MEETING WITH NRC ON MIDLAND PLANT FILL STATUS AND RESOLUTION

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1.0 INTRODUCTION

On August 22, 1978, Consumers Power Company notified the NRC Resident Inspector that there was larger than expected settlement of the diesel generator building foundation. On September 7, 1978 the NRC was notified that it was considered reportable. The first 50.55(e) Interim Report was on September 29, 1978 with the latest Interim Report submitted on June 25, 1979. On March 21, 1979 a 50.54(f) request was issued by H R Denton. Consumers Power Company replied on April 24, 1979 and revisions were submitted on May 31, 1979 and July 9, 1979. Meetings with the Staff and Inspection and Enforcement have taken place at Glen Ellyn and at the site. In addition we have received several questions on this subject from the Staff.

Initially, in September 1978 there were several options considered to correct the problems and these included modified mat, preloading, a combination of these, underpinning and removal and replacement of the structure and soil. From that time to the present, there have been many meetings between Consumers Power Company, Bechtel and the Consultants. Based upon these meetings, a decision has been made to delete the chemical grout option and to go to a site dewatering concept. This is discussed in more detail later.

2.0 PRESENT STATUS OF SITE INVESTIGATIONS

2.1 Meetings with Consultants and Options Discussed

The investigative program conducted to date has included: meetings with consultants to discuss the options for remedial action as noted in the introduction, discussions concerning the NRC findings, investigation of the various remedial actions and preparations of 50.55(e) Reports. As part of the investigative program, approximately 31 meetings have been held on this subject since September 1978. Various consultants participated in 11 of these meetings while the NRC attended approximately 8 of these meetings. Consumers Power Company attended a majority of the meetings also. During this time the causes of the problem were also investigated. Responses were also prepared to the 50.54(f) questions.

2.2 Investigative Program

The major portion of the investigative program was the investigation of the entire site soil conditions, which included approximately 161 soil borings, 14 dutch cone tests and 5 test pits. (Figures 1 and 2 show locations for soil borings and typical soil boring cross sections. Note: Sequential figure numbers have been added to show sequence in which they were presented at the July 18, 1979 meeting.) During this period of time, an investigative program was also launched to monitor all cracks in major Class I structures associated with plant area fill. Strain gauges were also utilized. (See Figure 3 on typical section through Service Water Building.)

It should also be noted that an independent firm Goldberg-Zoino-Dunncliff & Associates (GZD) was utilized for profiling pipes to determine settlement. (See Figure 4 on pipe profiling typical section.) A rabbit check of electrical duct work was also utilized for assuring continuity. (See Figure 5.) During this period of time the frequency of settlement monitoring of the Diesel Generator Structure was also increased.

2.3 Settlement

It is very important to note that the Diesel Generator Building is the only Class I structure that was observed to have excessive settlement; however, as a result of the boring program we did find some areas with questionable soils beneath the structures. These areas were: Diesel Generator Building, Service Water Building overhang portion only, Auxiliary Building electrical penetration rooms and Feedwater Isolation Valve Pits. To correct the problems with the Diesel Generator Building it was decided to preload to consolidate the soils and accelerate the total settlement. (See Figure 6 on overall site layout of the power block.) Figure 7 shows the settlement of the four Diesel Generator pedestals vs the application of the surcharge. It can be seen that at the completion of the surcharge application the settlement appeared to be leveling out. Figures 8 and 9 show the settlement for the Diesel Generator Building. These figures are profiles looking north and looking in the east-west direction. Figure 10 shows settlement vs log time. Figure 11 highlights the elevation contours and differential settlement between the northwest and southeast parts of the structure. Figure 12 represents the various utilities beneath the building. It should be noted that the Diesel Generator Building was initially partially hung up on these utilities and that after they were freed the building settled in a more or less uniform fashion over the last few months. Figure 13 shows the location and types of instrumentation utilized to monitor the settlement of the building and instruments that were utilized during the preload program to determine when the pore pressure had decreased to normal.

2.4 Recent Revisions

For the areas of questionable soil discussed previously it has been decided to provide vertical support for the Service Water Building Overhang and to improve the support of the Electrical Penetration areas and Feedwater Isolation Valve Pits.

The investigative program pointed out that certain sand areas were not adequately compacted. This presented a potential for liquefaction under the action of SSE. The initial remedial action plan was to chemically grout the loose sands. After further review of this remedial action, it appeared that while the grouting would sufficiently remedy the situation, it would be difficult to prove that all areas had been uniformly grouted. It was noted that there were discontinuous sand lens and fine grain sands and, furthermore, there were access problems for grouting. Underpinning of the Diesel Generator Building as another remedial action presented problems with shoring, support of utilities and schedule. It was decided recently that better remedial action would be to dewater the entire site on a permanent basis. This will provide a conservative solution since any liquefaction questions would be eliminated in any site area in the power block whether or not it was determined that there was a potential for liquefaction. More details of the basic plan discussed above are described in subsequent sections.

3.0 REMEDIAL WORK IN PROGRESS OR PLANNED

3.1 Diesel Generator Structures

The diesel generator building is a box-shaped structure. (See Figure 14.) Its main purpose is to provide a housing for the four emergency diesel generators. Structural walls are very rigid and are supported on strip footings. The building and the generator pedestal are founded on approximately 30 feet of fill. During the summer of 1978, settlements more than anticipated values were observed and a detailed soil investigation was conducted. The backfill was found to consist of soft to very stiff clay with pockets and layers of very loose to dense sand backfill. The conclusion of the investigation was that the fill was not adequately compacted. Based upon the recommendation of our soil consultants, Professors Peck and Hendron, the remedial measure chosen was to preload the existing backfill by layers of sand surcharge.

Figure 15 shows in plan the extent of sand surcharge. The surcharge was gradually applied in steps. To date, the backfill under the diesel building is subjected to 20 feet of sand surcharge. Figure 16 shows a cross-section of the building and the surcharge. The surcharge produces stresses in the fill greater than the amount the fill would experience when the structure is operational. This surcharge will remain until excess pore pressures are essentially dissipated and the rate of residual settlement becomes small and can be predicted conservatively by extrapolation.

The preload consolidates soft areas of clay fill; however, will not significantly improve the quality of loose sands. The potential of liquefaction of these sands and aerial dewatering of the plant site as a remedial measure for this problem will be presented later in detail.

Figure 17 shows plan and cross-sectional elevations of a typical diesel generator pedestal. This is a reinforced concrete structure having a minimum compressive strength of 4000 psi. The fill beneath the pedestals have also consolidated resulting in differential settlement. Differential settlement of the pedestals will have no effect on alignment of the engine and the generator because they are both mounted on the same foundation. Furthermore, because of the enormous stiffness of the pedestal, no significant warping is expected and the top of the pedestal will generally lie within one plane. The diesel generator will be set in a level position irrespective of the amount of differential settlement between the corners of the pedestal. It will be achieved either by a suitable layer of grout on the pedestal or by chipping a few inches of the top concrete and refinishing it to the required level.

The machine itself has considerable tolerance limits for tilt and roll. DeLaval Turbines, the manufacturer of the diesel generator, stated that a 5° combined backward tilt and roll of the pedestal or a forward tilt of 1.4° and roll of 5° combined will not affect the performance of the engine and the generator. Furthermore, during the operation of the plant, if further differential settlement causes this tolerance to be exceeded, the manufacturer states that the generators can be shimmed back to level position. In summarizing, for the diesel generator building the remedial work of preload is in progress and dewatering of site is being planned for implementation soon. No further remedial work on the pedestal than that mentioned above is anticipated.

3.2 Service Water Pump Structure

The service water pump structure is located in the southeast end of the power block area adjacent to the cooling pond. (See Figure 6.) Figure 18 shows a plan view of the structure. The cooling pond is on the southern side. The major portion of the structure is founded on natural soil material except for the northern portion which is founded on fill. Figure 19 shows a cross-sectional view of the structure. As mentioned earlier, the northern section, which is cantilevered off the main building, is founded on backfill material. As a follow-up to the investigation of all Class I structures on fill, several borings were taken in this area. The borings indicated that the backfill consists of soft to very stiff clay and loose to very dense sand. The conclusion was that some areas of the fill material under the northern part of the structure were not sufficiently compacted. However, no significant settlement of the structure has been noted. The reason for this is that the existing dead loads from this portion are being partially supported by the rest of the structures through cantilever action. The remedial measure chosen is to support the north wall on piles driven to hard glacial till. The choice of piles is an economical and expedient solution with minimal impact on the schedule.

Figure 20 shows in plan the layout of piles. A total of 16 piles is planned at this time. The piles will have a capacity of 100 tons and are designed as bearing piles to carry only vertical load. The piles will be pipe piles filled with concrete. They will be predrilled through the fill and driven into the glacial till. The length of piles is expected to be approximately 50 feet.

Figure 21 shows the method of transferring vertical load from the wall to the piles by a system of reinforced concrete corbels.

As shown in Figure 22 the concrete corbels will be anchored to the wall by a system of anchor bolts. The pipe piles in turn will be jacked against the corbels to effect the transfer of load.

A test pile will be load tested to determine its capacity.

3.3 Tank Farm

Figure 23 shows the tank farm in plan. There are two borated water storage tanks (BWST), a utility tank and a primary storage tank. Of these, only the BWSTs are safety-related. Each BWST has a capacity of 500,000 gallons and is 52 feet in diameter and 32 feet in height.

As shown in Figure 24, a short concrete ring girder foundation with a strip footing is provided for each BWST. The tank is supported on the ring girder and the soil within the foundation. The tank by itself is quite flexible.

Adjoining the ring girder for each tank is a small box-shaped structure called valve pit. This houses valves and other controls. At present, construction of ring girder and valve pits are complete and installation of piping is in progress. As a follow-up to the investigation of all Class I structures founded on fill, several borings and test pit examinations were completed in the tank farm area. The results of the investigation indicated that the tanks are supported on medium to very stiff clay backfill with occasional medium to very dense sand layers. The condition of the fill is suitable for the support of the tanks. To confirm this, the tanks will be constructed and filled with water in order to make a full-scale test of the foundation soil.

Figure 25 shows the layout of borated water lines entering the tank through the valve pit. The piping connections are being made to allow startup, flushing, filling and testing of the tank. Selected points on the piping between the BWSTs and the auxiliary building will be monitored for settlement during construction phase. Any differential settlement measured will be analyzed in accordance with established procedures.

In summary, the backfill material on which the BWSTs are founded is satisfactory and will be confirmed by a load test. Borated water lines will be monitored and evaluated for any differential settlements. Therefore, no remedial action is anticipated for these structures.

3.4 Diesel Oil Storage Tanks

The diesel oil storage tanks are located in the southeast end of the power block area and near the condensate storage tanks. There are 4 diesel oil storage tanks, each 12 feet in diameter and 44 feet in length. (See Figure 6.)

Figure 26 shows a cross-sectional view of a tank. There is six feet of earth covering each tank. The tanks are supported at three points anchored to concrete pedestals. The tanks are founded on backfill and results of the boring program indicated that the tanks are supported on medium to stiff sandy clay backfill. This soil condition is adequate to support the tanks. Moreover, the weight of the tanks is approximately equal to the fill that it replaced. In order to verify that the fill is satisfactory, these tanks have been filled with water and settlements are being monitored. It has been three months since the tanks have been filled with water and no appreciable settlements have been noted yet. Therefore, the backfill is considered adequate and no remedial measures are anticipated.

3.5 Underground Facilities

The underground facilities that will be discussed are Seismic Category I piping and electrical duct banks. Figure 6 shows safety-related piping, namely Service Water Lines, from the auxiliary building to the service water structure and from the diesel generator building to the service water structure, borated water.lines from the auxiliary building to BWST, and diesel oil lines from the diesel oil storage tanks to the diesel generator building. Also shown are electrical duct banks.

To evaluate the present condition of piping, a representative group of piping was selected and profiled by a Nold Aquaducer Profile Settlement Gauge. Figure 27 shows for illustrative purposes a plot of one of the lines profiled. All the pipes profiled were reanalyzed taking into account the measured differential settlement in accordance with the provisions of current codes. The analyses showed that the calculated stresses due to differential settlement are within allowable limits.

In summary, the pipes are very ductile and calculations show that there are no adverse effects of differential settlement. Therefore, no remedial work is anticipated with regards to buried piping.

Electrical Duct Banks

The duct banks are reinforced concrete elements enclosing PVC and rigid steel conduits, thus, providing voids for the cables. Continuity checks that are being performed by passing a rabbit through all the voids was discussed previously. This program establishes the fact that, to date, the duct banks are intact. Furthermore, the duct banks are reinforced with nominal amount of steel, therefore, possesses a considerable amount of ductility in bending.

As shown in Figure 28, a preliminary calculation indicated that a typical duct bank of 100 feet in length can undergo a maximum of 12" of central deflection in pure bending at ultimate load.

In summary, the integrity of the duct bank is established by passing a rabbit through during the construction phase and the duct bank by itself is ductile and can absorb a considerable amount of differential settlement without significant stresses. Therefore, no remedial measures are anticipated for duct banks.

3.6 Auxiliary Building and FW Valve Pits

The following describes the proposed remedial measures for the electrical penetration areas of the auxiliary building and the adjacent feedwater isolation valve pits. The objective of the remedial measures is to replace questionable bearing capacity as evidenced by soil sampling data. The design of the remedial measure has the objective of replacing the suspect soil bearing capacity with structural elements which extend from the existing concrete foundations to underlying undisturbed glacial till while minimizing disturbances to existing structures and construction operations. In order to accomplish this it is planned to utilize the structural capacity of the electrical penetration rooms to bridge over some of the questionable underlying materials by providing caissons at the extremities of the electrical penetration rooms. These caissons shall have sufficient capacity to support approximately one-half of the dead and live loads of the electrical penetration rooms with the remaining one-half being supported by the control tower. The proposed method for supporting the isolation valve

pits is to temporarily support them in place, totally undermine them by removing all materials to a depth at which undisturbed glacial till is encountered and filling the excavation with lean concrete.

The plan of attack for performing the work is as follows: (See Figures 29 thru 33)

1. Locally dewater the soil above the glacial till in the affected areas. It is essential that the loose granular soils be dewatered to permit excavation under the structures without significant loss of ground. The dewatering system shall be installed and the water drawn down in advance of any excavation. The dewatering system is a curtain cut-off type. A majority of the eductors will be installed from the lower basement of the turbine building. The discharge will be monitored for piped fines.

2. Temporarily support the isolation valve pit by the use of needle beams spanning between the buttress access shaft and turbine building foundation wall at the ground surface.

3. Excavate an access shaft adjacent to the isolation valve pits to a depth of approximately 7 feet below the bottom of these pits. The excavation would then proceed laterally as a drift until the excavation reaches the extreme edge of the electrical penetration area.

4. Install jacked caissons at this location utilizing the electrical penetration rooms foundation as the reaction. The jacked caisson method has been selected for the following reasons:

a. It will be possible to jack through loose sands and soft clays without excavating material from within the caisson thus preventing loss of ground from under the electrical penetration rooms, turbine building and buttress access shaft.

b. It is known that there are sizable concrete obstructions in the backfill area which will be encountered by the caissons. A caisson provides man-size working room for demolition of the concrete obstructions.

c. Likewise, the man-size working room of the caisson will permit direct excavation of highly compacted sands and/or clay as well as the glacial till (caissons penetrate the glacial till a minimum of 5 feet).

d. The caisson provides access for direct visual inspection of the glacial till for the initial determination of bearing capacity (final bearing capacity is by load test).

5. Concrete the caisson and load test same.

a. Load test one caisson under each electrical penetration room at 2.0 times design capacity.

b. Load test each caisson individually at 1.5 times design capacity.

c. Load test all caissons as a group at 1.0 times design capacity or 1/4" of vertical structure movement, whichever occurs first.

d. Upon completion of any tests the caissons are to be left in a prestressed state to prevent any settlement.

6. Install support of excavation system along the turbine building foundation wall and connect it to the access shaft and the jacked caissons. The jacked caissons which were previously installed under the electrical penetration rooms will temporarily act as support of excavation for the excavation under the isolation valve pit. The containment structure and the buttress access shaft form the remainder of the excavation enclosure under the isolation valve pit.

   The support of excavation system along the turbine wall foundation will also act to:

   a. Support the temporary additional load imposed on the foundation wall by the needle beams which support the isolation valve pit at the surface.

   b. Support the turbine building vertical loads within the zone of influence of the excavation under the isolation valve pit.

7. Excavate all material from underneath the isolation valve pits to a depth at which undisturbed glacial till is encountered.

8. Fill the excavation under the isolation valve pits with lean concrete backfill to within 7 feet of the existing foundation.

9. Place structural concrete in the drift under the isolation valve pits and the access area used for installation of caissons underneath the electrical penetration rooms.

10. Dry pack and transfer isolation valve pit load to the lean concrete backfill.

The design of the caisson is based upon a very conservative caisson tip pressure of 25 kips per square foot (KSF) for straight sided caissons. This provides a tip load intensity of approximately one-tenth that normally associated with jacked piling, and will bring the long term settlement into line with expected settlements of the balance of the auxiliary building. The bearing strata pressure is limited to 20 KSF for straight sided caisson. If the bottom of the jacked caissons are belled in the glacial till, the design tip pressure is reduced to 17.7 KSF. The bearing strata pressure associated with belled caissons is not relevant. The steel shells for the jacked caissons are neglected in calculating the structural capacity of the caisson.

The bearing pressure on the glacial till below the isolation valve pit is only nominally increased by the substitution of concrete for earthen fill.

3.7 Liquefaction Potential

Figure 34 presents a summary of the predominant fill condition (material type and density) below various category I structures supported on plant area fill. The figure shows the fill under all category I structures supported on plant fill consists of both sand and clay except for the borated water and diesel fuel tanks where the fill is predominantly clay. Liquefaction evaluations were made for the auxiliary building-control tower area, auxiliary building-railroad bay and the diesel generator building. No liquefaction analyses were made for other areas. The

liquefaction evaluation was based on experience at sites where liquefaction did or did not occur and access to pertinent information regarding earthquake magnitude, distance from the source, ground surface acceleration were either known or possible to estimate.

Figure 35 is a plot of the cyclic shear stress ratio causing liquefaction versus the standard penetration blowcount corrected to an equivalent overburden pressure of 2,000 pounds per square foot. The figure correlates the shear stress causing liquefaction in the field and the penetration resistance of the sand. Utilizing this figure, if the standard penetration resistance is known at a certain site along with other pertinent information regarding the soil column, the structure and ground surface acceleration, a point can be plotted on this graph. The horizontal coordinate of this point will be the standard penetration resistance after correction to an equivalent overburden pressure of 2,000 psf and the vertical coordinate will be the shear stress ratio induced during the earthquake. If the point falls below the line, this will indicate liquefaction would not occur. On the other hand, if the point plots above the line, this would indicate that liquefaction is possible. This can be illustrated in terms of factor safety as follows.

$$\text{Factor of safety} = \frac{\text{cyclic shear stress causing liquefaction}}{\text{induced cyclic shear stress}}$$

The liquefaction evaluation was based on ground water table at elevation 627 and ground surface acceleration of 0.12g and did account for surcharge from the structure. It is noted that figure 35 is based on data for magnitude 7.5 earthquake which constitutes a very conservative basis for evaluation of liquefaction at Midland.

Utilizing this information the line representing a safety factor of 1.5 has been calculated and superimposed upon the standard penetration blowcount versus depth for the northwest and northeast areas of the diesel generator building as shown in Figure 36 and 37. The figure also shows the line representing a factor of safety of 1.1. It is seen from Figure 36 that a good number of the standard penetration blowcounts are less than those required for the acceptable factor safety of 1.5. Evaluation of the sands in the northwest area of the building indicates that some of these loose sands may be connected. Figure 37 shows that the great majority of the penetration tests indicate a safety factor well in excess of 1.5 with the exception of three cases below 1.5.

Figure 38 is a similar plot for the auxiliary building railroad bay showing that all except a few of the standard penetrations values are well in excess of the required safety factor of 1.5. Some blowcounts in borings AX-1 and AX-10 between elevations (619-623) show a factor of safety slightly below 1.5, but these occur within a limited thickness and the neighboring boring AX-2 indicate much higher factors of safety within the same depth range.

Figure 39A illustrates that the standard penetration blowcounts from boring AX-9, AX-6 and AX-18 under the control tower indicate a factor of safety in excess of the required 1.5 in all cases. Figures 39B, C and D show the relationship between standard penetration resistance, relative density, and effective overburden pressure for the three areas indicated.

In conclusion, liquefaction analyses show that there could be a liquefaction problem at the diesel generator building. Borings also indicate liquefaction is very unlikely in the railroad bay and that there is no liquefaction problem in the control tower area.

In order to eliminate liquefaction questions anywhere at the site in Midland, a general dewatering scheme has been adopted. In this scheme the ground water table will be lowered to the approximate elevation of 600.

Settlement Due To Earthquake Shaking

With elimination of liquefaction potential the remaining factor to be considered in settlement of sand due to ground shaking. Analysis was conducted on the basis of studies by Seed and Silver (1972) and Finn and Byrne (1975) which considered relative density, number of earthquake cycles, ground surface acceleration level, thickness of the sand, effects of multi-directional shaking, and the presence of the structures. Relative density was evaluated on the basis of Gibbs and Holtz relationships. The number of earthquake cycles were taken as 10 in the Seed and Silver analysis. Finn and Byrne analysis was based on the recorded El-Centro earthquake. Acceleration level was taken as 0.12g for the SSE and 0.06g for the OBE. Thickness of the sands were based on the soil borings. Multi-directional shaking effects were counted for the multiplying the calculated uni-directional settlements by a factor of 2.5. The structure was accounted for as if it was a uniform surcharge.

Preliminary analysis based on these parameters indicated a settlement range of ½ inch to 1 inch for the diesel generator building area. It is noted that these estimates are conservative since they are based on the assumption that the sand is dry. Because the sand will be moist, the presence of capillary force will reduce actual settlements below those predicted.

3.8 Dewatering

Figure 40 is a Plan View of Area Dewatering System. The soil as described before by others generally consists of sand and or clay fill placed on the original sand or clay strata. The original sand generally extends from elevation 570 to elevation 600 with clay beneath the sand - though in a few areas the underlying clay extends to the original ground surface.

The present ground water level is about elevation 627 - the cooling pond level.

As part of the original dike construction, an impervious cutoff wall has been installed around the West, North and East sides of the area. The cut-off wall, a slurry trench or clay core, extends into the original clay till. The sources of recharge for ground water within the Q listed area are rain-fall and the cooling pond water from the South side of the area.

The coefficient of permeability of the soil as determined from the initial pumping test conducted in Auxiliary Building area is less than 0.007 feet per minute. Additional data about the permeability of the soil and total yield will be obtained during temporary dewatering of the Valve Pits and Electrical Penetration Rooms. Also there are considerable grain size data available from the extensive boring program that has been carried out at the site.

The present conception is to enclose the Q listed area with a permanent exterior dewatering system. The dewatering system would consist of

submersible deepwells that would extend to the original clay till. Approximately 200 to 300 deepwells would be installed. The number required to maintain the ground water at the desired level would be operated and the remainder would be redundant. There would be sufficient redundancy to provide for interruption of parts of the system. Also there will be 100% standby generation availability.

The pumps would be wired electrically such that they are staggared and sectioned so that one interruption does not affect a continuous length of the dewatering system.

The permanent interior dewatering system would be used to mop up ground water remaining within the area enclosed by the perimeter dewatering system. The wells would be pumped as required to remove ground water that collects within the exterior perimeter system because of the recharge from rain, shut down etc.

The ground water removed would be monitored to assure that no fines are being removed from the soil.

After an initial pumping period of about six months the basin that is dewatered should be large enough that the permanent dewatering system could be down completely from one to two weeks before a significant rise in the water level within the dewatered area would occur. The principal source of recharge is the cooling pond and the rate the ground water flows through the soil from the pond is low.

Piezometers would be located at key points to monitor the ground water level and alert the plant when the ground water has risen above a predetermined elevation.

Figure 41 is a north-south section through the area to be dewatered. The deepwells would extend to the original clay till, they would be spaced close enough to cut off the flow of water into and remove the water from within the Q listed area.

Figure 42 indicates that the dewatering system would be buried below the frost depth. The necessary disconnections would be provided to permit screening the deepwells. In area of heavy traffic a manhole would be provided for access to the deepwells.

The capacities of the well screens (6" diameter) are considerably in excess of the anticipated equilibrium flow of 1 to 10 gpm per well. The well screen diameter, 6 inches, is necessary to provide the clearance required for the submersible pump.

The well screens would extend the full depth of the soil to be dewatered and they would be encased in a select sand filter for their full depths.

Figure 43 shows that for areas where there is no objection to having a slight protrusion above the ground surface, pitless adaptors would be used to provide access to the wells and pumps instead of manholes.

Figure 44 is a sketch of an interior permanent deepwell. Smaller diameter wells would be used to remove the water perched within the Q listed area. These wells would be pumped initially and occasionally therefore as required.

4.0 ANALYTICAL INVESTIGATION

The following is a brief overview of:

4.1 Structural Investigation

4.2 Seismic Analysis

4.3 Structural Adequacy With Respect to PSAR, FSAR, Etc

Structural analysis is defined as static analysis when the various loadings are applied to the structure as static loads and then the design forces are determined for sizing reinforcing steel. Whereas, seismic analysis is defined as the dynamic analysis that is used to determine structural response.

Figure 45 shows the various items that were reviewed in the structural investigation. For the diesel generator building, the original design was governed by tornado missile impact and a 3 psi vacuum loading. The seismic response for this structure was relatively small. As an indication, the calculated shear stress in the east-west direction was 40 psi and 25 psi in the north-south direction.

The new analyses that are being performed will involve using a finite element model to investigate the variable foundation properties. Up to now, the maximum cracking observed in this structure has been approximately 30 mils and this occurred in the short walls from the vertical duct bank loadings during construction.

The structural investigation of the service water pump structure revealed the following: The original design for this structure was governed by tornado missile impact and the 3 psi vacuum loading. Seismic response was relatively low with a calculated shear stress in the major walls of about 20 psi. The new analyses that will be used for this structure will involve

conventional techniques considering the walls and slabs with the piling that will be used to support the portion of the structure on top of fill. Cracking in this structure to date has not exceeded 20 mils. This cracking occurred in the walls and the roof. Up to now there has been no detectable settlement for this structure.

The structural investigation of the auxiliary building penetration areas revealed the following: The original design was governed by the safe shutdown earthquake and the pipe break. The original, analysis was conservative since it was based on a system of beams and columns to simulate the large walls and floors. As far as the seismic response, the structure was near capacity using this original model. A new analysis is being performed which will involve a finite element analysis of the structure, this will include the caissons which will be used for end support. In this structure the cracking as measured to date has not exceeded 15 mils. This has occurred in the walls and there has been no detectable settlement.

