UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	Docket Nos. 50-329-OM
CONSUMERS POWER COMPANY	)	50-330-OM
	)	50-329-OL
(Midland Plant, Units 1 and 2)	)	50-330-OL

TESTIMONY OF

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CONCERNING

MIDLAND PLANT SERVICE WATER PUMP STRUCTURE

VOLUME 1 - TEXT

SERVICE WATER PUMP STRUCTURE

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SERVICE WATER PUMP STRUCTURE

1.0 BACKGROUND

1.1 SCOPE OF TESTIMONY

This testimony presents evidence regarding the remedial measures to be undertaken at the north end of the service water pump structure (SWPS) as a result of the detection of certain areas where the fill material which was placed for foundation support under the overhang section at the north end of the building was insufficiently compacted.

1.2 STATUS OF DESIGN EFFORT FOR REMEDIAL MEASURE

The design and analysis procedures and details for the remedial measure for the SWPS are described in detail in Sections 5.0 through 7.0 below. The status of the underpinning design and structural reanalysis is discussed in Section 7.3. The information herein provides an adequate and reasonable basis for assurance that upon completion of the proposed remedial action the SWPS will be fully capable of performing its intended safety function under all postulated conditions.

1.3 FUNCTION AND DESCRIPTION OF BUILDING

The SWPS is a reinforced concrete structure located approximately 500 feet east of the diesel generator building. (See Fig. SWP-1.) The structure contains three water-filled reservoirs and five pumps which together provide cooling water for various components during normal plant operation and which supply several safety-related cooling systems which are required to function during a design basis condition, such as a postulated safe shutdown earthquake (SSE). Because of its safety-related function, the SWPS must maintain its structural integrity during and after a design basis condition. Consequently, the building is required to be designed as a Seismic Category I structure.

The SWPS is rectangular in plan, with upper and lower sections of different plan dimensions. The upper section is 106 feet long and 86 feet wide. The lower section is approximately 72 feet long and 86 feet wide. The upper section thus has an overhang section at the north end which is supported by a separate base slab. The lower section base slab is situated approximately 47 feet below grade level. The upper section base slab is situated approximately 17 feet below grade level. The structure measures about 69 feet in total height from the lower base slab to the roof, with approximately 22 feet of the building extending above grade. (Figs. SWP-2 and SWP-3). The water-filled reservoirs are located in the deeper section of the structure. The south wall of the deeper section abuts the cooling pond.

The two reinforced concrete base slabs supporting the structure are located at elevations 587' and 617'. The lower slab is 5 feet thick and is constructed on undisturbed glacial till. The upper slab is 3 feet thick and is constructed on a triangular wedge of backfill soil with a maximum depth of 30 feet. Both slabs are locally thickened near sumps.

All walls and slabs are of reinforced concrete. Exterior walls are 2 to 4 feet thick and the interior walls vary from 1 foot, 6 inches to 2 feet in thickness. The roof slab is 1 foot, 9 inches thick.

1.4 IDENTIFICATION OF POSSIBLE UNSATISFACTORY FOUNDATION CONDITIONS

As a result of settlement measurements on another building in August 1978 (see G. S. Keeley, prepared testimony following Tr. 1163), the Applicant undertook a subsurface soil investigation in the vicinity of the SWPS utilizing soil borings. On November 7, 1978, the Applicant submitted a 10 C.F.R. § 50.55(e) interim report that disclosed that soil borings had been made in plant fill areas in the vicinity of the SWPS.

1.4.1 Test Borings

Eleven soil borings were taken in the area of the SWPS. Two borings were taken inside the building and nine in the surrounding area. (See Fig. SWP-4.) These borings indicated that some localized areas of the heterogeneous backfill material underneath and adjacent to the overhang section of the structure had not been sufficiently compacted.

1.4.2 Measurement of Building Settlement

The Applicant established a Foundation Data Survey Program (FDSP) to monitor settlement of Seismic Category I buildings at the site in May 1977 in anticipation of a commitment to do so in the Midland Project Final Safety Analysis Report (FSAR).

Pursuant to the FDSP, settlement markers were attached to the four corners of the SWPS by the summer of 1978. (See Fig. SWP-5.) Field personnel have surveyed the elevations of three of these markers bimonthly from about July 1978 to December 1980 and biweekly from January 1981 to the present. The initial reading for the fourth marker occurred in September 1978 but bimonthly resurvey did not commence until November 1979. The accuracy of these measurements is approximately ± 0.005 foot (1/16 inch).

Fig. SWP-6 shows plots of observed settlement against time for the four SWPS settlement markers. Figure SWP-7 shows dewatering activities in the vicinity of the SWPS during this period. As this figure indicates, the maximum movement since initiation of the FDSP occurred at marker SW-1 attached to the northwest corner of the building. This movement is about 1/2 inch.

In order to relate the net 1/4-inch settlement since program initiation to the total settlement experienced since essential building completion, additional measurements of markers other than those in the FDSP have been undertaken. Six construction survey control points were installed at elevation 639.5' a short time after concrete placement. (A complete history of the placing of concrete in the SWPS is shown in Fig. SWP-8.) These construction control points have been resurveyed in an effort to determine total changes in elevation.

The six control points were originally installed using bench mark PBM-1, which is a stable bench mark located outside the dike area, as a reference. The locations of these points are depicted in Fig. SWP-9. Four of these points, numbers 1, 3, 4 and 6, lie on the inside faces of the concrete walls in the same general regions as the four permanent settlement markers which are located on the outside of the building walls. The six construction control points were resurveyed in October 1982, using PBM-1 as a reference control bench mark. The net changes in elevation as of October 1982 for the six construction control points are shown in Fig. SWP-10. The accuracy of these measurements is $\pm$0.01 foot (1/8 inch).

Several important observations can be made from examination of these data. First, the values from the six control points are, as expected, generally larger than the values from the four permanent markers, which were placed 2 to 6 months later than the control points. Second, within the tolerances associated with the readings, the two sets of data correlate well. Third, the north-south differential building settlement is minimal; as determined by the six control points, the maximum differential is 0.02 foot (1/4 inch); the maximum value from the four FDSP settlement markers is .03 foot (approximately 3/8 inch). These measurements indicate that the building is very stable by conventional standards.

1.4.3 <u>Significance of Cracks</u>

For a discussion and evaluation of the significance of cracks in the SWPS, see the testimony of W. Gene Corley.

2.0 CORRECTIVE ACTION

The Applicant chose to undertake a remedial structural measure which would compensate for unsatisfactory compaction of backfill material rather than to attempt to demonstrate satisfactory fill material under the overhang portion of the building. The Applicant selected a remedial measure involving supporting the overhang section with a continuous perimeter underpinning wall founded on undisturbed glacial till*/ soil as the preferred measure for assuring proper foundation support for the SWPS.

3.0 CONCEPTUAL DESCRIPTION OF CONTINUOUS PERIMETER UNDERPINNING

In this concept, a continuous perimeter underpinning wall will be constructed under the north end of the existing structure. The underpinning will consist of three reinforced concrete walls which will be connected to one

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*/ As is more fully described in Sections 8.1 and 8.2, the underlying glacial till is typically located at and below elevation 590'. The underpinning piers will be founded at or below elevation 587' and, therefore, it is anticipated that the foundation material for these piers will be the glacial till. However, it is possible that small pockets of sandy very dense alluvium (see Section 8.1.2) may be encountered at that elevation. As described in Section 4.3, if sandy alluvium material is encountered in shallow layers, Applicant will remove the sand and replace it with concrete. If the layers are determined to be deep, Applicant will remove any disturbed sand, but will otherwise construct the pier footing on undisturbed alluvium. The sandy alluvium, as described in Section 8.1.2, is an acceptable foundation material and, in fact, has strength characteristics equal to or greater than those of the undisturbed glacial till. All references in this testimony which refer to the bearing material as undisturbed glacial till should be read to include the possibility of sandy alluvium.

another and to the main building to form a box structure. The reinforced concrete underpinning wall will exte . from the underside of the upper foundation slab to undisturbed glacial till (approximately elevation 587'). The underpinning walls will be 4 feet thick. Because the largest fraction of the applied load is on the north wall, the base of the underpinning wall under the north wall will be enlarged to 6 feet to maintain bearing pressures within allowable limits.

When the underpinning structure is complete, predetermined jacking forces will be applied at :he interfaces between the overhang perimeter and the underpinning wall to provide for load transfer from the structure to the underpinning and thence to the undisturbed glacial till. As soon as predetermined settlement criteria are met, the spaces between the tops of the walls and the bottom of the base slab will be firmly wedged and grouted, and the walls tied to the overhang structure and to the deeper section of the existing building. (See Section 4.4 below.) The completed underpinning wall structure will provide a structural foundation resting on undisturbed glacial till. (See Figs. SWP-11, 12, and 13).

4.0 CONSTRUCTION OF THE UNDERPINNING WALL

4.1 HISTORY AND APPLICATION OF UNDERPINNING CONCEPT

The general rationale for and procedures used in the technique of underpinning are set forth in Section 4.1 of the prepared testimony of Burke, Corley, Gould, Johnson, and Sozen on the Auxiliary Building (following Tr. 5509) and will not be repeated here.

4.2 CONSTRUCTION SUPPORT ACTIVITIES

4.2.1 Post-Tensioning Ties

As a preventive measure against possible building distress due to loss of buoyancy during construction, post-tensioning ties were installed along the tops of the east and west exterior walls of the SWPS in November 1981 (Fig. SWP-14). These ties, which consist of two tendon groups on each side of the building, apply a compressive force of about 500 kips to the upper portion of each east and west exterior wall. This force is intended to compensate for additional loading on the overhang section resulting from the loss of buoyancy which will be caused by the temporary dewatering required to construct the underpinning.

4.2.2 Dewatering

An important consideration in constructing the underpinning wall is the necessity for dewatering the underpinning construction area. To lower the water table temporarily within the construction area, construction dewatering wells will be installed in the immediate area of the SWPS. The plan for water level control and monitoring fines will be as described on p. 2-51 of Supplement No. 2 to the Midland Safety Evaluation Report (NUREG 0793, September 1982).

4.2.3. Access Cofferdam

An access cofferdam will be constructed adjacent to the northwest corner, the north wall and the northerly 34 feet of the east side of the SWPS. The access cofferdam will be constructed to provide access for workers and equipment for the underpinning work.

The cofferdam will be excavated in two stages for underpinning access purposes. Initially, it will be excavated adjacent to the building, to elevation 618' to permit installation through approach pits of the initial underpinning piers Numbers 1a and 2a on the west side and Numbers 1 and 2 on the east side beneath the SWPS base mat. When this initial underpinning is completed, the access cofferdam will be lowered locally at the northwest corner to elevation 609' to provide access for excavation of a tunnel beneath the west wall of the SWPS.*/ The underpinning piers along the remainder of the structure will be constructed from Elevation 618' by means of approach pits.

The cofferdams typically will be constructed using standard methods. First, auger holes about 2 feet in diameter will be excavated down to elevation 600'. The top 6 to 8 feet of each hole will be lined with a steel casing. The holes will then be filled with a slurry mixture, and steel beams, called soldier piles, will be inserted into the hole. Concrete will then be tremied into the hole to displace the slurry mixture. In some locations on the north side, where congested areas of underground utilities are encountered, hand-excavated and lagged piers will be constructed, a steel

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*/ A tunnel must be used along the west wall because of the location of the circulating water intake structure. The details of the tunnel construction are shown in Fig. SWP-12.

soldier beam inserted, and the pier filled with concrete. The distance between soldier piles will be about 5 to 7 feet around the access cofferdam perimeter. As excavation progresses downward, flat tubular steel lagging will be installed. The trimming of soil, trimming of the lean concrete in the auger holes, placement of lagging, and backpacking behind them with soil will be done by manual labor. At predetermined intervals, horizontal beams, called wales, will be installed to support the soldier piles. Support for the adjacent earth around the perimeter is provided in this manner at the same pace as the excavation in the cofferdam progresses downward. The excavation progress will be coordinated with the groundwater removal so that the measured groundwater levels will always be at least 2 feet below the permitted excavation level, including that encountered in the hand-dug pits.

4.3 INITIAL UNDERPINNING

Wall construction will begin from the access cofferdam at the northeast and northwest corners of the structure. Working from the cofferdam, construction of three piers at each corner and five piers at the center of the wall will be performed. The piers will be constructed in the sequence indicated in Plan Section A of Fig. SWP-11.

A typical pier is 5 feet long, 4 feet wide, and 30 feet deep. The piers along the north wall will be belled to six feet at the bottom. The pier areas will be excavated to undisturbed bearing material. The full depth of the pit excavation will be supported by steel tubular lagging which forms the sides of the pier (see detail in Fig. SWP-12).

It is anticipated that glacial till will be encountered at final subgrade. However, pockets of alluvial sand may be encountered. In such cases, it is anticipated that the sand will be removed down to undisturbed glacial till, provided that the excavation of the sand will be relatively shallow. The shallow pockets will be filled with concrete. If deep alluvial sand is encountered (deeper than 18 inches), it will be accepted for the foundation footing, provided it is undisturbed and of the quality described in Section 8.2 below. The onsite geotechnical engineer will determine when suitable foundation material has been reached using a combination of visual inspection, probing and penetrometer readings. (See Section 8.1.3.). After the foundation material for the pier has been approved, a lean concrete mat will be placed. Then reinforcing steel bar with couplers will be placed, pier instrumentation (see Section 4.6) will be installed, and concrete for the pier will be poured. The concrete will be cured to 2000 psi minimum strength before the initial jacking load is applied.

The principal consideration in the first stage of the construction is to provide initial support to the north end of the building. To compensate for the possible loss of support under the base slab caused by the underpinning operations and to further counteract loss of buoyancy, the underpinning construction procedure requires jacking an initial load into each pier.

The initial jacking load will be maintained as follows:

i) The pier will be loaded to 125% of the specified initial jacking load and continued at that load until the relative movement between the top of the pier and the underpinned structure is less than 0.01 inch for a continuous one hour period. When this condition is satisfied;

ii) The pier load will be reduced to 110% of the specified initial jacking load and continued at that load until the relative movement between the top of the pier and the underpinned structure is less than 0.01 inch in a continuous 24 hour period. When this condition is satisfied, the jacking load will be reduced to 100% of the specified jacking load.

For pier 1, however, a load test to 130% of the load corresponding to the maximum allowable bearing pressure will be conducted prior to reducing to the specified jacking load for this pier. In addition, if any piers are to be founded on alluvial sand, then the first pier to be so constructed will also be load tested. After jacking a pier, the underpinning work will be advanced to the next pier to be constructed.

Each group of three corner piers (Piers 1, 2 and 3, on the east and 1A, 2A and 3A on the west) has been assigned a total of 465 kips of load. However, the piers and underlying glacial till have the capacity to support at least 2,000 kips at each corner, with a factor of safety of 2.0, should additional unplanned load be transferred to these piers. Based on calculations, it has been determined that the north structural wall above the underpinning is sufficient to transfer these loads safely to the end groups of piers.

After the corner piers are completed, the remaining piers which receive an initial jacking load are constructed in the sequence shown on Fig. SWP-11.

The initial jacking load for each pier has been calculated so that most of the initial load will be distributed evenly along the north wall. The remaining load will be distributed to piers 3 and 9 on the east and piers 3A, 8 and 9A on the west wall. The loads will be monitored and adjusted for any shift of load caused by pier settlement. During the period that the initial jacking load is maintained on the piers, frequent checking of jacking load will be performed, and the wedges will be periodically retightened. In effect, the tight wedging will be a safeguard for the structure's support should a jack or its hydraulic line fail while loaded. In addition, screw collars which can be tightened on each jack after stressing will be installed as an added safety factor against sudden downward jack movement should the jack pressure lower inadvertently.

4.4 FINAL UNDERPINNING

After Piers 1 through 9A (refer to Plan Section A of Fig. SWP-11) have been initially jacked, the final jacking forces as specified in the jacking load table (Fig. SWP-11) are applied. The final jacking loads will be applied by simultaneously jacking all piers and sustaining this load long enough to assure that pier deflections are occurring at a satisfactory rate. (See Section 8.3 for a description of the criteria for acceptance of final jacking.)

After the above acceptance criteria have been met, the wedges will be driven tight and welded in place (See Detail 2 of Fig. SWP-12). The jacks will then be removed and the space between the top of the piers and the underside of the SWPS base slab will be closed with concrete and grout.

While the final load is being maintained in the jacks, the two pier areas at the interface with the main portion of the SWPS (Piers 10 and 10A), as well as Piers 11 and 11A, can be excavated. Reinforcing bar dowels will be drilled and grouted into the vertical face of the lower portion of the existing structure. (See Detail 5 of Fig. SWP-12.) After the final jacking is completed, the casting of Piers 10 and 10A will encase the dowels, thereby tying the vertical face of the underpinning wall to the existing structure. At the same time, Pier 11 and the portion of Pier 11A up to the bottom of the tunnel can be placed.

To tie the top of the underpinning wall to the existing structure, anchor bolts embedded in the concrete of the piers will extend through holes drilled through the upper foundation slab of the SWPS.

When pier interfaces are completed, the access tunnel at the west side will be filled with lean concrete. (Because the north end of the structure will be entirely supported by the underpinning, a nonstructural concrete can be used.) The tunnel will be filled and an exit made at Pier 11A. The remainder of Pier 11A is then cast, and its jacking load applied.

The top of the underpinning wall will then be fastened to the existing structure by tightening and grouting the previously placed anchor bolts (See Fig. SWP-13). Because concreting of Piers 10 and 10A and tensioning of the anchor bolts do not occur until after the final jacking load is locked off, these connectors do not transfer dead loads or jacking loads at the interfaces at the time of lockoff. At this stage, the underpinning of the SWPS is complete.

4.5 WALL CONTINUITY

The underpinning wall is constructed in pier segments and the piers are jacked at different times. To obtain wall continuity, the piers must be tied together with continuous reinforcing steel and shear keys. Splicing the reinforcing steel with reinforcing bar couplers placed at the interface of adjoining piers will provide continuity in the reinforcing steel. (See Detail 1 of Fig. SWP-12.)

Shear keys lock the concrete of adjoining piers together to enable the piers to act as a structural unit. The keys are created by forming a void area at the face of the first of the two piers constructed. This void is filled by the concrete cast in the second pier. As a result of the use of shear keys and coupled reinforcement, the piers together form a continuous wall which will resist lateral and vertical forces in the same way that a continuously constructed wall resists those forces.

4.6 INSTRUMENTATION OF UNDERPINNING PIERS

During underpinning installation, each pier will be instrumented to monitor deflection of the pier tops and bottoms. Pier top movement is monitored by an extensometer dial gage with readings taken between the underside of the foundation slab and the pier top. Monitoring will begin the day after pier concrete is placed and will include measurements during and after initial jacking.

