplishing restoration of any unacceptable conditions found within the zone of concern between elevation 640 and the top of rock (approximate elevation 615) for elements B and D as shown in Figure 4-1. Recommendations are also provided to establish the means of filling any holes made as a part of the exploration activity that do not encounter any unacceptable conditions. The method employed for both of these activities will be pressure grouting as described in the following sections.

B.2 Equipment and Materials

Equipment for grouting should be capable of satisfactorily supplying, mixing, stirring, and pumping the grout. Grout mixers should be colloidal type. Grout pumps should be the helical screw rotor type which will deliver a continuous supply of grout to the area of concern with minimum pressure fluctuation. The equipment should be of such a size as to be transported by manual means and installed at potentially congested locations with limited access.

Use of the following materials should be permitted:

(1) Portland cement, Type II, conforming to ASTM C-150

(2) Microfine cement (MC-500, Geochemical Corporation, or equivalent)

(3) Fluidifier for microfine cement (NS-200, Geochemical Corporation, or equivalent)

(4) Shrinkage-compensating admixture (Intrusion Aid, Type LS, Concrete Chemicals Company, or equivalent)

B.3 Personnel

Grouting activities, including preparation, should be under the general supervision of a grouting engineering specialist who has prior experience in remedial grouting on major tremie concrete structures. The grouting engineering specialist should have the responsibility to design the grout mixes, approve the grouting procedures, and provide for supervision as appropriate during the execution of grouting operations. The grout mixes and procedures should provide for appropriate adjustments to be made during the grouting process so that the maximum amount of filling is accomplished using the densest grout possible.

Grout logs should be kept for each hole by an experienced grouting engineer. Logs should include hole number, depth interval, time, quantity injected, water/cement ratio, injection pressure, communication, as well as, drilling and water testing data.

B.4 Grouting Program

Pressure grouting should be required to backfill all drill holes, to inject open fissures, and to consolidate loose materials. Additional holes may be required at the direction of the grouting engineering specialist in order to allow for displacement of water and unconsolidated material such as sand and laitance. All holes and segments designated to receive grout should be flushed with water to wash out as much fine material as is practicable prior to grouting.

Under the direction of the grouting engineer, permeability testing using water should be performed on zones isolated by double packers. Dye may be injected with water or grout in order to observe any communication between drill holes and interconnected zones of weak material.

Consistency of grout should be sufficiently fluid to ensure penetration into coarse sands.

Pressure grouting should be performed using approved procedures produced for the work at hand which may require grouting in stages isolated by packers. The procedures should require sufficient grouting to assure filling of all major lenses of loose, segregated material with grout for elements B and D between elevation 640 and top of rock. Above elevation 640, pressure grouting should be performed to backfill any zone of unacceptable material that can be reached using the existing network of holes available at that time. Further drilling solely for the purpose of reaching all potential lenses above elevation 640 is not needed. The progress of the grouting program should be reviewed by engineering to assure that the condition of concrete after the completion of this program is consistent with the design requirements as discussed in this report.

Recommended injection pressures should be established by the grouting engineering specialist taking into consideration grout communication, grouting medium, quality injected, and distance of the hole from the cell boundary. In general, grout pressures should not exceed one psi per foot of depth in the foundation rock and should not exceed one and one-half psi per foot of depth in concrete. A maximum pressure of 35 psi should be considered, as fracture of the structure can initiate above 40 psi.

A general criteria for determining the need for additional grouting in the areas below elevation 640 is as follows:

Grout Take (cu ft) per Foot Depth of Hole	Action
0 - 0.125	Accept
0.125 - 0.25	Questionable
> 0.25	Unacceptable, additional grouting required in secondary holes near this location

The grouting of a hole should not be considered complete until the hole or grout interval takes less than one cubic foot of grout in fifteen minutes at the pressure established for that interval of the hole.