For a review of the seismic analyses, refer to Figure 46. A general review is as follows: The ground response spectra is presented in the FSAR and this is based on an OBE of .06 g's and an SSE of .12 g's. Stick mass models with foundation springs were used. Material damping values are presented in the FSAR; modal damping was limited to 10% except for rigid body modes. The analysis technique used both the response spectrum and the time history methods.

For the diesel generator building the original analysis used a shear wave velocity of 1,360 fps. One analysis was performed and equipment response spectra was widened by ± 15 percent. A new analysis has been completed using

a lower limit shear wave velocity of 500 fps. The new spectra will envelop both the 500 and the 1,360 fps analyses values.

Referring to Figure 47, the seismic analysis for the service water building involved an original analysis which used 1,360 fps as a base case. Then the foundation shear modulus was varied by $\pm$ 50 percent. These three analyses were used to generate equipment response spectra and the spectra used was the envelop of all three. A new seismic analysis is being done which will use a shear wave velocity of 1,360 fps. The piling will be modeled in this analyses, but only to resist loads in the vertical direction. Torsion will also be considered in this model. The equipment will tnen be reexamined for the response spectra from both the original and the new analyses.

For the auxiliary building, including the control tower and electrical penetration areas, the original analysis used composite foundation springs with the equipment response spectra widened by $\pm$ 15 percent. The composite springs were used to represent different foundation materials for various parts of the structure. A new analyses will be performed including the caissons under the electrical penetration areas. The equipment response spectra will be widened by 15 percent and equipment will be checked, if this response spectra is greater than the original in any frequency range.

The different types of loads are shown in Figure 48. The first types of loads are primary loads. This type of load results in stress. As an example, the most critical type of loads would be what are considered mechanical loads. These would be dead load, pressure, wind. All these types of loads have a constantly applied force.

The next type of load, but of lesser severity, would be seismic inertia load, however, these are of a short duration.

The third type of load of lesser severity would be missile impact or pipe rupture loads. These types of loads have a limited energy input.

The next classification of load would involve what is known as secondary loads. This term is quite common in ASME codes. This type of load merely results in strain. They can result from internal self-constraint. As an example, if a pressure vessel has the bottom restrained, bending moments would develop which would be secondary in nature because they are due to internal self-constraint.

Seismic displacements in piping systems would be of a secondary nature since different support points would only move a set amount relative to each other and induce strain. However, these types of loads can be cyclic in nature.

Another type of secondary load would be a thermal load, such as a thermal gradient through a wall. This type of load is also cyclic.

Settlement is the least effective type of secondary loading because it primarily has only one/half cycle of load with a limited input. Settlement is similar to forming materials which are also half cycle. Forming is used for manufacturing pressure vessels and steel piping. Pipes are rolled to a particular shape. They exceed yield in this process, however, due to the low strain rates relative to ultimate, there is an undetectable reduction in the ultimate strength. It is also common to form reinforcing steel. As an example, in reinforced containments the major hoop bars are bent to

shape and this involves a yielding of the steel. This also does not lead to any detectable reduction in strength and, of course, hooks are commonly used in reinforcing steel.

Figure 49 shows a summary of the Midland design criteria. The first category is what is in the FSAR. The first is primarily dead and live load, the second combines the small earthquake with live and dead, the third combines live and dead load plus wind, and the fourth combination involves dead load, live load plus the safe shutdown earthquake. The final load combination is dead load and live load and the tornado loading.

After discovering the settlement problems on the diesel generator building at the Midland jobsite, it was decided to add some additional criteria. As a reference, ACI 318-1977 was used and it should be noted that in this code they recognized the fact that settlement only affects serviceability. This means it would induce some additional cracking, which if then exposed to a corrosive environment, could result in corrosion of reinforcing steel. Therefore, in ACI, settlement loads are only combined with normal operating type of loads such as live load and dead load. Using this as a base, the additional criteria shown in Figure 49 were created. The first combination involves dead load, live load and settlement. The second combination considers 1.4 x dead load plus 1.4 x settlement. These are based on serviceability.

Since the design wind and the small earthquake are postulated to occur more than once at the site, two load combinations have also been added as shown which include live load, dead load, settlement and either design wind or the operating basis earthquake.

In summary, either the source of load has been removed, or additional supports have been added for the various structures that are founded fully or partially on fill at the jobsite. For the diesel generator building, the duct banks have been cut loose, removing the source that caused the cracking. The service water pump structure will be supported by adding piling. In the auxiliary building electrical penetration areas, caissons will be added. So again, either the source of load has been removed or additional support has been supplied.

With respect to the significance of what has happened to date, the cracking only affects serviceability, cracks over 15 mils will be sealed in the future. As far as present and future actions are concerned, new seismic analyses are being performed and new static analyses checking the structural design will also be performed. For the diesel generator building, the building will be analyzed for variable foundation conditions. This will be the only building that will involve applying the additional criteria since variable foundation properties will be investigated.

In conclusion, the structures are box type, reinforced concrete, with high strength and good ductility. If it were not for the diesel generator building settlement the concrete cracking of the structures would probably not be of any concern, since all reinforced concrete structures do crack under service, and that is the reason why reinforcing steel is used. With the original FSAR criteria, and the additional criteria, together with the modifications, the structures will be able to safely resist all normal type of loads and postulated events.

4.4 Soils Summary

The diesel generator building settlement noted in August of 1978 was larger than expected. An exploration program was initiated to investigate the seat of the settlement and Drs. Peck and Hendron were consulted to discuss the evaluations and corrective actions required. Based on the exploration and the consultants recommendations it was decided to surcharge the building and surrounding area with a load exceeding the operating load. Instrumentation was installed to evaluate rate of soil consolidation and settlements of the structure and supporting soils. The preload was completed to a height of 20 feet in April 1979.

Figures 50 through 53 illustrate locations of the various instruments associated with the preload program. Figure 50 shows the locations of building survey settlement markers and pedestal settlement rods. Figure 51 shows the location of surface settlement plates and borros anchors installed in the fill primarily at three different elevations to monitor the movement of the soil as a result of the surcharge. The figure also shows locations of 4 deep (elev 535) borros anchors installed for use as reference points for the precise measurements during secondary compression where the movement has subsided to a very small rate. Figure 52 illustrates locations of piezometers installed primarily at three different elevations below the building to monitor the dissipation of pore water pressure during consolidation. Figure 53 illustrates the locations of Sondex instruments intended for measuring soil rebound in order to estimate the modulus of elasticity below the building to check the range used in dynamic analysis.

Figure 54 illustrates typical results of the settlement and pore water measurements for the building. It is seen that within a short time after the completion of the surcharge the settlements of both the soil and the building has subsided to a very low rate and the piezometer water levels have declined significantly. At present the piezometers indicate approximately the same water level as the general ground water level (elev 627). This indicates essentially total dissipation of pore water pressure.

A preliminary plot of the building settlement during secondary compression based on survey measurements indicates that the residual settlement of the building should be less than 1.5 inches during its service life.

The exploration program below the diesel generator building has indicated that the fill is quite variable both in the material type and quality. Therefore, additional explorations were made in the remaining plant site fill to evaluate its condition. The expanded exploration program indicated that although there was no settlement elsewhere, there were certain areas that the fill was of a quality requiring corrective action of the structure involved. These areas are the auxiliary building, electrical penetration rooms, valve pits, and the fill supported portion of the service water structure.

Figures 55 and 56 summarize the fill type (sand clay) below the structures and the planned remedial measures for the various structures supported on plant area.

Liquefaction evaluations based on published experience at sites where liquefaction did or did not occur showed that in certain areas of the sand fill,

under the maximum ground water level of elevation 627 and the SSE of 0.12g, the factor of safety was less than the acceptable value of 1.5. These areas are primarily in the diesel generator building.

As a result of these evaluations consideration was given to grouting of the sands and also to permanent area dewatering. The latter approach of dewatering was proven most beneficial in that it could be monitored simply.

Settlements of the sands following an SSE event would be on the order of ½ to 1 inch in the area of the diesel generator building.

Regarding the subject of estimated settlements for plant structures supported on fill, these settlements will be re-evaluated utilizing the following information:

1. Settlement of the own weight of the fill based on borros anchors installed in areas where no structures are involved
2. Measurements on existing structures and foundations
3. Soil boring information
4. Laboratory test information
5. Diesel Generator Building surcharge experience

These analyses will account for additional induced settlements due to dewatering. These evaluations will be made and reported in the FSAR as part of the current committment.

5.0 CONSULTANT'S STATEMENT (Dr R B Peck)

I have been a consultant to Bechtel on the Midland Project, together with Professor A J Hendron, beginning shortly after the settlements were noted in the Diesel Generator Building. I speak for myself and, I hope, for Professor Hendron, who is unable to be here because he is out of the country. I will not discuss anything that you have not already heard this morning. It is my intention, however, to review the proposed remedial measures and to emphasize those aspects that, in my judgment, are of greatest importance.

The investigations at the Diesel Generator Building rather quickly showed that the seat of settlement was in the clay fill underlying the structure. They also showed that the clay fill was extremely variable with respect to its density, its water content, and even its composition. Furthermore, the investigations showed that it would be feasible to surcharge the area in such a way as to stress the subsoil of the structure to levels exceeding the final stresses that would exist under operating conditions.

After consideration of a number of alternatives, it was decided to prestress the subsoil by means of a surcharge. In my view, this procedure had several important advantages. One of these is the opportunity to provide instrumentation, principally piezometers and subsurface settlement gages, that could furnish data permitting a reliable upper-bound settlement forecast. Furthermore, the procedure automatically prooftested the subsoil with respect to its future settlement behavior. Therefore there would be no need, in determining the

acceptability of the foundation, to depend on the results of additional borings, samples, compaction tests, or other similar activities. Such tests would be likely to prove inconclusive on account of the heterogeneity of the fill material, but they would also be irrevelant in view of the knowledge of the actual behavior.

The results of the preload procedure have been convincing. The observed pore pressures were small, smaller than actually anticipated, and they dissipated rapidly. Hence, primary consolidation was accomplished quickly and the curve of settlement as a function of the logarithm of time became linear shortly after the completion of placement of the fill. Therefore, it is possible to forecast the settlement that would occur at any future time by simple extrapolation, on the assumption that the surcharge will remain in place. Even this amount of settlement would be acceptable. However, the projected settlement determined on this basis is an upper bound, because the surcharge will be removed and the real settlements will certainly be smaller. In my judgment, the foregoing circumstances eliminate any uncertainties concerning the settlement behavior of the Diesel Generator Building resulting from the underlying clay fill.

The investigation at the Diesel Generator Building also showed, however, the presence of zones of sand, including some portions that were loose. This finding indicated a potential for liquefaction under severe earthquakes, and the possibility of settlement originating in the sands due to shakedown under seismic conditions. The surcharge would, of course, be ineffective to remedy this condition.

Of the various possible remedial measures, grouting, probably using chemicals, would, in my judgment, be feasible. Nevertheless, it would be difficult to be assured that all injected materials had been successfully treated, or that all loose zones had actually been injected. Thus, chemical grouting would at best be a piecemeal solution. It would be difficult to give a positive answer to the question whether all significant zones that might liquefy had been identified and treated.

The chosen alternative to grouting is general permanent dewatering of a large portion of the plant site. This solution has the advantage of being a positive solution to the liquefaction problem. Therefore, it provides positive answers to such questions as those just mentioned. The solution has the further advantage that it can be monitored effectively by simple procedures, primarily by the use of piezometers. In my view, one of the greatest advantages of general dewatering is the margin of safety inherent in the time lag that would be required for recharge of the dewatered zone if the pumps should cease to operate. That is, the beneficial effects of the dewatering would persist for a period on the order of weeks after pumping might be interrupted. Failure of the pumping system because of an earthquake would, therefore, not destroy the protection achieved by the dewatering.

In addition to being a positive solution to the liquefaction problem, wherever any such problem might exist in the dewatered area of the plant site, the drainage will reduce substantially any settlements that might be induced by compaction of the sands during an earthquake. The present methods of estimating settlements due to seismic shakedown

are overconservative, because they are based on the results of laboratory tests on dry sands. Even the settlements estimated on this basis would be acceptable. However the presence of capillary moisture in the soil would greatly reduce the freedom of the sand grains to assume a denser position during vibration. Therefore, I consider that dewatering will essentially eliminate any potential problems of seismic shakedown.

The continuing investigations of the plant area indicated other potential trouble areas. In my view, these potential trouble zones have now been adequately defined by the boring program and other investigations. One such area is the location of the Borated Water Tanks. Beneath these tanks the investigations have indicated better and more consistent subsurface conditions than beneath the Diesel Generator Building. It is proposed to fill the tank with water as a test load. The filling will constitute full-scale proof tests with respect to the bearing capacity of the subsoil. It is anticipated that the tanks will settle under the test load, and this settlement will increase the bearing capacity. Furthermore, by making settlement observations at various depths in the subsoil during and after the test loading and by combining this information with stress calculations and theory, it will be possible to make reasonable settlement predictions that take into account the actual subsurface conditions under realistic loadings.

The Electrical Penetration Structures extending from the Auxiliary Building, and the adjacent Valve Pits, are to be underpinned. This is a positive solution that will lead to satisfactory and predictable

results irrespective of the nature of the fill materials that may presently underlie these structures. The operations are expedient, in the sense that they are compatible with the general construction schedule. The nine caissons under each of the Electrical Penetration wings will be tested individually to 150 percent of the anticipated loading, and collectively to 100 percent of the anticipated working load. The latter procedure, in which all nine caissons are loaded simultaneously, constitutes a proof loading that will eliminate any doubts concerning the ability of the underpinning to support the structure without significant settlement.

The Diesel Fuel Tanks are buried structures that have already been subjected to a full-scale loading by filling them with water. The settlements under these test conditions were minimal. Whatever settlement of the tanks may occur will be associated primarily with settlement of the underlying and surrounding fill under its own weight. Since the tanks will be settling with fill, the differential movements between the tanks and the surrounding soil and piping will be minimal, and the connections can be expected to settle approximately equally with the tanks. Therefore, I do not consider that any unusual conditions exist with respect to the Diesel Fuel Tanks, and that attention to details providing reasonable flexibility will satisfy all requirements.

The Service Water Structure lies outside the area of planned permanent dewatering. Therefore the wing presently supported by fill will be picked up by a system of piles. The proposed procedure provides

positive support. The piles are to be designed to carry the structural loads at their buckling strength and will therefore be effective even in the event of liquefaction of the surrounding soil. Since these piles are not clustered in such a way as to stress highly a large mass of the bearing material, as in the case of the caissons proposed for the Electrical Penetrations of the Auxiliary Building, they are not to be proof loaded as a group, but will be loaded individually to 150 percent of the anticipated working load. This procedure is conservative.

In summary, my overall impressions and conclusions concerning the proposed remedial measures are as follows: The investigation has proceeded in a progressive fashion. Like most investigations of this kind, it has not always proceeded in a straightforward way, but has appropriately pursued various approaches. Although it is still continuing in some respects, I consider that it has now disclosed the significant conditions and potential problems associated with the foundation conditions of the site. As a result of the studies, a variety of solutions has evolved. Each solution is suited to the specific conditions and problems of a particular part of the facility. However, the potential for liquefaction has been eliminated once and for all, and many potential uncertainties have been eliminated by full-scale loading or proof testing where such procedures have been found advantageous. In my judgment, this is a strong advantage of the procedures adopted.

Finally, the proposed solutions do not require unreasonable maintenance or monitoring during the lifetime of the plant, and can therefore be adopted with confidence.

6.0 SCHEDULE

Figures 57 through 60 show the schedules of the four major remedial activities. The work on bearing piles for the Service Water Pump structure (Figure 57) will commence as soon as the administrative activities were completed, probably this fall, and should be completed sometime in early 1980. Since this is an independent activity it is expected to have no impact on the overall project schedule.

Figure 58 covering the Unit 1 and 2 Auxiliary Building Electrical Penetration areas and the Unit 1 and 2 Feedwater Isolation Valve Pits indicates that this work should complete about mid 1980; however, the actual schedule would probably extend 2-3 months beyond the dates shown. Again this is a separate activity and would not have an impact on the overall project schedule; however, it should be noted that this work would probably cause some additional work for construction due to congestion in the areas where other activities were taking place. It is not expected to be a major problem.

Figure 59 shows the borated water storage tanks activities however, this is a method of completing this activity and may not be the final method. This particular method includes a temporary cross tie between the two borated water storage tanks (Unit 1 and Unit 2) and would take until mid 1981 for final completion. This may be the most critical schedule activity as far as the overall project schedule is concerned, in that flushing activities and testing activities are taking place in the same time frame as the preload. After further evaluation, this schedule may be modified somewhat.

Figure 60 shows the permanent plant dewatering system. We had previously informed the NRC that because of the preloading activities there could be an overall impact of two months on the project schedule. At this time, because of a revised testing philosphy, the Unit 1 and 2 Diesel Generator turnovers need not take place until November of 1980 and August of 1980 respectively. This actually allows some float time in the schedule.

Approximately six months had been allocated in the schedule for dewatering the power block area to the design depth and about three months had been allowed after that time for recharge rate testing. This would allow all activities to complete prior to Unit 2 fuel load, and again, would not impact the overall project schedule. The major problem being that of site congestion and interference with other site activities. This is a construction problem and one that does not seem to be a major obstacle at this time.

7.0 CAUSE INVESTIGATION

The investigation into the cause of insufficient compaction of plant area fill was made by Bechtel using a problem analysis technique known as the Kepler-Tregoe (K-T) method. This approach involved the following steps and is shown on Figures 61 through 71.

(1) Identify deviation, in this case insufficiently compacted plant area fill.
(2) Develop criteria for determining in which plant area fill the deviation exists.
(3) Identify distinctions and changes which might have caused the deviation considering the subject of the deviation, where it occurred, time factors, and the extent.
(4) Develop list of possible causes using all distinctions and changes.
(5) Test possible causes for most probable causes.

It should be noted that although all areas were included in the investigation where deviations were identified by the soils investigation, some deviations were thought to be insufficient to require corrective actions. Two examples of such areas are the borated water tank area and the auxiliary building railroad bay. In these areas the compacted fill is adequate despite some indications of localized insufficiently compacted material.

Seventeen distinctions or changes were found to have occurred which could have been possible causes and these have all been evaluated. Specifications, first identified as a possible cause, were not included in the most probable cause list because it was felt upon evauluation that variances from the PSAR and FSAR and the various relatively minor inconsistencies could not have been a cause of the problem under investigation. The investigation is still under way into soils testing methods, equipment, results, retests, reviews, and

evaluations, since these were found to have contributed to the cause.

The five most probable causes remaining after evaluating the possible causes are, not necessarily in order of importance:

(1) Lift thickness/compactive effort. Recent tests have shown that lift thicknesses in some cases exceeded the capability of equipment being used, verifying that equipment was not adequately qualified in all cases.

(2) Compaction equipment/qualification. Same comments as for (1) apply.

(3) Test procedures and results. This included representativeness of tests, procedures for comparison with standard proctor specimens, procedures for taking soil tests within a lift, calculation of relative density, and use of nuclear densimeter.

(4) Inspection procedures. This included the use of a surveillance type program in the power block area for at least part of the time.

(5) Reliance on test results. This included construction's reliance on test results for qualification of equipment during the work and for acceptance of the work by Construction and Quality Control personnel.

Personnel were not included as a most probable cause because a review of qualifications and experience of both Bechtel and U. S. Testing personnel had shown presence of sufficient education, experience, and training to carry out the tasks assigned.

8.0 QA/QC ASPECTS

8.1 Corrective Actions

This section discusses the QA/QC aspects including the probable causes identified and the corrective action taken and/or to be taken. The possible and most probable causes were discussed in Section 7.0. The matrix found on page 2 (Most Probable Causes per K-T Analysis) indicates the corrective action taken or to be taken.

The deficiencies and items of concern from the 50.54(f) Report and the IE Inspection Reports 78-12, 78-20 and 79-10 and corrective action taken or to be taken are provided in two matrices and a table. /"Deficiency Description (Items of Concern)," "Corrective Action Statu· for Deficiency Description (Items of Concern)" and "Corrective Actions on a Generic Basis."/ These are found on Pages 4, 6 and 11, respectively. The first of these matrices is a cross-reference showing the specific item of concern in IE Inspection Reports and in 50.54(f). The second matrix shows the status of action based upon 50.54(f) answers to date for Items 1 through 13. The second matrix also shows status of action on Items 14 through 18. A plan view of the Tank Farm (Tank Farm Boring Plan) is provided on Page 12 to aid in locating test and inspection pits, air bubbles mapped, borings completed and borings proposed.

Item No	Possible Causes Per K-T Analysis	Corrective Action
1. 2.	Lift Thickness/Compactive Effort and Compaction Equipment/Qualification	Onsite geotechnical soils engineer at the site. Also, geotechnical soils engineer from the Geo-Tech Dept in home office to give technical direction. Specification C-211 has been revised such that the uncompacted lift thickness of the backfill material shall be determined by the onsite geotechnical soils engineer after evaluation of the proposed compaction equipment. However, in no case shall the uncompacted lift thickness exceed 8" for heavy self-propelled equipment and 4" for hand operated equipment. This specification has also been revised to read, "The onsite geotechnical soils engineer shall verify that the equipment used for compacting the backfill materials be capable of obtaining the desired results and obtaining the same acceptable compaction effort achieved in the test pad area." This verification shall include, but not be limited to, the following: number of passes, speed, revolutions per minute (frequency), overlap per pass, lift thickness requirements and uniformity. Specification C-211 states, "Selection and approval of all the proposed compaction equipment shall be on the basis of demonstrated ability to accomplish adequate compaction without damage to, or overstressing of, the adjacent structural members".
3.	Testing Procedures & Results	
	a. Methods	Specification C-211 is revised such that Proctors are made with every field density test.
	b. Equipment	The nuclear densometer will not be used.
	c. Results/Reports	The onsite geotechnical soils engineer will review and approve each soil test report. This will include, but not be limited to, gradation, moisture and density tests. US Testing will be checking all field density tests for cohesive material against a zero-air-voids curve. Any field test result which plots on or to the right of the zero-air-voids curve shall be regarded as suspect and cause for retest. The onsite geotechnical soils engineer shall determine all density test locations.

Item No	Possible Causes Per K-T Analysis	Corrective Action
3.	d. Retests	All material represented by failing tests is to be re-worked until the specified density and/or moisture is obtained. No material will be placed on any known failing material until satisfactory tests are obtained.
	e. Reviews/Evaluations	See Item c above.
	f. Personnel	An onsite geotechnical soils engineer and a part-time Geo-Tech soils engineer have been added at the site. The onsite geotechnical soils engineer coordinates with craft superintendents and notifies QC of selected areas to be backfilled, monitors subgrade quality and preparation, calling for testing as required. He evaluates size of fill area to determine testing frequency, monitors material and lift thickness placement. Calls for tests in borrow areas for cohesive fill. Monitors compaction process including moisture control for clay. Calls for tests at proper frequency and designates location. Works with craft superintendents and QC to obtain effective remedial action on failing tests. The geotechnical soils engineer provides overview and inputs technical assistance as required.
4. 5.	Inspection Procedures and Reliance on Test Results	
	a. Different Inspection Methods	The Project Quality Control Instruction has been revised to include a daily soil placement report which is used for each area where soils work is being performed. This report includes sketch showing areas of soil placement, identification of equipment being used, identification of supporting personnel, recording lift thickness measurements which are representative of the fill being placed, compactive effort used, location by grid coordinates and elevation of all tests taken and testing frequencies, types of material placed (cohesive/cohesionless). A Quality Control Engineer will be assigned 100% of his time to soil placement. Consumers Power Company will perform over-inspection on a sampling basis of the soil placements. Also see Item 2.f. above.
	b. Placement Methods	See Item 1 above.

Deficie[illegible] Description
(Ite[illegible] Concern)

Item No	Deficiency Description (Items of Concern)	Location in 50.54(f) Page No (Item)	Location in 78-20 Page No	Location in 78-12 Page No (Item)
1.	Inconsistency between specifications and the D&M Report.	I - 1, 3 A & B (1)	9, 10, 16, 17	8
2.	Lack of formal revisions of Specs to reflect clarification of Spec requirements.	I - 1-3 A & B (2)	9-14	7-8 (4)
3.	Inconsistency of information within the FSAR relating to Diesel Generator Bldg fill material and settlement.	I - 2, 4 A & B (3)	6-8	6-7 (3)
4.	Inconsistency between basis for settlement calcuations for Diesel Generator Bldg & design basis.	I - 2-4 A & B (4)	20-21	--
5.	Inadequate design coordination in the design of the duct bank.	I - 3-5 A & B (5)	23-24	10 (8)
6.	Insufficient compactive effort used in backfill operation.	I - 10 A & B (1)	--	--
7.	Insufficient technical direction in the field.	I - 10 & 11 A & B (2)	24-26	--
8.	Inadequate Quality Control inspection of placement of fill.	I - 13, 14 A & B (1)	25-29	--
9.	Inadequate soil moisture testing.	I - 13, 15 A & B (2)	14-16	8 (4)
10.	Incorrect soil test results.	I - 13, 15 A & B (3)	--	--
11.	Inadequate subcontractor test procedures.	I - 13, 14 & 16 A & B (4)	--	--
12.	Inadequate corrective action for repetitive conditions.	I - 21 & 22 A & B (1)	17-20	--
13.	The Bechtel Quality Assurance Audit and Monitor Program failed to identify the problems relating to the settlement.	I - 21 & 22 A & B (2)	17-20	--

Item No	Deficiency Description (Items of Concern)	Location in 78-20 Page No	Location in 78-12 Page No (Item)	Location in 79-10 Page No (Para)
14.	Effect of ground water on DGB settlement - unresolved.	9	7 (3d) 10 (8)	
15.	Inadequate subgrade preparation after winter freeze -	16-17		
16.	(NRC Question No 362.2 on FSAR Section 2.5.4.5.1)	--	8-9 (5)	
17.	(Cracks in concrete structural wall & footing in the DG Bldg)		9 (6)	
18.	(Air bubbles in Tank Farm Area and lack of action)	--	--	6-7 (5)

Corrective Action Status for Deficiency Description (Items of Concern)

Item No	Deficiency Description (Items of Concern)	Corrective Action 50.54(f) Discussion Items Located on Page No (Item)	Action Status
1.	Inconsistency between specifications and the D&M Report.	I - 6-8 C & D (1)	a. The review of the Dames & Moore Report is complete. Specification C-211 revised accordingly. b. Resolution of the audit findings on the Design Requirement Verification Checklist Audit continues.
2.	Lack of formal revisions of Specs to reflect clarification of Spec requirements.	I - 6, 8 C & D (2)	a. Generic Corrective Action - Engineering Departmental Procedure 4.49.1 has been revised to incorporate clarifications and instructions for use of Specification Change Notices. b. Generic Corrective Action - Reviewing specifications for specificity completed. Resolution shortly.
3.	Inconsistency of information within the FSAR relating to Diesel Generator Bldg fill material and settlement.	I - 6, 8 C & D (3)	Complete review of pertinent portions of the FSAR Section 2.5 and 3.8 have been completed.
4.	Inconsistency between basis for settlement calculations for Diesel Generator Bldg and design basis.	I - 6-9 C & D (4)	a. Correct settlement calculations are to be made subsequent to Diesel Generator Building surcharge removal. b. Generic Corrective Action - Scheduled audits will be performed on Geo-Tech section on a six month basis. The first audit is scheduled for July 27, 1979. c. Generic Corrective Action - Also, audits are scheduled for each design disciplines calculations on a yearly basis.
5.	Inadequate design coordination in the design of the duct bank.	I - 7, 9 C & D (5)	Generic Corrective Action - Drawings have been reviewed for possible effect of vertical duct bank restrictions in other areas. Ten areas resolved, one still in process.