Pier bottom movement is monitored by devices called telltales to help differentiate between the deflection in the underlying soil and deflection of the top of the pier due to shrinkage and creep. The telltales consist of the following instrumentation. A steel plate with an attached rod will be placed at the base of the pier (see Section D of Fig. SWP-12). The rod will be enclosed in a small diameter pipe sleeve to allow free movement. The rod and sleeve, which extend to the top of the pier, will be put in place before the pier concrete is poured. The top of the rod is connected to an extensometer dial gage which also indicates movement relative to the base slab. Rod movements will be recorded simultaneously with monitoring of the pier top.

These instruments produce measurements relative to the position of the base slab. Absolute pier top and bottom movement values can be obtained by adding the measurements of movement, if any, of the base slab obtained from the deep bench mark monitoring.

Carlson-type stress gages will be embedded in the concrete of the three piers in each corner. These gages will be monitored during installation of the corner piers and the remaining north wall piers. This information, in addition to other monitoring instrumentation, will be used to adjust the jacking loads.

4.7 BUILDING MONITORING DURING CONSTRUCTION

4.7.1 Existing Building Vertical Movement Monitoring

For the past several years, level readings have been used to monitor settlement of the SWPS at four locations. (See Section 1.4.2). In addition, the Applicant has established additional settlement reading points at the mid-span of the north side of the building and on the east and west side at the mid-point of the deeper portion of the structure and at the location of the vertical interface with the main part of the structure. These points will be observed by means of optical level runs before and after significant events which may affect the settlement of the structure and, during the underpinning operation, on at least a weekly basis. Readings are made with an accuracy of approximately ±0.005 foot.

In addition, a minimum of four deep bench marks will be installed to monitor SWPS movement during underpinning. Two will be installed at the north corners, one along the east wall near the depth change and one at the southeast corner. These deep bench marks will extend to a depth of at least 150 feet below grade level (elevation 634') and will be grouted into the undisturbed glacial till. A steel bracket arrangement will be attached to the SWPS to position a dial gauge or linear variable differential transducer (LVDT) above the two deep bench marks.

For a period prior to underpinning work, the instruments will be read daily or more frequently to provide a base for subsequent readings. After the underpinning contractor begins his work, the instruments will be read daily or more frequently to monitor effects of construction on the building displacement. By plotting the measurements made along a north-south line of bench marks, the structural deflection of the building will be monitored, as well as the rotation of the SWPS during underpinning.

4.7.2 Strain Monitoring

Extensometers will be placed at two locations on the exterior east and west walls straddling the junction of the overhang and the main portion of the structure to monitor strain during the underpinning operation. Four 5' long extensometers, which will span a minimum of 20 feet, will be installed near the roof line of each wall and four 5' long extensometers will be located on each wall at Elev. 628'-0". Each 5' long extensometer will measure the strain primarily with a LVDT and secondarily with dial gages. Reading frequency with be as described in Section 4.7.1.

4.7.3 Building Crack Monitoring

During various stages of the underpinning work, accessible areas in the critical locations in the overhanging portions of the SWPS resting on fill will be monitored for cracks. The critical locations are the exterior east and west walls of the overhang and the roof slabs at the junction of the overhand portion of the building with the deeper portion of the building.

4.7.4 Acceptance Criteria

The acceptance criteria for relative displacement, strain monitoring and crack monitoring, as discussed earlier, are as follows:

		Alert Level	Action Level
(1)	Relative displacement [excluding rigid body motion] in the north-south direction.	50 mils	70 mils
(2)	Strain monitoring.	.001 in/in	.002 in/in
(3)	Crack mapping.	New cracks exceeding 10 mils or any crack exceeding 30 mils	Any crack reaching 60 mils.

The terms "alert level" and "action level" are defined as follows:

Alert Level:

All values up to the alert level are considered to be within normal working ranges.

Settlement, strain and cracking information will be reviewed by the resident structural engineer daily. In general, for readings below the alert level, attention should be focused on the value of the readings versus the construction progress and any indication of trends that would indicate the alert level will be exceeded.

Once the alert level is exceeded, the site resident structural engineer must inform engineering in Ann Arbor of the situation. All available data should be evaluated in total. Where trends exist that indicate the action level is likely to be reached, plans will be evaluated to remedy the situation. (Note: it is recognized that the evaluation may conclude that no changes are warranted.)

Action Level:

Values in excess of the action level must be reviewed by the resident structural engineer and by engineering in Ann Arbor.

Plans would be initiated to modify the condition that caused the reading to exceed the action level. Consumers Power Company must be informed of the revised plan so that the NRC can be advised of the situation. (Note: it is recognized that the evaluation may conclude that no changes are warranted.)

5.0 ACCEPTANCE CRITERIA

5.1 UNDERPINNED STRUCTURE

As discussed in Section 1.3, the SWPS is designated as a Seismic Category I structure. As such, the underpinned structure is analyzed in accordance with the design criteria and applicable loads and load combinations described in FSAR Section 3.8.6.3, Rev. 44. This includes checking for stability against overturning, sliding, and flotation. The operating basis earthquake (OBE) and safe shutdown earthquake (SSE) loads for the existing structure are specified in Section 3.7 of the FSAR.

The existing structure is also analyzed for various conditions corresponding to the construction of the underpinning wall.

The underpinned structure is required to satisfy the stability criteria as specified in the FSAR. The existing portion of the underpinned structure must satisfy the allowable stresses specified in the ACI 318 Code and AISC codes, as specified in the FSAR.

5.2 UNDERPINNING WALLS AND CONNECTORS

The design of the underpinning walls and connectors complies with the design criteria, loads, and load combinations specified in the ACI 349 Code as amended by NRC Regulatory Guide 1.142. The underpinning and connectors are designed to withstand the effects of the site-specific response spectra (SSRS) ground motion. The structural forces resulting from the FSAR SSE ground motion were multiplied by a factor of 1.5 for the design of the underpinning and connectors. The response from 1.5 times the FSAR SSE envelops the final SSRS response.

5.3 MATERIAL PROPERTIES

The underpinning wall will be constructed of reinforced concrete. The wall is constructed in segments and assembled into a continuous wall by utilizing standard Fox-Howlett couplers to splice the reinforcement where necessary. The walls are attached to the existing structure with grouted-in reinforcing bars and anchor bolts. The specified minimum strength of the materials used in the underpinned structure is as follows:

Concrete (existing structure)	f'c = 4,000 psi
Concrete (underpinning walls)	f'c = 6,000 psi
Reinforcing bars (ASTM A 615)	fy = 60,000 psi
Anchor bolts (ASTM 540)	fy = 120,000 psi

f'c is the compressive strength of concrete. fy is the minimum yield strength of steel.

Fox-Howlett couplers comply with requirements of ASTM 576.

6.0 DESIGN OF THE UNDERPINNING WALL

The underpinning wall is designed for the loads and load combinations discussed in Section 5.2. Included in these loads is the jacking load -- a load which is not unusual in underpinning work.

6.1 JACKING LOAD

The jacking load fulfills two objectives:

1. The initial jacking load supports the overhang portion of the existing structure during construction of the underpinning.
2. In combination with the anchor bolts, the final jacking load provides a means of adequately transferring loads from the existing structure through the underpinning to a suitable bearing stratum following completion of construction.

6.1.1 Initial Jacking Load

The construction of the underpinning wall requires a limited amount of excavation under the structure. To compensate for the possible loss of support, an initial level of jacking force is inserted between the top of each underpinning pier and the SWPS. A total initial stage jacking load of 3,130 kips is applied with 2,500 kips distributed to the north underpinning wall and the remainder to the side walls. The value of 3,130 kips compensates for the possible loss of support during pier and tunnel construction.

6.1.2 Final Jacking Load

In addition to providing the load transfer capability of the underpinning system, the final jacking load of 4,600 kips reduces the loading on the lower base mat. The final jacking load, combined with the building dead and live loads, produces a more uniform distribution of soil pressure, thereby improving the overall capability of the existing structure.

6.2 BEARING PRESSURES

The thickness of the underpinning wall is set at 4 feet, with the base of the north wall enlarged to 6 feet. Using the results of the finite-element analysis (explained in Section 7.2), bearing pressures at the underpinning walls and at the lower base slab are calculated and are shown in FSAR Table 2.5-14.

6.3 INTERFACE CONNECTORS

After the final jacking load is locked off, the underpinning wall is attached to the existing structure. At the vertical interface, #9 reinforcing bars are grouted into the existing structure and embedded in the underpinning wall. The attachment at the horizontal interface is made with 2 3/4-inch diameter anchor bolt assemblies, which connect the foundation slab of the existing structure to the underpinning.

ACI 349 load combinations, including the increased seismic forces referred to in Section 5.2, are used to obtain the shear and axial forces that the connectors must transmit. To more adequately transmit these forces, the contact surfaces of the existing structure are to be roughened in compliance with the ACI 349 code. For details of the described connections see Figures SWP-12 and 13.

7.0 STRUCTURAL EVALUATION

An evaluation is made to verify that the existing structure, underpinning walls and connectors, and the total underpinned structure meet the acceptance criteria explained in Section 5.0.

7.1 ANALYTICAL PROCEDURE

There are three steps in a structural evaluation. The first is to determine external loads that represent actual loads to which the structure may be subjected during its service life. The second stage involves calculating the magnitude and distribution of internal forces and displacements within the structure caused by application of the external loads. In the third stage, forces are converted to stresses and compared to allowables or directly compared to section capacities.

7.1.1 Determination of External Loads

The external loads are briefly addressed in this section. They consist of many common types of loads, such as dead, live, wind, seismic, and some special loads from the effects of jacking forces and differential settlement.

7.1.1.1 Dead Loads

Dead loads are determined from the self-weight of the structure, the weight of permanent equipment, and hydrostatic pressures.

7.1.1.2 Live Loads

Design live loads consider probable load variations during the normal function of the building and are applied to the floor and roof slabs. Lateral soil pressures on the walls are also included in the live load category.

7.1.1.3 Wind and Tornado Loads

Wind and tornado loads are determined from the external velocity pressure which varies as a function of the wind velocity and the shape of the building. The effects of tornado missiles are also included.

7.1.1.4 Buoyant Load

The buoyant load is determined from the volume of a submerged portion of the building during various conditions including the probable maximum flood.

7.1.1.5 Seismic Loads

The seismic accelerations are determined using a lumped-mass model with the response spectrum modal superposition technique. The computed seismic response accelerations are multiplied by the structural element masses to provide the seismic forces for the structural analysis.

Further information on the seismic analysis criteria, such as ground response spectra, damping values, etc., is contained in FSAR Section 3.7. The details of the dynamic model for the seismic analysis and associated soil-structure interaction formulation were addressed by Dr. R. P. Kennedy in previous testimony.

The resistance of the underpinned SWPS to earthquake loads associated with the SSRS will be evaluated in the seismic margin evaluation program. As stated in Section 5.2, the underpinning walls and connectors are designed to withstand the effects of the SSRS.

7.1.1.6 Thermal Effects

Thermal effects result from the existence of thermal gradients within the walls and slabs.

7.1.1.7 Jacking Preload

Before attaching underpinning to the existing structure, jacking loads are treated as external loads. After the underpinning is attached to the building, jacking preload effects consist of internal forces, moments, and deformations retained in the structure.

7.1.1.8 Settlement Effect

The long-term differential settlement effect is included in the analysis for sustained loads, using appropriate soil springs to reflect settlements predicted in Section 8.4 of this testimony.

7.1.1.9 Construction Loads

Construction loads are temporary and are applied to the existing structure during construction of the underpinning walls. They include post-tensioning loads, dewatering effects, reactions from struts placed against the structure during excavation, and jacking loads during various stages of underpinning wall construction.

7.1.2 Internal Force Distribution

Internal force magnitude and distribution and structural displacements are determined by solving a series of force-displacement equations. The three-dimensional, finite-element model representing the elastic behavior of the SWPS under load serves as the basis for the equations. The finite-element model is explained in Section 7.2. Further details of the analytical procedure are set forth in Appendix B of the previously submitted testimony of Burke, Corley, Gould, Johnson, and Sozen for the Midland plant auxiliary building.

7.1.3 Comparison to Allowable Stresses

Calculated stresses are compared to allowable stresses by selecting locations subjected to the highest internal forces and moments. Two options are generally used to verify adequacy:

1. Forces and moments are converted to stresses and compared to allowable stresses.
2. Forces and moments are compared to section capacities.

7.2 METHOD OF ANALYSIS

The underpinned structure is analyzed by the finite-element method using the Bechtel Structural Analysis Program (BSAP). The existing structure and underpinning wall are modeled by a set of finite elements. The model consists of approximately 1,200 elements. Plate elements representing the floors and walls of the structure form the largest group of elements. Plate elements are also used at the interfaces of the underpinning walls and existing structure to simulate the connection detail. Beam elements are used to represent beams and columns. Soil support to the structure is represented by boundary elements that act as springs. Different spring values are used to simulate different settlement and loading conditions.

Two model configurations are used. The first model is the disconnected model in which the underpinning wall is not connected to the structure. This model is used to analyze the construction conditions. The second model is the connected model in which the underpinning wall is connected to the structure. The second model is used to analyze all other conditions.

7.3 RESULTS

Approximately 95% of the principal structural elements of the SWPS are slabs and walls. The remaining 5% are beams and columns.

The evaluation of all slabs and walls of the SWPS has been completed. The existing structure, underpinning walls, and connectors meet the design criteria contained in the FSAR and described in Section 5 of this testimony. Typical results are given in Table SWP-1.

The beams and columns of the SWPS are being evaluated, and the results will be available for NRC Staff review by December, 1982.

In the Applicant's response to NRC Question No. 26 regarding plant fill, the Applicant committed to check the existing structure for the load combinations contained in ACI 349, as modified by Regulatory Guide 1.142. This check indicates that the existing structure generally meets ACI 349 with the exceptions of some localized areas where minor over-stresses occur. In our judgment, this condition does not affect the structural integrity of the building.

8.0 GEOTECHNICAL CONSIDERATIONS

8.1 CHARACTERISTICS OF THE UNDERLYING SOILS

The original site investigation and subsequent borings near the SWPS disclosed the presence of hard glacial till throughout the immediate vicinity. Table SWP-2 lists the successive boring programs pertinent to the SWPS, the dates the borings were made and the type of technical information developed from the boring and sampling.

In 1981 an investigation by Woodward-Clyde Consultants (WCC) added information on subsoil conditions (See Reference 1). These WCC borings included numbers COE-14, COE-15A, COE-16, and COE-16A. Borings Nos. COE-16 and 16A, made at the northeast corner of the SWPS, provided detailed properties of the glacial till.

A plan of the immediate area of the SWPS underpinning with locations of borings relevant to the underpinning design is shown in the upper panel of Fig. SWP-15. In the lower panel of Fig. SWP-15 is a geological section developed through the U-shaped underpinning wall with the wall unfolded as if it were being viewed in a single vertical plane. The borings are plotted at positions projected at right angles to the line of the underpinning wall. Standard sampler penetration resistance is shown at the borings where these values were obtained.

The borings near the planned underpinning wall revealed three general subsoil strata which are described in the following paragraphs in order of depth from the ground surface. The properties of these three strata are summarized in Table SWP-3.

8.1.1 Stratum F, Fill

This stratum consists of clay with lesser amounts of sand, extending from present ground surface typically to a depth of 34 feet, or from elevation 634' to 600'. The tests on WCC boring samples demonstrate that the fill is chiefly a clay soil of moderate plasticity with 65 percent passing the 200 sieve size. Median shear strength of the clay from nine undrained shear tests on clay in the WCC borings is 1.5 ksf. Beneath the overhang structure, sampler penetration resistance has a median value of 17 blows per foot in the clay and 16 blows per foot in sand fill at the southwest of the overhang. There are a number of thin layers of concrete.

8.1.2 Stratum A, Alluvium

This stratum consists of very dense sand mixed and interlensed with lesser amounts of silt and clay, extending typically to elevation 590'. In some locations it is in pockets within the upper portion of the glacial till. It is chi·fly classified as "SM, silty sand" with some amount of smal. gravel, with 28 percent passing the 200 sieve size. Standard sampler penetration resistance is medium to very high, generally between 40 and 120 blows per foot with a median of 90 blows. Test values of undrained shear strength from three CIU triaxial tests average 25 ksf under chamber pressures of about 2 ksf. Drained friction angles average 41 degrees. These exceptionally high strength and sampler penetration resistance values indicate that, after deposition by water action, the alluvium was overridden by the waning continental ice sheet and is therefore preconsolidated.

8.1.3 Stratum T, Undisturbed Glacial Till

This stratum, which consists of extremely compact sandy clay till, was encountered typically below elevation 590' down to the maximum depth explored in the borings. The presence of undisturbed glacial till will be determined in the field by the resident geotechnical engineer, utilizing the Waterways Experiment Station CN-973 penetrometer device.

Continuous sampling at the WCC borings indicates that the till is remarkably consistent for the full depth of those borings. Detailed testing at Boring Nos. COE-16 and COE-16A yielded the following average properties: 57 percent passing the 200 sieve size; liquid limit of 17; plastic limit of 11; natural water content of 9 percent. Standard sampler penetration resistance ranges typically from 50 to 120 blows per foot with a median value of 75 blows. Ten undrained triaxial tests performed on the WCC boring samples yielded a median undrained shear strength of 18 ksf. The preconsolidation stress evidenced in several WCC consolidation tests is at least 48 tons per square foot. For the purpose of settlement analysis, modulus of elasticity (E) of this extremely compact sandy clay till was assessed based on the following conventional correlation: E equals 500 times the undrained shear strength. Thus E must be at least 6,000 to 9,000 ksf.

The glacial till found at the SWPS location would have been deposited in the original advance of the continental ice by being pressed directly on an underlying resistant surface by the thrust of the ice sheet. It is one of the hardest and most stable glacial soils encountered in the northern and eastern United States. For example, its test properties are superior to the glacial till "hardpan" of New York City which serves as a supporting stratum for many of the largest buildings in the country and which is assigned a nominal allowable bearing capacity of 12 tons per square foot.

8.2 BEARING CAPACITY OF UNDERPINNING PIERS

The SWPS underpinning piers will be founded at or below elevation 587' on undisturbed glacial till. Ultimate bearing capacity is that value of unit loading on a foundation which will cause shear failure in the supporting soil, leading to continuous downward movement. The safety factor against such a failure equals the ultimate bearing capacity divided by the prescribed combinations of applied loading. The bearing capacity commitment in FSAR Subsection 2.5.4.10.1 for the foundation design requires a safety factor of at least 3 against dead load plus sustained live load and a safety factor of at least 2 for these loads plus the seismic load. In engineering practice these values represent a conservative selection.

