TECHNICAL REPORT ON UNDERPINNING
THE SERVICE WATER PUMP STRUCTURE
FOR
MIDLAND PLANT UNITS 1 AND 2
CONSUMERS POWER COMPANY
DOCKET NUMBERS 50-329 AND 50-330

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TECHNICAL REPORT ON UNDERPINNING
THE SERVICE WATER PUMP STRUCTURE

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TECHNICAL REPORT ON UNDERPINNING

THE SERVICE WATER PUMP STRUCTURE

1.0 INTRODUCTION

This report describes the design and construction requirements of the remedial action for the service water pump structure (SWPS) necessitated by the settlement potential of the plant fill underlying the structure.

2.0 PRESENT CONDITION

The SWPS is a two level, rectangular, reinforced concrete structure. Below el 617', it measures 86 feet by 71 feet 11 inches; above el 617' it measures 106 feet by 86 feet. The maximum overall height is 69 feet [See Figures 1 and 2 (FSAR Figures 3.8-56 and 3.8-57)].

The structure was designed to be supported by the two foundation slabs, one at el 587'-0" and the other at el 617'-0". The lower slab rests on undisturbed natural material and the upper slab rests on fill material placed during construction in 1977.

After discovering settlement of the fill under the diesel generator building, an investigation of the plant fill revealed some questionable areas under the upper base slab, el 617'-0", of the SWPS.

3.0 REMEDIAL ACTION

For the part of the structure resting on plant fill, a continuous underpinning wall, resting on undisturbed natural material, is provided to support the structure adequately under all design load conditions. The underpinning wall provides the necessary vertical and horizontal support to the affected part of the structure. To ensure adequate load transfer, the underpinned structure is jacked from the underpinning walls (Refer to Figure 3).

4.0 DESIGN FEATURES

The proposed underpinning is a 4-foot thick, reinforced concrete wall that is 30 feet high and is constructed to act as a continuous member under the perimeter of the structure overhang. The entire wall is founded on undisturbed natural material. The base of the north underpinning wall is belled out to a 6-foot thickness to limit bearing pressures to the allowable values, whereas the bases of the east and west side walls are 4 feet wide. The allowable bearing pressures

for the undisturbed natural material are based on safety factors of 2 for dynamic loading and 3 for static loading.

A predetermined jacking force is applied to the overhang perimeter to provide adequate load transfer from the structure to the underpinning.

The connection between the underpinning wall and the existing structure is made by 2-inch diameter rock bolts at the vertical interfaces and 2-3/4-inch diameter anchor bolt assemblies at the horizontal interfaces (Refer to Figures 4 and 5). The connectors are designed to transfer shear and tension forces to the underpinned wall. The connectors are not subject to stresses during the jacking procedures because the rock bolts have not yet been installed and the anchor bolts have not been tightened (Refer to Subsection 5.3.2). After the underpinning wall is connected to the existing structure, the connectors are stressed by loads applied to the underpinned structure.

5.0 CONSTRUCTION

The construction procedures discussed in this report are recommended for underpinning the SWPS. If subcontractor recommendations result in improved procedures, they will be incorporated. For details of construction and the construction procedures, refer to Figures 4 and 5.

5.1 DEWATERING

To construct the underpinning, the SWPS site is dewatered: The groundwater level is lowered to el 587 (approximately) by using temporary dewatering wells. These wells will be sealed after the underpinning wall is completed. The acceptance criteria for the dewatering system require that the system produces an effluent that has less than 10 parts per million of soil particles larger than 0.05 millimeters.

5.2 BUILDING POST-TENSIONING

Construction site dewatering removes the buoyancy force on the overhang portion of the structure, resulting in additional loading on the overhang. To compensate for this additional loading of the overhang, a temporary post-tensioning system applies a compressive force to the upper part of the building along each north-south wall. This post-tensioning allows the additional force to be transferred from the overhang by beam action to the adjoining walls which rest on undisturbed natural material (Refer to Figure 6). The post-tensioning system is removed after the initial jacking loads are applied.

5.3 CONSTRUCTION PROCEDURES

The underpinning is constructed as individual piers tied together by threaded reinforcing bar couplers and shear keys to form a continuous wall. Refer to details and procedures in Figures 4 and 5.

5.3.1 Initial Construction Activities

To preserve the structural integrity of the building, the underpinning wall is constructed in small sections (piers) from tunnels which are advanced simultaneously from access shafts located at the northeast and northwest corners of the building. The tunnels initially extend only far enough to construct an approximately 30-foot deep, 5 foot by 4 foot, sheeted pit at each corner of the overhang. The pit is hand dug. The shear strength of the subgrade soil is assessed with a Corps of Engineers cone penetrometer, model CN-973. Under a maximum force of 150 pounds, the cone should not penetrate the surface more than 1/2 inch. After the subgrade is inspected and approved by a geotechnical engineer, reinforcement, subgrade settlement monitoring instrumentation, and anchor bolt assemblies to tie the pier to the underside of the slab, are installed. The pier is then cast with concrete pumped from the access shaft. After at least 48 hours of curing, an initial jacking load is applied to the overhang from jacks placed on the pier top. To ensure adequate support to the building, the tunnel is not advanced to the next stage until the pier is jacked.

Simultaneously with applying the jacking force, the tunnels are advanced to the location of the next pier, which is constructed in a similar manner to the first pier. The piers are tied together with threaded reinforcing bar couplers and shear keys to form a continuous underpinning wall. The threaded reinforcing bar couplers (see Detail 1, Figure 5) conform to the requirements of Section III, Division 2 of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code, 1980 Edition, 1980 and 1981 Summer Addenda. The tensile strength of the splice system is not less than 125% of the specified minimum yield strength of the spliced bar.

A settlement monitoring program for the top and base of each pier begins immediately after pier construction. Instruments accurate to 0.001 inch are installed before the initial jacking is applied. The information from the monitoring program is used to evaluate the time required to dissipate shrinkage and creep of the concrete and creep of the undisturbed natural material. The rate of settlement decreases with time. At the proper point on the settlement-time curve (as determined by the geotechnical engineer), the final jacking operations (as described below) begins.

5.3.2 Final Jacking Stage

After Piers 10 (Figure 4) are constructed, the underpinning wall has progressed to within 6 feet of the vertical interfaces with the existing structure, and the final jacking load is applied. Settlements caused by this load are monitored. When the geotechnical engineer judges that the settlement rate has decreased to a proper value, the load is transferred from the jacks to wedges positioned between the top of the piers and the underside of the overhang, and the jacks are removed. Piers 11 are poured, encasing rock anchors that were previously drilled into the vertical face of the existing structure and thereby connecting the underpinning wall to the vertical face of the existing structure (Refer to Detail 5, Figure 5). The space between the top of the underpinning wall and the underside of the base slab is filled with nonshrink grout, and previously placed anchor bolt assemblies (which tie the top of the piers to the foundation slab) are tightened (Refer to Detail 7, Figure 4). The underpinning wall is connected to the structure at both the vertical and horizontal interfaces.

5.3.3 Completion of the Underpinning Wall

The tunnel is backfilled with lean concrete beginning at the vertical interface and at the north wall. The completion of the tunnel backfilling terminates at the locations of Piers 12. These piers are then constructed, completing the underpinning wall.

6.0 MONITORING REQUIREMENTS

During construction, the underpinning of the existing structure is monitored for settlement and crack propogation. The long-term surveillance program of the building after the construction of the underpinning is being evaluated.

6.1 SETTLEMENTS

The elevations of settlement markers attached to the structure are measured in accordance with a schedule based on construction procedures. Expected building movements during underpinning operations are small. These movements are recorded, and those exceeding 1/4 inch will be evaluated and reported to the NRC.