Item No	Deficiency Description (Items of Concern)	Corrective Action 50.54(f) Discussion Items Located on Page No (Item)	Action Status
6.	Insufficient compactive effort used in backfill operation.	I - 11 C & D (1)	a. Re-evaluation of construction equipment used for compaction is still in process. b. Generic Corrective Action - The review of other construction specifications and procedures to identify equipment requiring qualifications is still under way.
7.	Insufficient technical direction in the field.	I - 11, 12 C & D (2)	a. An onsite geotechnical soils engineer and a Geo-Tech soils engineer have been assigned to the job. b. Generic Corrective Action - Field Procedure FPG-3.000 has been reviewed to assure clarity and completeness and found adequate. c. Consumers Power Company to implement over-inspection for soils placement and US Testing activities in the soils area.
8.	Inadequate Quality Control inspection of placement of fill.	I - 16, 18-20 C & D (1), D (5)	a. Project Quality Control Instruction C-1.02 has been revised to provide inspection rather than surveillance and to record daily inspection reports. b. Generic Corrective Action - All active PQCI's have been reviewed for surveillance vs inspection callouts and are now being evaluated. c. Generic Corrective Action - Bechtel is working to incorporate scientific sampling plans for inspection areas instead of using percentage sampling (being used now). d. Consumers Power Company to implement over-inspection for soils placement and US Testing activities in the soil area on a sampling basis.
9.	Inadequate soil moisture testing.	I - 16-20 C & D (2), D (5)	The use of the nuclear densometer has been discontinued.

Item No	Deficiency Description (Items of Concern)	Corrective Action 50.54(f) Discussion Items Located on Page No (Item)	Action Status
10.	Incorrect soil test results.	I - 17-20 C & D (3), D (5)	a. The Project Quality Control Instruction C-1.02 has been revised from surveillance to inspection of the testing operation. b. The in-depth review of soil test results is still in process. c. Generic Corrective Action - The in-depth audit of US Testing has been completed. Two findings were a result of this audit. One, administrative problem by US Testing, the other by Bechtel Subcontracts. These audit findings will be closed prior to soil placement. d. Generic Corrective Action - PQCI's have been reviewed for adequacy of documentation callouts and are being resolved. e. Consumers Power Company will implement an over-inspection of US Testing activities in the soils area. f. Bechtel has directed US Testing to check all field density tests for cohesive material against a zero-air-voids curve. Any field test results which plots on or to the right of the zero-air-voids curve shall be regarded as suspect and cause for re-test. g. Bechtel Geo-Tech has re-emphasized to US Testing the importance of taking accurate tests.
11.	Inadequate subcontractor test procedures.	I - 17-20 C & D (4), D (5)	a. Generic Corrective Action - An in-depth audit of US Testing has been completed with no problems found in the area of the test procedures.
12.	Inadequate corrective action for repetitive conditions.	I - 22 C & D (1)	a. An in-depth review of the Bechtel Trend Program Data has been performed by Bechtel QA Management with no items indicating trends found.

Item No	Deficiency Description (Items of Concern)	Corrective Action 50.54(f) Discussion Items Located on Page No (Item)	Action Status
12.	(Contd)		b. Training sessions have been held in Ann Arbor, Jackson, and Midland site to all Consumers and Bechtel QA Engineers and auditors to increase their awareness of the settlement problem and discuss auditing and monitoring techniques to increase audit effectiveness.
13.	The Bechtel Quality Assurance Audit and Monitor Program failed to identify the problems relating to the settlement.	I - 22 C & D (2)	Same as 12 above.
14.	Effect of ground water on DGB settlement - unresolved.	--	As discussed in the K-T Analysis, the effect of ground water on the Diesel Generator Building settlement would be insignificant had the compaction of the material been to the proper density.
15.	Inadequate subgrade preparation after winter freeze -	--	This also has been discussed in the K-T Analysis and has been eliminated as a cause to the Diesel Generator Building Settlement.
16.	(NRC Question No 362.2 on FSAR Section 2.5.4.5.1)	--	This has been addressed.
17.	(Cracks in concrete structural wall & footing in the DG Bldg)	--	This has been addressed in a previous presentation.
18	(Air bubbles in Tank Farm Area and lack of action)	--	Air bubbles have been mapped as indicated in the sketch of the Tank Farm ARea. An inspection pit has been dug from 628' ± to 616' ± in the Tank Farm Area indicated with 3 in the sketch. The pit was approximately 20'x20' @ 628' and approximately 10'x10' @ 616'. The material from 628' to 624' was soft wet and disturbed material. The material from 624' to 622' was a transition area. The material from 622' to 616' was very good hard stiff clay with some sand pockets. There was no evidence of undermining from the air bubbles. The air pipe is approximately @ elevation 611'. The excavation was discontinued due to the adequate material between 622' & 616'.

Item No	Deficiency Description (Items of Concern)	Correc[illegible] Action 50.5[illegible](f) Discussion Items Located in Page No (Item)	Action Status
18.	(Contd)		Four borings are proposed in the areas of bubbles indicated on the sketch. Two of the borings are located where previous borings were taken during the soils investigation, to correlate the effect of the air bubbles. Two are in progress at this time. A new air line has been placed in the steam tunnel and the air line in the Tank Farm is no longer in use.

Corrective Actions on a Generic Basis

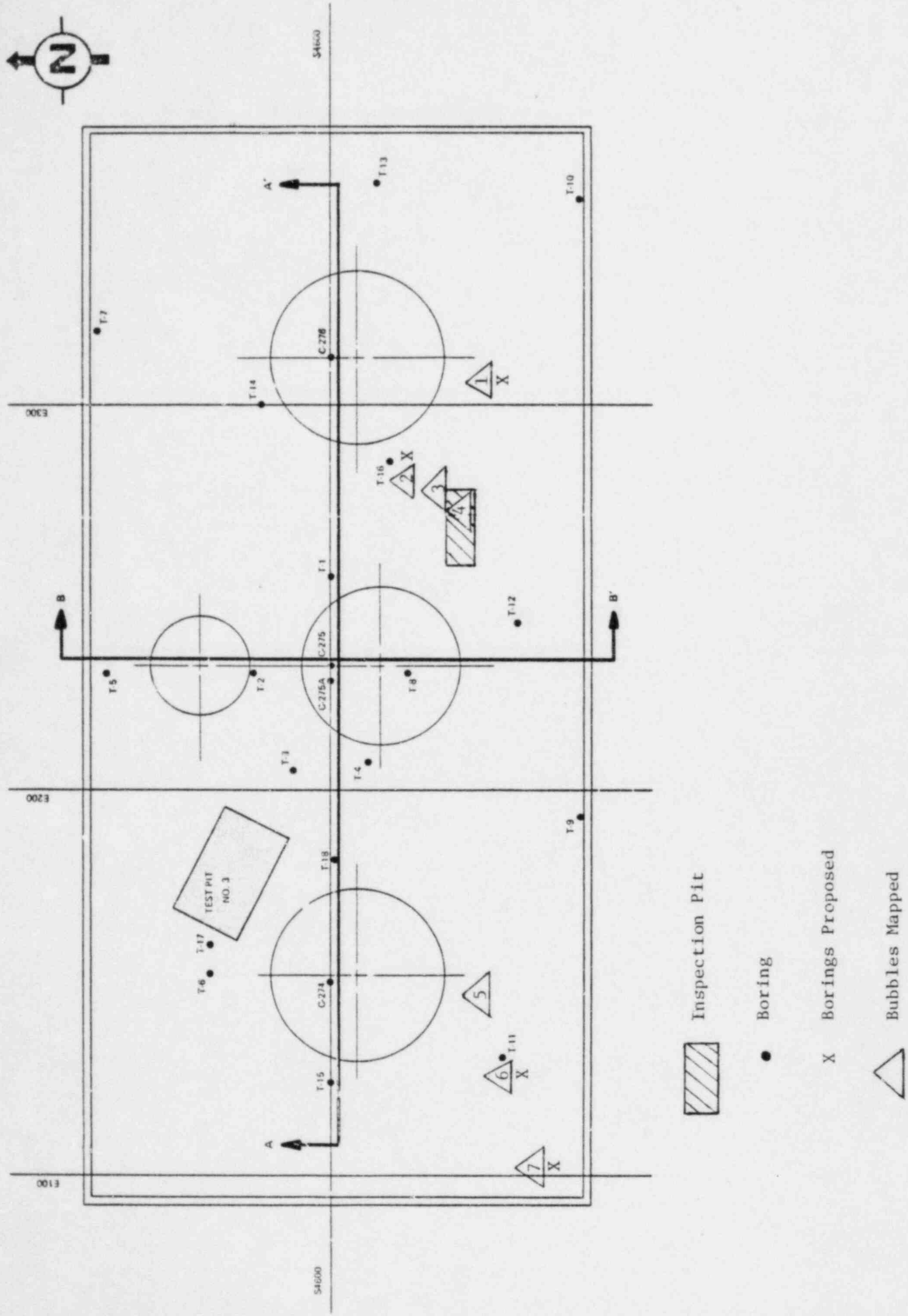
The final review and update of the PSAR commitment list continues and will be completed by January 1, 1980.

Review of Engineering Departmental Procedure 4.22 "Preparation and Control of Safety Analysis Reports" has been completed and no changes were required.

A review of sections of the FSAR is being performed.

A Quality Assurance audit will be made of these three activities.

7/13/79



TANK FARM BORING PLAN

8.2 Q-List Fill Resumption

The following figures (pages 14 through 19) describe those Consumers Power prerequisites which must be completed prior to resumption of Q-list backfill. Some of these prerequisites were referenced in IE Inspection Report 79-10 and are so indicated on these figures. Following these figures is a matrix showing the status of corrective action (Pages 20 through 22).

CPCo PREREQUISITES PRIOR TO RESUMPTION OF Q-LIST BACKFILL

Item No.	Prerequisites	79-10
1.	IDENTIFY CONFLICTS WITHIN FSAR	●
2.	IDENTIFY INCONSISTENCIES BETWEEN FSAR/ AND SPECIFICATIONS OR DRAWINGS	●
3.	IDENTIFY INCONSISTENCIES OR OMISSIONS WITHIN SPECIFICATIONS	
4.	RE-EVALUATE CONTINUED USE OF "RANDOM FILL" IN ZONE 2 AREAS	●

● = Located in Indicated Document

G 0095 24

CPCo PREREQUISITES PRIOR TO RESUMPTION OF Q-LIST BACKFILL (Cont.)

Item No.	Prerequisites	79-10
5.	PROVIDE:	
	Flow Diagram of Necessary Steps for Quality Control and Assurance of Soil Work	
	Specific Organization Responsible	
	Specific Procedure Used	
	Specific Acceptance Criteria	
6.	ASSURE THAT ALL "CLARIFICATIONS" AND "INTERPRETATIONS" ARE RESOLVED VIA OFFICIAL SPECIFICATION CHANGE NOTICES	

● = Located In Indicated Document

G 0695 25

8.0

15

CPCo PREREQUISITES PRIOR TO RESUMPTION OF Q-LIST BACKFILL (Cont.)

Item No.	Prerequisites	79-10
7.	APPOINT SINGLE INDIVIDUAL RESPONSIBLE FOR EACH OF THE FOLLOWING: Directing Construction Aspects of Soils Work Directing Design Aspects Directing Quality Control Aspects	●
8.	INSTITUTE 100% INSPECTION OF SOILS PLACEMENT WITH CORRESPONDING INSPECTION RECORD DOCUMENTATION OF SPECIFIC CHARACTERISTICS INSPECTED IN EACH CASE	

● – Located In Indicated Document

8.0

16

CPCo PREREQUISITES PRIOR TO RESUMPTION OF Q-LIST BACKFILL (Cont.)

Item No.	Prerequisites	79-10
9.	RE-EVALUATE CAPABILITY OF EQUIPMENT BEING USED IN RELATION TO MAXIMUM ALLOWABLE LIFT THICKNESS AND COMPACTION REQUIREMENTS	●
10.	RE-EVALUATE APPROPRIATENESS OF CONTINUED USE OF NUCLEAR DENSOMETER, WITH ITS MEASUREMENT ACCURACY BEING QUESTIONABLE RELATIVE TO MOISTURE CONTENT SPECIFICATION LIMITS OF "PLUS OR MINUS TWO PERCENT OF OPTIMUM"	

● = Located in Indicated Document

G 0695 27

CPCo PREREQUISITES PRIOR TO RESUMPTION OF Q-LIST BACKFILL (Cont.)

Item No.	Prerequisites	79-10
11.	RE-EVALUATE SARs, SPECIFICATIONS AND PROCEDURES RELATIVE TO THEIR ADEQUACY IN SPECIFYING: Points In Process at which Measurements or Test are to be made Frequencies of these Measurements or Tests Conditions under which New Laboratory Standards Must Be Acquired	●
12.	ASSURE THAT METHOD EXISTS THREE DIMENSIONAL AND VOLUMETRIC FOR IDENTIFYING SPECIFIC LIFTS WHICH ARE INSPECTED AND TESTED	●

● = Located In Indicated Document

G 0695 26

8.0

18

CPCo PREREQUISITES PRIOR TO RESUMPTION OF Q-LIST BACKFILL (Cont.)

Item No.	Prerequisites	79-10
13.	ASSURE NONCONFORMANCE REPORTS ARE DISPOSITIONED	●
14.	ASSURE THAT FIELD DENSITY/MOISTURE TEST THAT PLOT TO RIGHT OF ZERO AIR VOID CURVE ARE UNDERSTOOD	

● = Located in Indicated Document

G-0695 29

STATUS ATTACHMENT
OF 14 PREREQUISITES

Consumers Power Company Item Number*	Action(s) and Status
1. Identify all conflicts within PSAR, within the FSAR, or between the PSAR and the FSAR, and correct these inconsistencies via official changes to the appropriate documents.	Project Engineering and Geo-Tech performed a review of subsections FSAR section 2.5 pertaining to backfill operations to eliminate inconsistencies, etc. Project Engineering and Geo-Tech performed a review of the Dames & Moore Soil Report. Resolved CPCo-PMO comments on FSAR Section 2.5. Completed via Rev 7 to Spec C-211.
2. Identify any inconsistencies between the PSAR/FSAR and the detailed specifications or drawings, and correct these inconsistencies via official changes to the appropriate documents.	Resolved CPCo-QA comments on Specifications C-210 and C-211. Completed via Rev 7 to Spec C-211.
3. Identify any inconsistencies or omissions within the specifications and correct these inconsistencies via official Specification Change Notices.	Same as Item #2 above.
4. Re-evaluate the appropriateness of the continued use of "random fill" in Zone 2 areas.	Specification C-211 revised to redefine random fill with special emphasis on soils supporting structure. Completed via REv 7 to Spec C-211. This will be accomplished through overview by the onsite geotechnical soils engineer.
5. Provide a flow diagram of the steps which are needed for the quality control and assurance of soils work and assure that for each step there is a designation as to the specific organization primarily responsible for the action; a designation of the specific procedure to be used; and a designation of the specific acceptance criteria for the step.	A combined flow chart has been prepared illustrating the backfill process and the responsibilities of the onsite geotechnical soils engineer, Geo-Tech soils engineer, Soils Quality Control Engineer and US Testing. This flow chart has been placed in Field Instruction FIC-1.100 "Q-Listed Soils Placement Job Responsibilities Matrix".

*Per: (1) Meeting minutes from the April 24, 1979 Bechtel/CPCo meeting on resumption of Q-listed backfill.
(2) Added action items at the April 26, 1979 Diesel Generator Task Group Meeting.
(3) JFNewgen letter to TCCooke BCCC-3995 dated May 4, 1979.

Consumers Power Company Item Number*	Action(s) and Status
6. Assure that all "clarifications" and "interpretations" are resolved via official Specification Change Notices.	Engineering Departmental Procedure Instruction 4.49.1 has been revised to incorporate clarifications and instructions for use of Specification Change Notices.
7. Establish a single individual at the site to be responsible for each of the following: directing the construction aspects of the soil work; directing the design aspects; and directing the quality control aspects.	The following positions have been established: a) Onsite geotechnical soils engineer. b) Geo-Tech soils engineer. c) Soils QC Engineer. Their responsibilities are defined in the flow chart described in '5' above.
8. Institute 100 percent inspection of each lift placement with a corresponding Inspection Record documentation of the specific characteristics inspected in each case.	Bechtel QC has revised the Project Quality Control Instruction PQCI/QCIR for backfill placement. Revised PQCI/QCIR calls for inspection of backfill work by a full time Soils QC Engineer with generation of a daily report for each area of backfill worked.
9. Re-evaluate the capability of the equipment being used in relation to the maximum allowable lift thickness and the compaction requirements.	Hand held equipment has been qualified for the two sands to be used. Qualification of equipment to be used on cohesive materials are still in progress. All equipment will be qualified in specific soils prior to its use.
10. Re-evaluate the appropriateness of the continued use of the nuclear densometer, with its measurement accuracy being questionable relative to the moisture content specification limits of "plus or minus two percent of optimum".	The use of the nuclear densometer has been discontinued for record inspection use.
11. Re-evaluate the SAR's specifications and procedures relative to their adequacy in specifying the points in the process at which the measurements of tests are to be made, the frequencies of these measurements or tests, and the conditions under which new laboratory standards must be acquired.	Geo-Tech has performed this review. An audit has been performed on US Testing by Bechtel to determine the adequacy of their soils testing procedures. The Audit was performed on 4/25 - 26/79. Two findings on administrative policies were found. One against Subcontracts and one against US Testing. Corrective action will be taken prior to starting backfill.

*Per: (1) Meeting minutes from the April 24, 1979 Bechtel/CPCo meeting on resumption of Q-listed backfill.
(2) Added action items at the April 26, 1979 Diesel Generator Task Group Meeting.
(3) JFNewgen letter to TCCooke BCCC-3995 dated May 4, 1979.

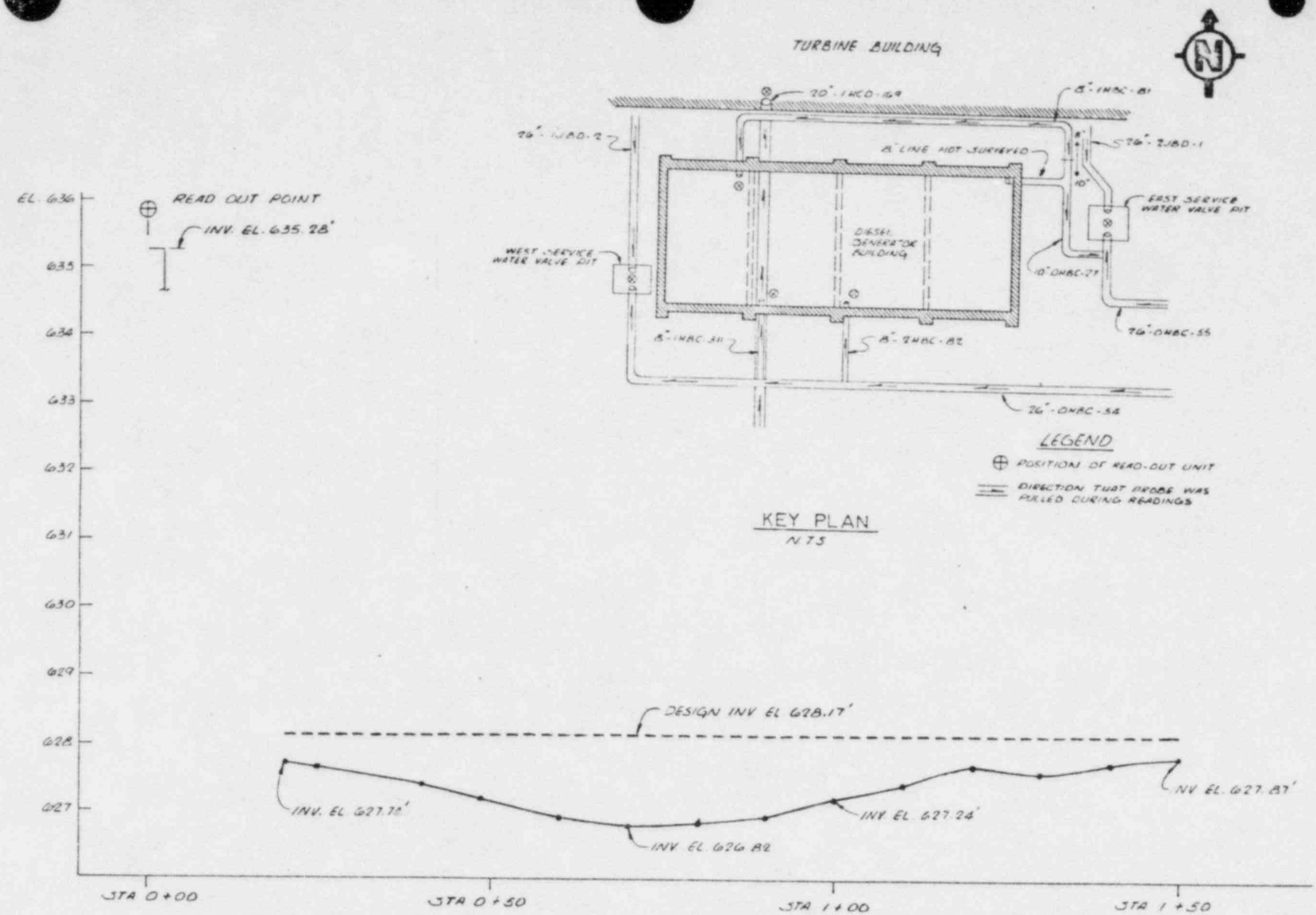
Consumers Power Company Item Number*	Action(s) and Status
12. Assure that there is a method, on a three dimensional and volumetric basis, for identifying the specific lifts which are inspected and tested.	Bechtel QC has revised the Project Quality Control Instruction PQCI/QCIR C-1.02 to cover this.
13. Assure that each nonconformance report (regardless of the type of report) is dispositioned.	For each Q-listed area all Discrepancy Reports and NCR's (Bechtel and CPCo) will be fully dispositioned and closed out prior to placement of backfill. This will be covered on case-by-case basis prior to backfill starting in a particular area. Additionally, P.E. will release areas for backfill which are listed in MCAR 24 as questionable areas on a case-by-case basis by memo or TWX.
14. Understanding the field density/moisture test in the Oily Waste Area that plotted to the right of the zero-air-void curve.	Bechtel has directed US Testing to check all field density tests for cohesive material against a zero-air-void curve. Any field test result which plots on, or to the right of the zero-air-voids curve, shall be regarded as suspect and cause for retest. Bechtel Geo-Tech has re-emphasized to US Testing the importance of taking accurate tests.

*Per: (1) Meeting minutes from the April 24, 1979 Bechtel/CPCo meeting on resumption of Q-listed backfill.
(2) Added action items at the April 26, 1979 Diesel Generator Task Group Meeting.
(3) JFNewgen letter to TCCooke BCCC-3995 dated May 4, 1979.

9.0 LICENSING ACTIVITIES AND CHANGES TO FSAR

With respect to the Site Fill problems at Midland, Consumers Power Company has received several documents from the NRC requesting information. This includes questions via 50.54 and the FSAR. There are still some questions yet to be answered and it is our intent to answer these by amendments to these documents. We will be keeping the NRC informed by means of further 50.55e reports. Upon completion of the corrective actions and answering all questions, the FSAR will be changed to update it to the as built condition of the plant.

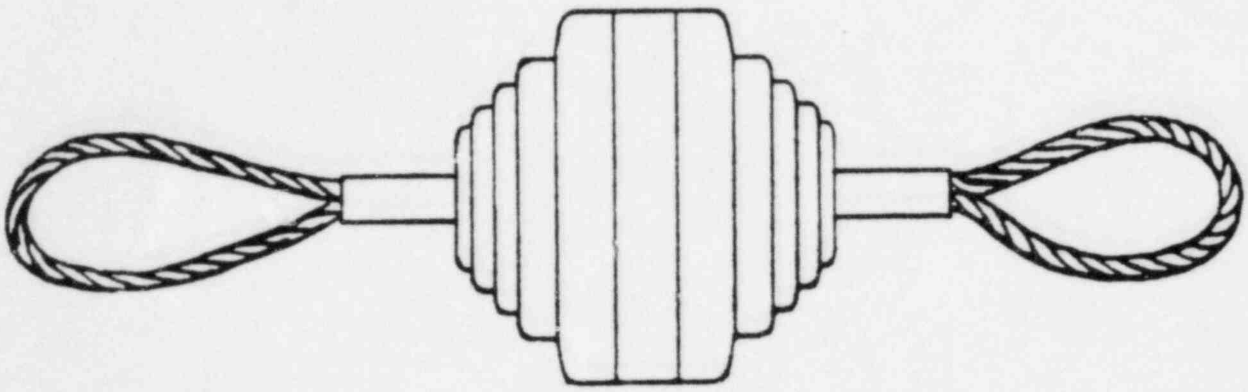
As indicated in reply to 50.54, the FSAR is being re-reviewed for technical consistency with respect to project design documents, consistency between FSAR subsections, and documenting the PSAR commitments have been dispositioned. The re-review is scheduled to be completed by January 1, 1980.