For purposes of computing the ultimate bearing capacity for the SWPS underpinning it is appropriate to multiply the till's undrained shear strength by a "bearing capacity factor." Based on the testing conducted at the SWPS location as well as a review of other relevant soil boring information contained in Fig. SWP-15 and Table SWP-3, an undrained shear strength value of 8 ksf has been conservatively selected for the glacial till. The shear strength properties of the alluvial material are even more favorable than those determined for glacial till.

The bearing capacity factor is a parameter which relates cohesive shear strength and ultimate bearing pressure. As demonstrated by A. W. Skempton in Reference 2, it is a function of the shape of the footing and its depth of embedment in the supporting soil. A distinctly conservative bearing capacity factor of 6.5 was selected for this analysis on the basis of a depth of embedment which is at least equal to the 6-foot width of the base of the north underpinning wall. With this bearing capacity factor and an undrained shear strength of 8 ksf, the ultimate bearing capacity is 52 ksf.

An evaluation was also performed using the drained strength parameters from WCC testing set forth in Table SWP-3 ($C'=0.73$ ksf and $\emptyset = 36°$). The ultimate bearing capacity with drained shear strength is approximately 160 ksf.

Since the undrained shear strength is smaller than the drained shear strength, the ultimate bearing capacity of 52 ksf, based on undrained shear strength, is used. Safety factors are determined by dividing the ultimate bearing capacity by the various applied loads and are summarized in FSAR Table 2.5-14. These factors of safety exceed the required values.

8.3 ESTIMATE OF SETTLEMENT OF THE UNDERPINNING PIERS

The anticipated total settlement of the underpinning wall was computed utilizing elastic theory and a conservative selection of undrained modulus of elasticity of 4,000 ksf. The particular equation employed is that given on Figure 11-9 of Reference 3, which contains factors to allow for the shape and embedment of the permanent underpinning. The total settlement thus computed is estimated to be between 0.4 to 0.5 inch over the 40-year life of the SWPS. This includes the immediate settlement, settlement due to volume change from primary consolidation and long-term, delayed secondary compression settlement.

The underpinning scheme with its load applied by jacks will prestress the till into "secondary compression," which is that long-term gradual settlement which takes place under load in fine-grained strata after the hydrostatic excess pore water pressures have been dissipated. It is manifested as a straight line relationship between settlement and log of time in a semi-log plot. Secondary compression has also been referred to as "secondary consolidation."

It is a fundamental provision of the underpinning scheme that the immediate settlements and consolidation, if any, will occur during the jacking phase and only the secondary compression will remain to take place in the 40-year life of the structure. It is intended that the final jacking operation be continued until the following criteria are satisfied:

1. The jacking load for the permanent underpinning will be maintained at the specified value for at least 30 days and;
2. On a semi-log time plot, the progression of settlement in the later stage of jacking will approximate a straight line,
3. No more than 0.05 inch of settlement will occur in the last 30 days of jacking, and
4. No more than 0.01-inch settlement will occur in the last 10 days as measured by extensometer dial gages.

After these criteria are satisfied, it is assured that secondary compression alone remains to occur. Once this condition has been reached, sufficient data will be available to make a prediction of future settlements by an extrapolation of the straight line trend of secondary compression.

The secondary compression portion of the total settlement value has been estimated by weighing the following items of information:

1. The WCC testing report (Reference 1) yields a coefficient of secondary compression in the stress range associated with the underpinning piers equal to 0.0005 units of strain per log cycle of time. The underpinning piers will cause significant stress increase within a depth equal to one foundation width or 6 feet. Therefore, the settlement from secondary compression in each log cycle of time would equal 0.0005 times 6 feet times 12 to convert to inches, a value of 0.04 inch per log cycle. In the two log cycles of time from the completion of jacking to the 40-year life of the structure, secondary compression would total 0.1 inch by this computation.

2. Actual observations of settlement extending over several years at the SWPS indicate that the portion of this large structure founded on the sandy clay till settled in the range of 0.2 to 0.3 inch per log cycle of time. From this it would be reasonable to conclude that the smaller SWPS underpinning units would settle typically 0.1 to 0.2 inch per log cycle.

3. General experience of settlement of large structures on heavily preconsolidated clay, as illustrated by data presented by A. W. Skempton (Reference 2), indicates that long-term delayed settlement is typically one-fifth to one-third of the total settlement of the structure.

Based on the above considerations and on experience with similar controlled loading of spread footings on glacial till, it is estimated that two-thirds to three-quarters of the total computed settlement of 0.4 to 0.5 inch will be completed in the jacking period, leaving 0.1 to 0.2 inch of long-term settlement of the underpinning piers to take place in the 40-year life of the structure.

8.4 DIFFERENTIAL SETTLEMENT BETWEEN UNDERPINNING AND MAIN SWPS

Settlement measurements at the corners of the main SWPS structure founded on undisturbed glacial till commenced in the summer of 1978. These measurements defined semi-log straight lines which yield estimated projected settlement between the present date and the 40-year life of the structure equal to about 0.2 to 0.3 inch if loading conditions remained unaltered. The present loading is essentially the buoyant dead load of the structure with no water contained in the building's reservoirs and is equal to 1.6 ksf. The predicted long-term settlement for the bottom of the underpinning piers after jacking was estimated (see Section 8.3) to be about 0.1 to 0.2 inch. Hence, the potential for differential settlement between the portion of the building to be underpinned and the main portion of the building presently founded on undisturbed glacial till is on the order of 0.1 to 0.2 inch, if the loading conditions on the latter portion of the building remain as they are now.

The long-term loading of the main portion of the SWPS and the sequence of operating conditions which could influence this future settlement trend are as follows:

8.4.1 Filling SWPS Reservoirs

The average distributed static design unit load of the main SWPS building, including dead and sustained live load, over the entire slab founded on undisturbed glacial till equals 3.2 ksf. This includes an allowance for the buoyancy from hydrostatic uplift with the groundwater level maintained at elevation 627' and it also includes the weight of the water contained in the reservoir of the SWPS.

Of that 3.2 ksf, a total of 1.6 ksf, or one-half of the sustained loading is contributed by the weight of water in the building's water reservoirs. It is intended that the SWPS water reservoirs be totally filled before underpinning is completed. During regular operations of the SWPS, maintenance and cleaning will be accomplished by unloading reservoirs individually. Therefore, the water reservoirs' load will never be removed or reapplied all at one time. It is estimated that changes in loading after filling will amount to only one-quarter of the total of 1.6 ksf, or 0.4 ksf.

In April 1980 the SWPS reservoirs were filled to the extent of adding a unit load of 1.2 ksf. This loading was maintained until October 1980. The observed settlement in that period was no more than 0.2 inch. Full filling of the tanks would therefore be expected to cause a settlement of approximately 0.3 inch and partial emptying and refilling would cause up or down movement of about 0.1 inch. All of these movements refer to the main portion of the SWPS building and would be expected to have insignificant effects at the north wall underpinning.

8.4.2 Dewatering and Drawdown

Piezometer observations in September 1982 indicate that a groundwater level at about elevation 612' now exists beneath the SWPS structure. It is expected that there will be a drawdown during underpinning to approximately elevation 585' along the north underpinning wall of the SWPS that could create lowering of piezometric levels in the glacial till beneath the structure. On the basis of the assumed flow field and drawdown within the underlying glacial till, it is estimated that the drawdown for underpinning could cause a settlement of about 0.2 inch beneath the north underpinning wall and less than about 0.1 inch beneath the south wall of the main SWPS. This drawdown settlement will occur with the underpinning construction and will be recovered to some extent at the end of that operation.

8.4.3 Summary of Differential Settlement

In summary, fluctuations of the water loading in the reservoir will be the principal influence on movements of the southern main body of the SWPS whereas changes in drawdown of the plant area will chiefly affect settlements of the piers. An exact prediction of the settlements due to these effects acting concurrently cannot and need not be made. A distinctly conservative evaluation of these possible effects can be achieved by assuming a differential movement of 0.3 inch between the center of the main block of the SWPS and the north wall underpinning elements and that differential settlement could occur with maximum settlement either at the underpinning or at the main SWPS block.

9.0 CONCLUSION

As a result of these investigations and analyses as presented herein, it is concluded that the underpinning can be constructed without damaging the existing structure. With the underpinning resting on sound foundation material, the structural analysis of the modified structure proves that the SWPS will safely perform its intended function for the 40-year life of the plant.

As previously discussed in this proceeding, certain details of Applicant's testimony may be modified as construction proceeds. Any major changes would be brought to the attention of the NRC Staff pursuant to the Work Authorization Procedure established in August 1982.

REFERENCES

1. Woodward-Clyde Consultants, Test Results, Auxiliary Building Soil Boring and Test Program, Midland Units 1 and 2, Midland, Michigan, October 26, 1981

2. A. W. Skempton, "The Bearing Capacity of Clays," Building Research Conference Congress Proceedings, 1951

3. NAVFAC Design Manual DM-7, Soil Mechanics, Foundations, and Earth Structures, Department of the Navy, Naval Facilities Engineering Command, March 1971

TABLE SWP-1

TABLE OF REINFORCING BAR MAXIMUM STRESSES
FSAR LOAD COMBINATIONS

Location	Maximum Stress in Reinforcement (ksi)[(1)]	Load Combination
South Wall	49.007	$1.0(D + L + P_L + T_o + E')$
North wall, el 620'-0" to el 656'-0"	20.378	$1.0(D + L + P_L + T_o + E')$
East wall	48.410	$1.0(D + L + P_L + T_o + E')$
West wall	44.618	$1.4(D + L + E) + 1/0(P_L + T_o)$
Interior wall	52.638	$1.4(D + L + E) + 1.0(P_L + T_o)$
Roof slab	50.549	$1.4(D + L + E) + 1.0(P_L + T_o)$
Floor slab at el 634'-0"	50.939	$1.4(D + L + E) + 1.0(P_L + T_o)$
Underpinning walls(2)	42.752	$0.9(D) + 1.7(L) + 1.0(P_L) + 1.9(E)$
Base slab at	15.779	$1.0(D + L + P_L + T_o + E')$

D = Dead Load
L = Live load
P_L = Final jacking load
T_o = Operating thermal load
E = Operating basis earthquake
E' = Safe shutdown earthquake

(1) Allowable reinforcement stress = 54 ksi

(2) Designed for ACI-349 load combinations

TABLE SWP-2 SUMMARY OF TEST BORING SERIES IN VICINITY OF THE SERVICE WATER PUMP STRUCTURE UNDERPINNING

Boring Series	Date Performed	Purpose of the Borings	Technical Data
C	Oct. 1969	In original preconstruction investigation	Standard sampler penetration resistance (N values), no laboratory testing
D	June 1970	In original preconstruction investigation	Standard sampler penetration resistance (N values), no laboratory testing
SWP	Oct. 1974	Preconstruction investigation	N values, no laboratory testing
SW	Oct. 1978 to March 1979	Investigate character of fill beneath and around overhang	N values, grain size analyses
Test Pit	June 1979	To investigate fill condition at NE corner of SWPS	Material identification test including, moisture content, density, limits, sieve analysis, specific gravity, and compaction testing
CH	July 1979	Determine seismic velocities in fill and till	N values, no laboratory testing
PD	Dec. 1979 to Feb. 1980	Groundwater investigation	N values, grain size analyses

Boring Series	Date Performed	Purpose of the Borings	Technical Data
COE	April 1981 to May 1981	COE-16 and 16A: to obtain samples for test to support underpinning design	Continuous undisturbed sampling for identification and engineering properties tests - See WCC Report 10/1/81
WF	May 1981	Groundwater investigation for Permanent wells	Continuous undisturbed sampling for identification and grain size analysis

TABLE SWP-3 SERVICE WATER BUILDING UNDERPINNING - PROPERTIES OF FILL AND SANDY CLAY TILL

Stratum F: Fill

Median values of standard sampler penetration resistance, "N" values, in blows per foot; taken in underpinning zone below overhang foundation:

For Clay Fill, N = 17 blows per foot
For Sand Fill, N = 16 blows per foot
(Sandy material concentrated at the southwest corner and west wall of underpinning)

Stratum A: Sandy Alluvium

Median Standard Penetration Resistance:

N = 90 blows per foot;
28% passing the No. 200 sieve size;
Average undrained shear strength from three CIU traxial tests by WCC in 1981 equals 25 ksf.

Stratum T: Hard Clay Till

Median Standard Penetration Resistance:

N = 75 blows per foot
57% passing the No. 200 sieve size;
liquid limit = 17
plastic limit = 11
Natural water content = 9%

Median Shear Strength from undrained triaxial tests

Testing Grouping	Median Value of Shear Strength (ksf)
Three UU triaxial tests	16
Seven CIU triaxial tests	22
All 10 undrained triaxial tests	18

Average drained strength parameters:

From 2 series of CIU triaxial tests by WCC, 1981:

$C' = 0.73$ ksf $\emptyset' = 36^{\circ}$

Typical Consolidation Properties:

From 2 consolidation tests by WCC, 1981:

Recompression Ratio: $C_r/(1 + e_0) = 0.004$ (strain per log cycle of pressure)

Coefficient of Secondary Compression: 0.0005 (strain per log cycle of time)

Coefficient of Consolidaton: $C_v = 0.01$ cm per second.

SS: STATE OF MICHIGAN
COUNTY OF WASHTENAW

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329-OM
	)	50-330-OM
CONSUMERS POWER COMPANY	)	50-329-OL
	)	50-330-OL
(Midland Plant, Units 1 and 2)	)	

AFFIDAVIT OF ALAN J. BOOS

My name is Alan J. Boos. I am Assistant Project Manager for the Midland Project for Bechtel Power Corporation. In this capacity I am responsible for providing overall management direction for soils work for Bechtel as well as for project services (procurement, administrative services, cost and schedule). I have a B.S. and M.S. in Civil Engineering from the University of Michigan. I am a registered professional engineer in the states of Michigan and California. My resume is attached.

In connection with my role as Assistant Project Manager, I have been assigned the responsibility for the Service Water Pump Structure Testimony. I am personally familiar with the events leading up to the decision to implement remedial work on the building, and I participated in the decision process related thereto. I have reviewed in detail the proposed design of the remedial work and I am jointly responsible with Edmund M. Burke, James P. Gould and Palanichamy Shunmugavel for the testimony.

I swear that the statements contained in this affidavit, the attached resume, and the Service Water Pump Structure Testimony are true and correct to the best of my knowledge and belief.

Alan J. Boos
ALAN J. BOOS

SIGNED AND SWORN TO BEFORE
me this 18th day of
October, 1982.

Eleanor S. Ivanson
NOTARY PUBLIC
My Commission Expires January 14, 1983

ALAN J. BOOS

POSITION	Assistant Project Manager
EDUCATION	BS, Civil Engineering, University of Michigan MS, Civil Engineering, University of Michigan
PROFESSIONAL DATA	Registered Professional Civil Engineer in Michigan Registered Professional Engineer in California
SUMMARY	1-1/2 years: Assistant project manager 4 years: Project field engineer 2-1/2 years: Area engineer 1-1/2 years: Senior civil design engineer 2-1/2 years: Civil design engineer 2 years: Civil field engineer

EXPERIENCE

Mr. Boos is currently serving as an assistant project manager on the Consumers Power Company Midland project. In this position he is responsible for providing overall management guidance for remedial soils work and general project services (procurement, administrative services and cost/schedule).

Prior to this assignment Mr. Boos served as project field engineer for the Midland project. He was responsible for all field engineering activities at the jobsite in this position.

Prior to his assignment as project field engineer, Mr. Boos was an area engineer at the Midland jobsite where he coordinated and directed all field engineering activities for the construction of the auxiliary building.

Mr. Boos came to the Ann Arbor Power Division in December 1972 as a senior civil design engineer. He responsibilities included reviewing project civil designs and acting as the civil department's licensing engineer reviewing civil safety analysis report sections prior to submittal to the NRC.

Mr. Boos joined Bechtel in July 1968 as a civil design engineer for the Power and Industrial Division in San Francisco. His assignments as a civil design engineer and a civil field engineer sent him to Russellville, Arkansas, and to Homestead, Florida, where, among other activities, he supervised the repair of a post-tensioned concrete containment dome.

February 1982

SS: STATE OF MICHIGAN
COUNTY OF WASHTENAW

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	Docket Nos. 50-329-OM
	)	50-330-OM
CONSUMERS POWER COMPANY	)	50-329-OL
	)	50-330-OL
(Midland Plant, Units 1 and 2)	)	

AFFIDAVIT OF PALANICHAMY SHUNMUGAVEL

My name is Palanichamy Shunmugavel. I am a Engineering Specialist in the civil/structural department of Bechtel Power Corporation in Ann Arbor. In this capacity I am responsible for providing consultation to civil/ structural engineers working for Bechtel and for reviewing their work. I have a B.E. in Civil Engineering, M.Tech. in Structural Engineering and a Ph.D. in Civil Engineering. I am a registered professional engineer in the state of California. My resume is attached.

In connection with my role as Engineering Specialist, I have been assigned the responsibility for the Service Water Pump Structure Testimony. I have reviewed in detail the proposed design of the remedial work, and the related structural evaluations and I am jointly responsible with Edmund Burke, James P. Gould and Alan J. Boos for the testimony.

I swear that the statements contained in this affidavit, the attached resume, and the Service Water Pump Structure Testimony are true and correct to the best of my knowledge and belief.

Palanichamy Shunmugavel
P. Shunmugavel

PALANICHAMY SHUNMUGAVEL

SIGNED AND SWORN TO BEFORE
me this 13th day of
October, 1982.

Eleanor S. Evanson

NOTARY PUBLIC

My Commission Expires January 14, 1983

October, 1982

PALANICHAMY SHUNMUGAVEL

POSITION	Engineering Specialist, Civil/Structural Staff
EDUCATION	BE, Civil Engineering, Madras University, India M. Tech, Structural Engineering, Indian Institute of Technology, Bombay Ph.D., Civil Engineering, University of Illinois, Urbana
PROFESSIONAL DATA	Registered Civil Engineer in California Member, American Concrete Institute Member, American Society of Civil Engineering
SUMMARY	4 years: Engineering Specialist, Civil/Structural Staff 1 year: Senior Engineer, Civil/Structural Staff 1 year: Senior Engineer, Pilgrim 2 Nuclear Project 3 years: Senior Analyst, Sargent & Lundy, power projects 1 year: Analyst, Sargent & Lundy, power projects
EXPERIENCE:	Dr. Shunmugavel is currently a member of the civil/structural staff in the Bechtel Ann Arbor Power Division. His primary responsibilities, in connection with fossil and nuclear power plant structures, include providing consultation to civil/structural engineers engaged in the performance of design and analysis and in the preparation of specifications, reports and drawings; reviewing typical project work; developing methods and procedures for the solution of special structural problems; solving special structural problems; and supervising three staff engineers. He is the chairman of the structural committee, the main purpose of which is to establish and maintain a good technical quality with uniformity among various offices of the Bechtel Power Corporation. He has participated in an extensive concrete masonry shear wall test program.