6.2 CRACKS

Monitoring of existing or new cracks appearing during the underpinning construction is scheduled. Because of the

sequencing of construction procedures, it is not anticipated that existing cracks will significantly widen or new cracks will appear. However, any new structural cracks or changes in existing structural crack widths exceeding 0.010 inch will be evaluated and reported to the NRC.

7.0 ANALYSIS AND DESIGN

The SWPS was originally designed in accordance with FSAR requirements for Seismic Category I structures. A preliminary analysis of the underpinned structure was made which complied with these FSAR requirements, and added a jacking load to the load combinations. The seismic loads used in this analysis were extrapolated from the seismic loading from a previous underpinning design based on piles. When the final seismic loads become available, they will be incorporated in the final design.

In the final design, seismically induced forces and instructure response spectra of the structure are generated in accordance with FSAR Section 3.7. The revised model portrays the structural behavior including the effects of the underpinning and associated foundation modification.

The mathematical seismic model and a description of the soil-structure interaction coefficients to be used in the seismic analysis will be submitted to the NRC in September 1981.

The static structural analysis uses an analytical model capable of representing the structure behavior. The interface between the existing structure and the underpinning wall is modeled to transfer direct loads without providing rotational restraint. The soil media are represented by springs of appropriate stiffness at the base of the structure. The detailed analysis will be performed by conventional methods such as beam theory and/or plate theory or by using the computer program Bechtel Structural Analysis Program (BSAP). For details of the BSAP computer program see FSAR Subsection 3.8.3.4.

7.1 STRUCTURE BEHAVIOR

The vertical loads of the structure are transmitted to the foundation medium through the existing base slab at el 587'-0" and the underpinning wall bearing area. The lateral forces due to seismic and tornado loads are resisted by the shear walls in the structure. These lateral loads are transferred to the foundation medium by the combined action of the base slab at el 587'-0" and the underpinning wall bearing area. To ensure this action, the underpinning walls are connected to the existing structure by rock anchors and anchor bolts capable of transferring all direct loads. This connection is a pinned connection that is consistent with the analysis method.

7.2 DESIGN CRITERIA AND APPLICABLE CODES

The underpinned structure is designed as a Seismic Category I structure. The design complies with the requirements of ACI 318-71 and the 1969 edition of the AISC.

7.3 LOADS AND LOAD COMBINATIONS

The underpinning structure rests entirely on undisturbed natural material. The preliminary analysis of the underpinned structure utilizes the same load combinations used in the original design. However, each load combination is modified by adding the jacking load (P_L). For each loading combination, the jacking load was evaluated with two load factors: a value of 1.0, and the load factor associated with the dead load for that load combination.

For the design of the underpinning and the connections to the existing structure, the safe shutdown earthquake (SSE) forces were increased by 50% to provide for a possible future increase in this loading. The 50% increase was applied to the seismic response of the structure corresponding to the analytical model with the mean soil properties. The existing structure was checked for a 0.12g SSE.

The long-term settlement of the underpinning wall after it is connected to the existing structure will be calculated. The calculation is based on properties of the supporting soil. The long-term settlement effects will be considered in the final analysis of the structure. To provide for these effects, the final analysis is governed by four additional load combinations. These load combinations are discussed in the response to Question 15 of the NRC Requests Regarding Plant Fill (September 1979) and were used in the diesel generator building reanalysis. The load combinations are modified by the addition of the jacking load.

Table 1 lists 26 load combinations, modified for jacking loads. For the preliminary analysis of the underpinned SWPS, the following load combination was most critical:

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L$$

where

D = dead loads

L = live loads

E' = safe shutdown earthquake

T_o = thermal effects during normal operating conditions

H_o = force on structure due to thermal expansion of pipes under operating conditions

R = local force or pressure on structure or penetration caused by rupture of any one pipe

P_L = load on structure due to jacking preload

In addition to this load combination, the underpinned structure was checked for stability using the load combinations specified in FSAR Subsection 3.8.6.3.4.

A complete analysis of the underpinned structure, using all applicable load combinations, will be made when the final seismic loads become available.

7.4 STRUCTURAL ACCEPTANCE CRITERIA

The acceptance criterion for analyzing the underpinned structure is in accordance with FSAR Subsection 3.8.6.5.

8.0 QUALITY ASSURANCE REQUIREMENT

This project work is a combination of Q- and non-Q-listed work. The construction of the permanent structures such as the underpinning wall and the connectors are Q-listed, as well as any other activity or structure necessary to protect the SWPS. Construction of temporary structures such as the access shafts and tunnels is non-Q-listed. A detailed quality plan shall be prepared by the subcontractor to identify those specific activities which are required to have a safety "Q" quality program applied along with the major quality program elements for these activities. This quality plan shall be approved by Bechtel and Consumers Power Company prior to the start of any Q-listed work.

9.0 ADDITIONAL NRC REQUIREMENTS

For information purposes, an analysis of the critical sections of the underpinned structure will be made conforming to the provisions of ACI 349-76 as supplemented by NRC Regulatory Guide 1.142.

TABLE 1

LOAD EQUATIONS FOR THE SERVICE WATER PUMP STRUCTURE MODIFIED TO INCLUDE PRELOAD

Responses to NRC Requests Regarding Plant Fill, Question 15

a. Normal Operating Condition

$U = 1.05D + 1.28L + 1.05T + P_L$ (1)

$U = 1.4D + 1.4T + P_L$ (2)

b. Severe Environmental Condition

$U = 1.0D + 1.0L + 1.0W + 1.0T + P_L$ (3)

$U = 1.0D + 1.0L + 1.0E + 1.0T + P_L$ (4)

Loading Under Normal Conditions

a. Concrete

$U = 1.4D + 1.7L + P_L$ (5)

$U = 1.25 (D + L + H_o + E) + 1.0T_o + P_L$ (6)

$U = 1.25 (D + L + H + W) + 1.0T_o + P_L$ (7)

$U = 0.9D + 1.25 (H + E) + 1.0T_o + P_L$ (8)

$U = 0.9D + 1.25 (H + W) + 1.0T_o + P_L$ (9)

For ductile moment resisting concrete frames and for shear walls

$U = 1.4 (D + L + E) + 1.0T_o + 1.25H_o + P_L$ (10)

$U = 0.9D + 1.25E + 1.0T_o + 1.25H_o + P_L$ (11)

Structural Elements Carrying Mainly Earthquake Forces, Such as Equipment Supports

$U = 1.0D + 1.0L + 1.8E + 1.0T_o + 1.25H_o + P_L$ (12)

b. Structural Steel

$D + L + P_L$ (stress limit = f_s) (13)

$D + L + T_o + H_o + E + P_L$ (stress limit = $1.25f_s$) (14)

Table 1 (Continued)

$$D + L + T_o + H_o + W + P_L \text{ (stress limit } = 1.33f_s) \qquad (15)$$

In addition, for structural elements carrying mainly earthquake forces, such as struts and bracing:

$$D + L + T_o + H_o + E + P_L \text{ (stress limit } = f_s) \qquad (16)$$

Loading Under Accident Conditions

a. Concrete

$$U = 1.05D + 1.05L + 1.25E + 1.0T_A + 1.0H_A + 1.0R + P_L \qquad (17)$$

$$U = 0.95D + 1.25E + 1.0T_A + 1.0H_A + 1.0R + P_L \qquad (18)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25H_o + 1.0R + P_L \qquad (19)$$

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_A + 1.0H_A + 1.0R + P_L \qquad (20)$$

$$U = 1.0D + 1.0L + 1.0B + 1.0T_o + 1.25H_o + P_L \qquad (21)$$

$$U = 1.0D + 1.0L + 1.0T_o + 1.25H_o + 1.0W' + P_L \qquad (22)$$

b. Structural Steel

$$D + L + R + T_o + H_o + E' + P_L \text{ (stress limit } = 1.5f_s) \qquad (23)$$

$$D + L + R + T_A + H_A + E' + P_L \text{ (stress limit } = 1.5f_s) \qquad (24)$$

$$D + L + B + T_o + H_o + P_L \text{ (stress limit } = 1.5f_s) \qquad (25)$$

$$D + L + T_o + H_o + W' + P_L \text{ (stress limit } = 1.5f_s) \qquad (26)$$

where

U = required strength to resist design loads or their related internal moments and forces

For the ultimate load capacity of a concrete section, U is calculated in accordance with ACI 318-71.