PROFILE 8"-1HBC-81

SCALE: VERT. 1" = 1'
HORIZ. 1" = 10'

Figure 4



INSIDE DIAMETER OF CONDUIT = 4¼"
OUTSIDE DIAMETER OF MANDREL = 3¾"

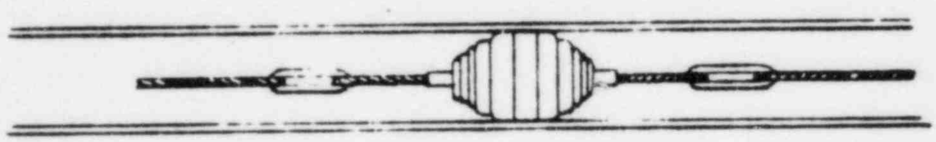
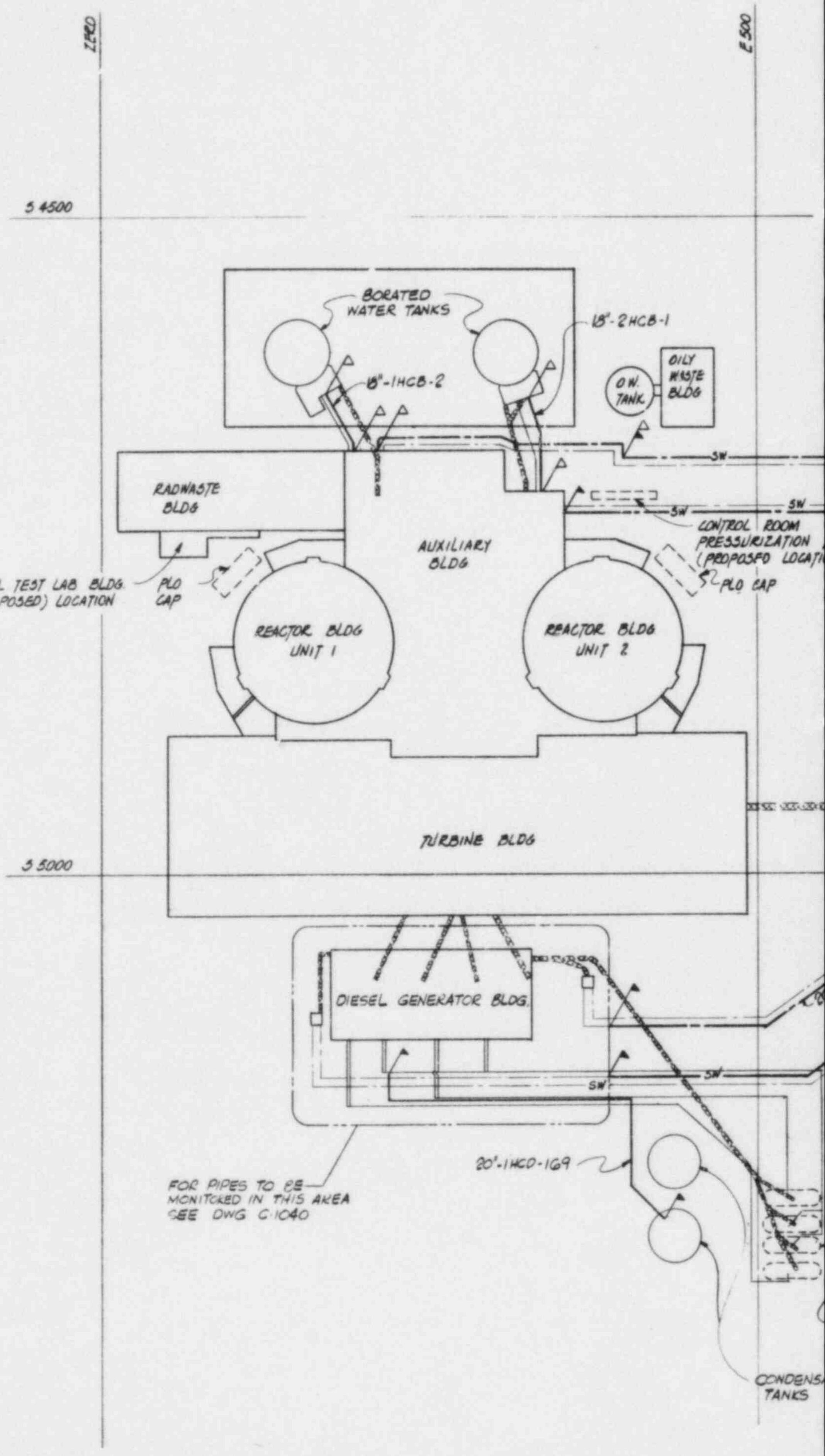


DIAGRAM OF MANDREL (RABBIT) USED TO CHECK CONDUITS

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

RABBIT
FOR ELECTRICAL DUCT

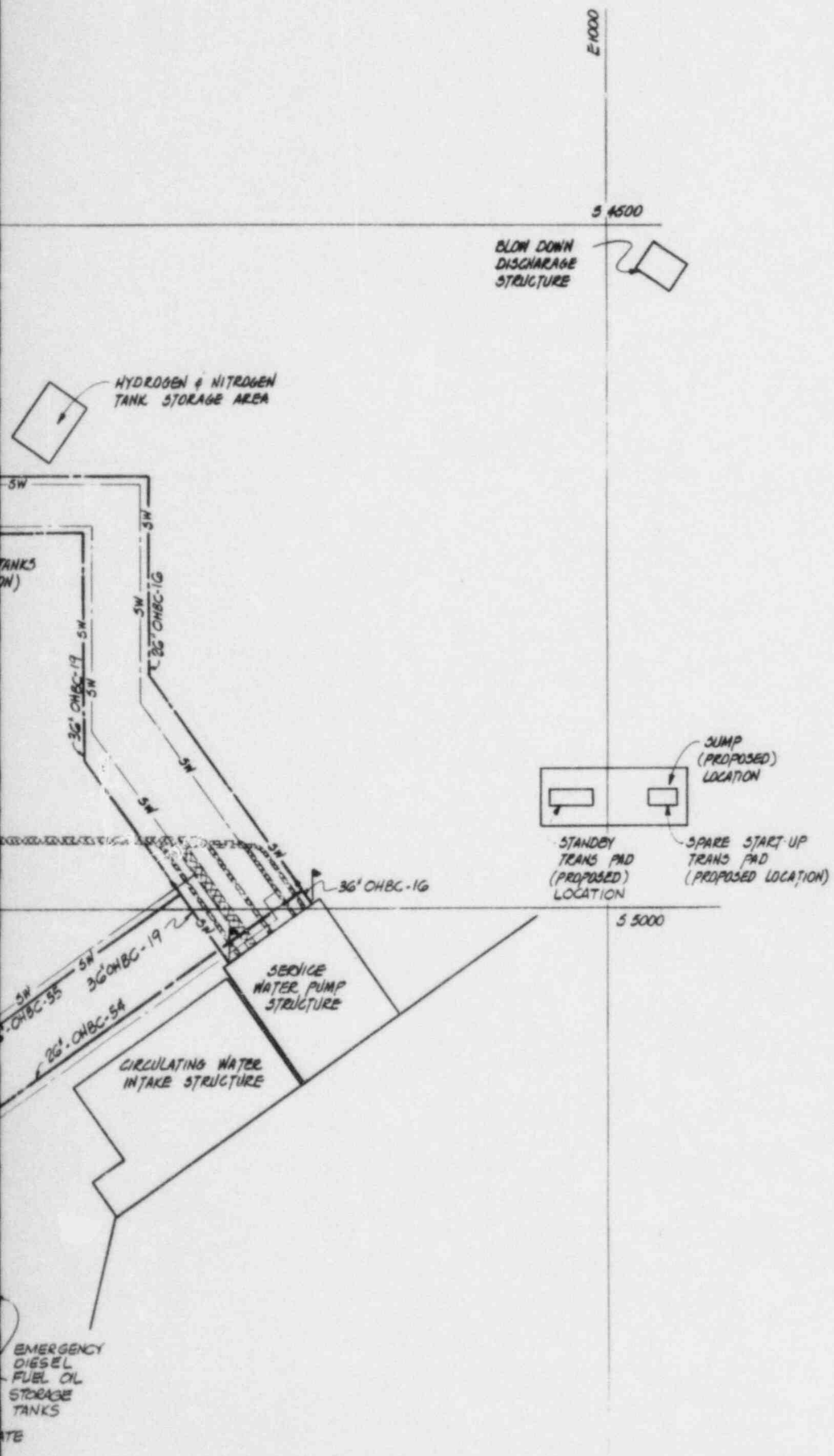
FIGURE 5 DATE 4 24 79



ZERO
E 500
S 4500
S 5000
BORATED WATER TANKS
18"-2HCB-1
18"-1HCB-2
O.W. TANK
OILY WASTE BLDG
SW
RADWASTE BLDG
AUXILIARY BLDG
CONTROL ROOM PRESSURIZATION (PROPOSED LOCATI
PLO CAP
WELL TEST LAB BLDG (PROPOSED) LOCATION
PLO CAP
REACTOR BLDG UNIT 1
REACTOR BLDG UNIT 2
TURBINE BLDG
DIESEL GENERATOR BLDG.
20"-1HCD-169
FOR PIPES TO BE MONITORED IN THIS AREA SEE DWG C-1040
CONDENS TANKS

1. PROFILE MEASUREMENTS OF THE PIPE INVERTS SHOWN AS A HEAVY LINE ON THIS DWG. SHALL BE PERFORMED IN ACCORDANCE WITH SPECIFICATION 7220-C-82(Q) SECTION 8.0

2. PROFILE MEASUREMENTS OF THE PIPES SHOWN AS HEAVY LINES SHALL BE PERFORMED BY OPTICAL METHODS (STANDARD TRANSIT AND LEVEL)



LEGEND

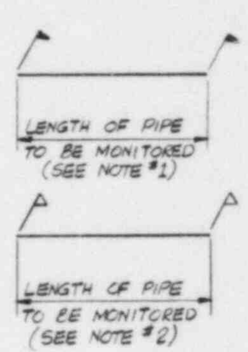
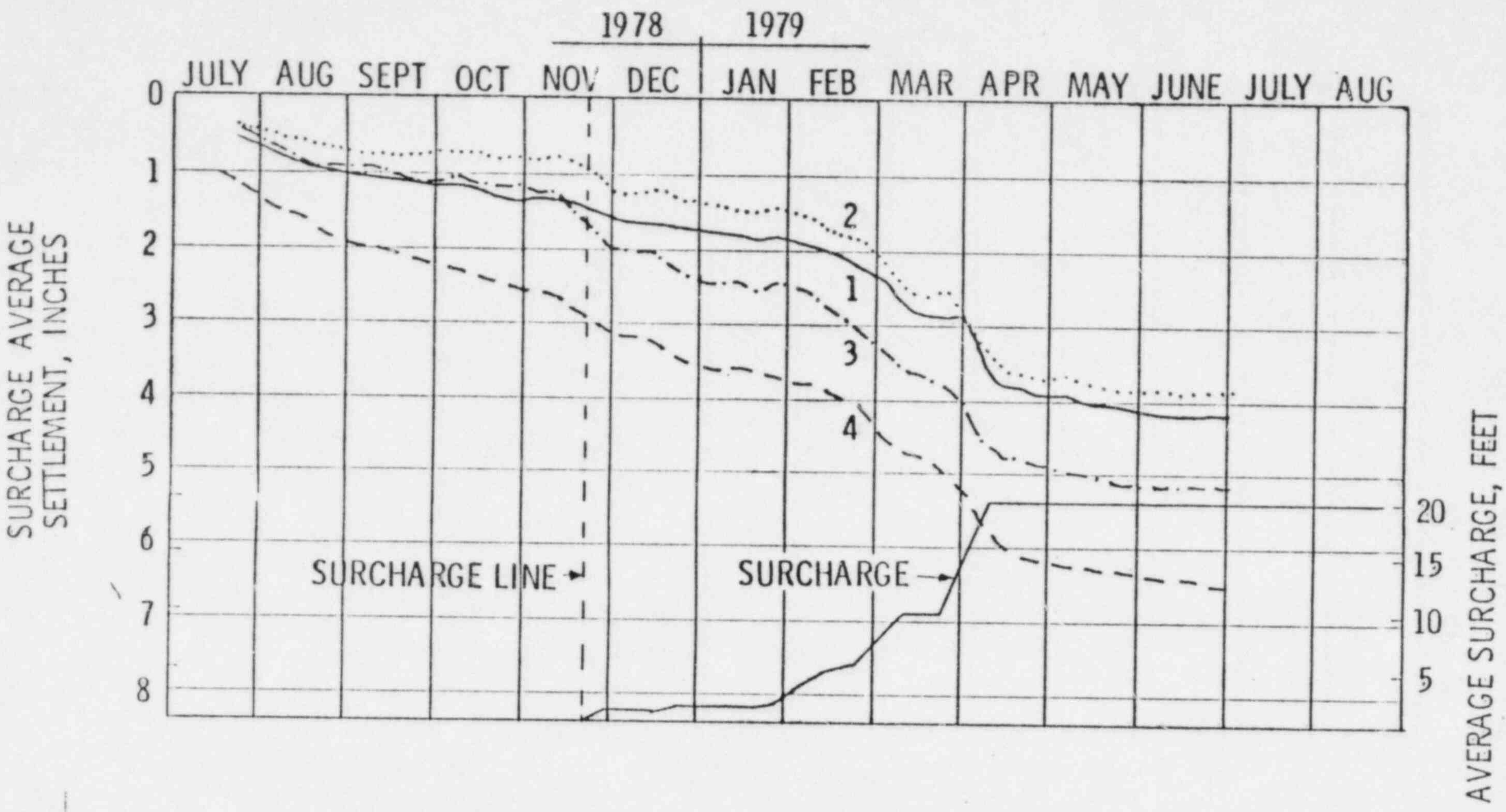
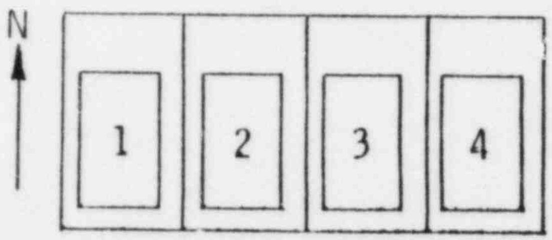


Fig. 6

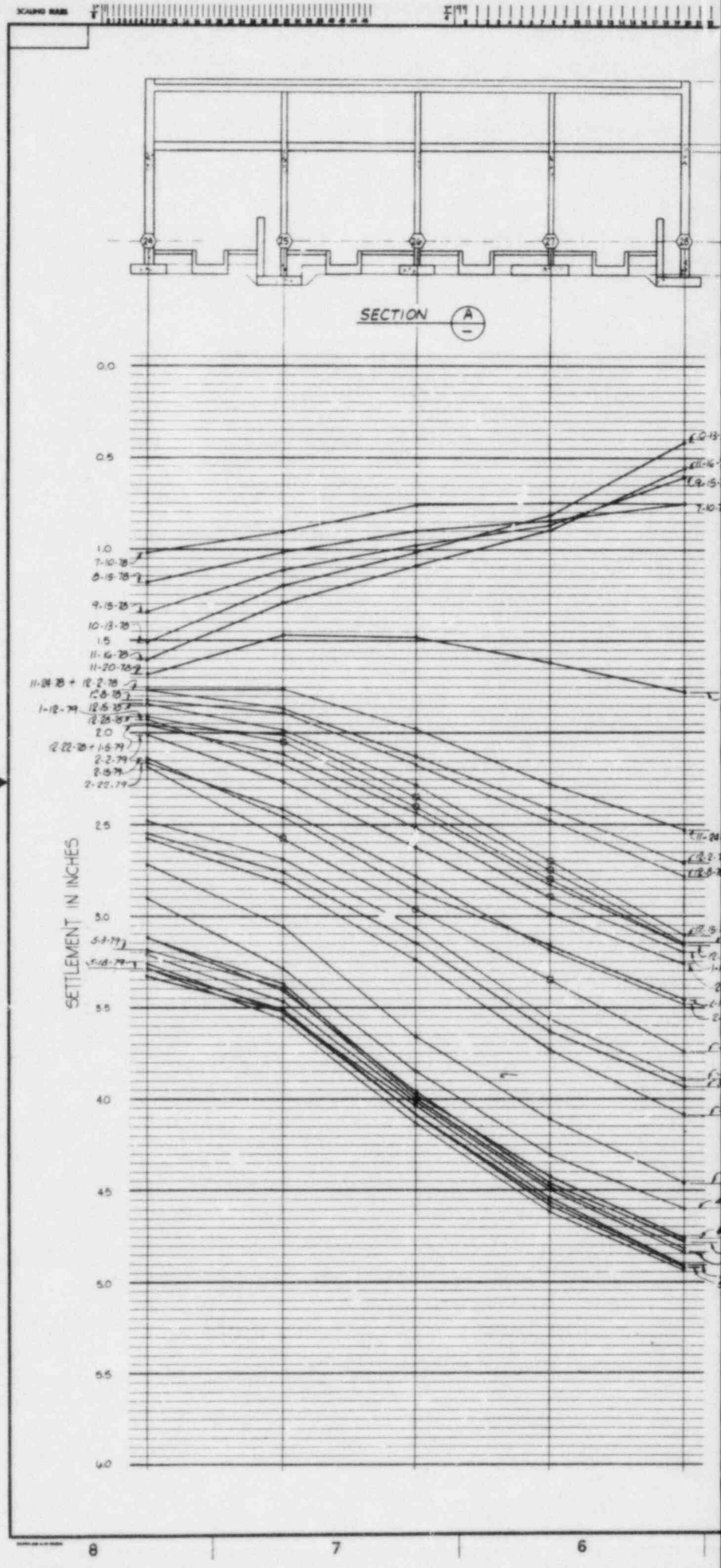




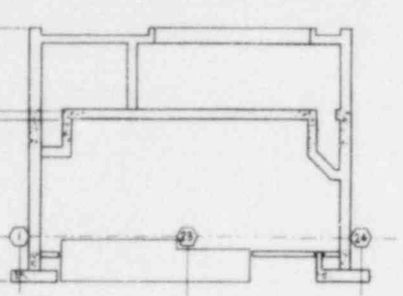
DIESEL GENERATOR BUILDING

FIGURE 7

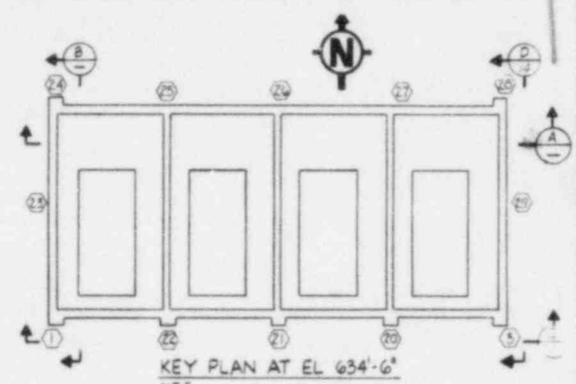
DIESEL GENERATOR BUILDING
AVERAGE PEDESTAL SETTLEMENT VS TIME


SCALING RULER
SECTION A
24
25
26
27
28
SETTLEMENT IN INCHES
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
7-10-78
8-15-78
9-15-78
10-13-78
11-16-78
11-20-78
11-24-78 + 12-2-78
12-8-78
1-12-79
12-15-78
12-28-78
12-22-78 + 1-5-79
2-2-79
2-15-79
2-22-79
5-1-79
5-18-79
8
7
6

SIZE E

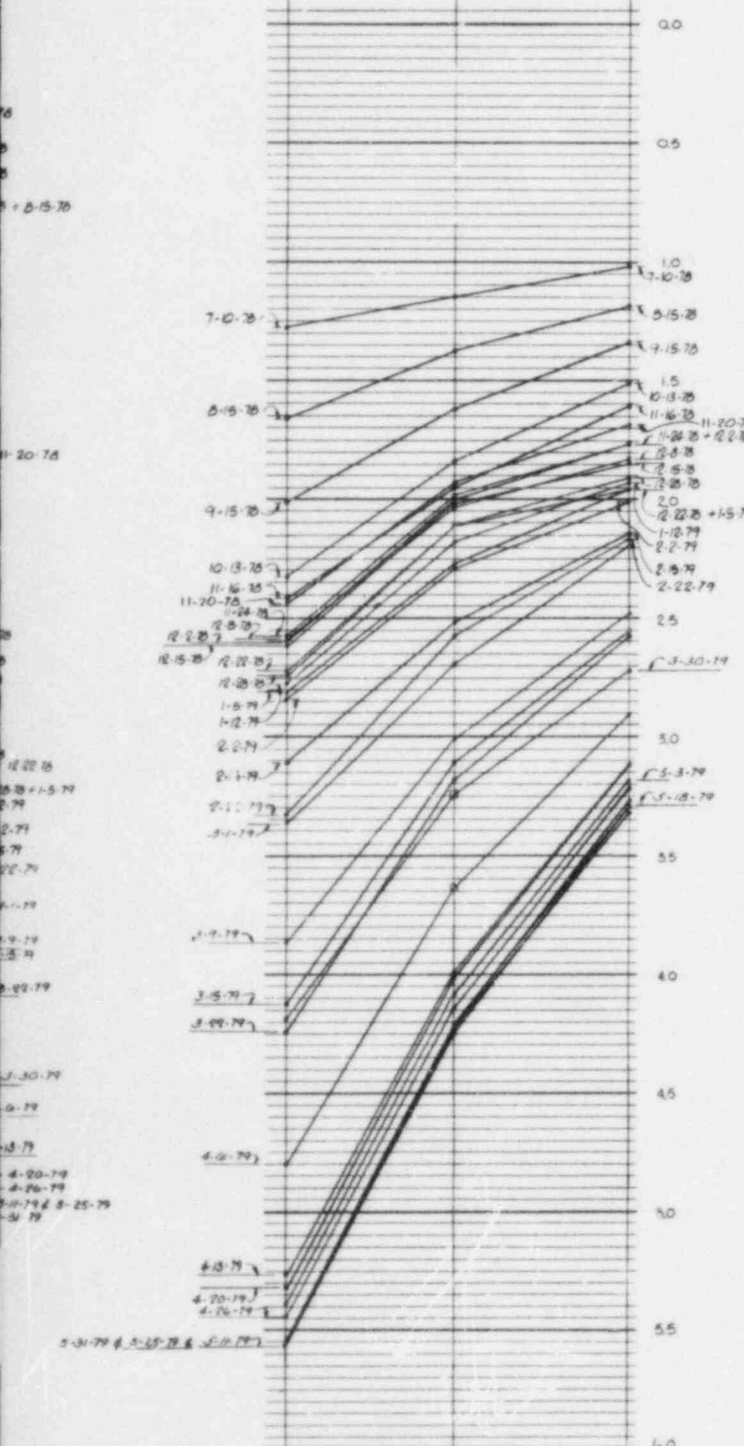


SECTION (B)



KEY PLAN AT EL 634'-6"

N.T.S.



SIGNIFICANT EVENTS				
NO	EVENT	BAY NO	DATE	REMARKS
1.	BUILDING FDN PLACEMENT TO EL. 630.5'		10-5-77 TO 1-25-78	
2.	WALLS TO EL. 635'		12-13-77 TO 2-20-78	
3.	DIESEL GEN. PEDESTAL FDN	1 2 3 4	3-23-78 3-8-78 2-14-78 1-6-78	
4.	WALLS TO EL 654'		3-14-78 TO 4-28-78	
5.	HVAC CHAMBER SLABS @ EL. 656.5'		7-10-78 TO 8-3-78	
6.	ISOLATE ELEC DUCT BANKS	3 1 4 2	11-16-78 11-18-78 11-21-78 11-24-78	
7.	BEGAN PLACING APPROX 4' OF FROST PROTECTION		11-28-78	
8.	PLACE MEZZANINE FLOOR TO ELEV. 664'-0"	4 3 1 2	12-18-78 12-19-78 12-20-78 12-28-78	
9.	WALLS TO EL. 678.25'		1-5-79 TO 2-20-79	
10.	ISOLATE CONDENSATE LINES		1-22-79 TO 1-30-79	
11.	COMPLETE PLACING 10 FT OF SURCHARGE		3-3-79	
12.	ROOF AND PARAPET TO EL. 681.5'		3-8-79 TO 3-22-79	
13.	COMPLETE PLACING 20 FT OF SURCHARGE		4-7-79	

NOTES

1. PERMANENT DIESEL GENERATOR BUILDING (DG) SETTLEMENT MARKERS WERE INSTALLED TO REPLACE THE TEMPORARY CONSTRUCTION SCRIBES ON 11-15-78.
2. THE FINAL SETTLEMENT READINGS FOR THE SCRIBES WERE TAKEN ON 11-24-78 AT THAT TIME A CORRELATION BETWEEN THE SCRIBES AND SETTLEMENT MARKERS WAS ESTABLISHED.