His areas of responsibility include containment, seismic analysis, computer analysis, computer applications and other general structural analysis and design.

Previously, Dr. Shunmugavel was assigned to the Pilgrim 2 Nuclear Power Plant project in Bechtel's San Francisco office. His responsibilities included the performance of containment analysis and design, the preparation of related specifications and drawings, the evaluation of contract packages, and the supervision of about ten engineers and drafters.

Prior to joining Bechtel, Dr. Shunmugavel had about 4 years experience in fossil and nuclear power projects. His assignments included the development of computer programs for seismic analysis of power plant structures, components and piping systems; the application of statistical and probabilistic methods to structural problems; the analysis and design of special structures such as nuclear steam supply system supports, machine foundations, pipe whip restraints, and buried pipes; and the analysis of structures for extreme loads such as tornado, explosion, aircraft impact and hydrodynamic effects in boiling water reactor containments.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	Docket Nos. 50-329-OM
CONSUMERS POWER COMPANY	)	50-330-OM
	)	50-329-OL
(Midland Plant, Units 1 and 2)	)	50-330-OL

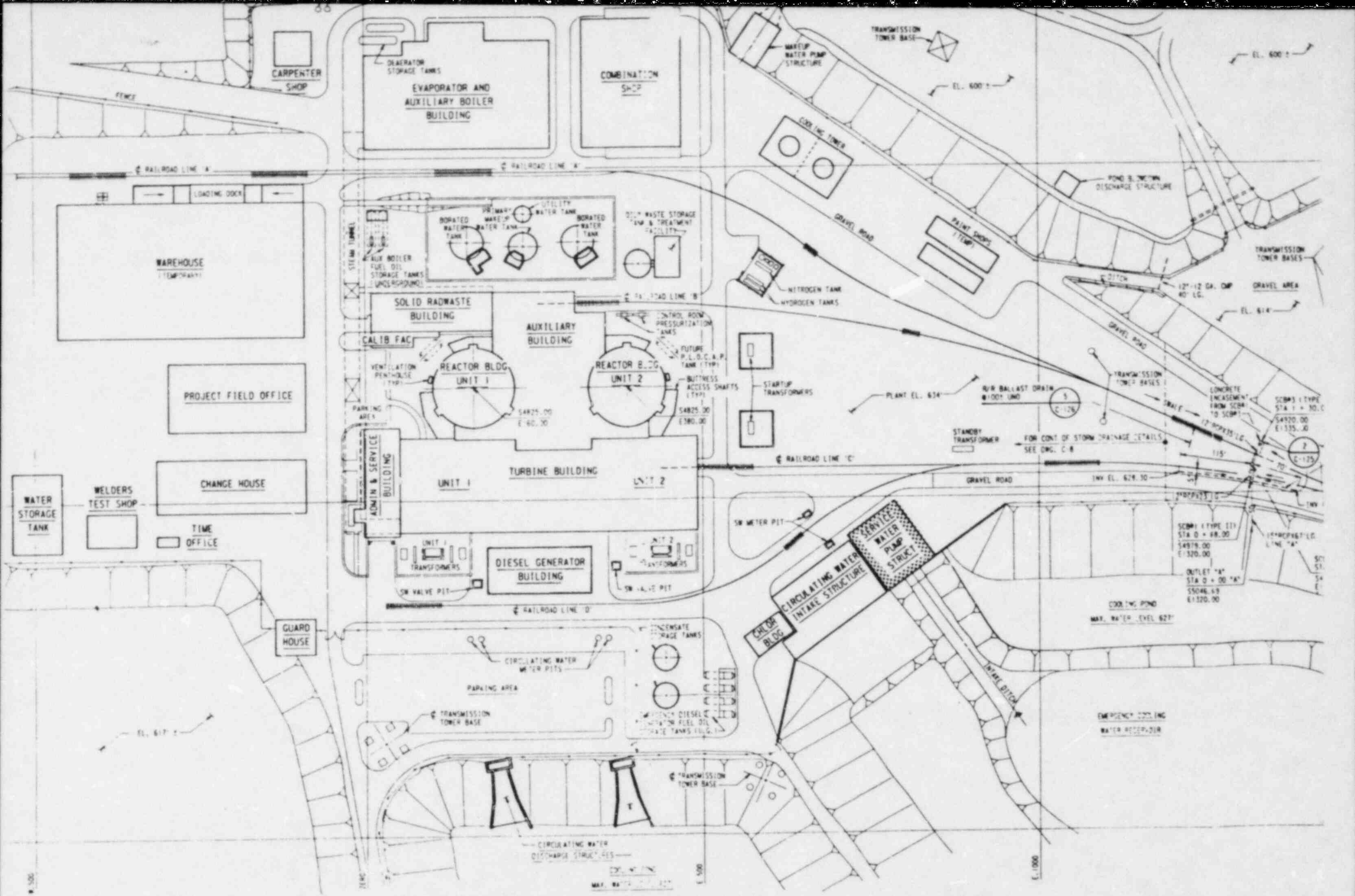
TESTIMONY OF

ALLEN J. BOOS, EDMUND M. BURKE, JAMES P. GOULD, and PALANICHAMY SHUNMUGAVEL

CONCERNING

MIDLAND PLANT SERVICE WATER PUMP STRUCTURE

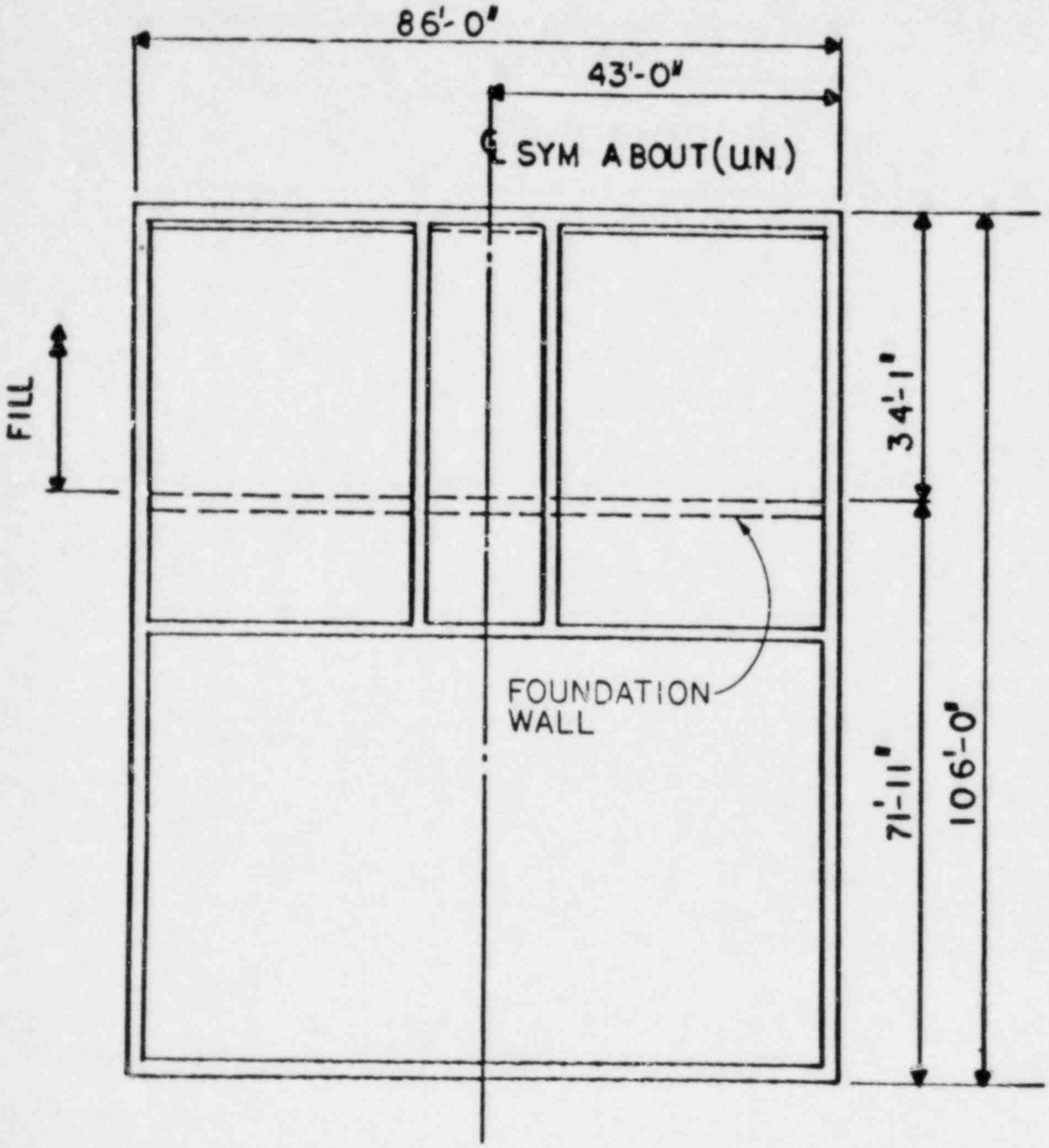
Volume 2 - Figures



SITE PLAN OF MIDLAND UNITS 1 AND 2 POWER PLANT

FIG SWP-1

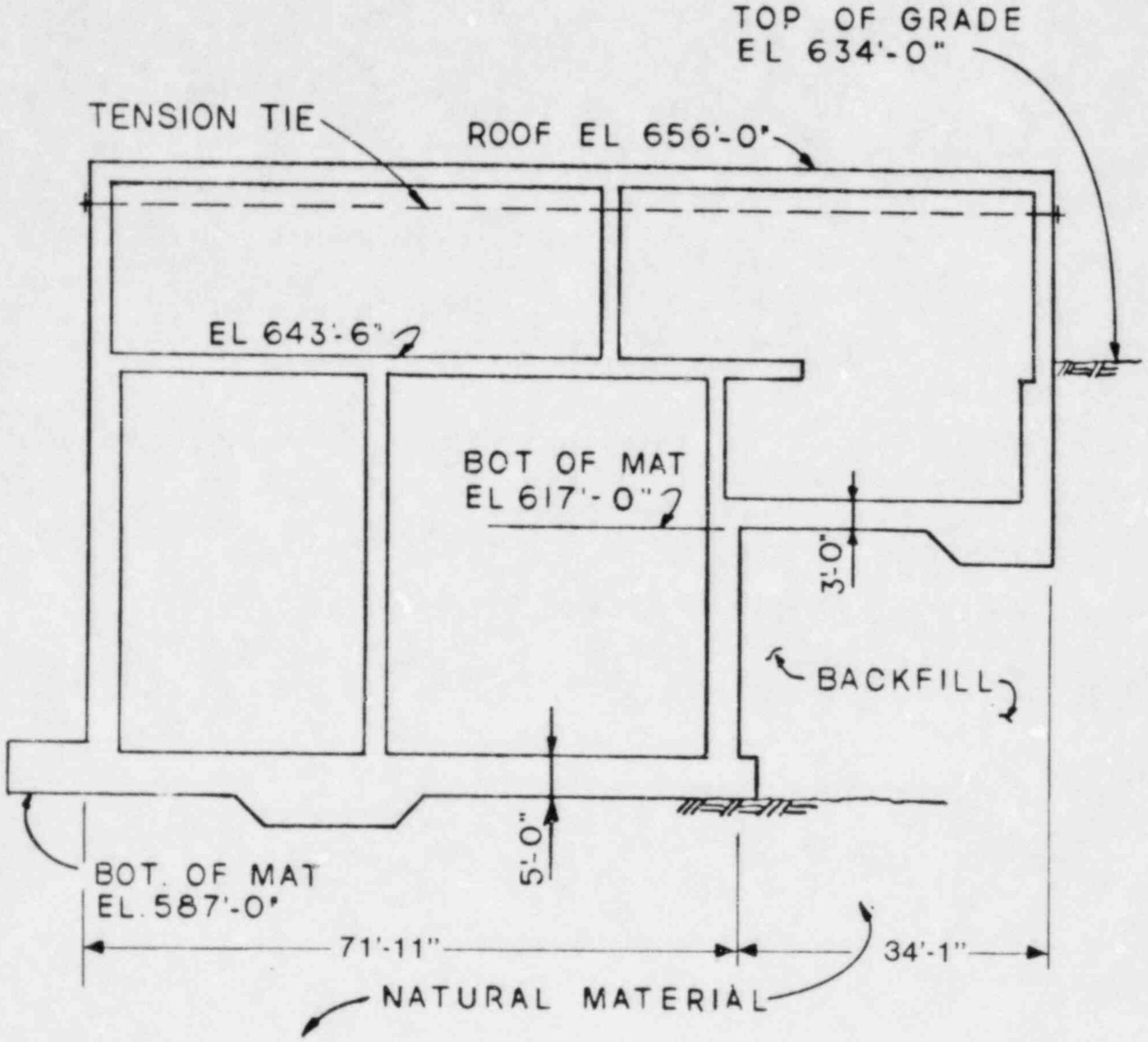
G-2611-01



PLAN OF SERVICE WATER PUMP STRUCTURE AT EL 634'-6"

FIG SWP-2

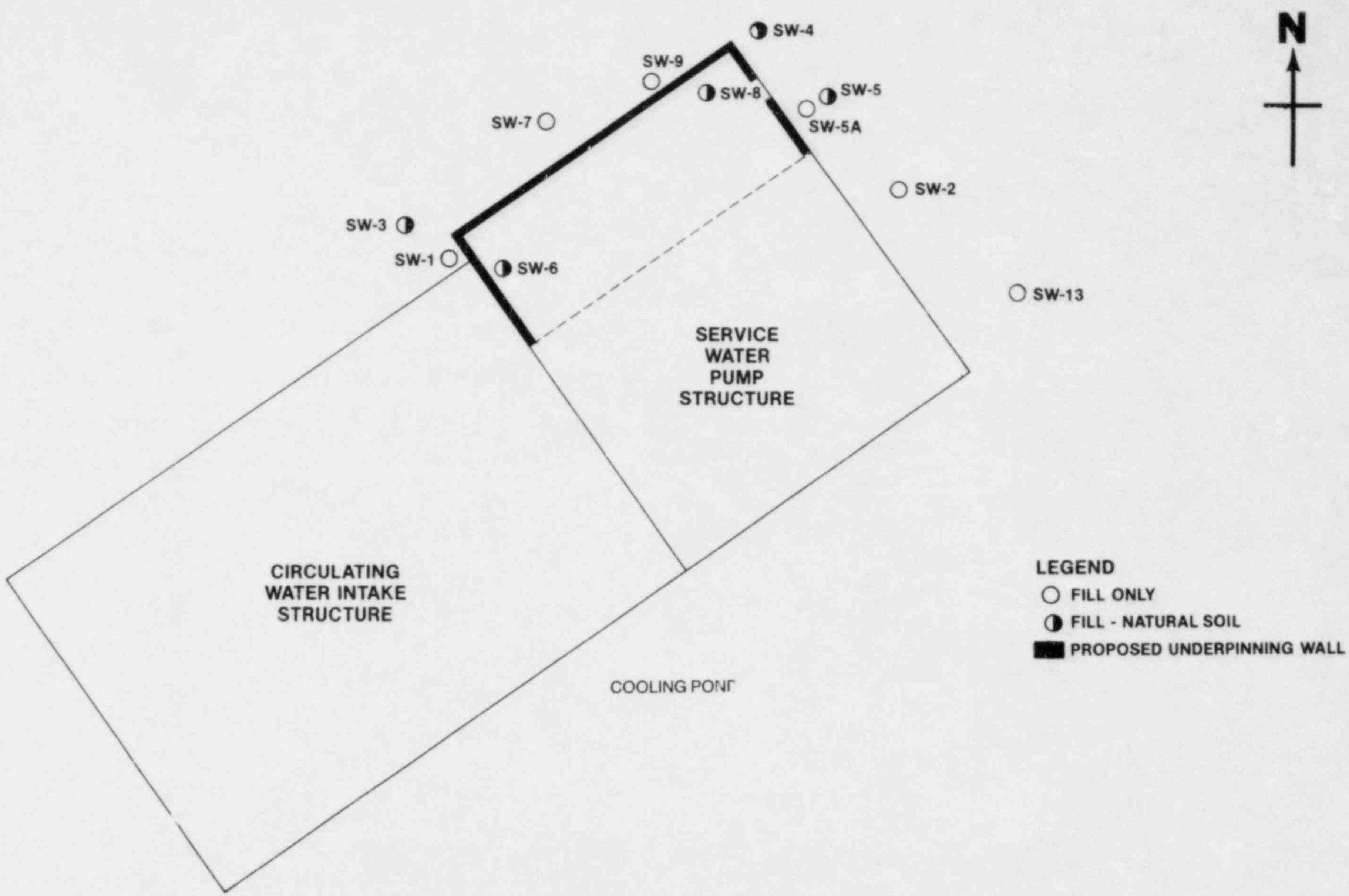
G-2611-02



TYPICAL SECTION OF SERVICE WATER PUMP STRUCTURE
(Looking West)