F_y = specified minimum yield strength for structural steel

f_s = allowable stress for structural steel; f_s is calculated in accordance with the AISC Code, 1963 Edition for design calculations initiated prior to February 1, 1973.

f_s is calculated in accordance with the AISC Code, 1969 Edition, with Supplements, 1, 2, and 3 for design calculations initiated after February 1, 1973.

Table 1 (Continued)

D = dead loads

L = live loads

P_L = load on structure due to jacking preload

R = local force or pressure on structure or penetration caused by rupture of any one pipe

T_o = thermal effects during normal operating conditons

H_o = force on structure due to thermal expansion of pipes under operating conditions

T_A = total thermal effects which may occur during a design accident other than H_A

H_A = force on structure due to thermal expansion of pipes under accident condition

E = operating basis earthquake (OBE)

E' = safe shutdown earthquake load (SSE)

B = hydrostatic forces due to the postulated maximum flood (PMF) elevation of 635.5 feet

W = design wind load

W' = tornado wind loads, including missile effects and differential pressure

$\emptyset$ = capacity reduction factor

The capacity reduction factor ($\emptyset$) provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, although individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity.

NOTES:

1. In the load equations, the following factors are used:

$\emptyset$ = 0.90 for reinforced concrete in flexure

$\emptyset$ = 0.75 for spirally reinforced concrete compression members

$\emptyset$ = 0.70 for tied compression members

$\emptyset$ = 0.90 for fabricated structural steel

Table 1 (Continued)

Ø = 0.90 for reinforced steel in direct tension

Ø = 0.90 for welded or mechanical splices of reinforcing steel

2. Unity load factor is shown for P_L. An alternative load factor to be considered in all load combinations is the load factor associated with dead load (D) in that loading combination.

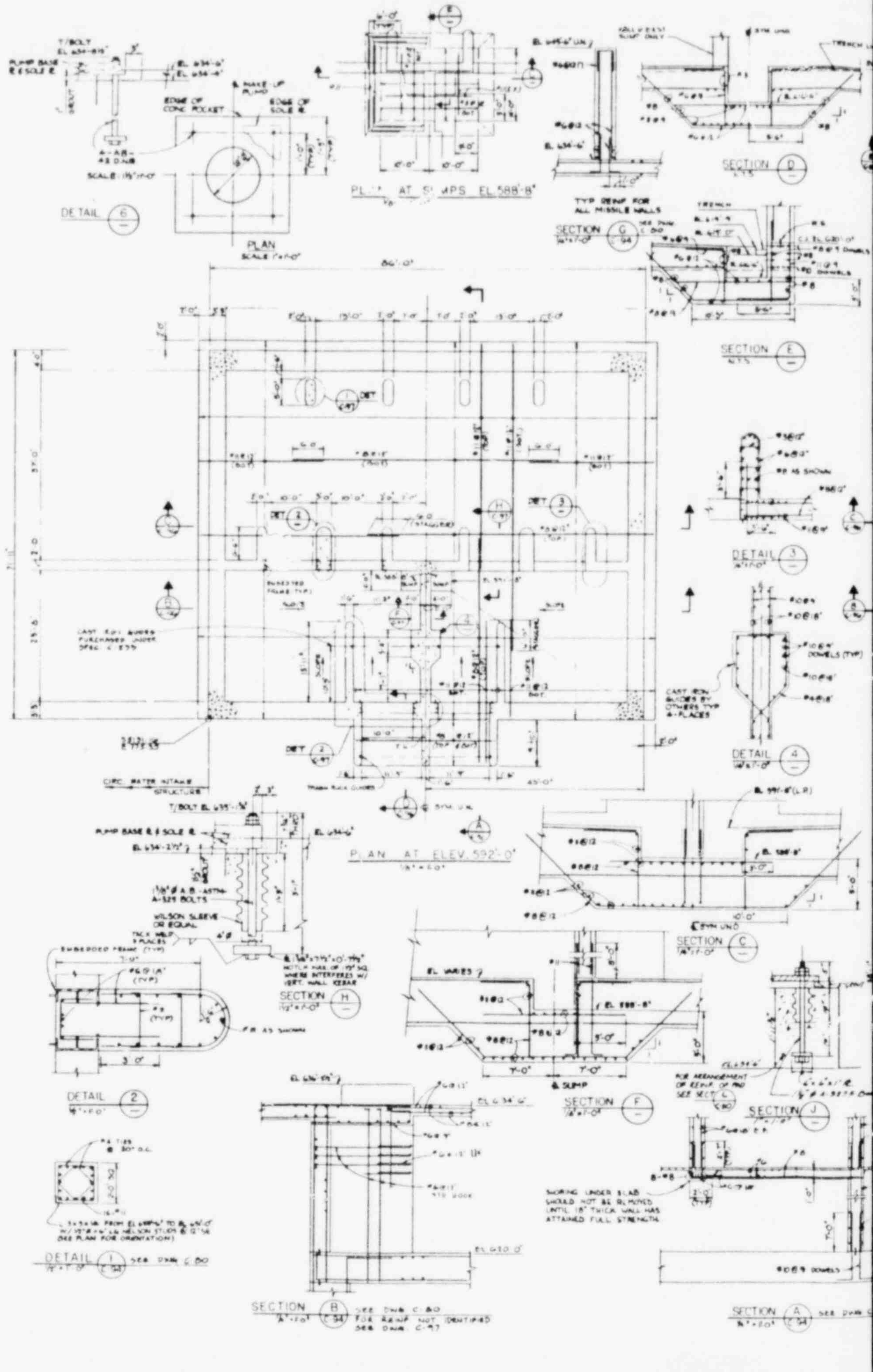
For load combinations 23-26:

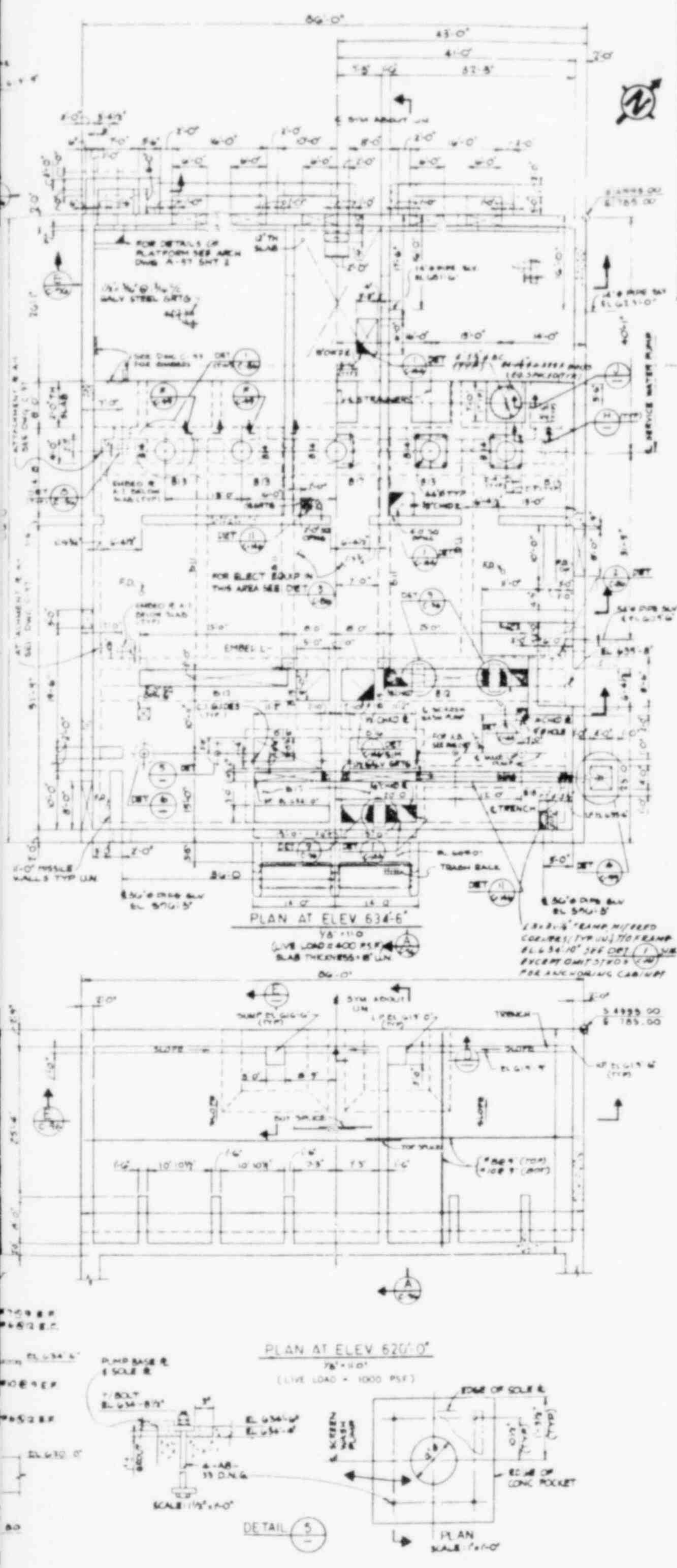
Maximum allowable stress in bending and tension is 0.9 F_y. Maximum allowable stress in shear is 0.5 F_y.

For these load combinations, the maximum allowable stress except for local areas that do not affect overall stability is limited to 0.9 F_y for bending, bearing, and tension and 0.5 F_y for shear. The time phasing between loadings is used where applicable to satisfy the above equations.

Structural components subjected to postulated impulse loads and/or impact effects are designed in accordance with BC-TOP-9-A, Rev 2, using ductility ratios not exceeding 10.

Structural members subjected to missile and pipe break loads are designed in accordance with Bechtel's BC-TOP-9-A, Rev 2, and Bechtel's BN-TOP-2, Rev 2.


DETAIL 6
SCALE: 1½"=1'-0"
PLAN
SCALE 1"=1'-0"
EDGE OF CONC POCKET
EDGE OF SOLE PL
MAKE-UP PUMP
PLAN AT SUMPS EL. 588'-8"
SECTION D
N.T.S.
TYP REINF FOR ALL MISSILE WALLS
SECTION G
SEE DWG C-80
TRENCH
SECTION E
N.T.S.
86'-0"
DETAIL 3
DETAIL 4
CAST IRON GUIDES BY OTHERS TYP 4-PLACES
DOWELS (TYP)
CAST IRON GUIDES PURCHASED UNDER SPEC
CIRC. WATER INTAKE STRUCTURE
TRASH RACK GUIDES
PLAN AT ELEV. 592'-0"
T/BOLT EL. 635'-5½"
PUMP BASE PL & SOLE PL
EL. 634'-6"
WILSON SLEEVE OR EQUAL
TACK WELD 3 PLACES
NOTCH MAX. OF 1½" SQ. WHERE INTERFERES W/ VERT. WALL REBAR
SECTION H
EMBEDDED FRAME (TYP)
DETAIL 2
SECTION C
EL. VARIES
SUMP
SECTION F
SECTION J
FOR ARRANGEMENT OF REINF OF PAD SEE SECT. C
SHORING UNDER SLAB SHOULD NOT BE REMOVED UNTIL 18" THICK WALL HAS ATTAINED FULL STRENGTH
SEE PLAN FOR ORIENTATION
DETAIL 1
SEE DWG C-80
EL. 630'-0"
SECTION B
SEE DWG C-80
FOR REINF NOT IDENTIFIED SEE DWG C-97
SECTION A
SEE DWG



GENERAL NOTES

1. SEE DRAWING C-4 FOR ADDITIONAL NOTES.
2. SEE DRAWINGS C-140, C-141, C-144, C-145, C-146 & C-147 FOR STANDARD DETAILS & NOTES UNLESS OTHERWISE NOTED ON DRAWINGS.
3. FOR PIPE SLEEVES SEE MECHANICAL DRAWINGS.
4. ALL CONCRETE SHALL HAVE A COMPRESSIVE STRENGTH OF 4000 PSI, CLASS C-2.
5. FOR LAP SPLICE & DEVELOPMENT LENGTHS OF REINF STEEL SEE TABLE ON DWG C-96 UNLESS OTHERWISE NOTED.
6. ALL EMBEDDED MISC METAL ITEMS SHALL BE GALVANIZED U.N. ON DWGS.
7. NO STIFFENERS ARE REQUIRED FOR CHECKERED PL COVERINGS U.N.
8. ALL SLUICE GATES SHALL BE INSTALLED IN ACCORDANCE W/ MANUFACTURERS INSTRUCTIONS UNDER P.O. 7220-C-84 (EXCEPT USE 1/2 + 5/8 STAINLESS STEEL HEX HEAD BOLTS)
9. ALL SLUICE GATES EXCEPT THE TWO OUTER SLUICE GATES SHALL BE FIELD TESTED FOR LEAKAGE WHEN THE WATER LEVEL IN THE COOLING POND IS AT 627'-0". THE TWO OUTER SLUICE GATES SHALL BE TESTED FOR UNSEATING HEAD ONLY WITH WATER EL 627'-0".
10. ALL STUD BOLTS FOR ATTACHING SLUICE GATE TO THIMBLE SHALL BE REPLACED WITH HEX HEAD BOLT 1 1/2" LONG W/MIN. 1 3/16" THREAD ENGAGEMENT.