LEGEND

⬡ DENOTES SETTLEMENT MARKERS

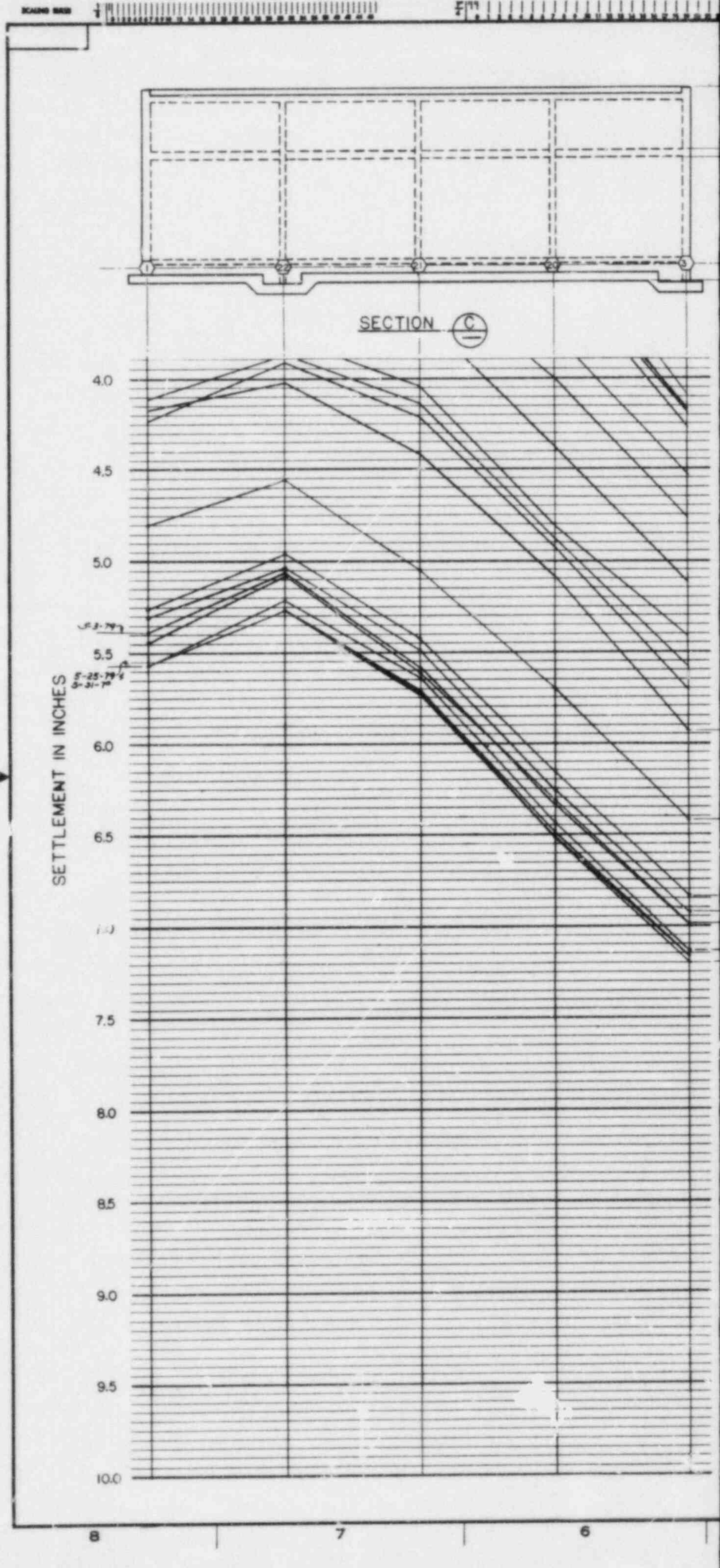
□ ESTIMATED SETTLEMENT

UPDATED AS OF 6-1-79

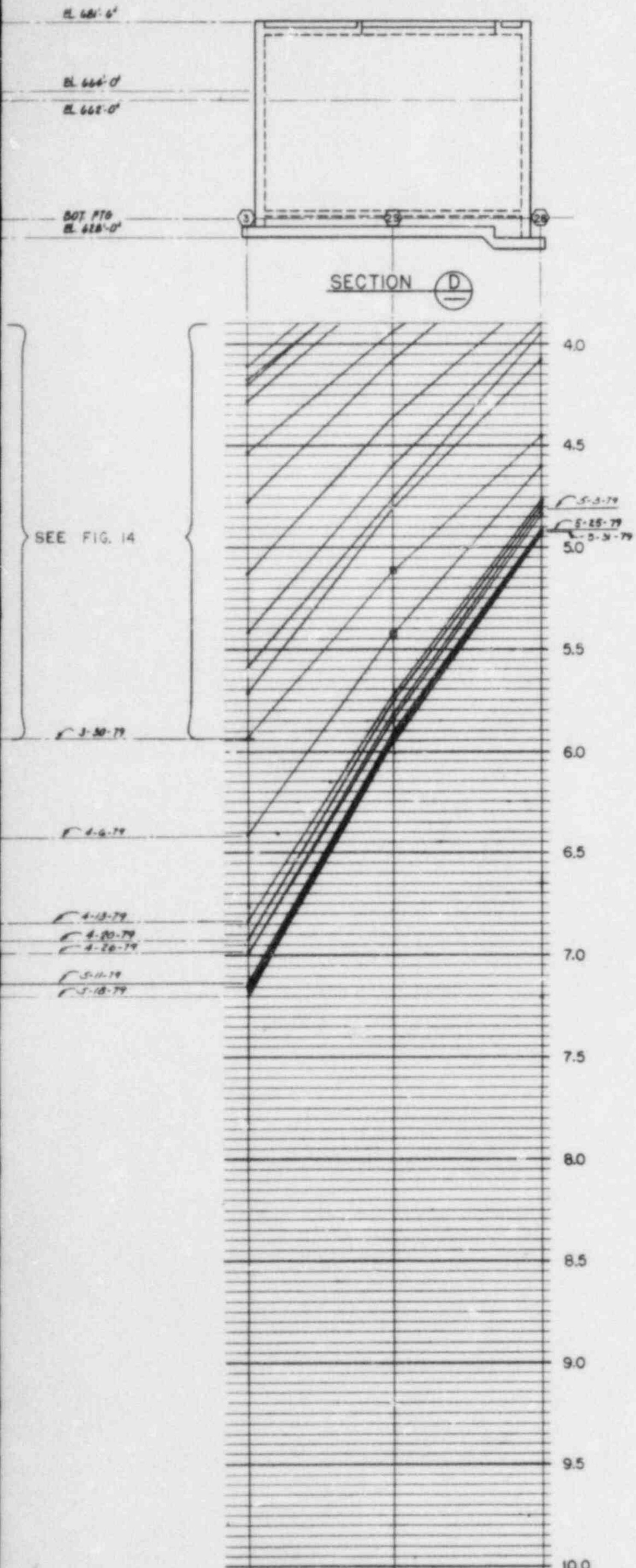
MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

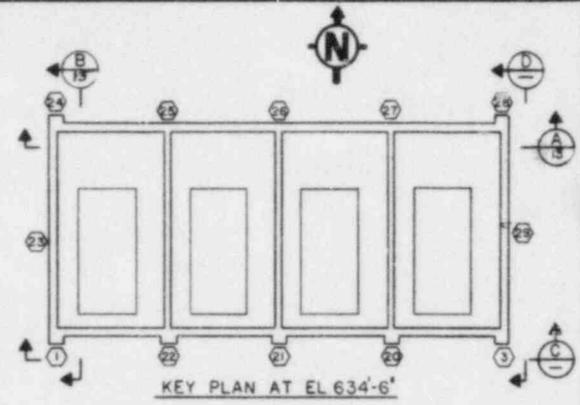
DIESEL GENERATOR BLDG.
SETTLEMENT DATA

FIGURE NO.	REV.
8	6B



SECTION C
SETTLEMENT IN INCHES
4.0
4.5
5.0
5.5
6.0
6.5
7.5
8.0
8.5
9.0
9.5
10.0
8
7
6





NOTES

1. FOR NOTES SEE FIGURE 13

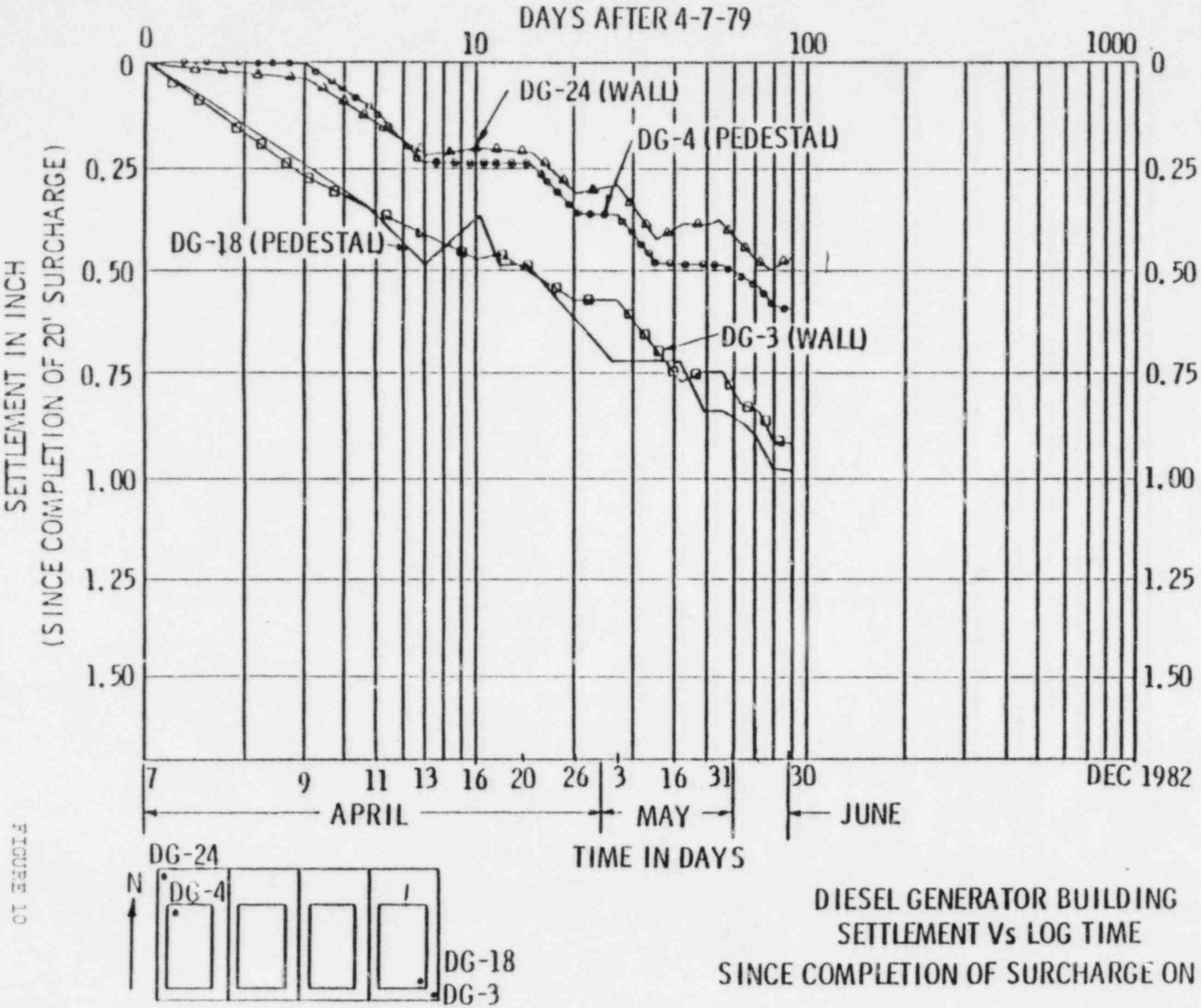
UPDATED AS OF 6-1-79

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

DIESEL GENERATOR BLDG
SETTLEMENT DATA SH. 2

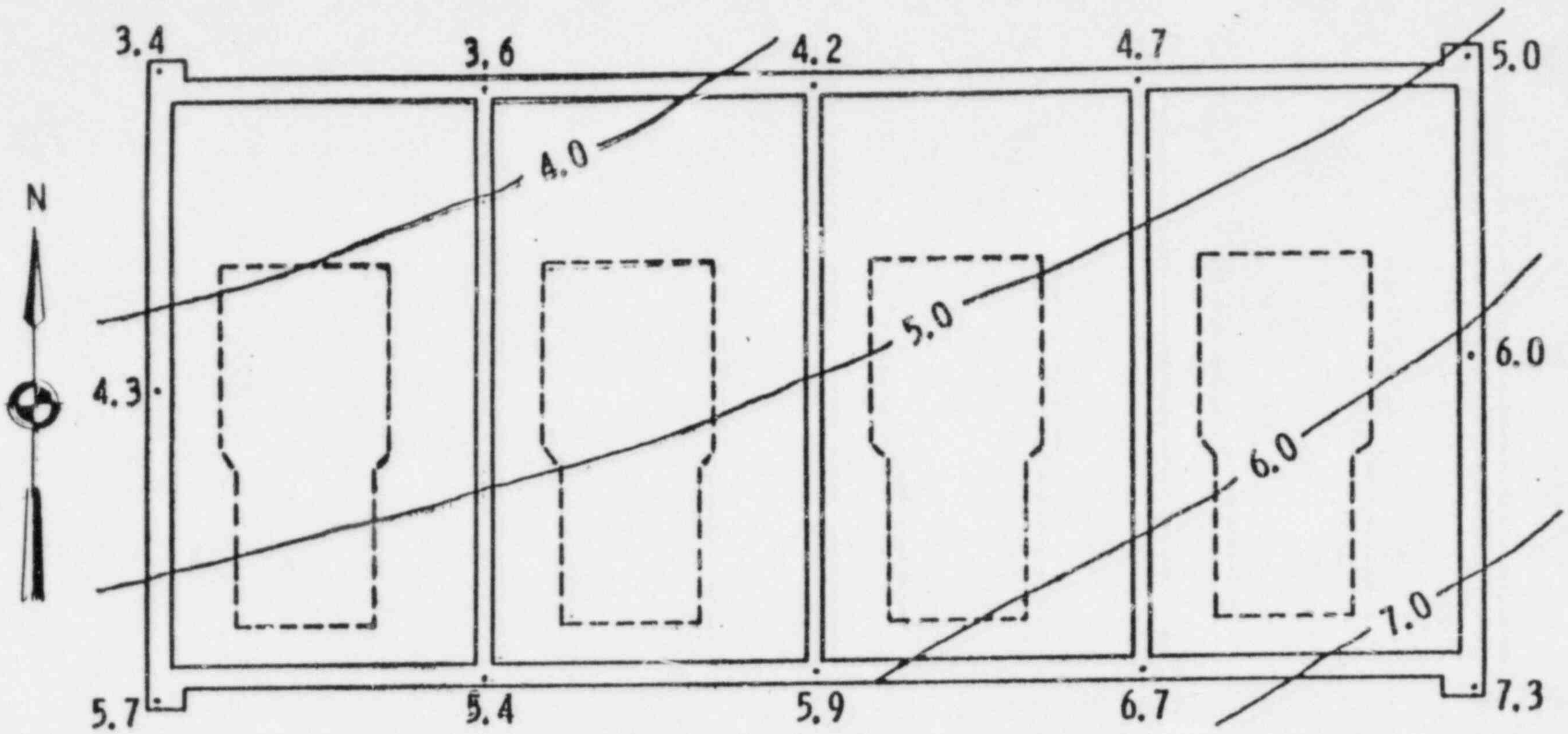
FIGURE NO. 9 | REV. 6R

SIZE E



DIESEL GENERATOR BUILDING
SETTLEMENT Vs LOG TIME
SINCE COMPLETION OF SURCHARGE ON 4-6-79

FIGURE 10



DIESEL GENERATOR BUILDING

TOTAL SETTLEMENT OF WALLS FROM 7-14-78 TO 6-29-79 IN INCHES

(20 FEET OF SURCHARGE)

FIGURE 11

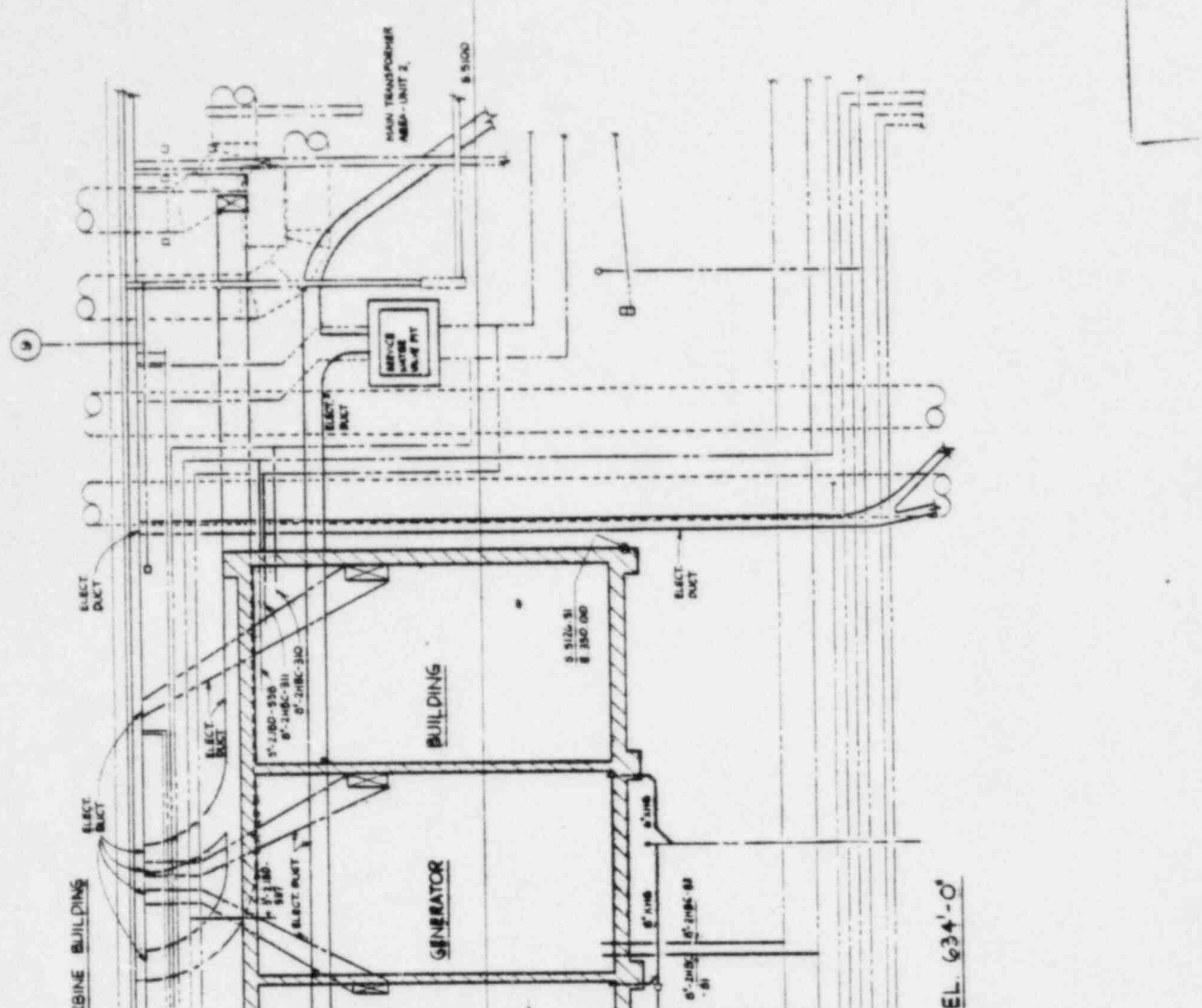

TURBINE BUILDING
DIESEL
GENERATOR
BUILDING
ELECT. DUCT
CONC. CASING
SERVICE WATER VALVE PIT
MAIN TRANSFORMER AREA - UNIT 1
MAIN TRANSFORMER AREA - UNIT 2
PLAN AT EL. 634'-0"

FIGURE 12

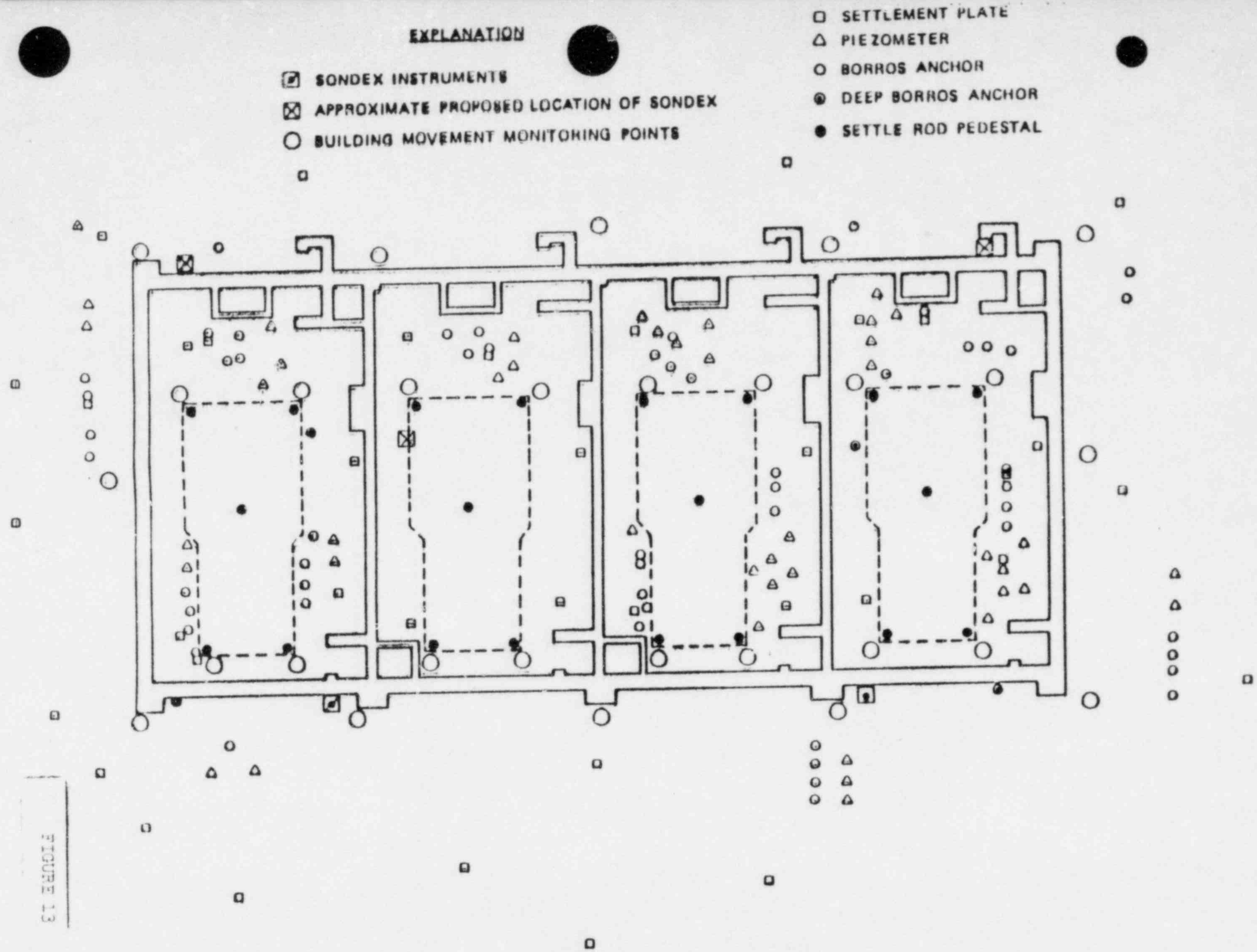
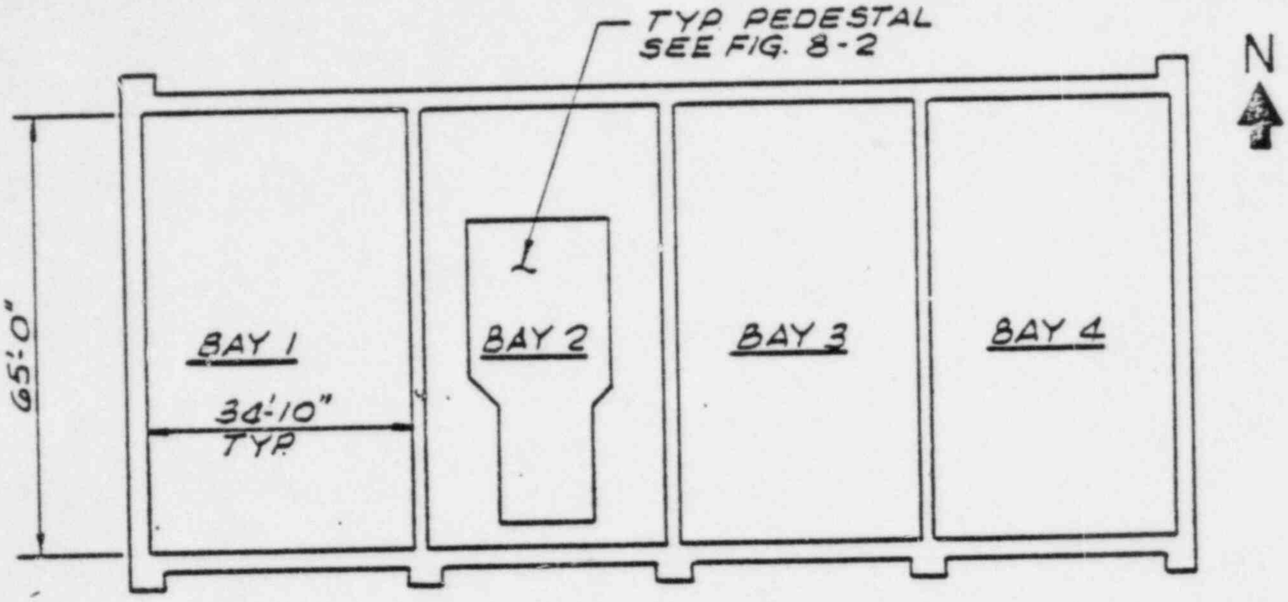
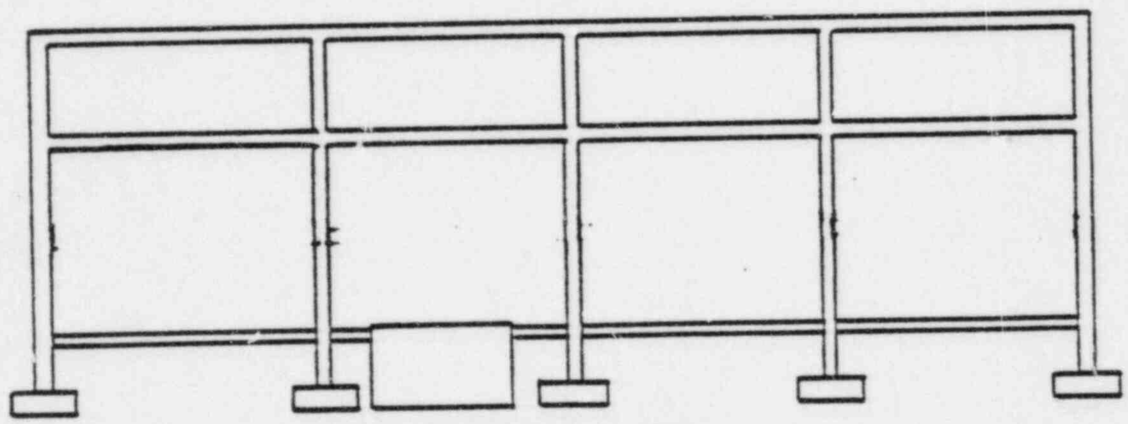

EXPLANATION
SONDEX INSTRUMENTS
APPROXIMATE PROPOSED LOCATION OF SONDEX
BUILDING MOVEMENT MONITORING POINTS
SETTLEMENT PLATE
PIEZOMETER
BORROS ANCHOR
DEEP BORROS ANCHOR
SETTLE ROD PEDESTAL