FIG SWP-3

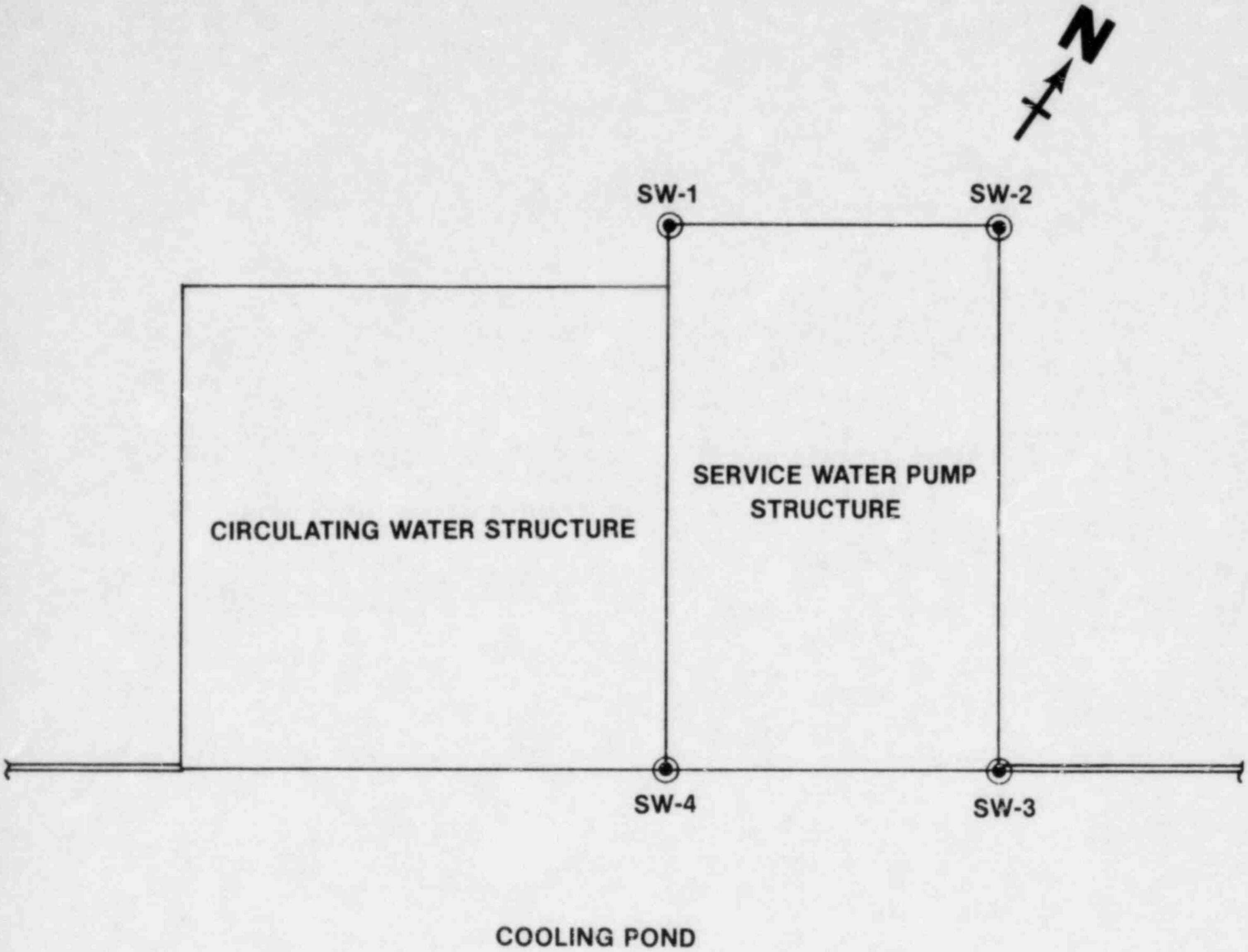
G-2611-03



BORING LOCATION PLAN

FIG SWP-4

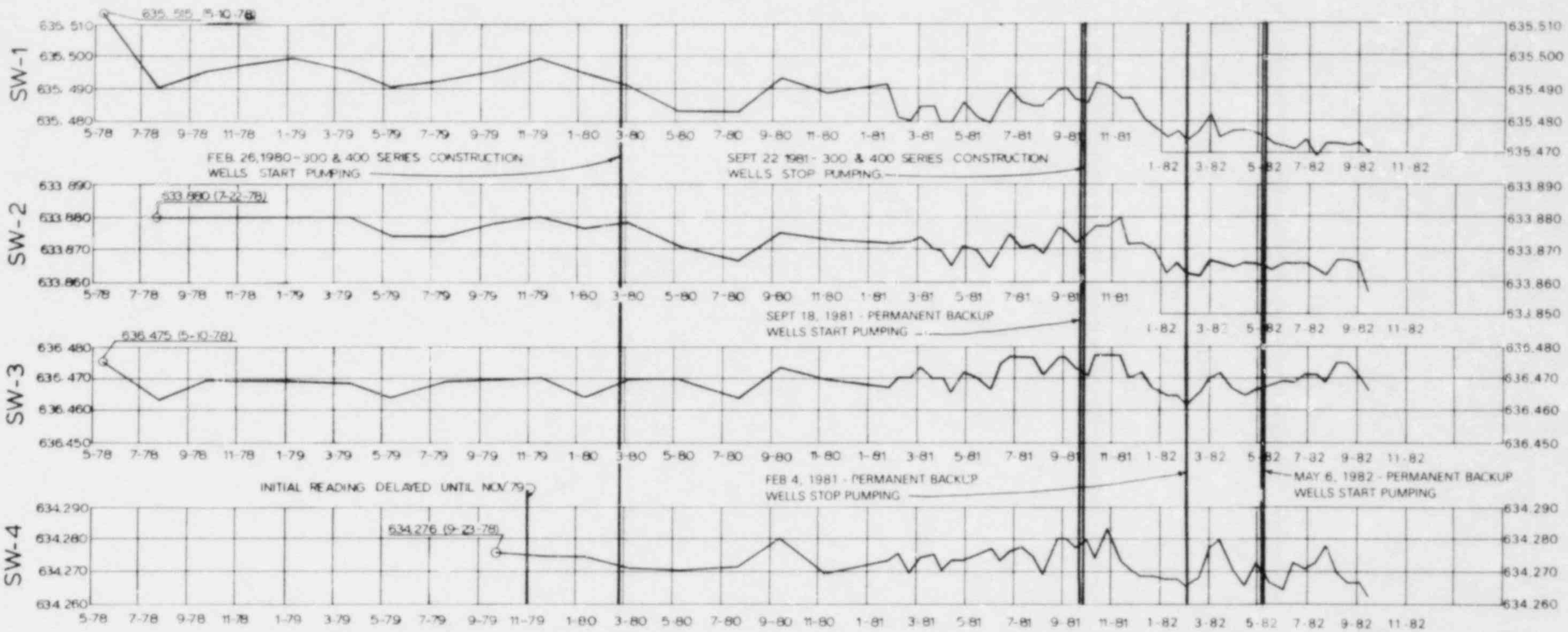
G-2611-04



SERVICE WATER STRUCTURE SETTLEMENT MARKER LOCATIONS

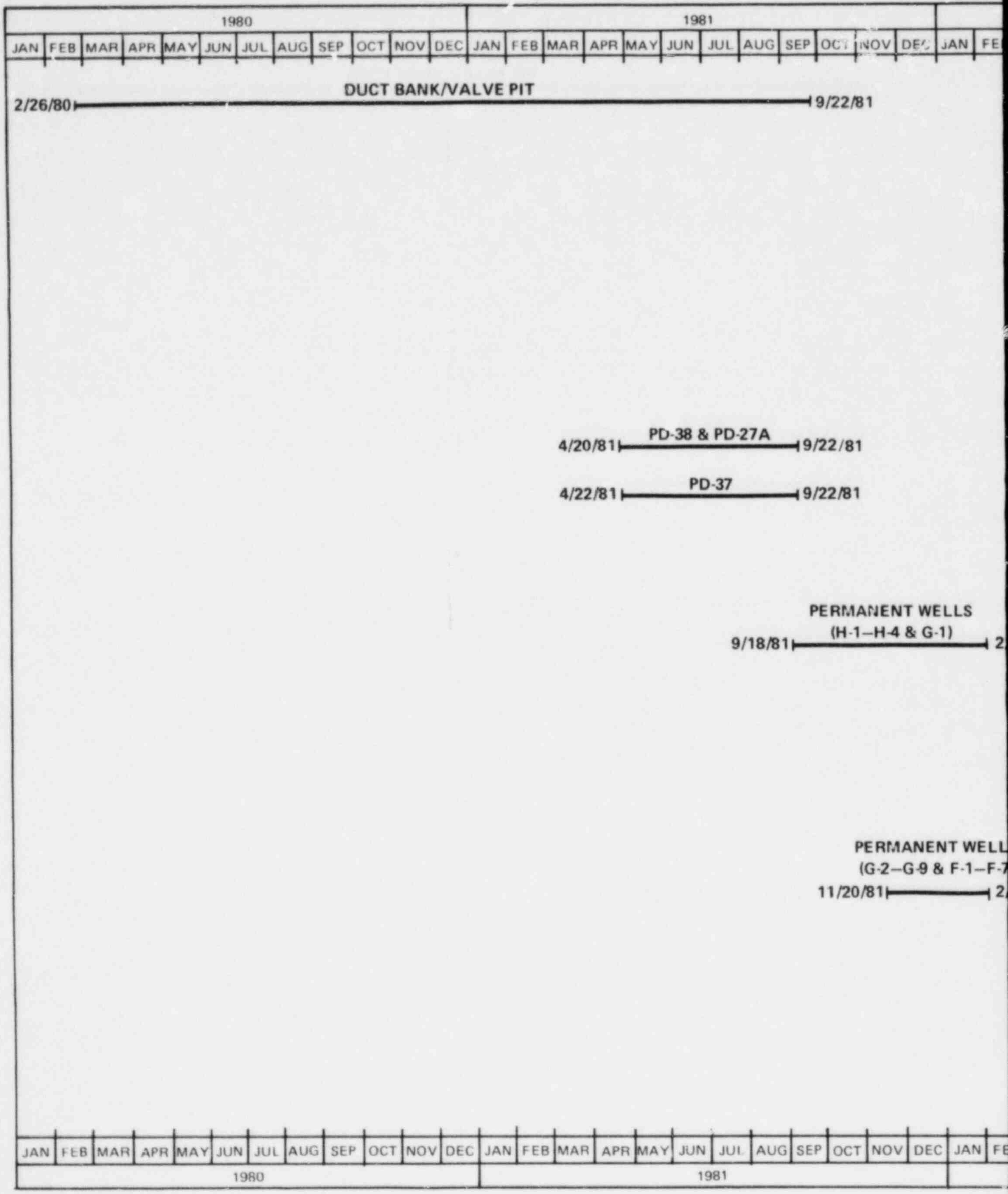
FIG SWP-5

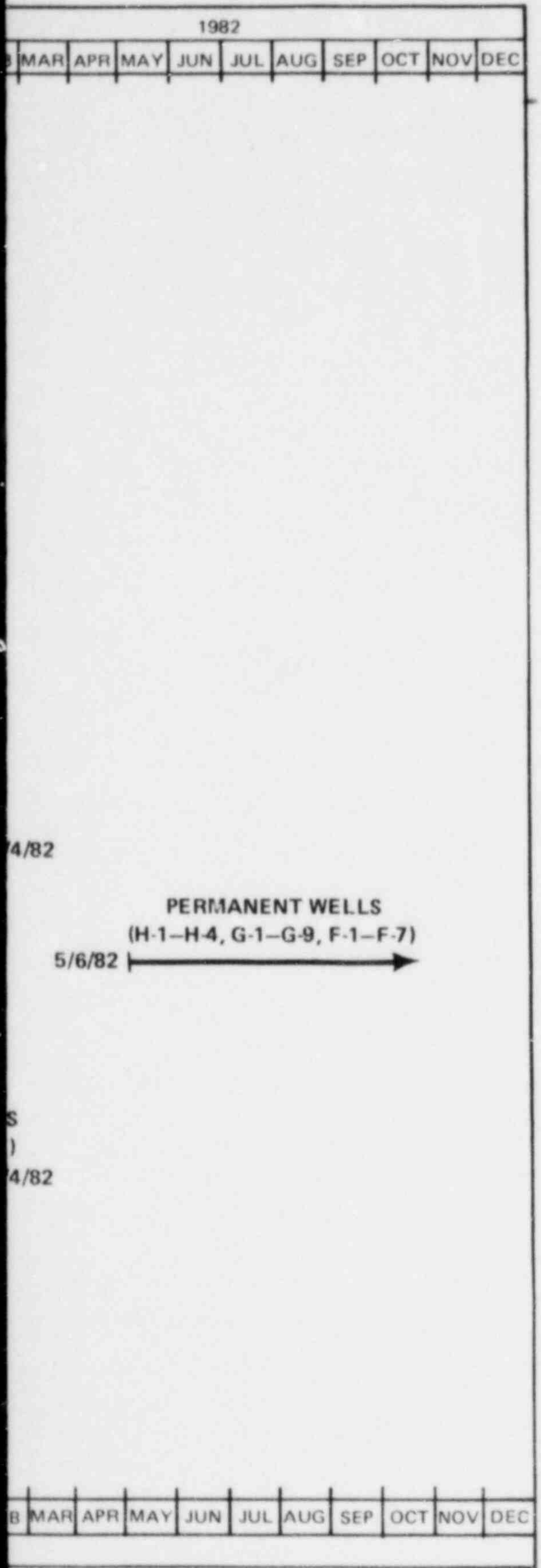
G-2611-05

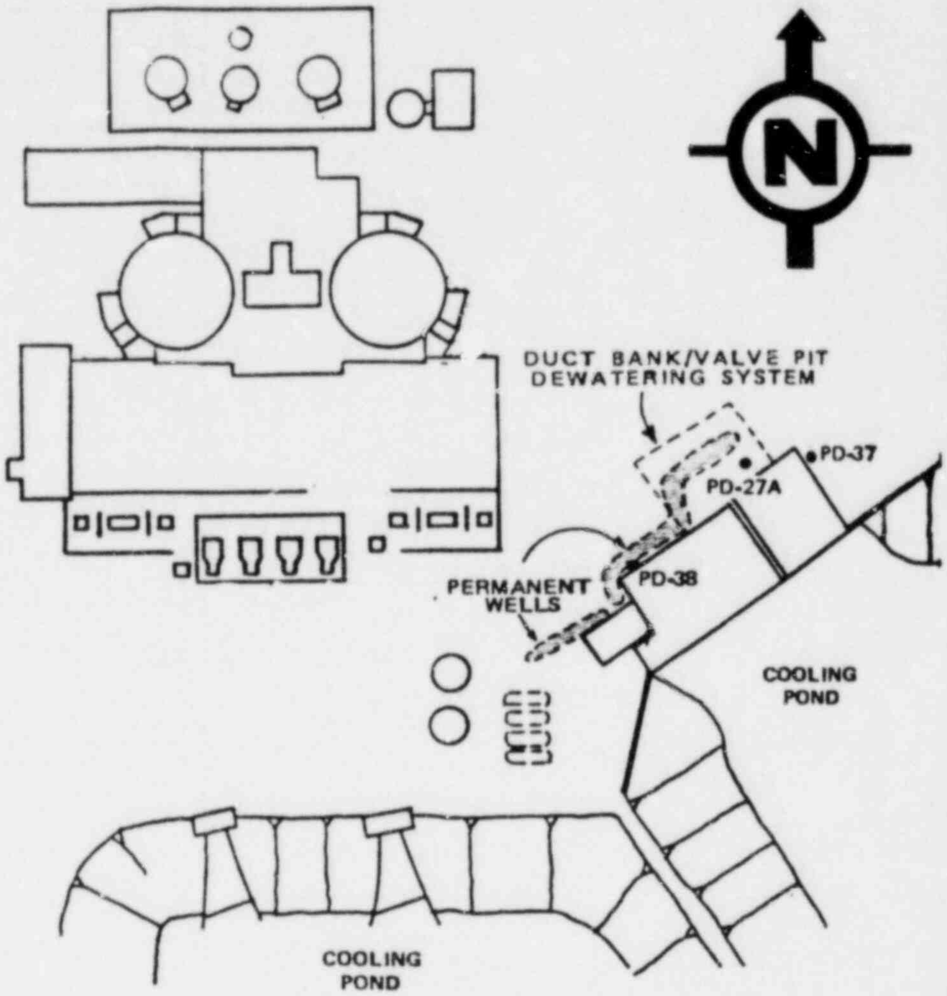


SERVICE WATER PUMP STRUCTURE
SETTLEMENT MARKER PLOTS
FIG. SWP - 6

G-2611-06


1980
1981
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FE
DUCT BANK/VALVE PIT
2/26/80
9/22/81
PD-38 & PD-27A
4/20/81
9/22/81
PD-37
4/22/81
9/22/81
PERMANENT WELLS
(H-1–H-4 & G-1)
9/18/81
PERMANENT WELL
(G-2–G-9 & F-1–F-7
11/20/81
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC JAN FE
1980
1981





SITE PLAN
LOCATIONS OF DEWATERING WELLS
(NOT TO SCALE)

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

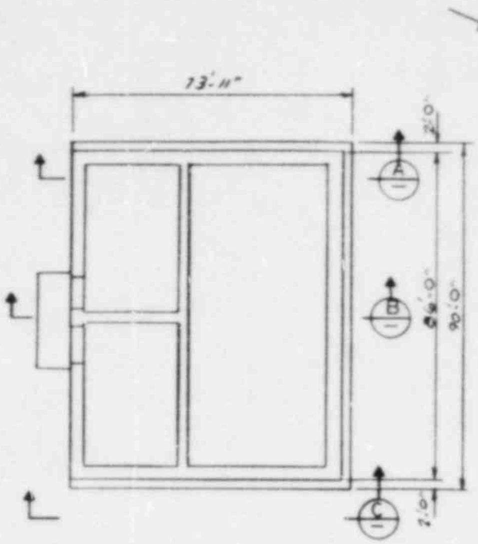
DEWATERING EVENTS IN SERVICE WATER PUMP STRUCTURE AREA

FIGURE SWP-7

G-2611-07

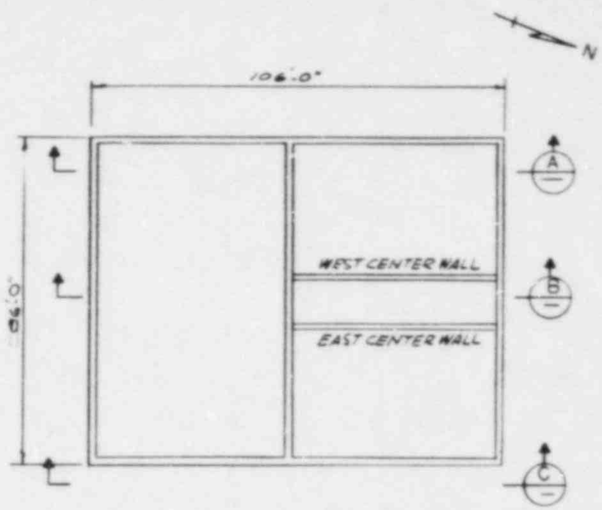
NOTE:

DATES SHOWN ON SECTIONS INDICATE DATE CONCRETE WAS PLACED.