REFERENCE DRAWINGS

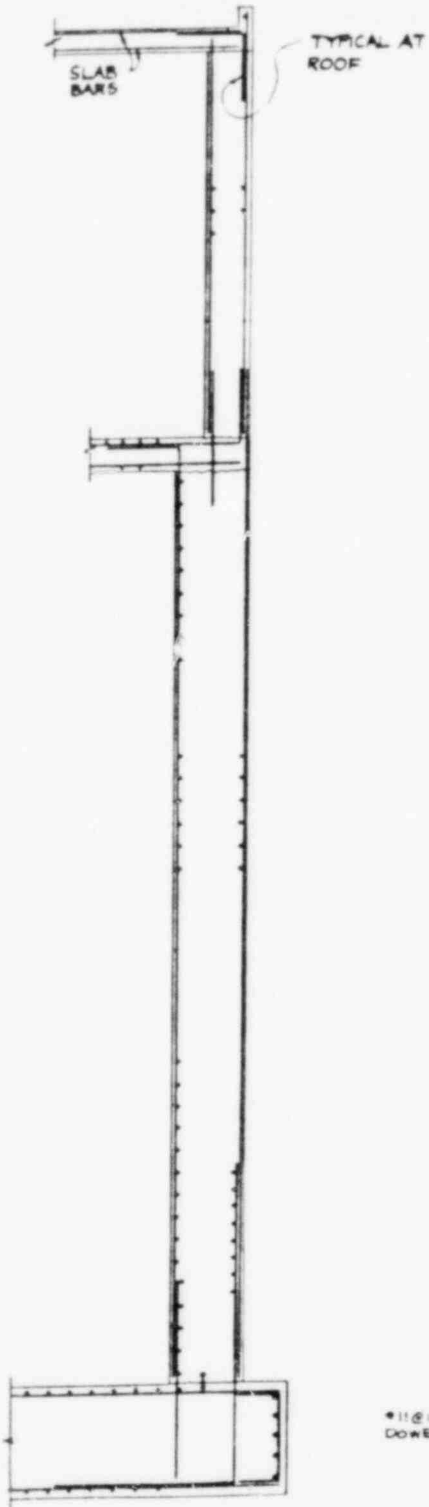
- C-80 REINFORCING SLAB - EL. 634'-6"
- C-86 MISC STEEL & CONC DETAILS - SH 2
- C-94 CONCRETE PLAN AT EL. 592'-0" & EL. 634'-6"
- C-95 CONCRETE PLAN AT EL. 656'-0" & SECTIONS
- C-96 CONCRETE SECTIONS
- C-97 CONCRETE SECTIONS & DETAILS
- C-99 MISC STEEL & CONC DETAILS - SH 1
- M-83 SHT 1 PLAN @ EL. 634'-6"
- M-83 SHT 2 PLANS BELOW 634'-6"
- M-84 SHT 1 SECTIONS
- M-84 SHT 2 SECTIONS
- M-244 FLOOR & ROOF DRAINS

FIGURE 1

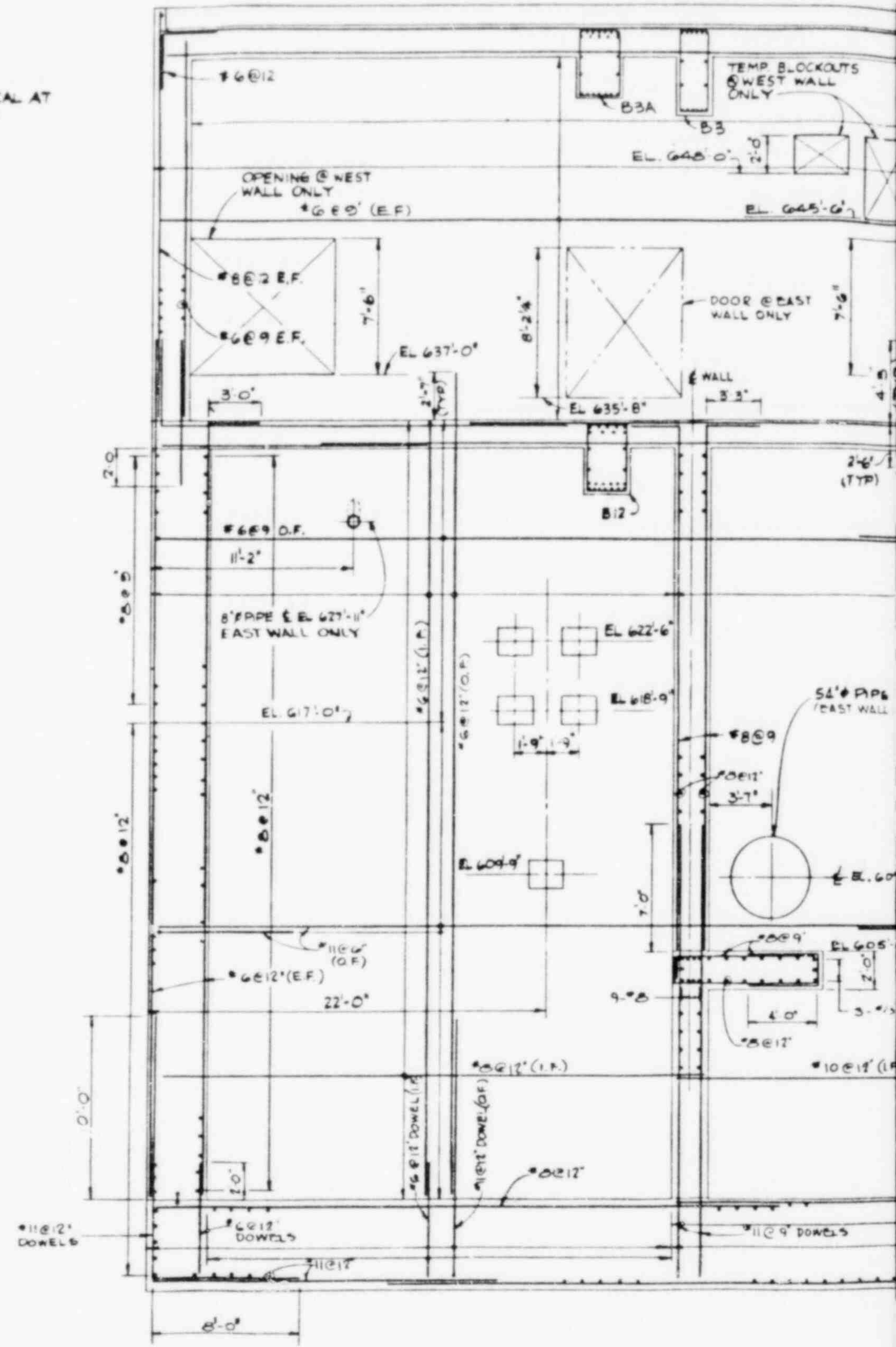
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Service Water Pump Structure
Concrete Floor Plans at El. 592'-0" and El. 634'-6"
(C-94, Rev 8)