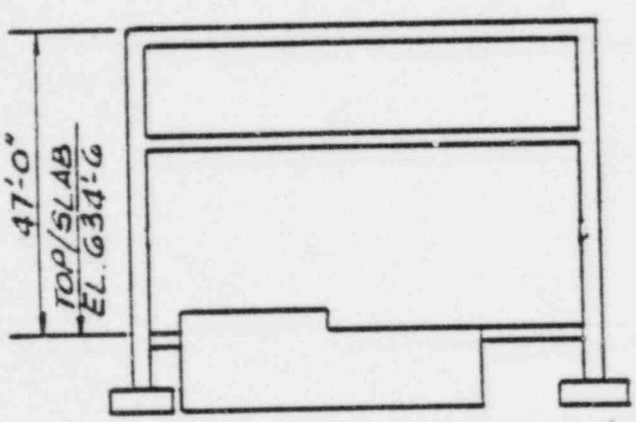
FIGURE 13



PLAN



SECTION
LOOKING NORTH



SECTION
LOOKING WEST

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

DIESEL GENERATOR BLDG
PLAN & SECTIONS

FIG.14

DATE: 4/24/79

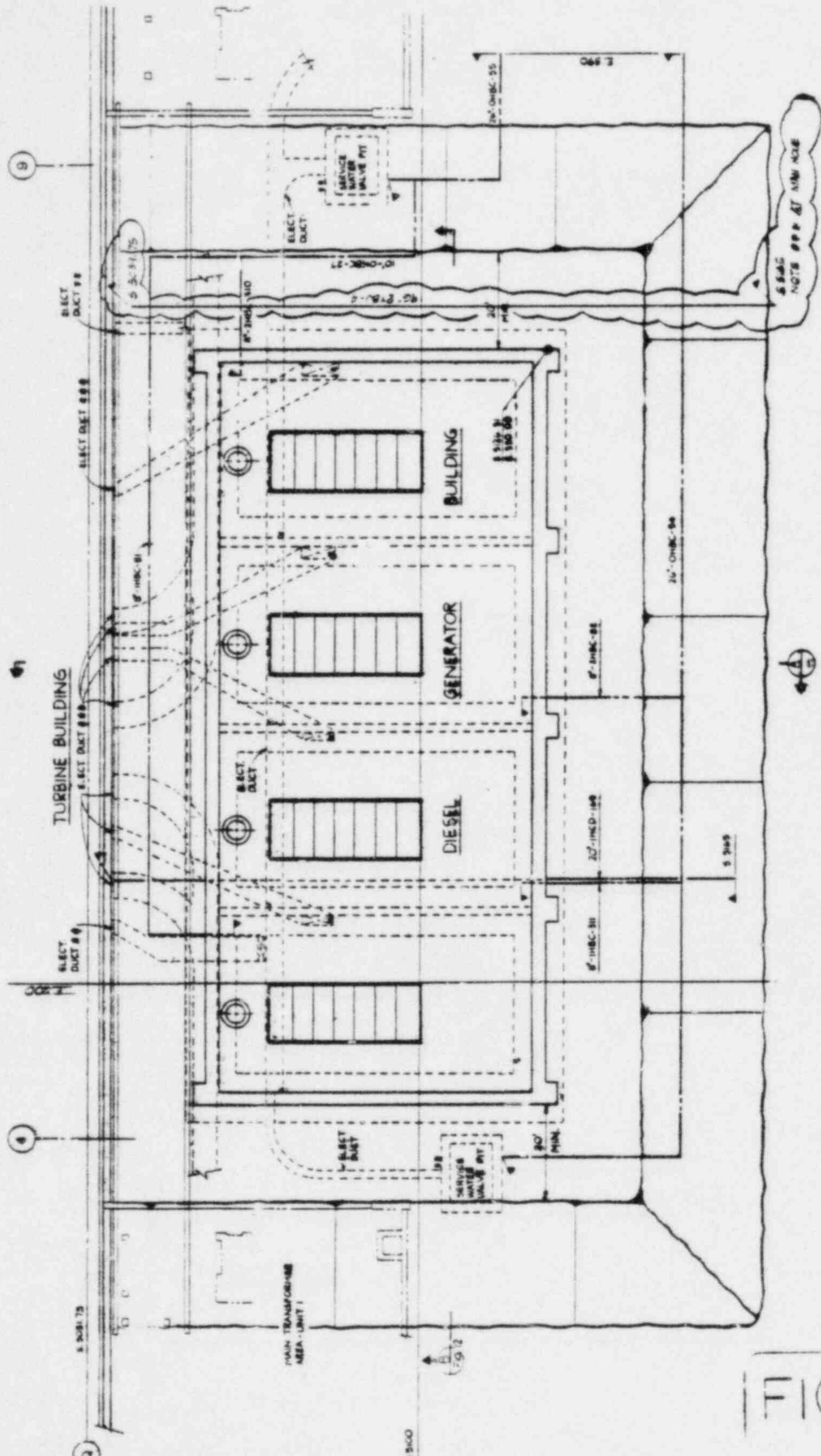
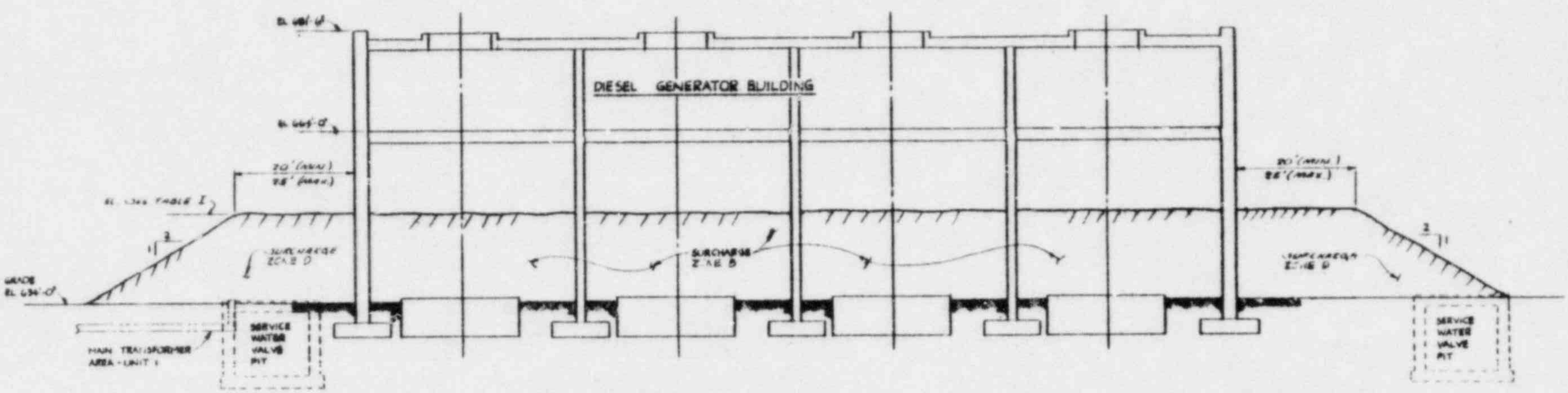
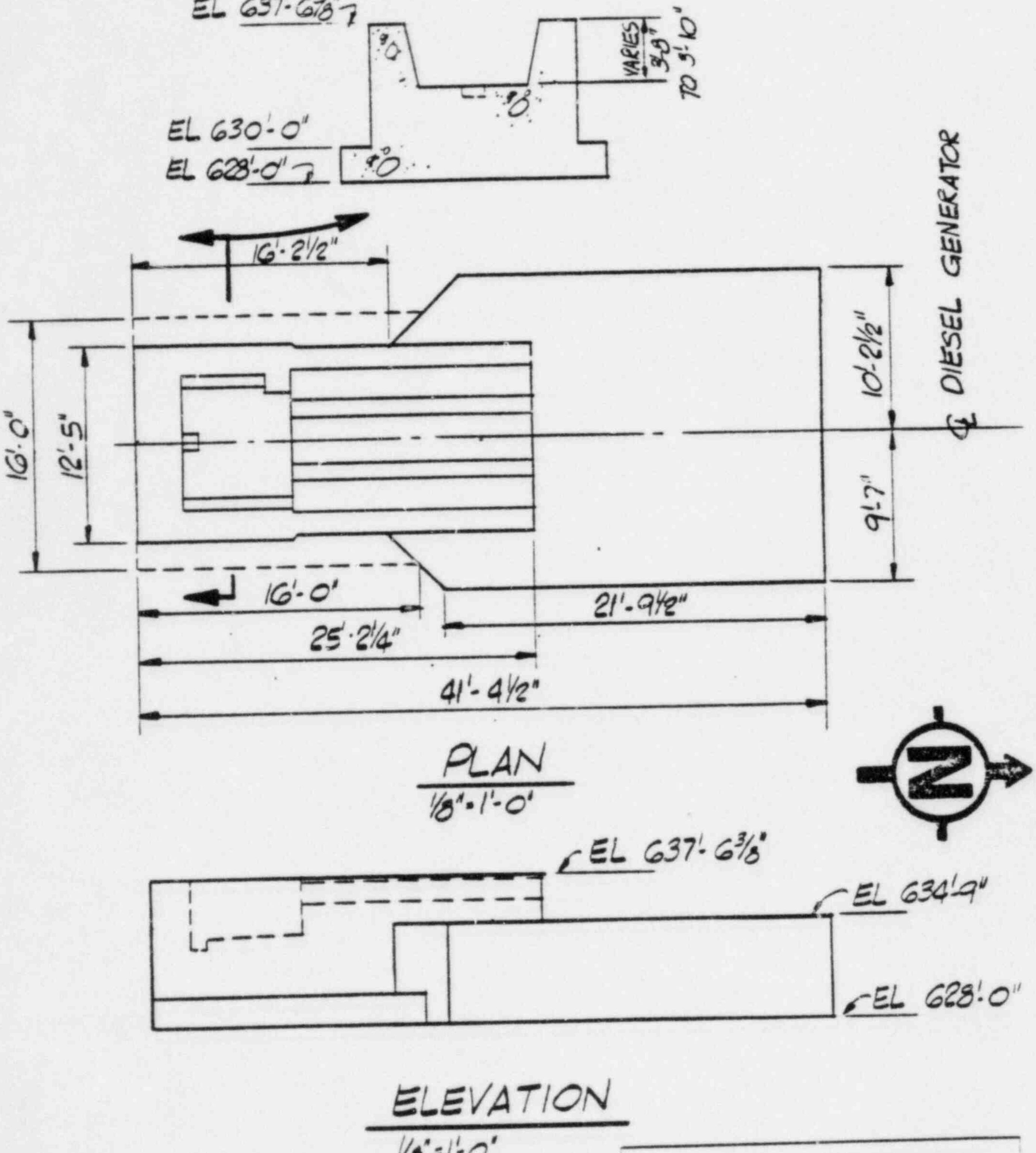


FIG.15



SECTION B

FIG.16



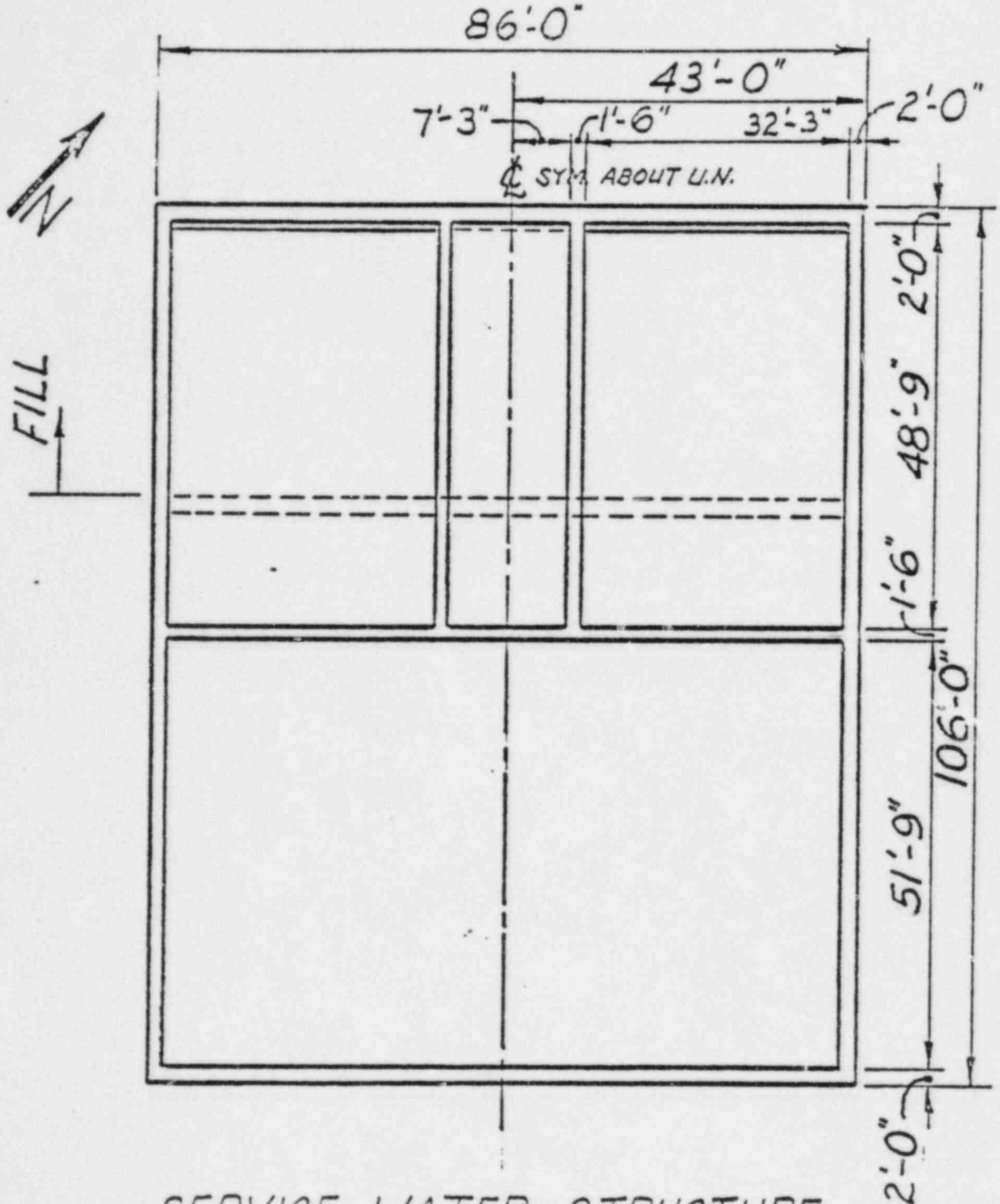
DIESEL GENERATOR PEDESTAL

MIDLAND PLANT UNITS 1 & 2 CONSUMERS POWER COMPANY
DIESEL GENERATOR PEDESTAL

FIG. 17

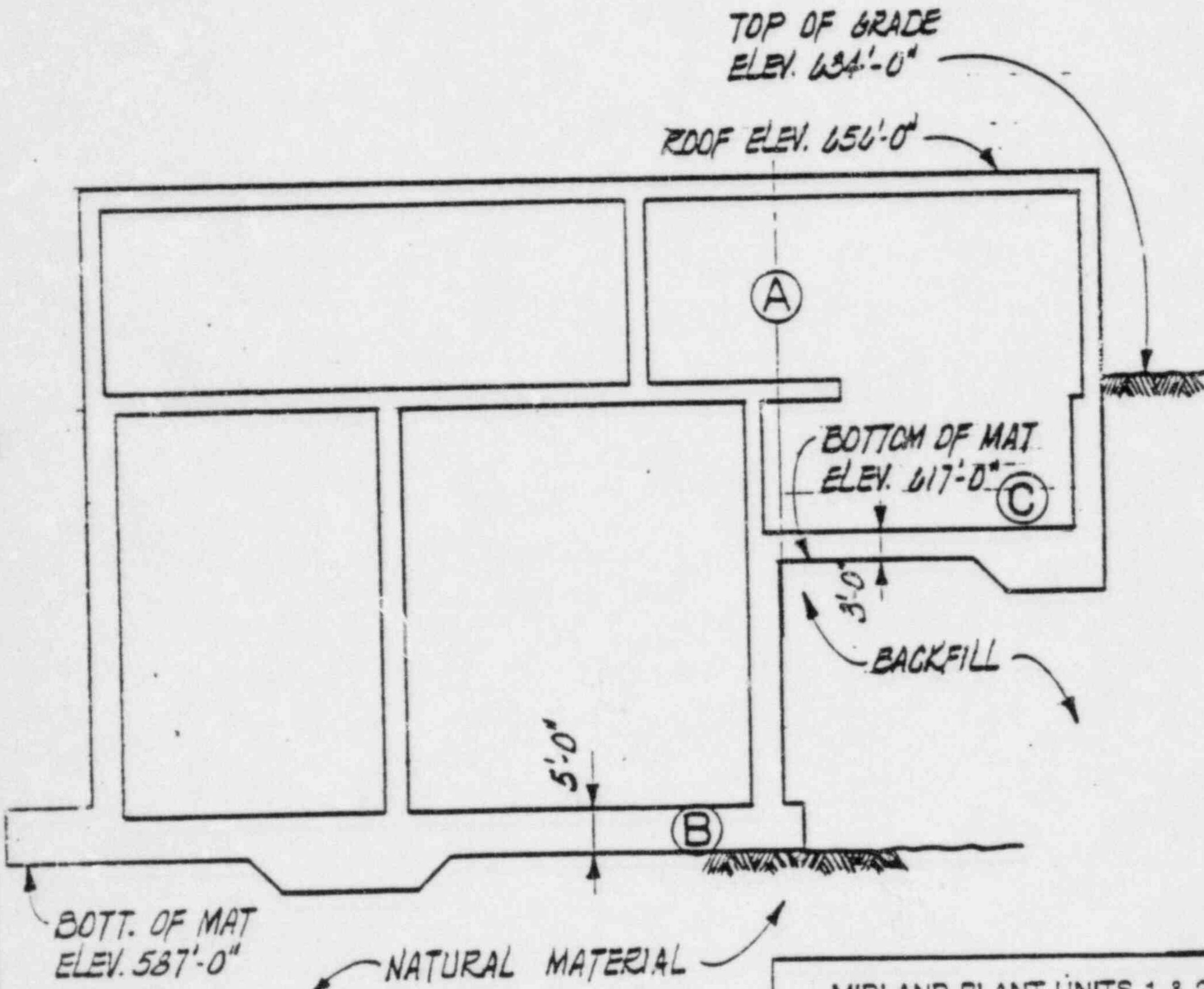
DATE: 4.24.79

G-0588



SERVICE WATER STRUCTURE
PLAN AT EL. 634'-6"

FIG. 18



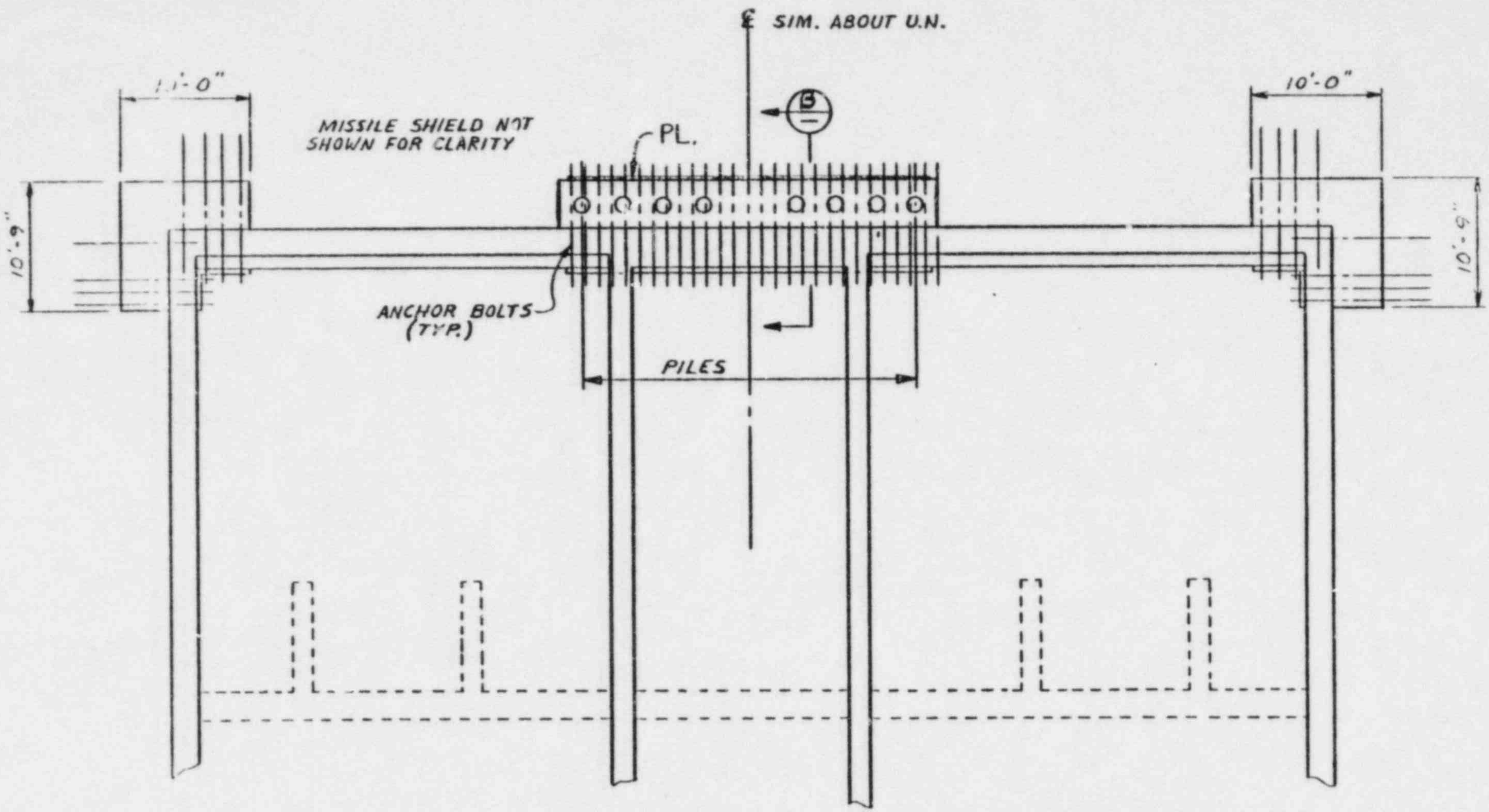
TYPICAL SECTION
(LOOKING WEST)
SERVICE WATER
STRUCTURE

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

SERVICE WATER STRUCTURE
TYPICAL SECTION

FIG. 19

DATE: 4/24/79

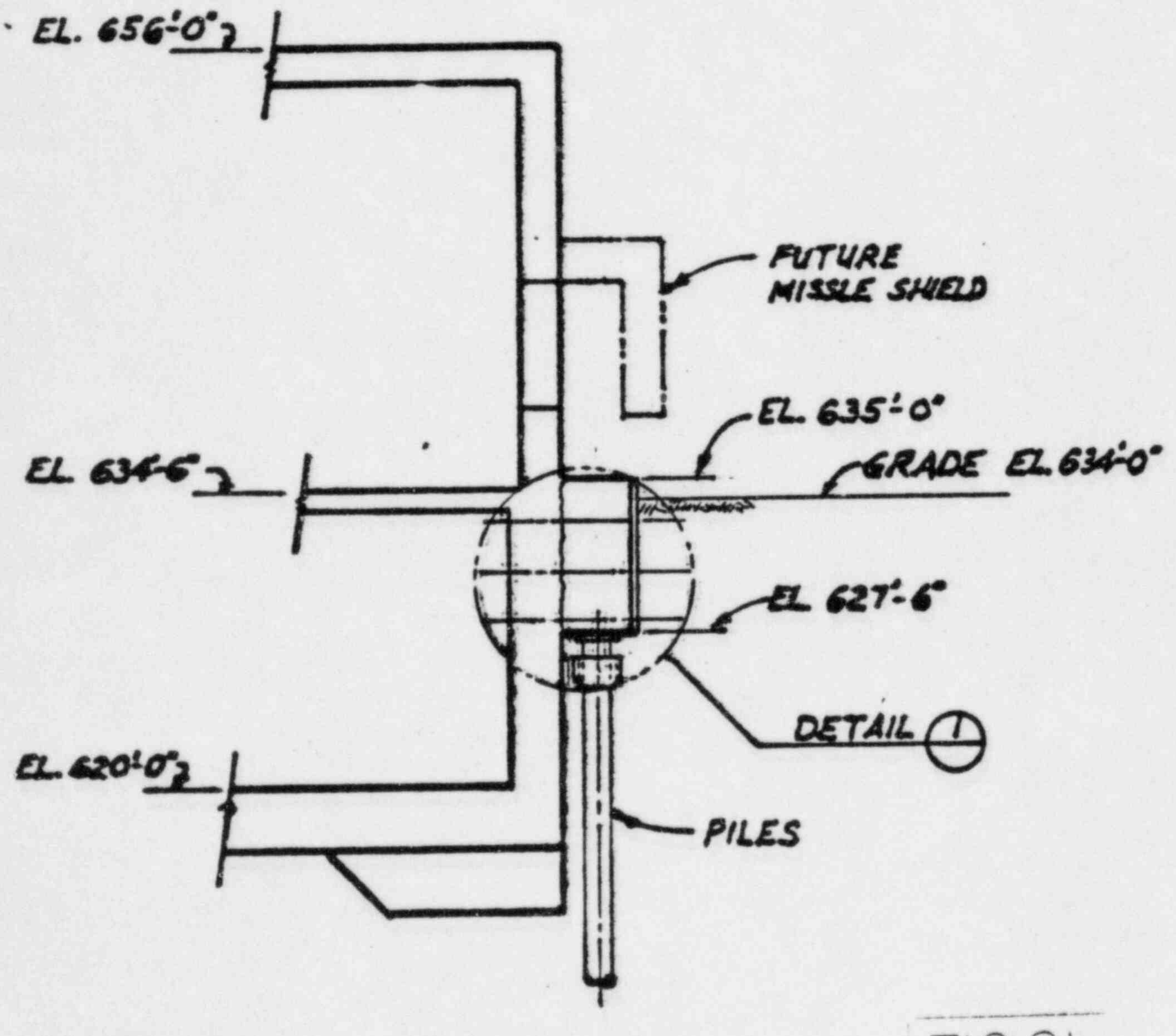


PLAN AT EL. 634'-6"

FIG. 20

$DL + LL + EQ = 2790^K$

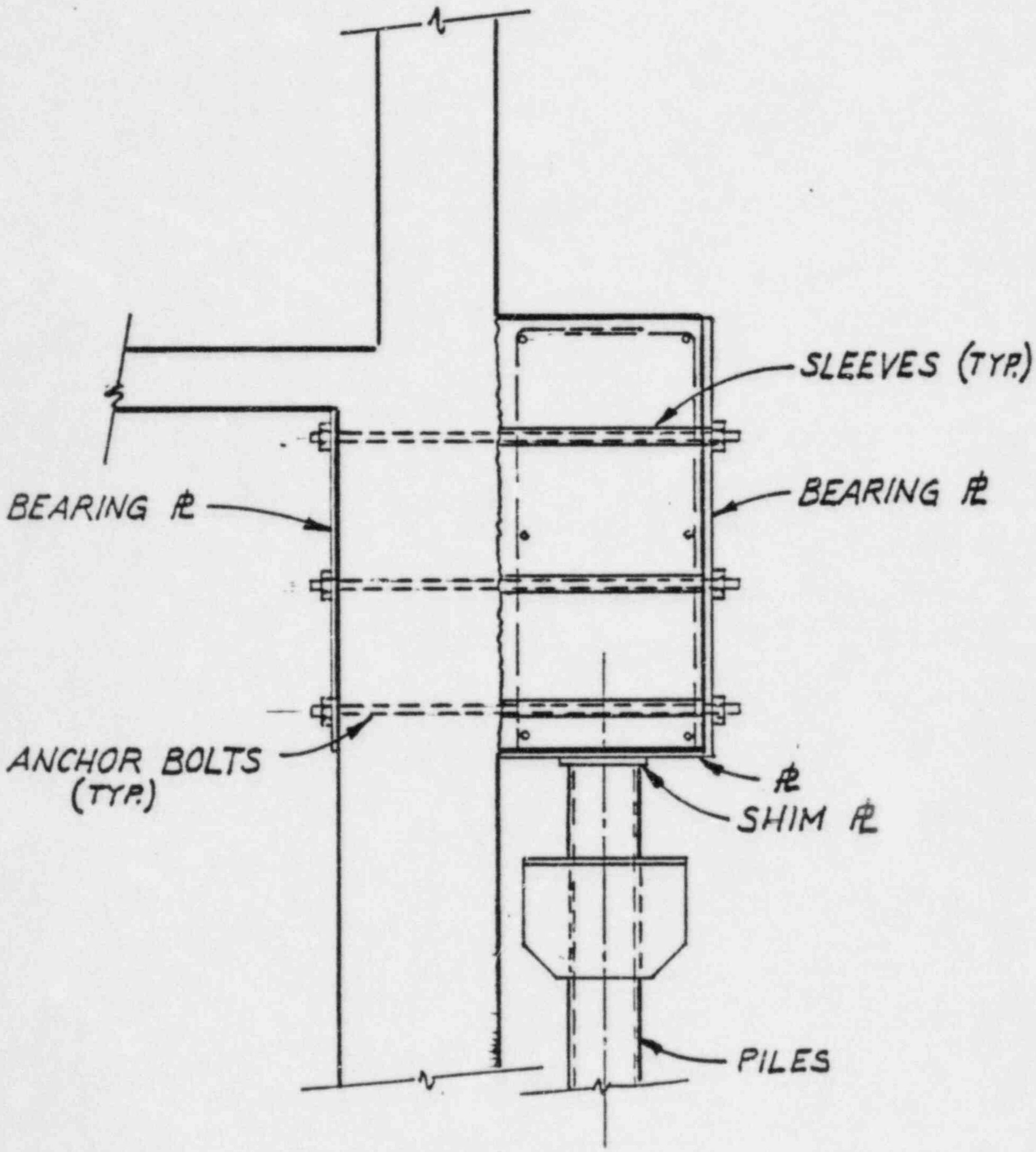
16 PILES @ 100^{TON}/PILE = 3200^K



SECTION B

FIG. 21

REDUCE CAPACITY OF A-540 BOLTS (T.S. = 155 [illegible])
BY 25% TO ACCOUNT FOR STEEL RELAXING, CONCRETE CREEP AND ELASTIC SHORTENING.