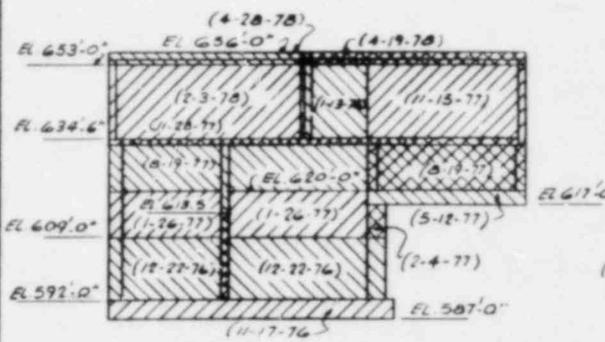
PLAN AT EL. 592'-0"

N.T.S.



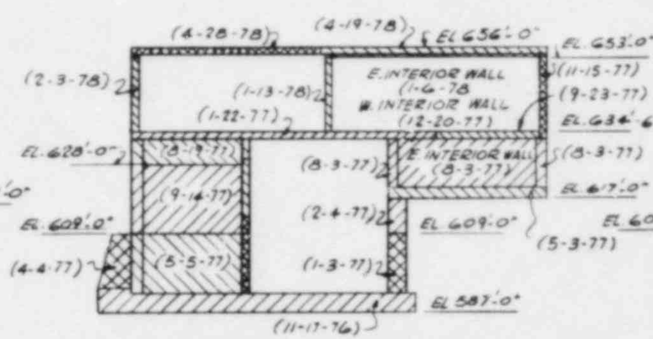
PLAN ABOVE EL. 634'-6"

N.T.S.



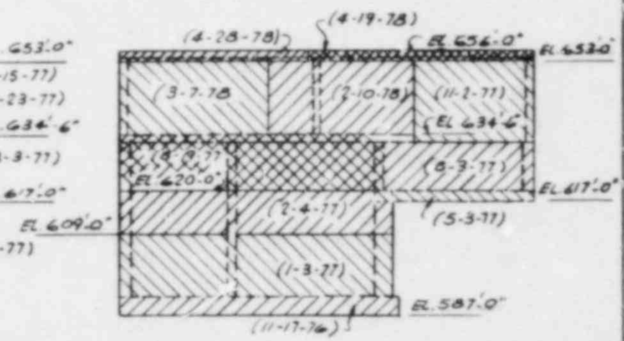
SECTION A

N.T.S.



SECTION B

N.T.S.

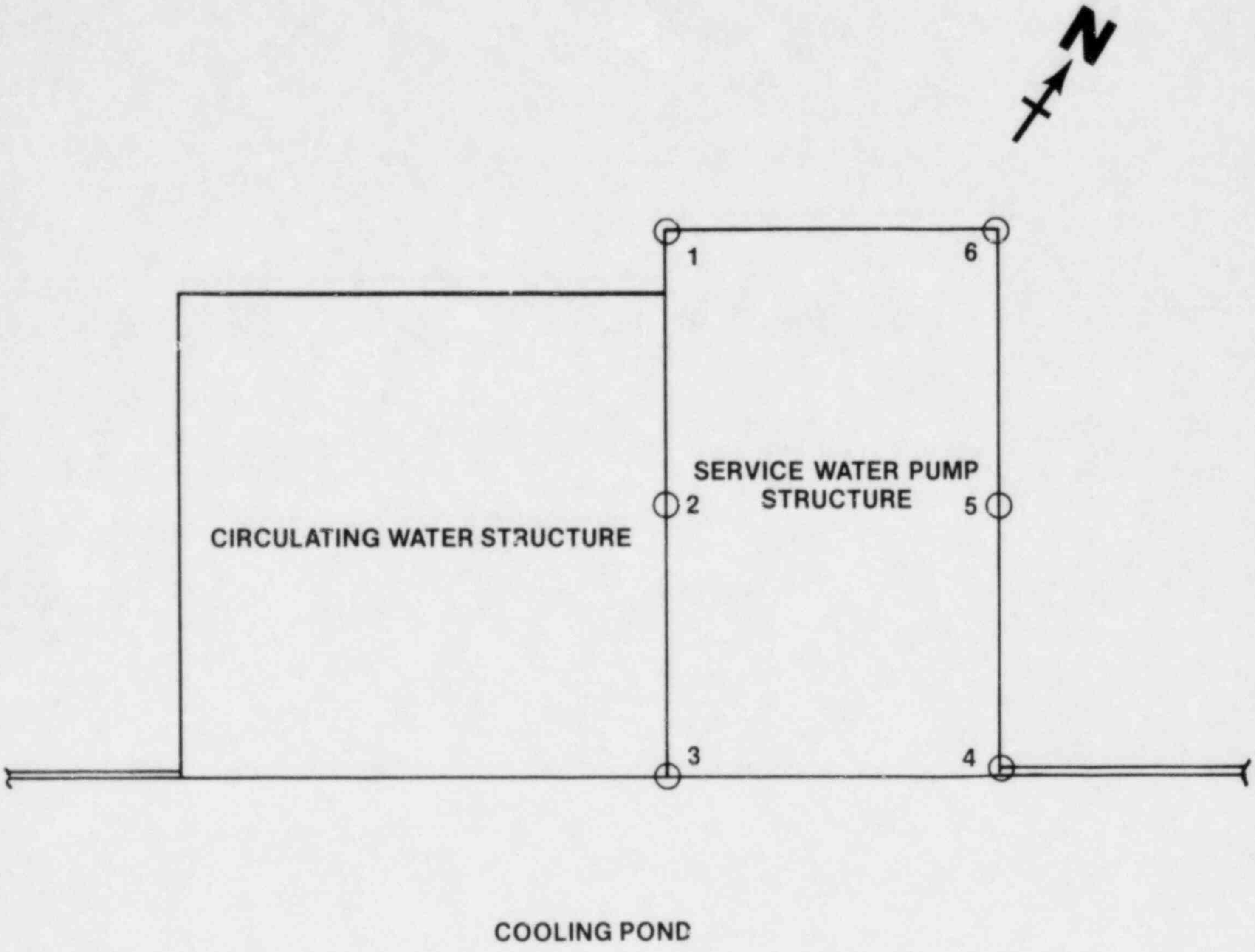


VIEW C-C

N.T.S.

SERVICE WATER PUMP STRUCTURE
CONCRETE PLACEMENT SEQUENCE
FIG. SWP - 8.

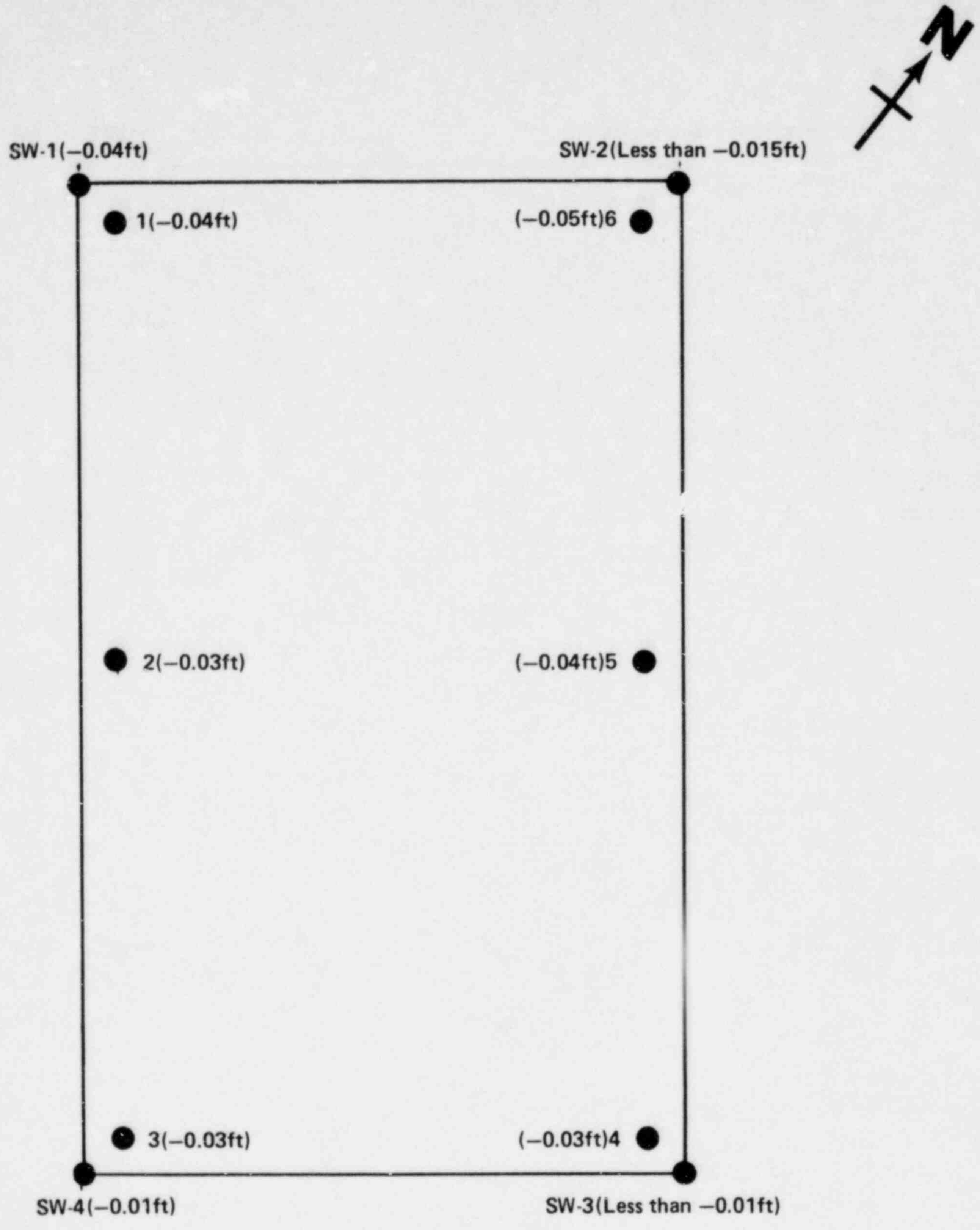
G-2611-08



SERVICE WATER PUMP STRUCTURE
CONSTRUCTION SURVEY CONTROL POINT
LOCATIONS

FIG SWP-9

G-2611-09

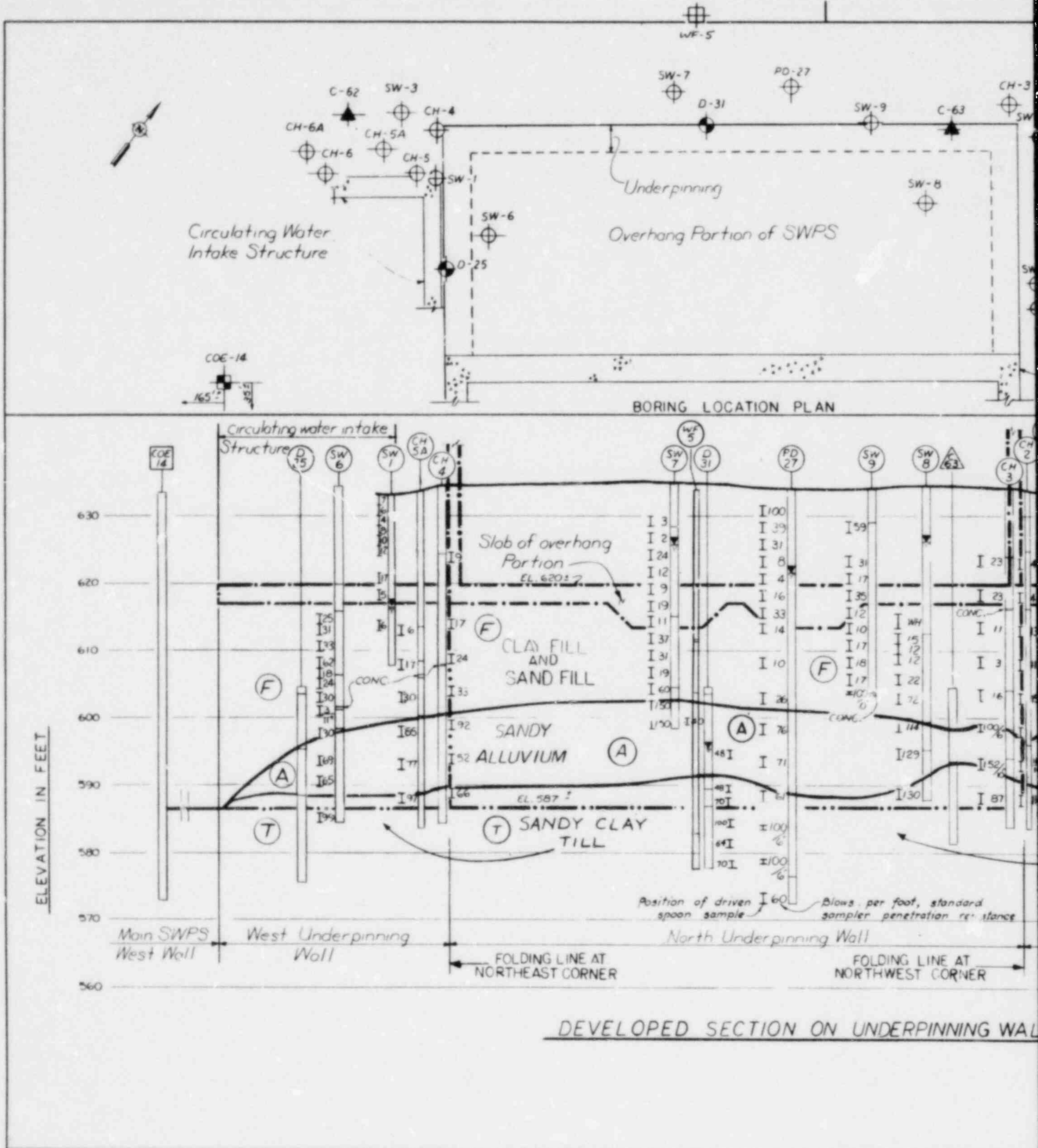


SERVICE WATER PUMP STRUCTURE CHANGES IN ELEVATION

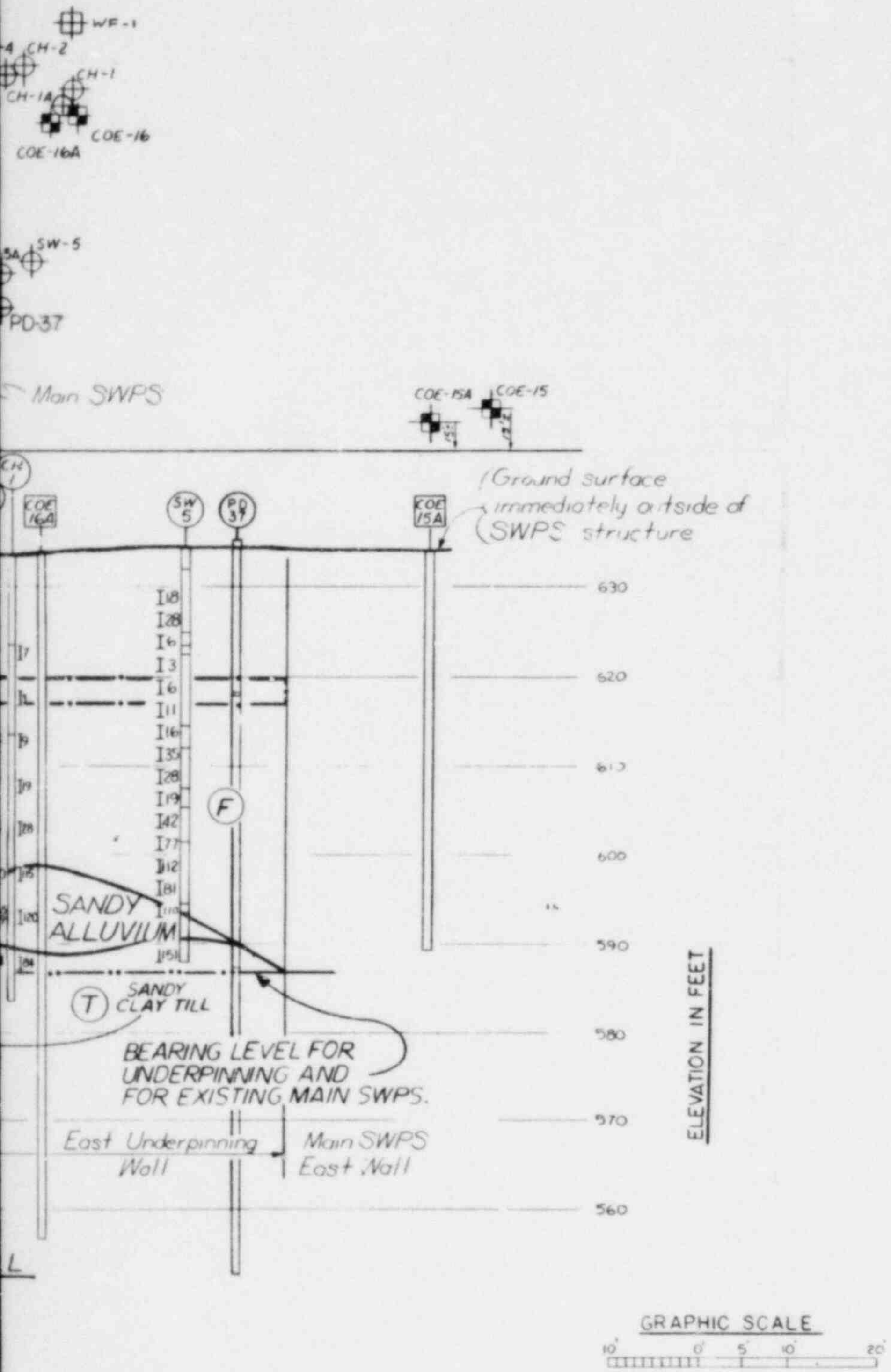
NOTE:
ettlements in ( ) at markers 1 through 6 are measured from March 1978 and indicate movement rough October 1982. Figures at markers SW1 through SW4 are measured from later dates (see ure 6)

FIG SWP-10

G-2611-10



WF-5
SW-7
PD-27
C-62
SW-3
CH-4
D-31
SW-9
C-63
CH-3
CH-6A
CH-5A
CH-6
CH-5
SW-1
Underpinning
SW-8
SW-6
Circulating Water Intake Structure
Overhang Portion of SWPS
D-25
COE-14
BORING LOCATION PLAN
Circulating water intake Structure
Slab of overhang Portion
EL. 620±
CLAY FILL AND SAND FILL
SANDY ALLUVIUM
SANDY CLAY TILL
EL. 587±
CONC.
ELEVATION IN FEET
Position of driven spoon sample
Blows per foot, standard sampler penetration resistance
Main SWPS West Wall
West Underpinning Wall
North Underpinning Wall
FOLDING LINE AT NORTHEAST CORNER
FOLDING LINE AT NORTHWEST CORNER
DEVELOPED SECTION ON UNDERPINNING WAL



BORING LEGEND

- "C" SERIES BORINGS MADE IN OCT, 1969
- "D" SERIES BORINGS MADE IN JUNE, 1970
- "SW" "CH" AND "PD" SERIES BORINGS MADE BETWEEN OCTOBER, 1978 AND FEB. 1980
- "COE" SERIES BORINGS MADE DURING APRIL AND MAY, 1981
- "WF" SERIES BORINGS MADE, MAY 1981

NOTES

1. ELEVATIONS REFER TO PLANT DATUM.
2. GROUND WATER ELEVATIONS SHOWN BY SYMBOL ARE THOSE MEASURED AT THE DATE OF THE BORINGS. PIEZOMETERS NEAR SWPS INDICATE THAT IN NOVEMBER, 1981 GROUND WATER LEVELS ARE TYPICALLY AT ELEVATION 620 NEAR SWPS.