FSAR Figure 3.8-56



SECTION A



ELEVATION - WEST WALL
ELEVATION EAST WALL (O

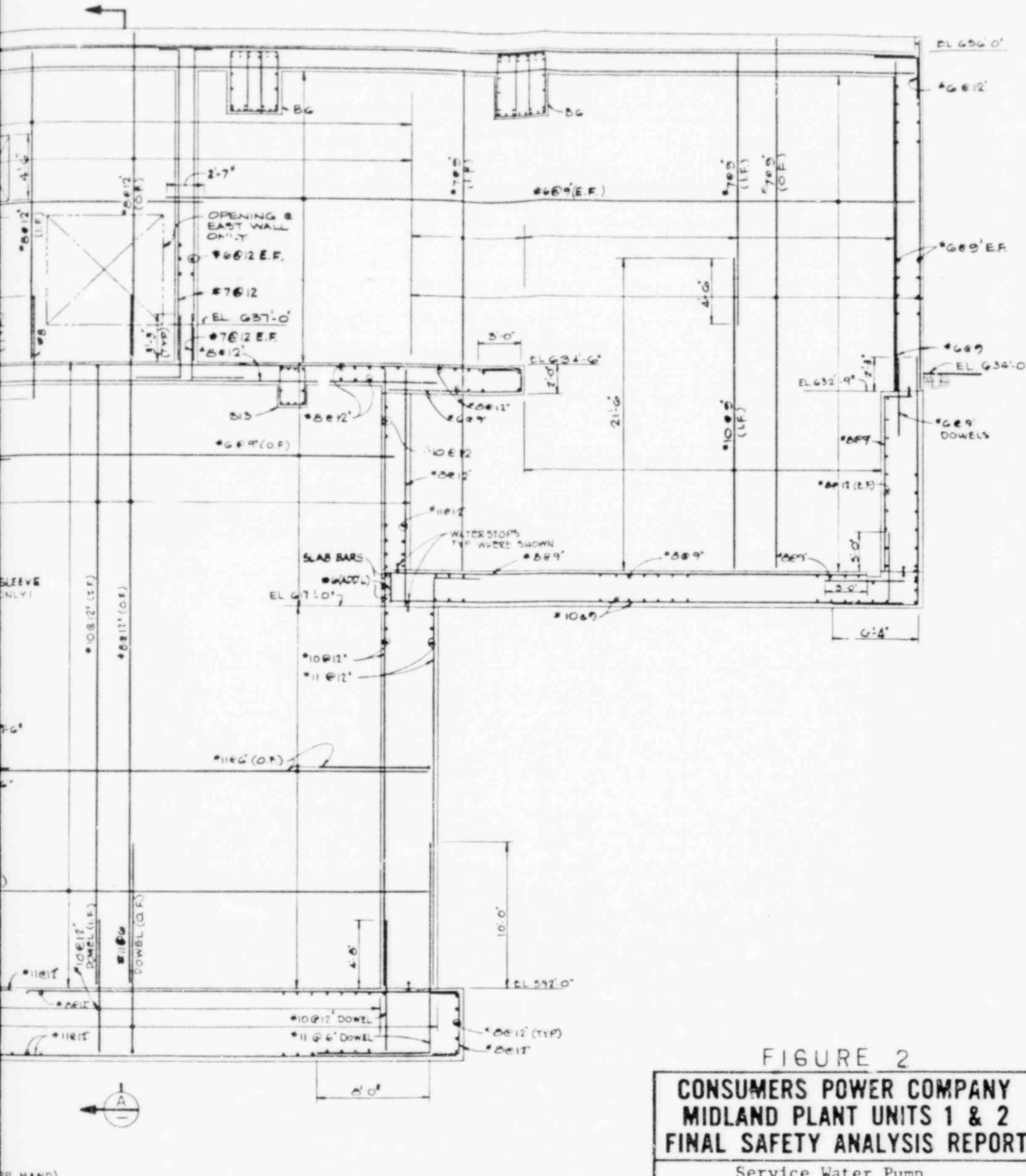


FIGURE 2

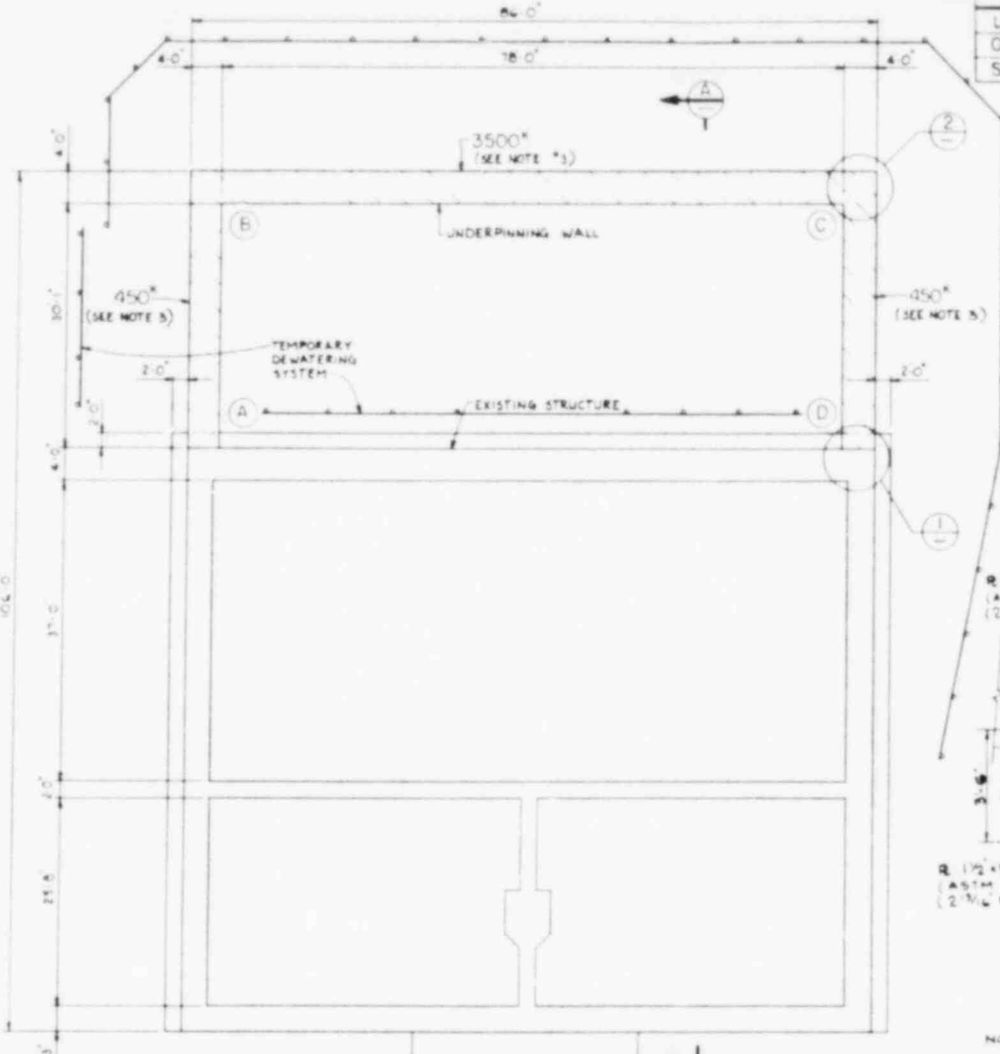
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Service Water Pump
Structure Section

(C-97, Rev 2)

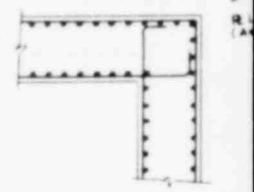
FSAR Figure 3.8-57

SEISMIC LOAD · PRELIM				
LOC.	A	B	C	D
OBE		14 KLF	14 KLF	
SSE		40 KLF	40 KLF	



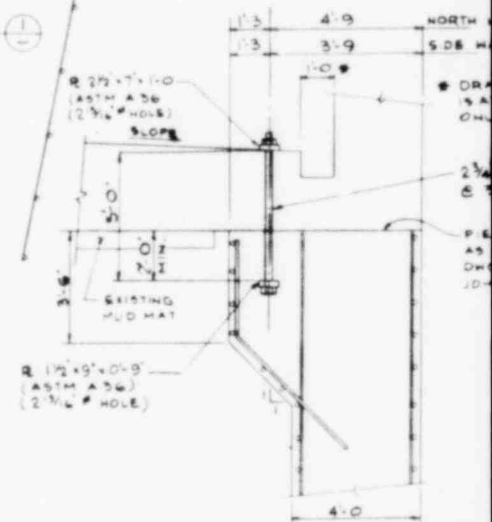
PLAN AT EL 612'-0"

1/8" = 1'-0"



DETAIL (2)

1/8" = 1'-0"