DETAIL (1)

FIG.22

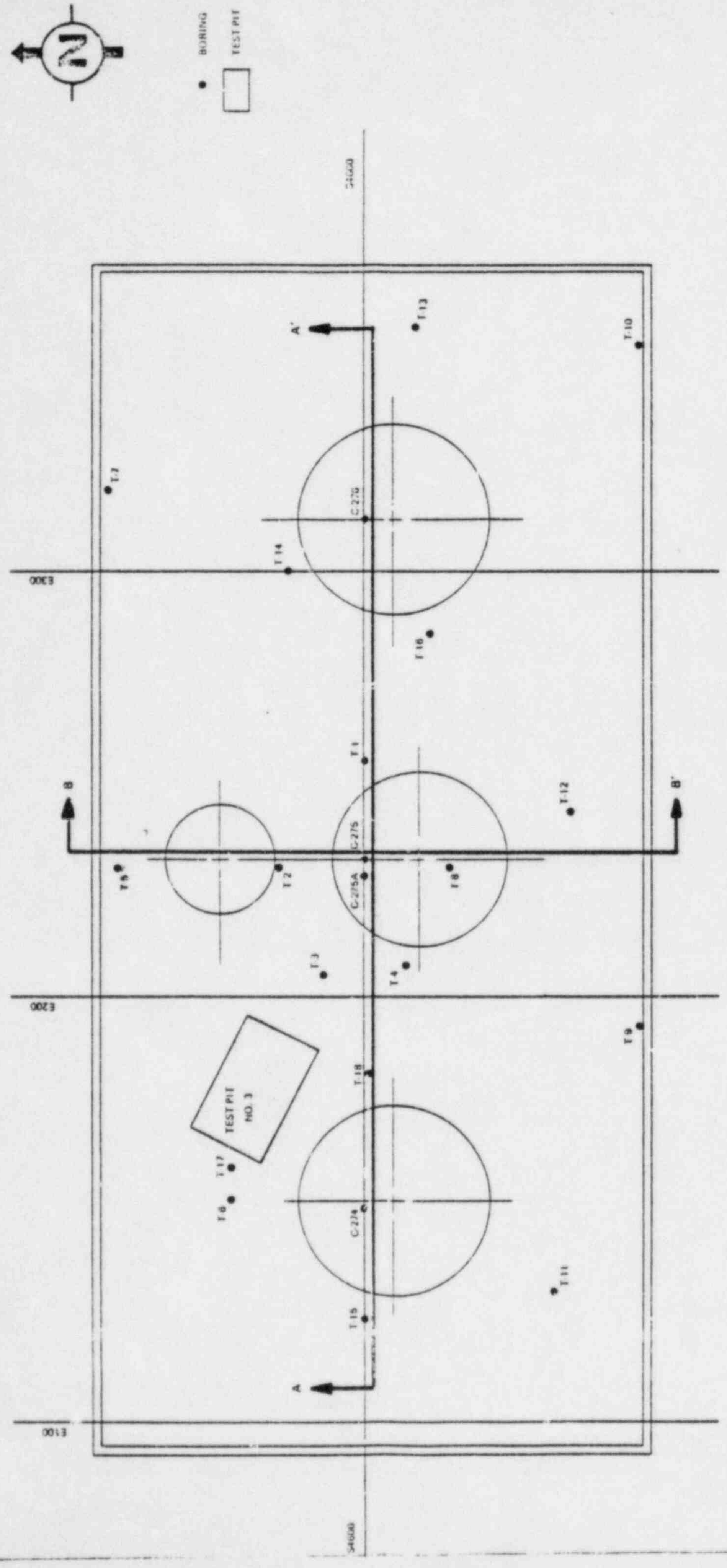
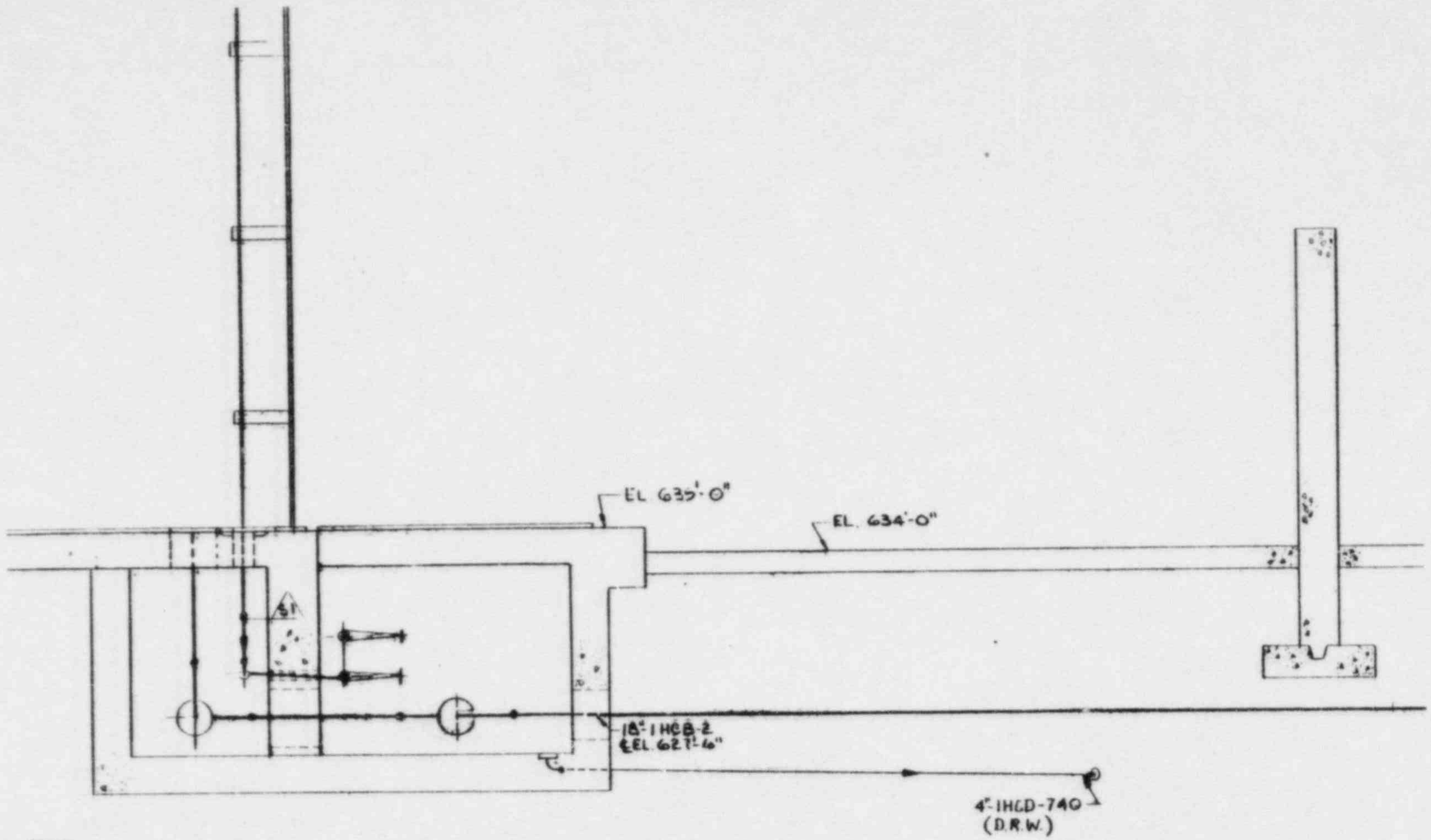

BORING
TEST PIT
TEST PIT NO. 3
C-274
C-275
C-275A
C-276
E100
E200
E300
A
A'
B
B'
T-1
T-2
T-3
T-4
T-5
T-6
T-7
T-8
T-9
T-10
T-11
T-12
T-13
T-14
T-15
T-16
T-17
T-18

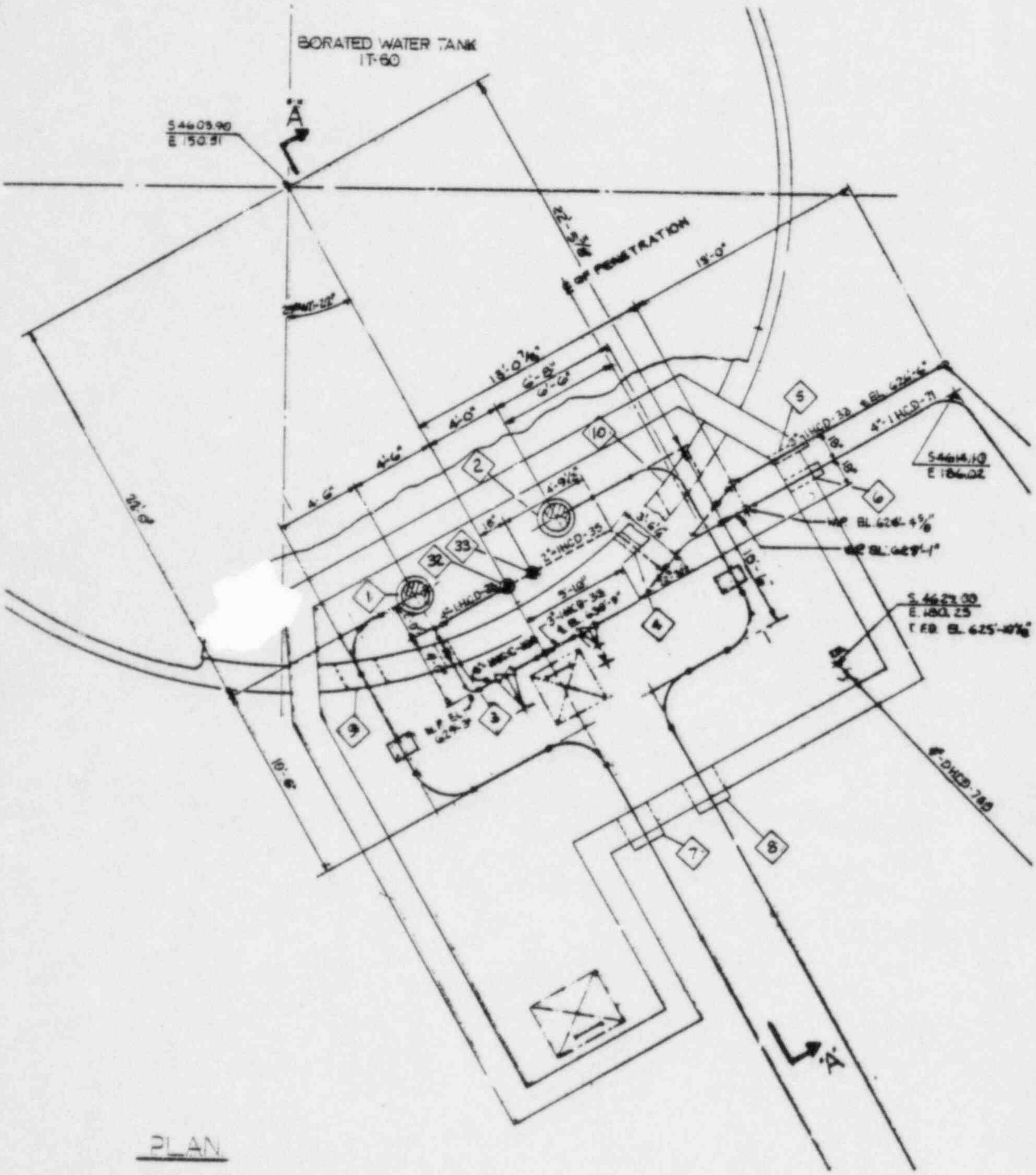
FIG.23



SECTION "A-A"

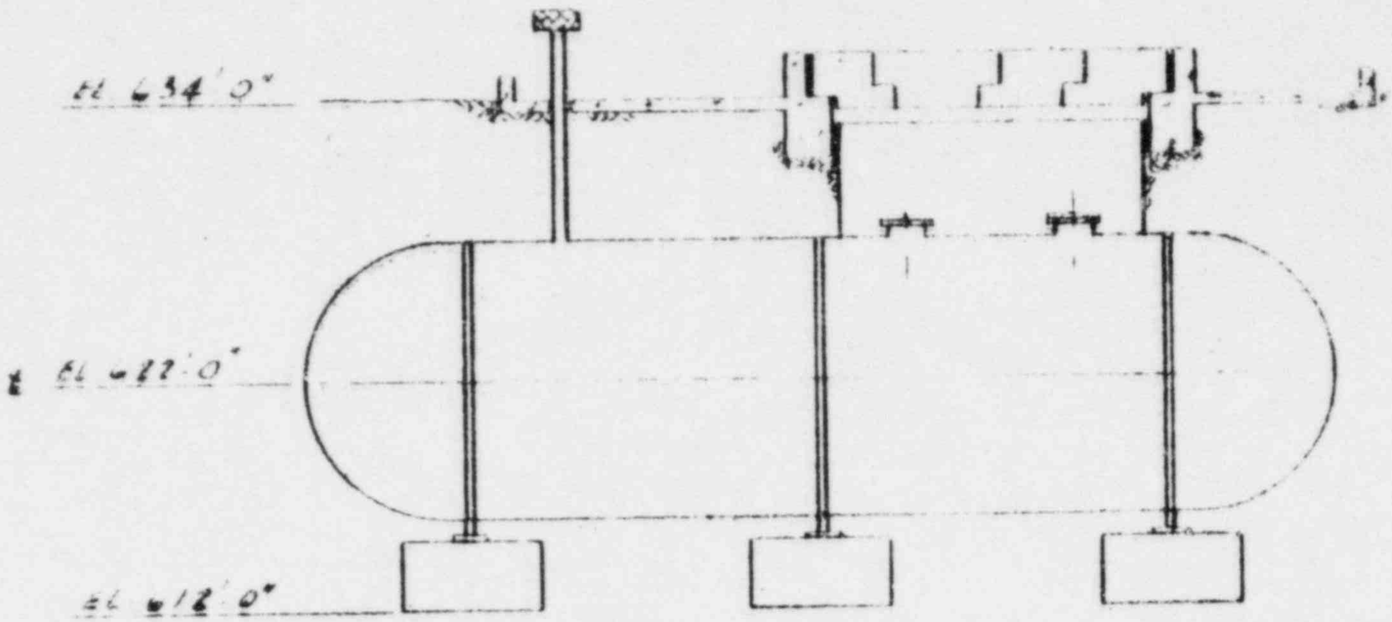
BORATED WATER STORAGE TANK
FOUNDATION & VALVE PIT

FIG.24


BORATED WATER TANK
1T-60
S4609.90
E150.51
A
A
15'-0"
18'-0"
6'-8"
6'-0"
4'-0"
4'-6"
4'-6"
22'-0"
10'-6"
S4614.10
E186.02
S4624.00
E180.25
4"-1HCD-71
6"-DHCD-788

PLAN

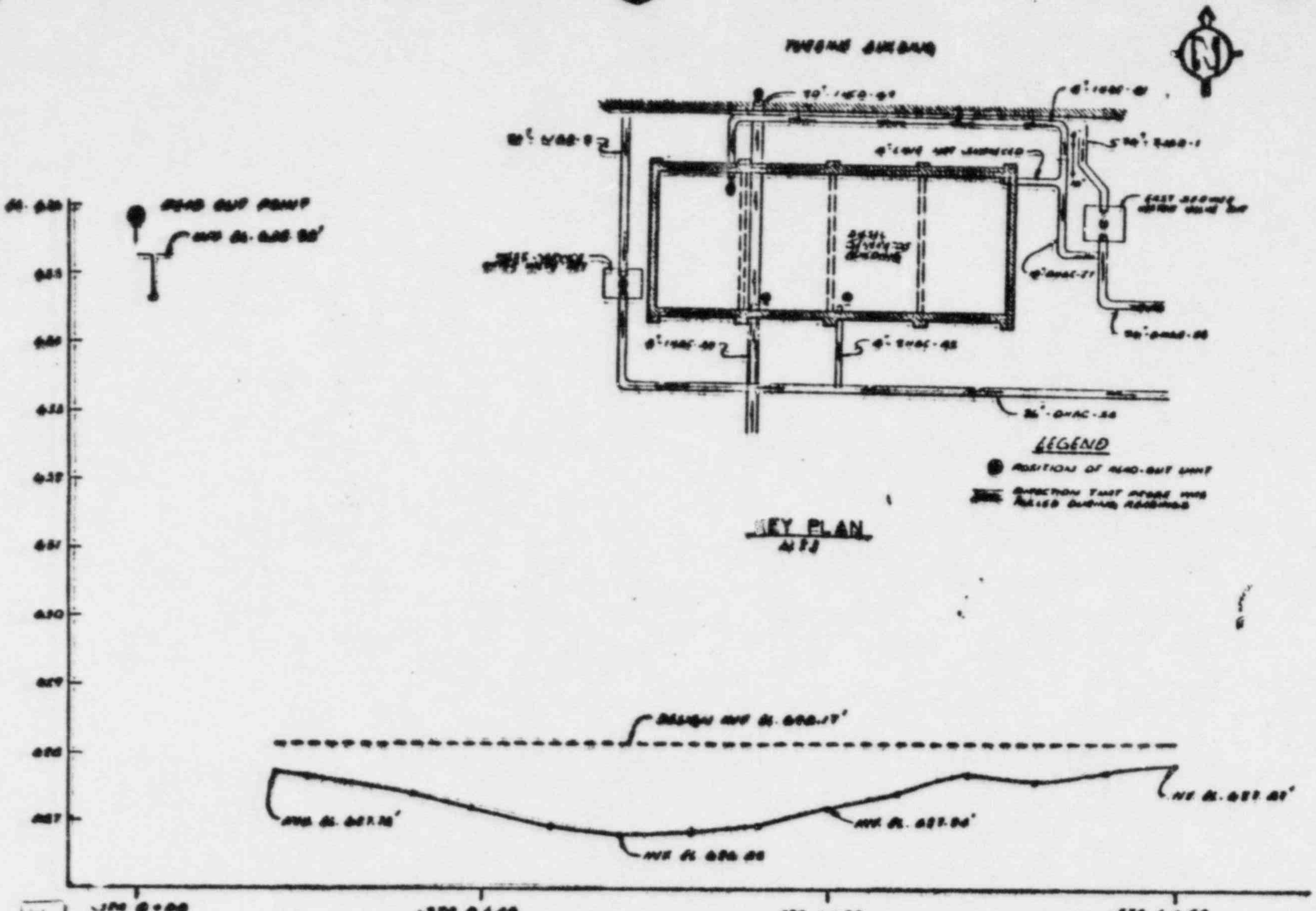
FIG.25



ELEVATION

EMERGENCY DIESEL FUEL OIL STORAGE TANKS (Q)

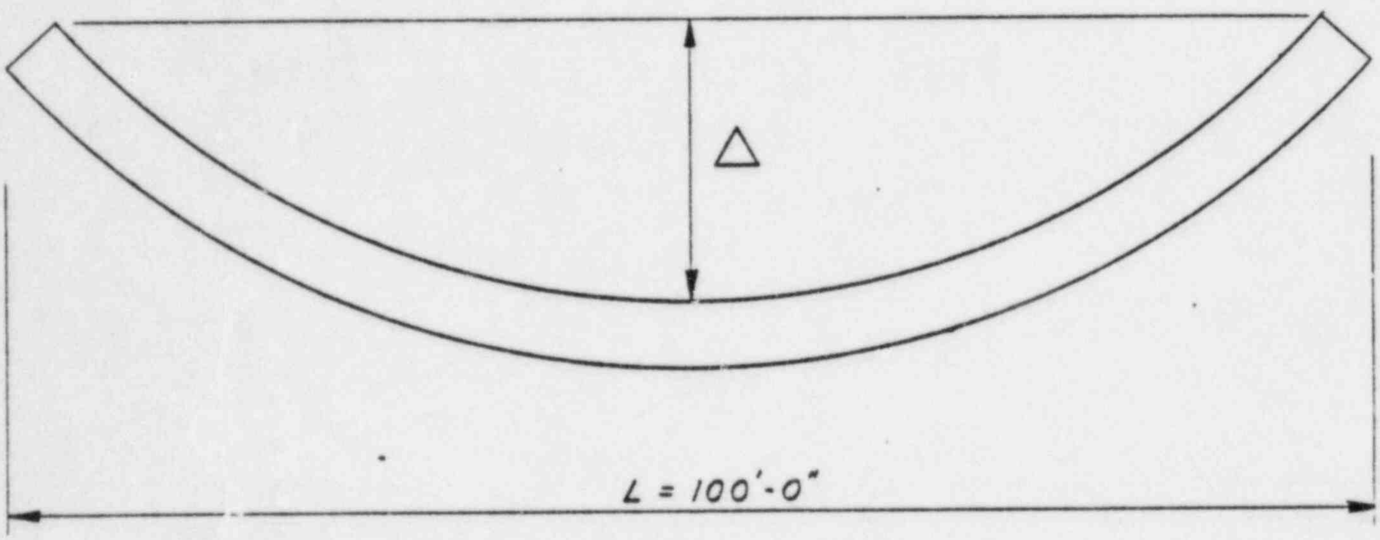
FIG.26



PROFILE 8"-1HBC-81

FIGURE 27

DUCT BANK DEFLECTION



CONSTANT RADIUS OF CURVATURE IS ASSUMED

f'_c = 3000 PSI

E_c = 1,734 KSI
(MODIFIED FOR LONG TERM DEFLECTION PER ACI 318-77 SECTION 9.5.2.5)

FOR A DUCT BANK 43" x 18" DEEP
△ WHEN STEEL YIELDS = 43"

FOR A DUCT BANK 54" x 35" DEEP
△ WHEN STEEL YIELDS = 15"

FIG.28

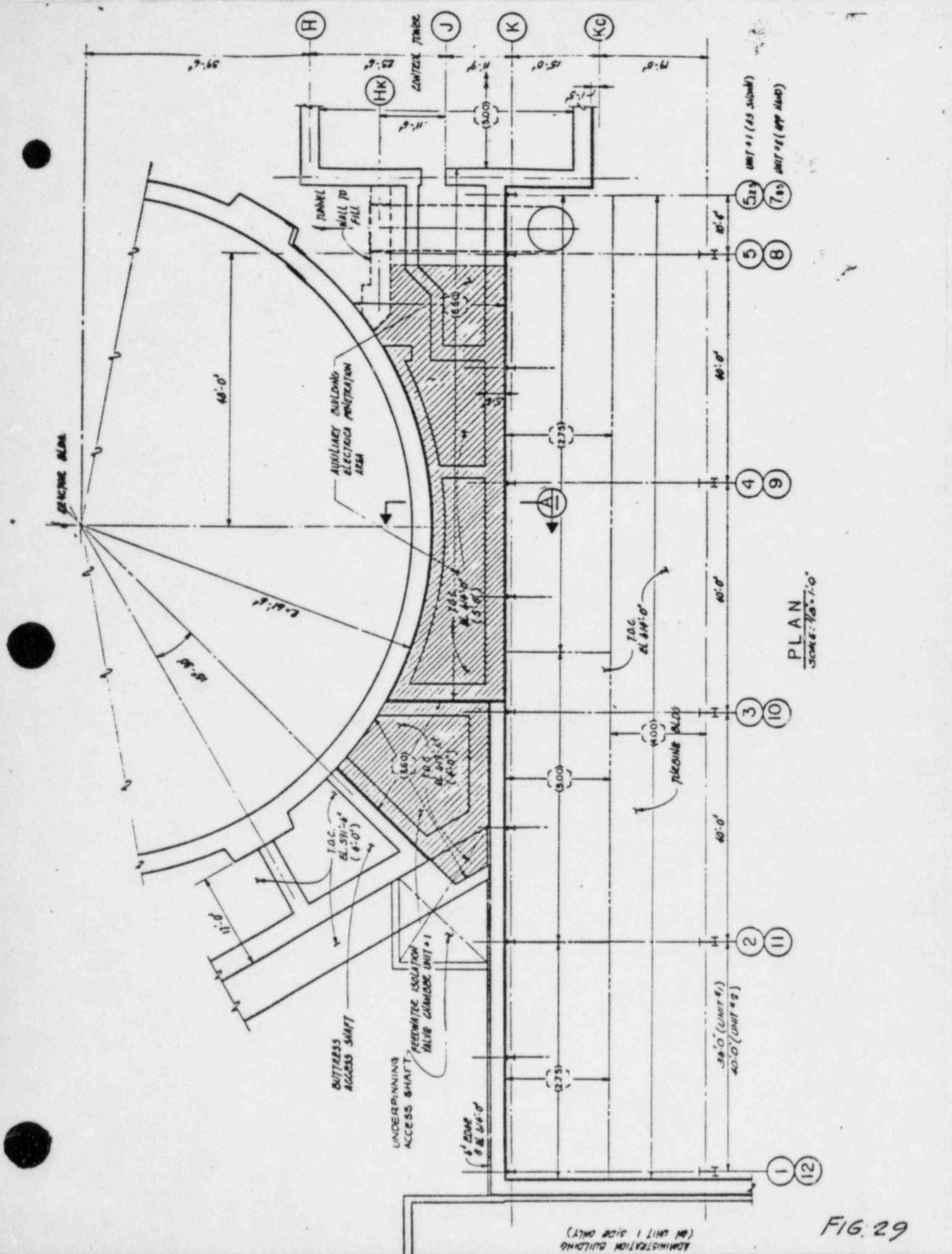

REACTOR BLDG
AUXILIARY BUILDING ELECTRICAL PENETRATION AREA
TUNNEL
WALL TO FILL
CONTROL TOWER
TURBINE BLDG
BUTTRESS ACCESS SHAFT
UNDERPINNING ACCESS SHAFT
FEEDWATER ISOLATION VALVE CHAMBER UNIT #1
ADMINISTRATION BUILDING (ON UNIT 1 SIDE ONLY)
H
HK
J
K
KC
PLAN
SCALE: 1/8"=1'-0"