GENERAL STRATA DESCRIPTIONS

STRATUM

(F) **FILL**, CHIEFLY OF SILTY OR SANDY CLAY BUT ALSO INCLUDING NUMEROUS CONCRETE MUD MATS AND VERY COMPACT SAND FILL AT WEST.

(A) **SANDY ALLUVIUM**, COMPACT TO EXTREMELY COMPACT SAND INTERLENSED WITH SILT AND CLAY.

(T) **SANDY CLAY TILL**, EXTREMELY COMPACT OR VERY HARD MIXTURE OF CLAY OF LOW PLASTICITY, SOME SAND, NUMEROUS GRAVEL FRAGMENTS; MAY ALSO INCLUDE LENSES OR POCKETS OF ALLUVIAL SAND.

NOTE:

STRATA LINES AT THE POSITION OF THE UNDERPINNING WALL WERE ESTIMATED BY WEIGHING VARIOUS ITEMS OF BORING LOG INFORMATION AND CONDITIONS REVEALED DURING EXCAVATION FOR THE MAIN SWPS FOUNDATIONS. BORINGS THAT DO NOT APPEAR IN PROFILE, WERE CONSIDERED REDUNDANT.

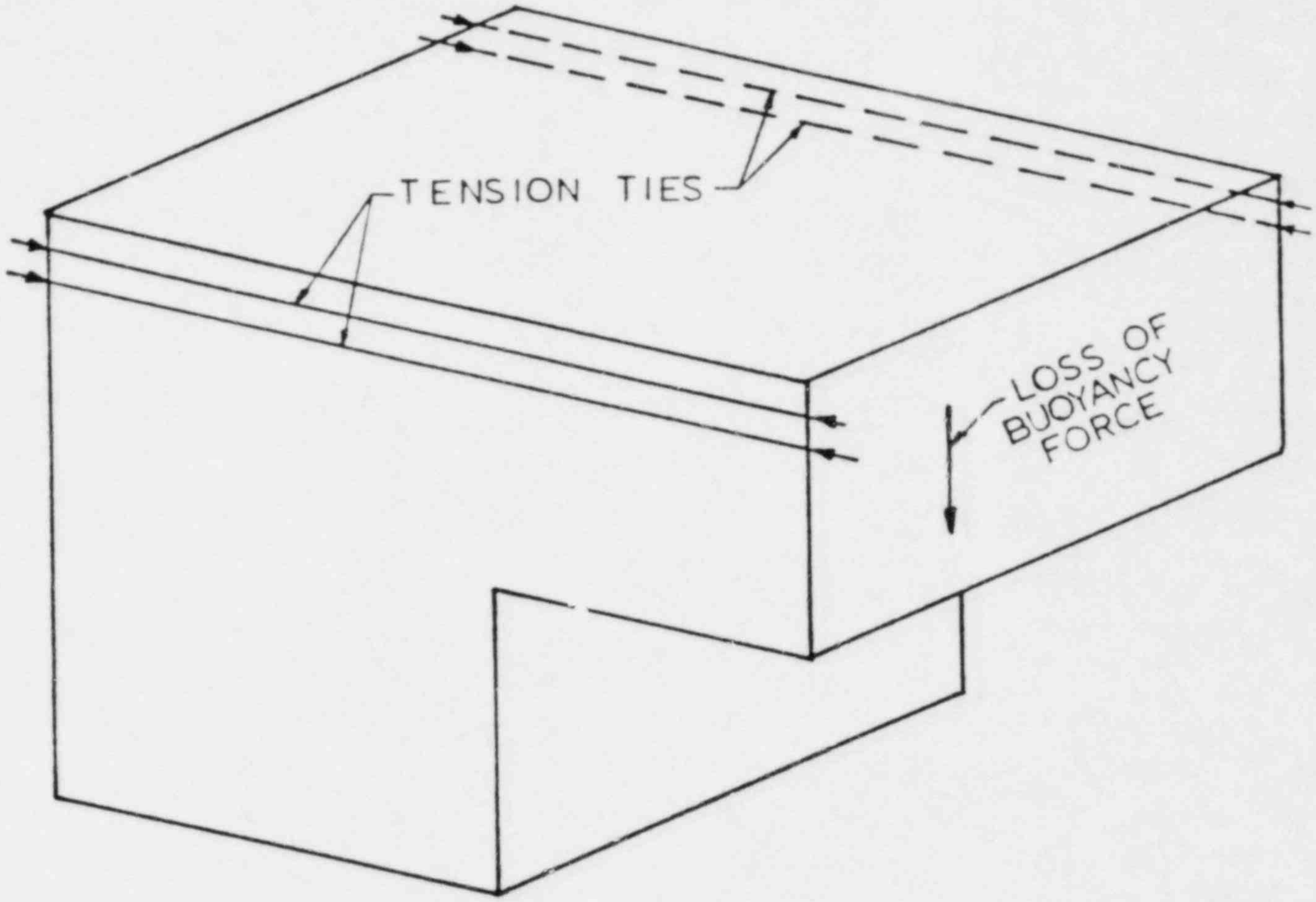
MIDLAND POWER PLANT

SERVICE WATER BUILDING UNDERPINNING

MUESER · RUTLEDGE · JOHNSTON & DESIMONE
CONSULTING ENGINEERS
708 THIRD AVENUE, NEW YORK, N.Y. 10017

SCALE 1" = 8'	MADE BY C.P. DATE 11-9-81 CH'KD BY J.P.G./S.L.T. DATE 11-9-81	FILE NO. 5464
BORING LOCATION PLAN AND GEOLOGICAL SECTION		FIGURE SWP 15

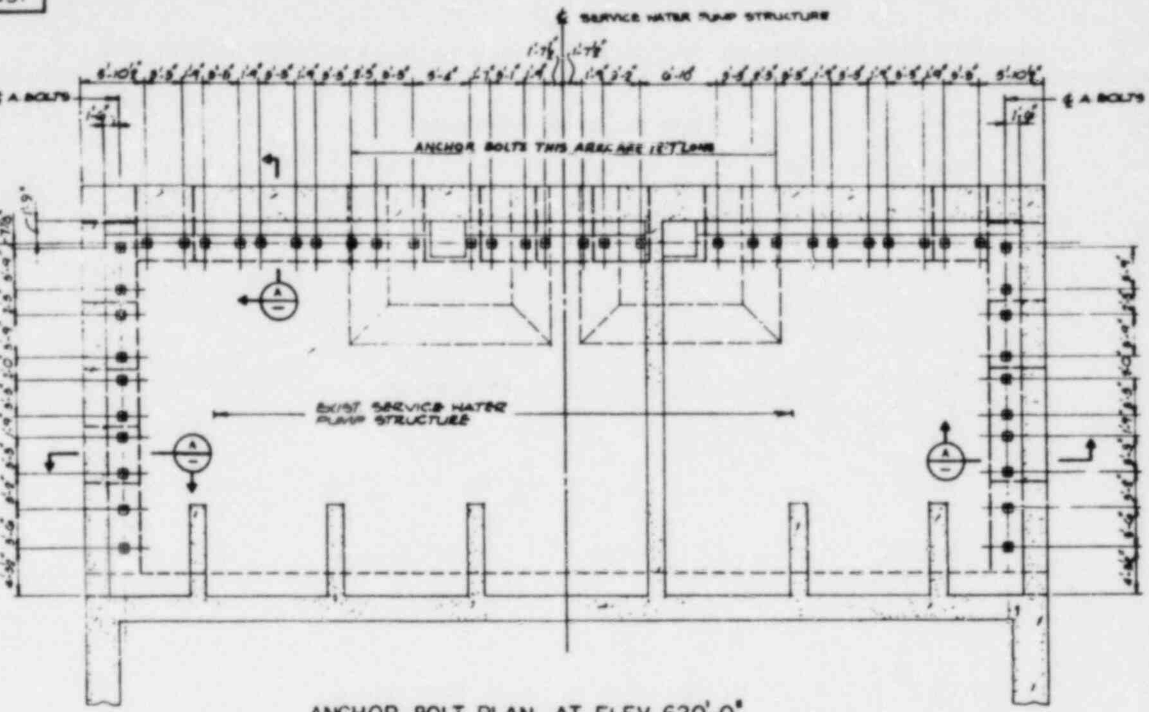
G-2611-15



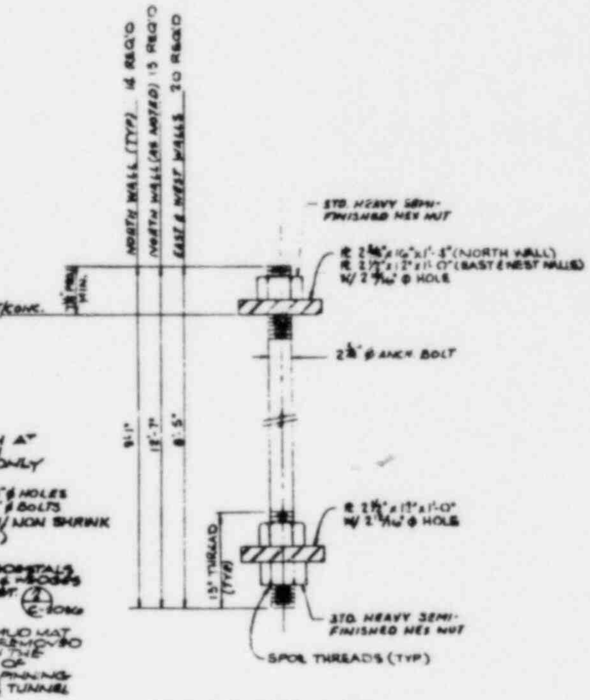
**SERVICE WATER PUMP STRUCTURE
TENSION TIES**

G-2611-14 FIG. SWP 14

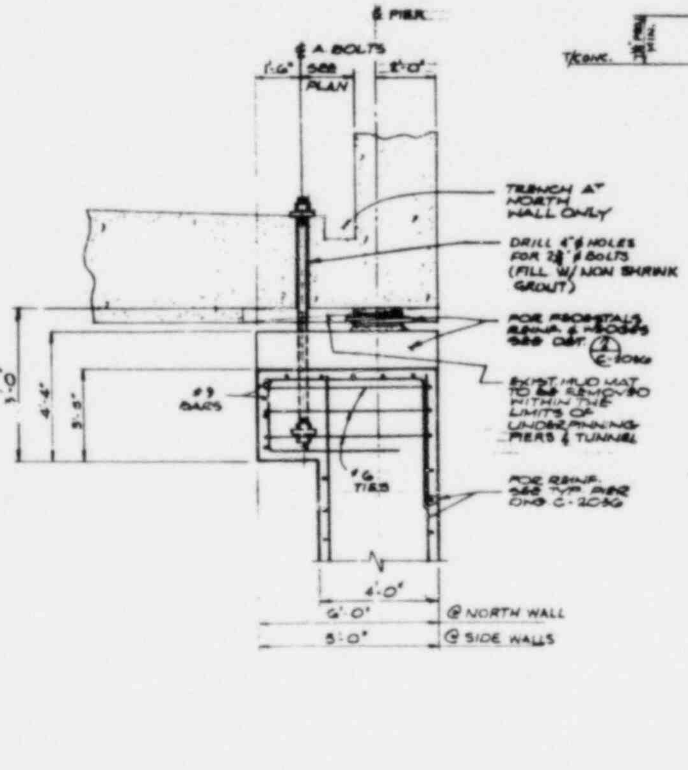
C-2037



ANCHOR BOLT PLAN AT ELEV. 620'-0"

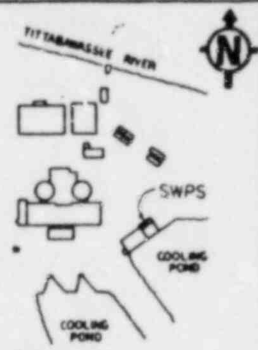


ANCHOR BOLT DETAIL
N.T.S.



TYP SECTION (A / C-2033) SEE DWG. C-2033

8 | 7 | 6 | 5



KEY PLAN

GENERAL NOTES

CONTINUED FROM DWG C-2036

13. CONCRETE FOR SOLDIER PILES SHALL BE:
    A. CLASS E-4 BOTTOM 6 FT.
    B. CLASS L-F REMAINDER

14. CONCRETE FOR SOIL REPLACEMENT AND FOR THE BRACED EXCAVATION ADJACENT TO THE SWPS & CWIS SHALL BE CLASS L-F.

15. BEARING PLATES, WEDGES, BRACING, SOLDIER PILES AND ALL OTHER MISCELLANEOUS METAL SHALL BE ASTM A-36 FURNISHED & DELIVERED IN ACCORDANCE WITH SPECIFICATION C-233 OR C-33

16. ANCHOR BOLTS SHALL BE ASTM A 540 GRADE B 21 CLASS 4, FURNISHED & DELIVERED IN ACCORDANCE WITH SPECIFICATION C-255, AND STRESSED TO 2 KSI (± 1 KSI)

17. FOR SEALING OF EXPANSION JOINT SEE DRAWING C-2036

18. ANCHOR BOLT LOCATIONS MAY BE VARIED BY ± 2" IN ORDER TO AVOID CUTTING REBAR

19. ANCHOR BOLTS, PLATES & NUTS SHALL BE GALVANIZED IN ACCORDANCE WITH SPECIFICATION C-255

20. ANCHOR BOLTS SHALL MEET THE FOLLOWING HARDNESS REQUIREMENTS:

	MIN.	MAX.
BRINELL	,269	331

21. THE NON-Q MATERIALS FURNISHED BY THE SUBCONTRACTOR SHALL INCLUDE THE FOLLOWING:
    A.) ALL MATERIAL REQUIRED FOR INSTALLATION OF VENTILATION & LIGHTING IN TEMPORARY ACCESS SHAFTS & TUNNEL.
    B.) ALL MATERIAL REQUIRED TO MEET THE TEMPORARY DEWATERING REQUIREMENTS AS SET FORTH IN SPEC. 7220-C-194 (Q)
    C.) MISC. MATERIALS AS FOLLOWS:
        1.) CHAIRS, BOLSTERS, SPACERS & TIE WIRE REQUIRED FOR REBAR PLACEMENT.
        2.) TIMBER PLANKS, AND WOOD FOR TEMPORARY FORMING OF PIERS.
        3.) JACKS & JACKING EQUIPMENT.
        4.) OBSERVATION WELL MATERIAL.

22. STARTER MIX SHALL BE CLASS E-STARTER IN ACCORDANCE WITH SPECIFICATION C-230

C-2037

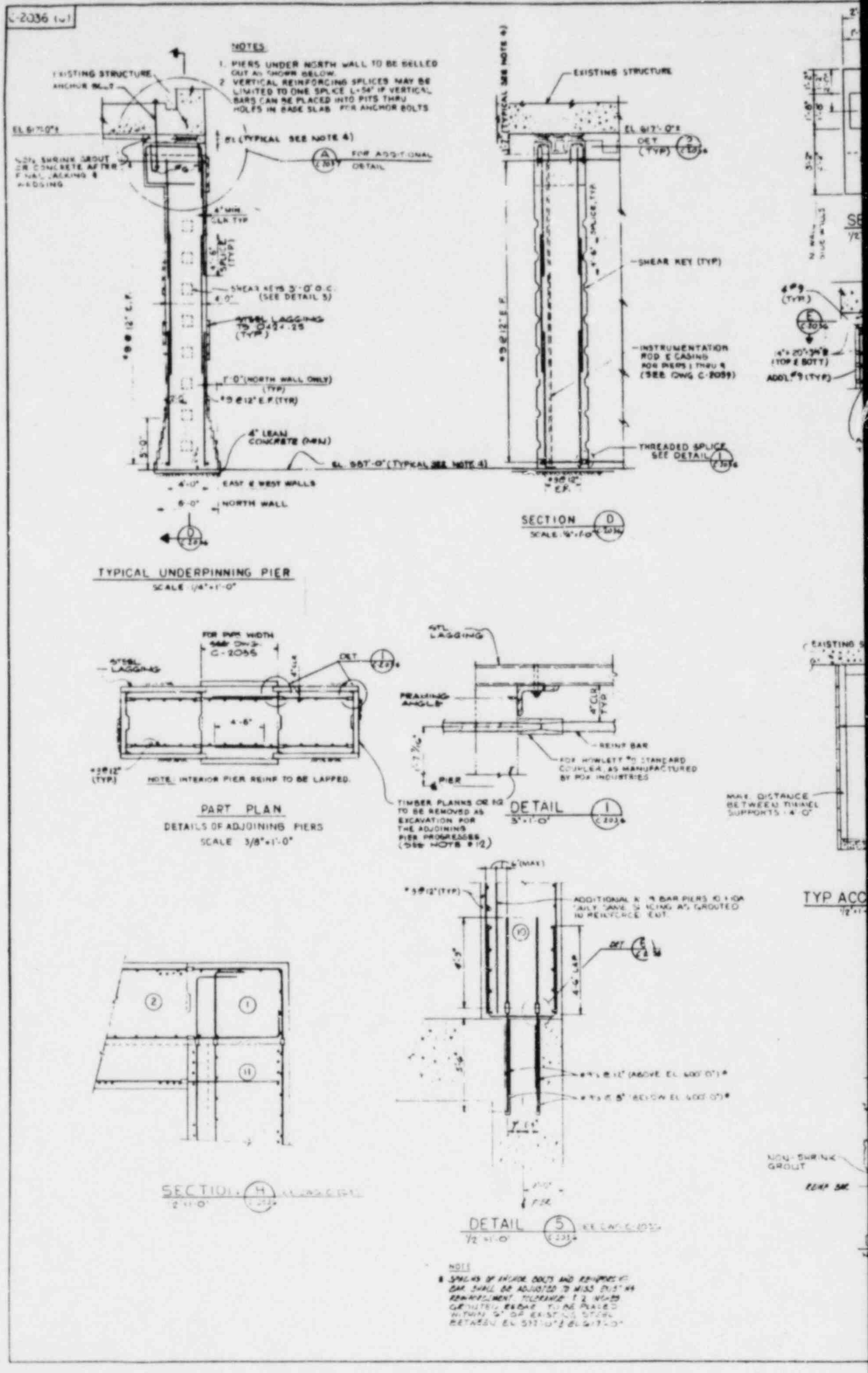
CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SWPS UNDERPINNING
SECT.'S & DET.'S SHEET 2