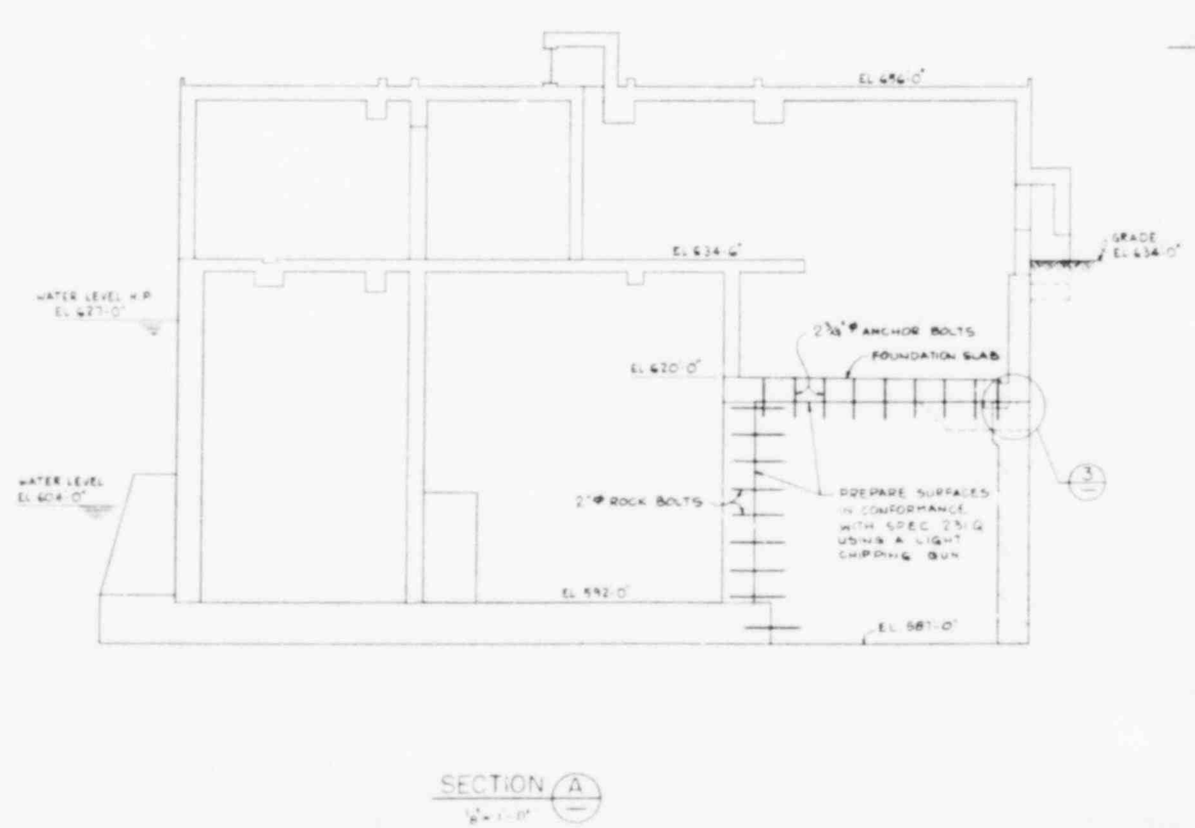


NOTE: ANCHOR BOLTS SHALL BE INSTALLED FOR LENGTH OF THE UNDERPINNING WALL AT A SPACING OF 3'-9. ANCHOR BOLTS SHALL POSITIONED NO CLOSER THAN 1'-3' TO A UNDERPINNING PIER.
ANCHOR BOLTS SHALL MEET THE FOLLOWIN HARDNESS REQUIREMENTS:

	MIN.	MAX
BRINELL	235	311
ROCKWELL C	22	33

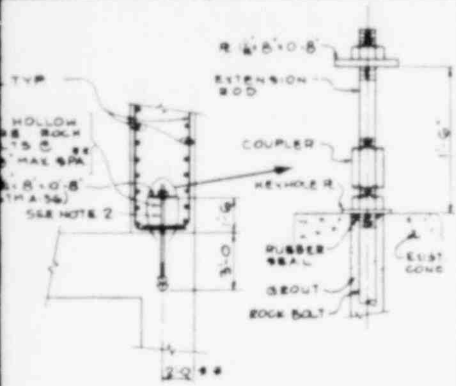
DETAIL (3)

1/2" = 1'-0"



SECTION (A)

1/8" = 1'-0"



DETAIL 1
1/4" = 1'-0"

** SPACING OF ANCHOR AND ROCK BOLTS SHALL BE ADJUSTED TO MISS EXISTING REINFORCEMENT.

...ALL
...ALL
...NAGE TROUGH
... NORTH WALL
...Y

...BOLT (A 354) GR BC
...-9" MAX SPACING **

...R TOP TO BE REINFORCED
... SHOWN IN DETAIL 2 OF
... U 2 BY MUESER-RUTLEDGE
...NSTON & DESIMONE

...HE FULL
... MAXIMUM
... BE
...Y EDGE OF

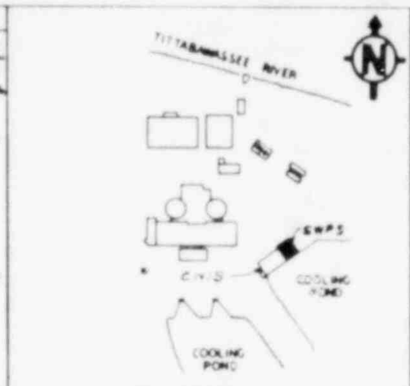
...G

ROCK BOLT INSTALLATION PROCEDURE

1. DRILL HOLE INTO WALL OF SWPS AS SHOWN ON SEC A AND DET. 1 REFER TO ROCK BOLT INSTALLATION PROCEDURE NOT FOR TOLERANCE OF HOLE SIZE AND SPACING. CLEAN ALL DEBRIS FROM HOLES.
2. INSERT ROCK BOLT ASSEMBLY (I.E. NUTS, WASHERS, KEYHOLE PLATE, GROUT AND DE-AIR TUBES) INTO HOLE WITH THE REQUIRED EMBEDMENT AS INDICATED.
3. TORQUE ROCK BOLTS TO 1050 FT-LBS ± 25 FT-LBS.
4. USING A HAND OPERATED JACK APPLY A PROOF LOAD OF 145 KIPS, HOLD FOR 5 MINUTES, THEN RELAX THE LOAD. THE LENGTH OF THE BOLT FROM THE CONCRETE SHALL BE MEASURED AND RECORDED PRIOR TO APPLICATION OF PROOF LOAD, AT MAXIMUM LOAD AND AFTER LOAD IS RELEASED. IF THE LATTER MEASUREMENT DIFFERS FROM THE PREVIOUS MEASUREMENT BY 10% OR MORE, THEN NOTIFY PROJECT ENGINEERING FOR FURTHER DIRECTIONS. IF UNDER 10% USE THE HYDRAULIC JACK TO TENSION THE ROCK BOLT TO 96 KIPS. TIGHTEN ROCK BOLT NUT TO TRANSFER THE LOAD FROM THE JACK TO THE ROCK BOLT.
5. APPLY GROUT (I.E. WILLIAMS WIL-X CEMENT GROUT, EMBECO 713 OR EQUAL) AS PER MANUFACTURERS PROCEDURES UNTIL A CONTINUOUS FLOW OF GROUT IS SEEN COMING OUT OF THE DE-AIR TUBE. ALLOW GROUT TO CURE FOR 48 HOURS.
6. THE ROCK BOLTS SHALL BE HOLLOW CORE "WILLIAMS" FOR 2" Φ ROCK BOLT PART NO. US 16-HC-LCSLF-350 OR EQUAL. KEYHOLE PLATES, NUTS, WASHERS, COUPLINGS AND EXTENSION RODS SHALL BE "WILLIAMS" OR EQUAL.
7. NEW HOLES DRILLED INTO CONCRETE SHALL HAVE A DEPTH TOLERANCE OF +3" UNLESS NOTED. THE ROCK BOLTS SHALL NOT BE SPACED CLOSER THAN 3'-0".
8. HOLES IN CONCRETE FOR 2" Φ ROCK BOLTS SHALL BE 3 1/2" Φ IN ACCORDANCE WITH THE MANUFACTURERS REQUIREMENTS.