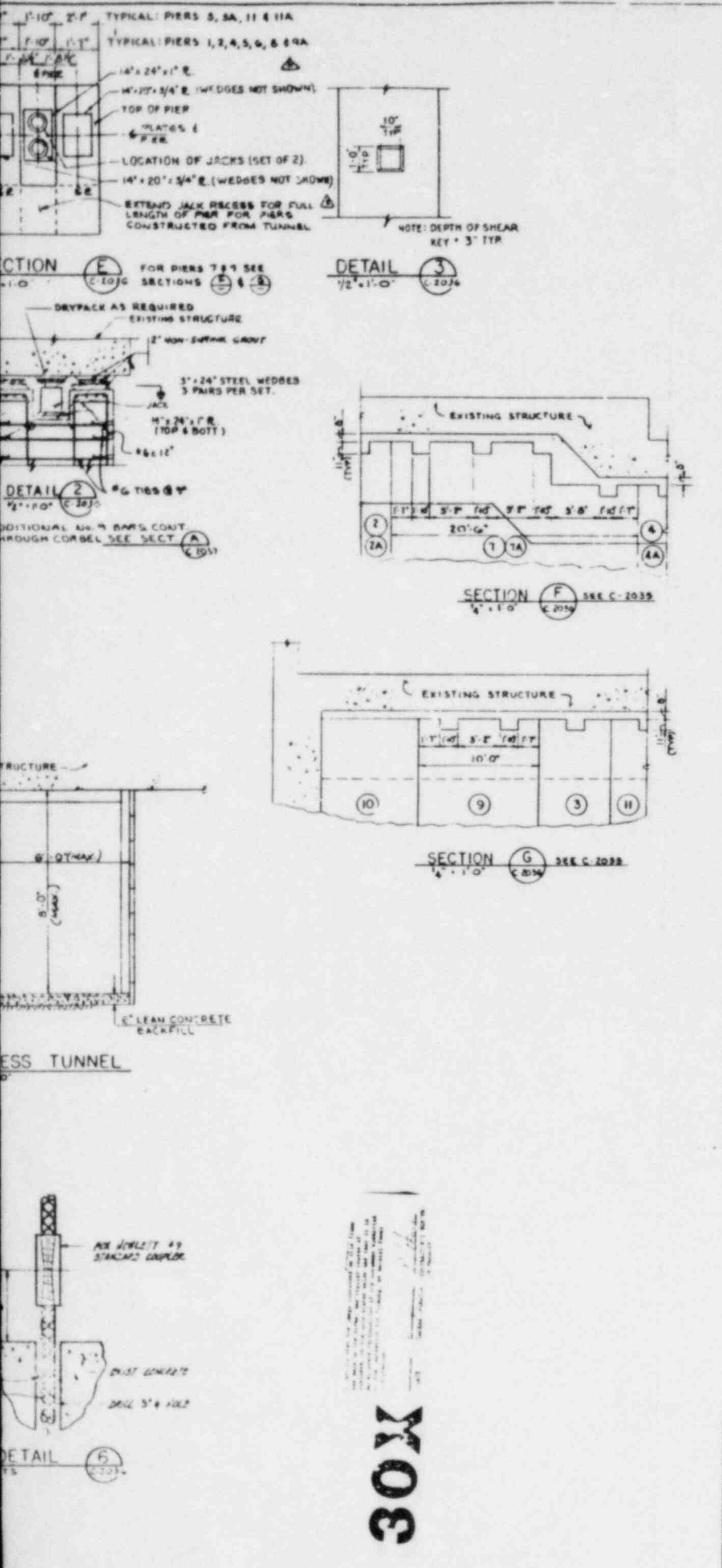
FIG SWP-13

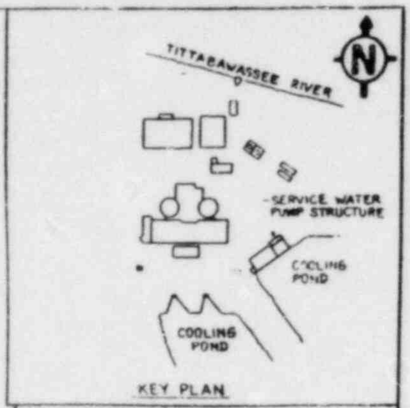
30X

G-2611-13



C-2036
NOTES
1. PIERS UNDER NORTH WALL TO BE BELLED OUT AS SHOWN BELOW.
2. VERTICAL REINFORCING SPLICES MAY BE LIMITED TO ONE SPLICE L=54" IF VERTICAL BARS CAN BE PLACED INTO PITS THRU HOLES IN BASE SLAB FOR ANCHOR BOLTS
EXISTING STRUCTURE
ANCHOR BOLT
NON-SHRINK GROUT OR CONCRETE AFTER FINAL JACKING & WEDGING
FOR ADDITIONAL DETAIL
SHEAR KEYS 3'-0" O.C. (SEE DETAIL 5)
STEEL LAGGING
1'-0" (NORTH WALL ONLY) (TYP)
4" LEAN CONCRETE (MIN)
EL. 587'-0" (TYPICAL SEE NOTE 4)
EAST & WEST WALLS
NORTH WALL
TYPICAL UNDERPINNING PIER
SCALE 1/4"=1'-0"
EXISTING STRUCTURE
SHEAR KEY (TYP)
INSTRUMENTATION ROD & CASING FOR PIERS 1 THRU 9 (SEE DWG C-2039)
THREADED SPLICE SEE DETAIL 1
SECTION D
PART PLAN
DETAILS OF ADJOINING PIERS
SCALE 3/8"=1'-0"
NOTE: INTERIOR PIER REINF TO BE LAPPED.
TIMBER PLANKS OR 1/2 TO BE REMOVED AS EXCAVATION FOR THE ADJOINING PIER PROGRESSES (SEE NOTE #12)
STL LAGGING
FRAMING ANGLE
REINF BAR
FOX HOWLETT #9 STANDARD COUPLER AS MANUFACTURED BY FOX INDUSTRIES
PIER
DETAIL 1
3"=1'-0"
MAX. DISTANCE BETWEEN TIMBER SUPPORTS = 4'-0"
TYP ACC
SECTION H
DETAIL 5
NON-SHRINK GROUT
REINF BAR
NOTE





GENERAL NOTES

1. ALL UNDERPINNING WORK SHALL BE PERFORMED UNDER SUB-CONTRACT C-194
2. ALL CONCRETE, GROUT AND DRYPACK SHALL BE FURNISHED AND DELIVERED IN ACCORDANCE WITH SPECIFICATON C-230 AND AS FOLLOWS
   a.) CONCRETE FOR UNDERPINNING PIERS SHALL BE CLASS E-4.
   b) LEAN CONCRETE BACKFILL SHALL BE CLASS A-1.
   c.) MATERIAL FOR SEALING DEWATERING WELLS AND OBSERVATION WELLS SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 100 PSI
3. TESTING OF CONCRETE AND GROUT SHALL BE IN ACCORDANCE WITH SPECIFICATION C-208 AND AS FOLLOWS:
   a. PIER CONCRETE AND GROUT SHALL HAVE ATTAINED A MINIMUM COMPRESSIVE STRENGTH OF 2000 PSI PRIOR TO THE IMPOSITION OF STAGE I JACKING LOADS
4. THE ELEVATIONS INDICATED FOR PIER TOP & BOTTOM ARE AT TIME OF CONCRETE PLACEMENT ONLY.
5. REINFORCING STEEL SHALL BE FURNISHED AND DELIVERED IN ACCORDANCE WITH THE REQUIREMENTS OF SPECIFICATION G-33 AND FABRICATED IN ACCORDANCE WITH SPECIFICATION C-33
6. THREADED COUPLERS AND ETHAFOAM SHALL BE FURNISHED AND DELIVERED IN ACCORDANCE WITH SPECIFICATION 7220-G-99
7. CONCRETE EXPANSION ANCHORS FOR TEMPORARY LAGGING & BRACING SHALL CONFORM TO SPECIFICATION C-305
8. SETTLEMENT MONITORING OF THE STRUCTURE SHALL BE IN ACCORDANCE WITH DRAWINGS C-2003 & C-2004
9. CRACK MONITORING OF THE STRUCTURE SHALL BE IN ACCORDANCE WITH DRAWING C-2040
10. MONITORING OF PIER AND BUILDING INSTRUMENTATION SHALL BE IN ACCORDANCE WITH THE PROVISIONS OF DRAWING C-2039.
11. STEEL BOLTING MATERIAL FOR TEMPORARY LAGGING AND BRACING SHALL BE ASTM A 307 & BE FURNISHED IN ACCORDANCE W/SPEC C-233 OR G-33
12. TIMBER BOARDS MAY BE USED FOR TEMPORARY SHEETING BETWEEN THE INDIVIDUAL UNDERPINNING PIER FACES WHERE THEY WILL BE REMOVED AS CONSTRUCTION PROGRESSES. IF USED, TIMBER BOARDS SHALL BE SOUTHERN PINE NO 2. DENSE 55

FOR CONTINUATION OF GENERAL NOTES SEE DWG. C-2037

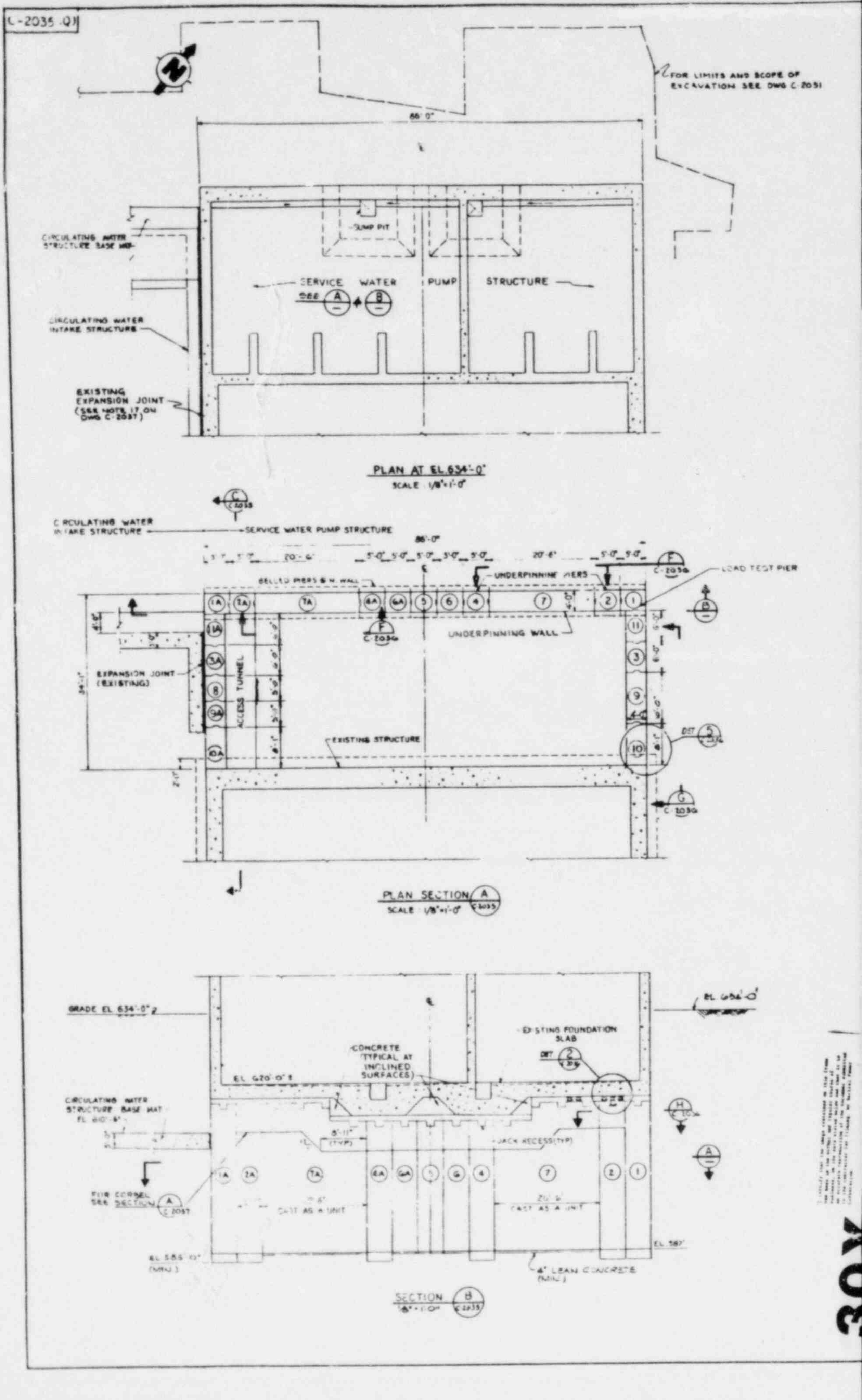
C-2036

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SWPS UNDERPINNING
SECT.'S & DET.'S SHEET 1

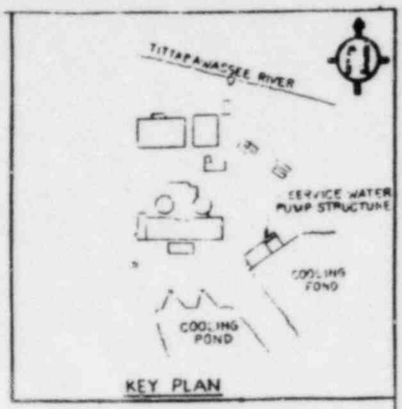
FIG SWP-12

G-2511-12

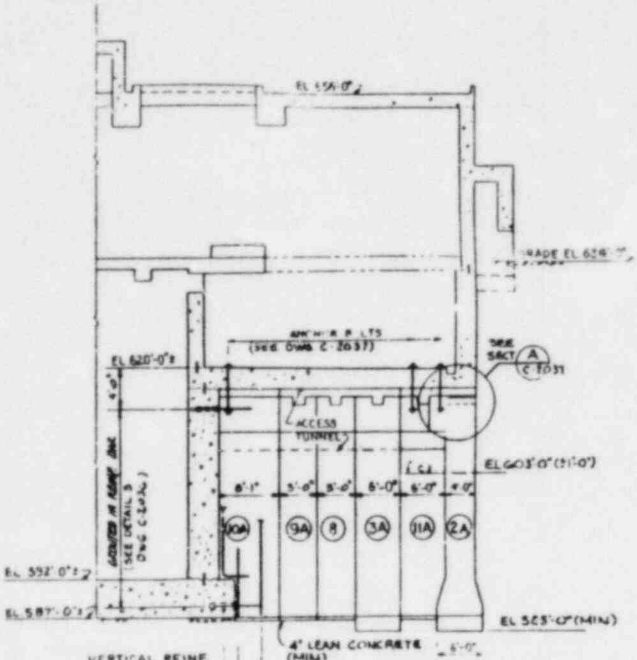


C-2035-Q1
FOR LIMITS AND SCOPE OF EXCAVATION SEE DWG C-2051
86'-0"
CIRCULATING WATER STRUCTURE BASE MAT
SUMP PIT
SERVICE WATER PUMP STRUCTURE
SEE A & B
CIRCULATING WATER INTAKE STRUCTURE
EXISTING EXPANSION JOINT (SEE NOTE 17 ON DWG C-2037)
PLAN AT EL.634'-0"
SCALE 1/8"=1'-0"
CIRCULATING WATER INTAKE STRUCTURE
SERVICE WATER PUMP STRUCTURE
BELLED PIERS & N. WALL
UNDERPINNING PIERS
LOAD TEST PIER
UNDERPINNING WALL
EXPANSION JOINT (EXISTING)
ACCESS TUNNEL
EXISTING STRUCTURE
PLAN SECTION A C-2035
SCALE 1/8"=1'-0"
GRADE EL 634'-0"±
EL 634'-0"
EXISTING FOUNDATION SLAB
CONCRETE (TYPICAL AT INCLINED SURFACES)
EL 620'-0"±
CIRCULATING WATER STRUCTURE BASE MAT
JACK RECESS (TYP)
FOR CORBEL SEE SECTION A C-2037
CAST AS A UNIT
EL 586'-0" (MIN.)
EL 587'
4" LEAN CONCRETE (MIN.)
SECTION B C-2035
1/8"=1'-0"

30W



KEY PLAN



SECTION C
3/8" = 1'-0" C-2031

NOTE: SHEAR KEYS BETWEEN PIERS NOT SHOWN. SEE DWG. C-2036

JACKING LOAD TABULATION (TOLERANCE: ±5%)

SEQUENCE NUMBER	LENGTH (L)	JACK LOAD (STAGE 1)	JACK LOAD (FINAL STAGE)	NUMBER OF JACK SETS PER PIER
1* & 1A	5'-0"	145 KIPS	204 KIPS	1
2 & 2A	5'-0"	145 KIPS	204 KIPS	1
3	6'-0	175 KIPS	150 KIPS	1
3A	6'-0	175 KIPS	150 KIPS	1
4 & 4A	5'-0	145 KIPS	204 KIPS	1
5	5'-1"	145 KIPS	204 KIPS	1
6 & 6A	5'-0"	145 KIPS	204 KIPS	1
7 & 7A	20'-6	596 KIPS	834 KIPS	4
8	5'-0"	70 KIPS	150 KIPS	1
9	10'-0	140 KIPS	300 KIPS	2
9A	5'-0"	70 KIPS	150 KIPS	1
10 & 10A	8'-1"	-	WEDGE ONLY	0
11	6'-0"	-	80 KIPS	1
11A	6'-0"	-	80 KIPS	1

* SEE APPENDIX E OF SPECIFICATION C-194 FOR LOAD TEST CRITERIA

LEGEND

(5) UNDERPINNING PIERS CONSTRUCTION SEQUENCE NUMBER
NEW UNDERPINNING PIERS
EXISTING STRUCTURE
C.J. CONSTRUCTION JOINT

C-2035

CONSUMERS POWER COMPANY
MIDLAND UNITS 1 AND 2

SWPS UNDERPINNING
PLAN & SECTIONS

FIG SWP-11

G-2611-1