ANCHOR BOLT INSTALLATION PROCEDURE

1. DRILL 4" Φ HOLE.
2. PREPARE LEVEL SEAT FOR BASE PLATE WITH NON-SHRINK GROUT (EMBECO 713 OR EQUAL).
3. INSTALL ANCHOR BOLT ASSEMBLY.
4. AFTER COMPLETION OF THE UNDERPINNING WALL AND APPLICATION OF THE FINAL PRELOAD, GROUT AROUND ANCHOR BOLT AND TENSION BOLT 1/2 TURN AFTER SNUG FIT.
5. PAINT PLATES, NUTS AND BOLTS WITH A ZINC RICH PAINT (CARBO ZINC 11 OR APPROVED EQUAL).



KEY PLAN

NOTES

1. ALL CONC AND REINF SHALL CONFORM TO THE REQM'TS OF ACI 318-71. CLASS C/C CONC SHALL HAVE A 28 DAY COMP STR OF 4000 PSI AND REINF STEEL SHALL BE GRADE 60.
2. FINAL CONNECTION OF REINF AND ANCHORS BETN THE EXIST STRUCTURE AND THE UNDERPINNING WALL SHALL NOT BE MADE UNTIL ALL JACKING OPERATIONS ARE COMPLETED.
3. STRUCTURAL LOADS SHALL BE TRANSFERRED TO THE UNDERPINNING WALL BY JACKING. JACKS SHALL BE SPACED UNIFORMLY ON EACH SEGMENT OF THE WALL AT 8'-0" O.C. MAXIMUM.
   THE NUMERICAL VALUES OF JACKING LOADS SHOWN ARE AFTER ALL LOSSES.
4. MINIMUM FACTOR OF SAFETY (F.S.) AGAINST ULTIMATE BEARING CAPACITY SHALL BE:
   (A) F.S. OF 3 FOR COMBINATIONS OF DEAD AND LIVE LOADS.
   (B) F.S. OF 2 FOR COMBINATION OF DEAD, LIVE AND SEISMIC LOADS.
5. SMALLER BUNDLED BARS OF EQUAL CROSS-SECTIONAL AREA MAY BE SUBSTITUTED FOR HORIZONTAL REINF SHOWN.
6. A PORTION OF THE UNDERPINNING WALL SHALL BE CONSTRUCTED TO ACT AS A TEMPORARY SUPPORT OF THE CANTILEVER PART OF THE STRUCTURE. AN INITIAL JACKING LOAD OF 2500 KIPS SHALL BE APPLIED TO THE BASE SLAB FROM THE UNDERPINNING PIERS UNDER THE NORTH WALL. THE LOADS SHALL BE APPLIED EQUALLY ABOUT THE STRUCTURE ℄. THIS WORK MUST BE DONE BEFORE ANY EXCAVATION FOR THE REMAINDER OF THE UNDERPINNING WALL CAN BE STARTED.
7. CONCRETE WILL BE FURNISHED BY BECHTEL.
8. STRESSES UNDER JACKS FROM INITIAL PRELOAD SHALL NOT EXCEED 1250 PSI.

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

SERVICE WATER PUMP STRUCTURE
UNDERPINNING REQUIREMENTS

Figure 3

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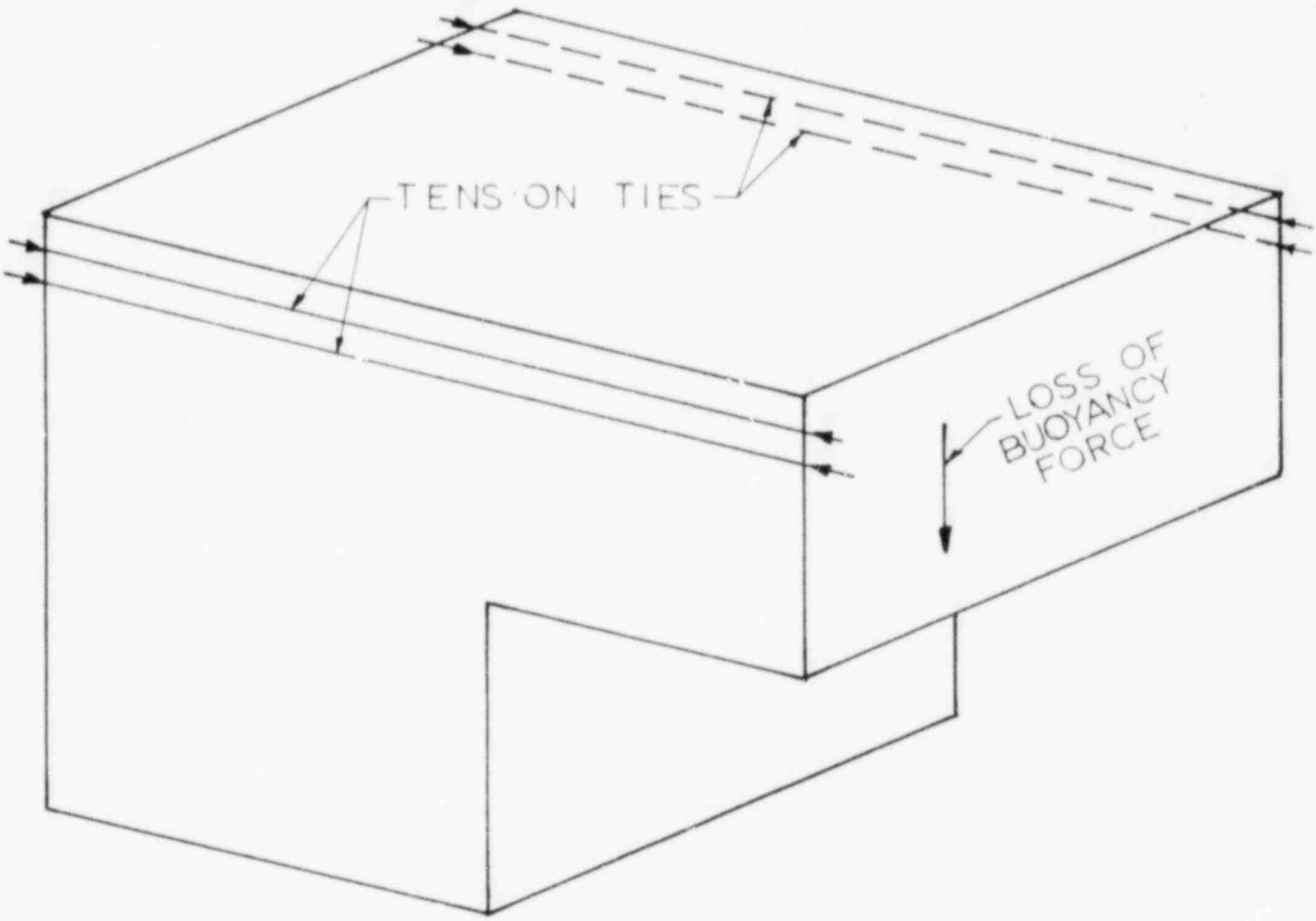

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SERVICE WATER PUMP STRUCTURE
TENSION TIES

Figure 6