FIG. 29

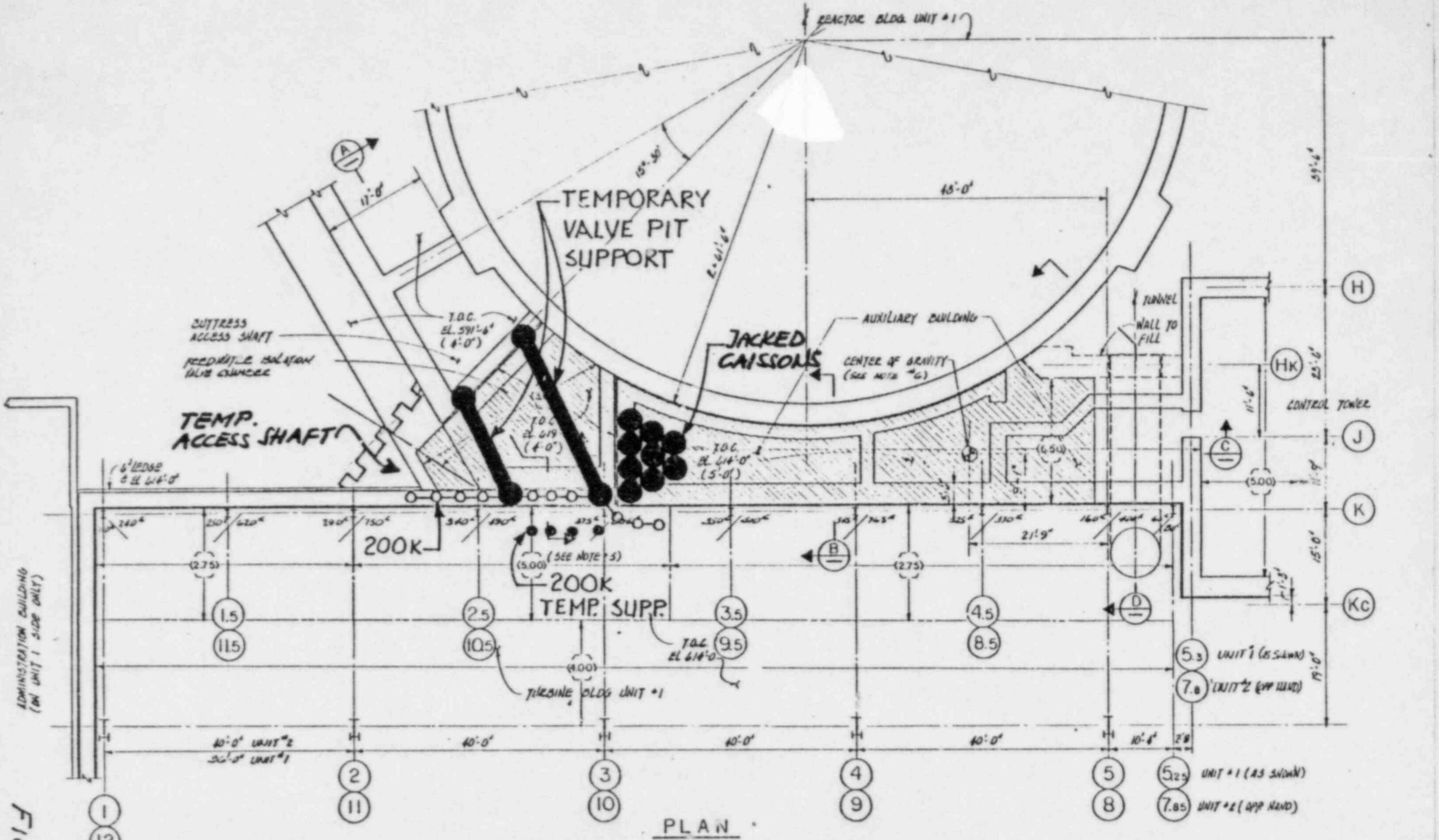
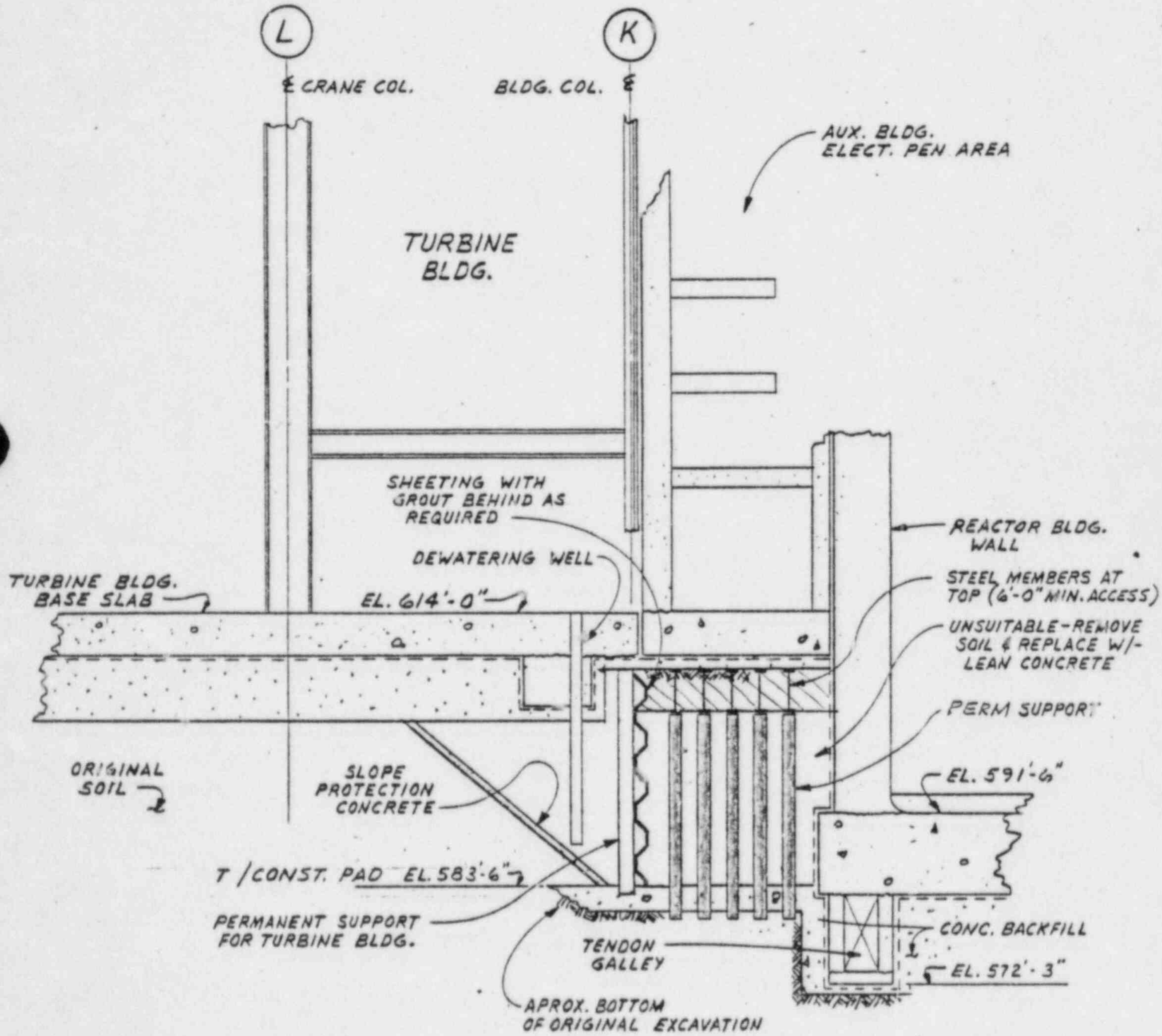
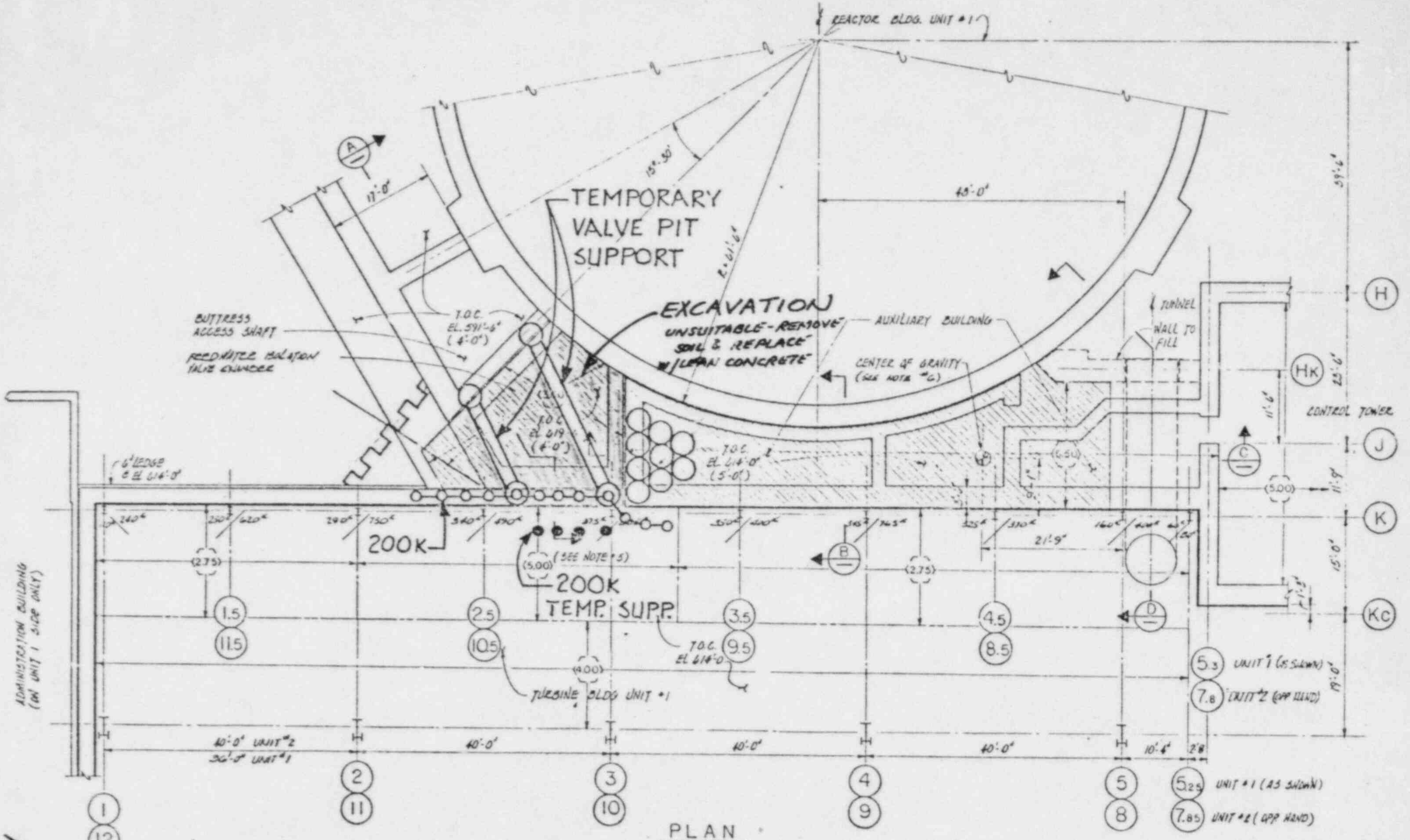

REACTOR BLDG UNIT #1
TEMPORARY
VALVE PIT
SUPPORT
JACKED
CAISSONS
TEMP.
ACCESS SHAFT
BUTTRESS
ACCESS SHAFT
FEEDWATER ISOLATION
VALVE CHAMBER
AUXILIARY BUILDING
CENTER OF GRAVITY
(SEE NOTE #6)
TUNNEL
WALL TO
FILL
CONTROL TOWER
200K
200K
TEMP. SUPP.
(SEE NOTE #5)
TURBINE BLDG UNIT #1
ADMINISTRATION BLDG UNIT 1 (MIRROR)
UNIT #1 (AS SHOWN)
UNIT #2 (OPP HAND)
PLAN

FIG 30



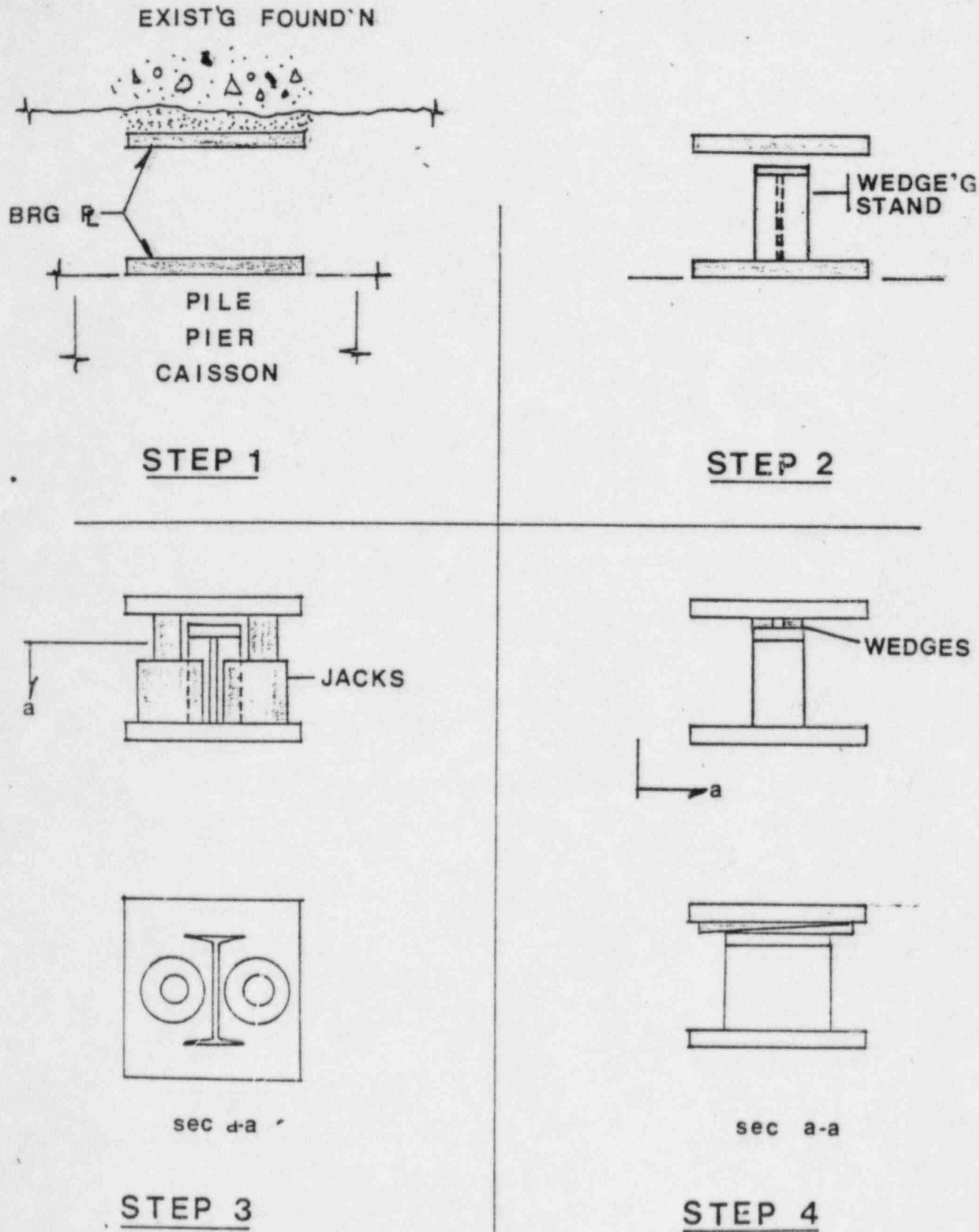
TYPICAL SECT.

FIG 31



REACTOR BLDG. UNIT #1
TEMPORARY VALVE PIT SUPPORT
EXCAVATION
UNSUITABLE - REMOVE SOIL & REPLACE W/LEAN CONCRETE
AUXILIARY BUILDING
CENTER OF GRAVITY (SEE NOTE #6)
BUTTRESS ACCESS SHAFT
FEEDWATER ISOLATION VALVE CHAMBER
T.O.C. EL 591'-6" (4'-0")
T.O.C. EL 619 (4'-0")
T.O.C. EL 614'-0" (5'-0")
T.O.C. EL 614'-0
6" LEDGE @ EL 614'-0"
TUNNEL
WALL TO FILL
CONTROL TOWER
200K
200K TEMP. SUPP.
(SEE NOTE #5)
TURBINE BLDG UNIT #1
ADMINISTRATION BUILDING (IN UNIT 1 SIDE ONLY)
UNIT #1 (AS SHOWN)
UNIT #2 (OPP HAND)
40'-0" UNIT #2
36'-0" UNIT #1
40'-0"
48'-0"
21'-9"
10'-4"
2'8
H
HK
J
K
KC
PLAN

FIG. 32



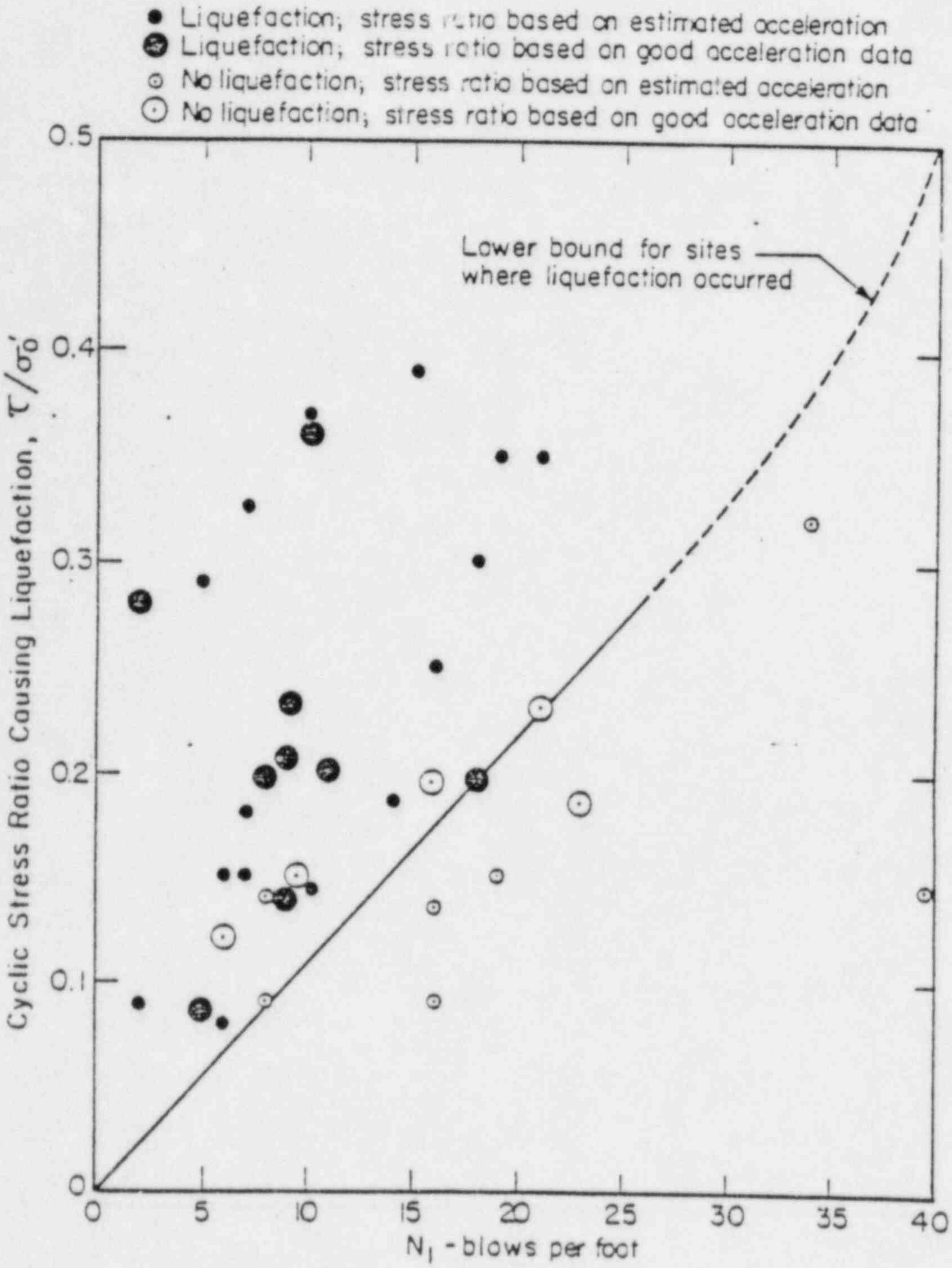
PRESTRESS PROCEDURE

FIG. 33

STRUCTURES	SUPPORTING SOIL TYPE
A. AUXILIARY BUILDING	
1). CONTROL TOWER	Medium dense to very dense sand.
2). UNIT 1 ELECTRICAL PENETRATION AREA	Dense to very dense sand with layers of loose sand and soft clay
3). UNIT 2 ELECTRICAL PENETRATION AREA	Medium dense to dense sand with medium stiff clay layers.
4). RAILROAD BAY	Medium to very dense sand.
B. FEEDWATER ISOLATION VALVE PITS	
1). UNIT 1	Loose to dense sand and medium stiff to very stiff clay.
2). UNIT 2	As UNIT 1.
C. SERVICE WATER PUMP STRUCTURES	Soft to very stiff clay and loose to very dense sand.
D. BORATED WATER TANKS	Medium to stiff sandy clay to clay.
E. DIESEL FUEL TANKS	Medium to stiff sandy clay to clay.
F. DIESEL GENERATOR BUILDING	Soft to stiff clay and loose to dense sand.

SUMMARY OF PREDOMINANT FILL TYPE AND CONDITION BELOW VARIOUS CATEGORY I STRUCTURES SUPPORTED ON PLANT AREA FILL

FIGURE 34



CORRELATION BETWEEN STRESS RATIO CAUSING LIQUEFACTION IN THE FIELD AND PENETRATION RESISTANCE OF SAND. (after Seed et al.)

FIGURE 35

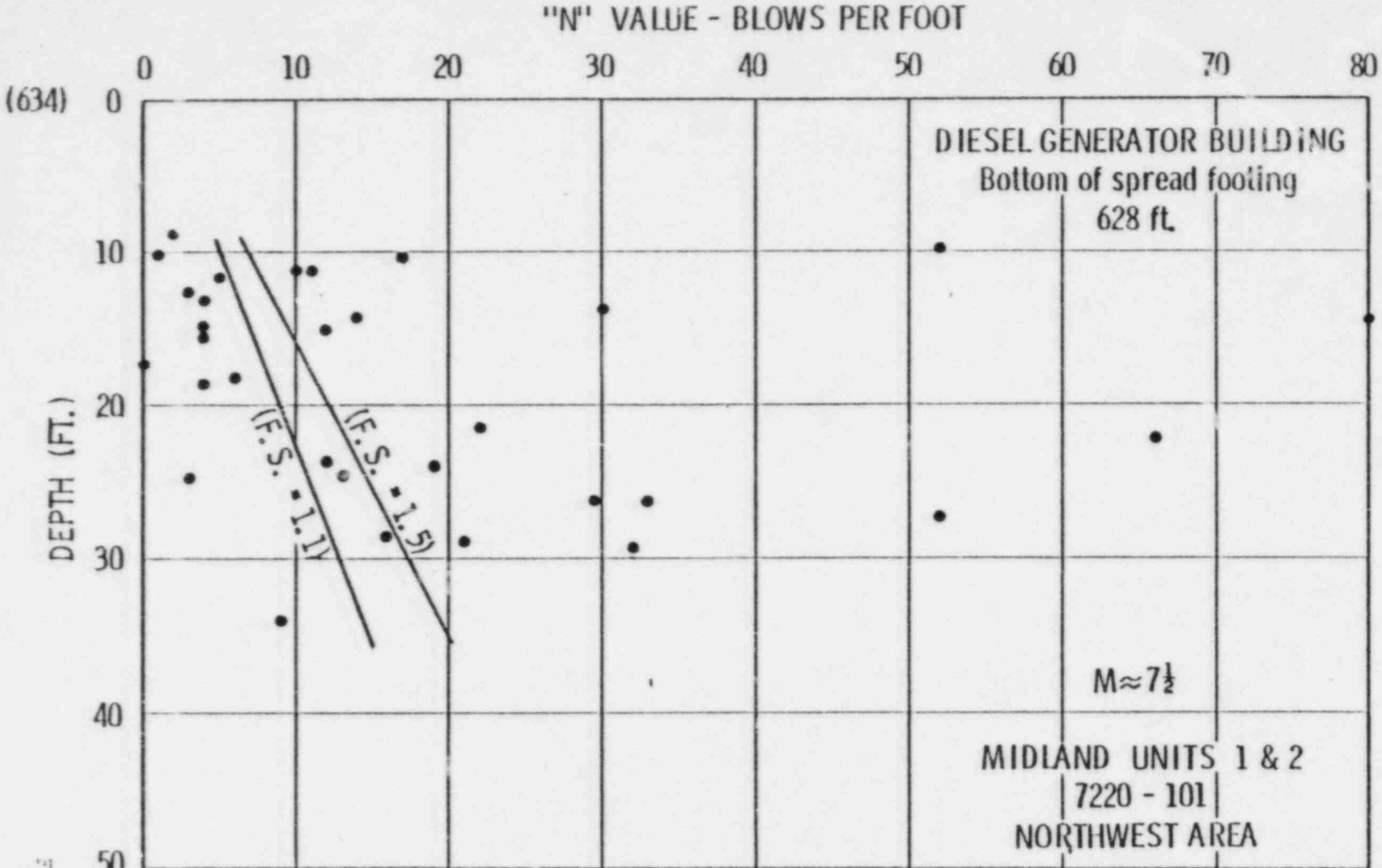
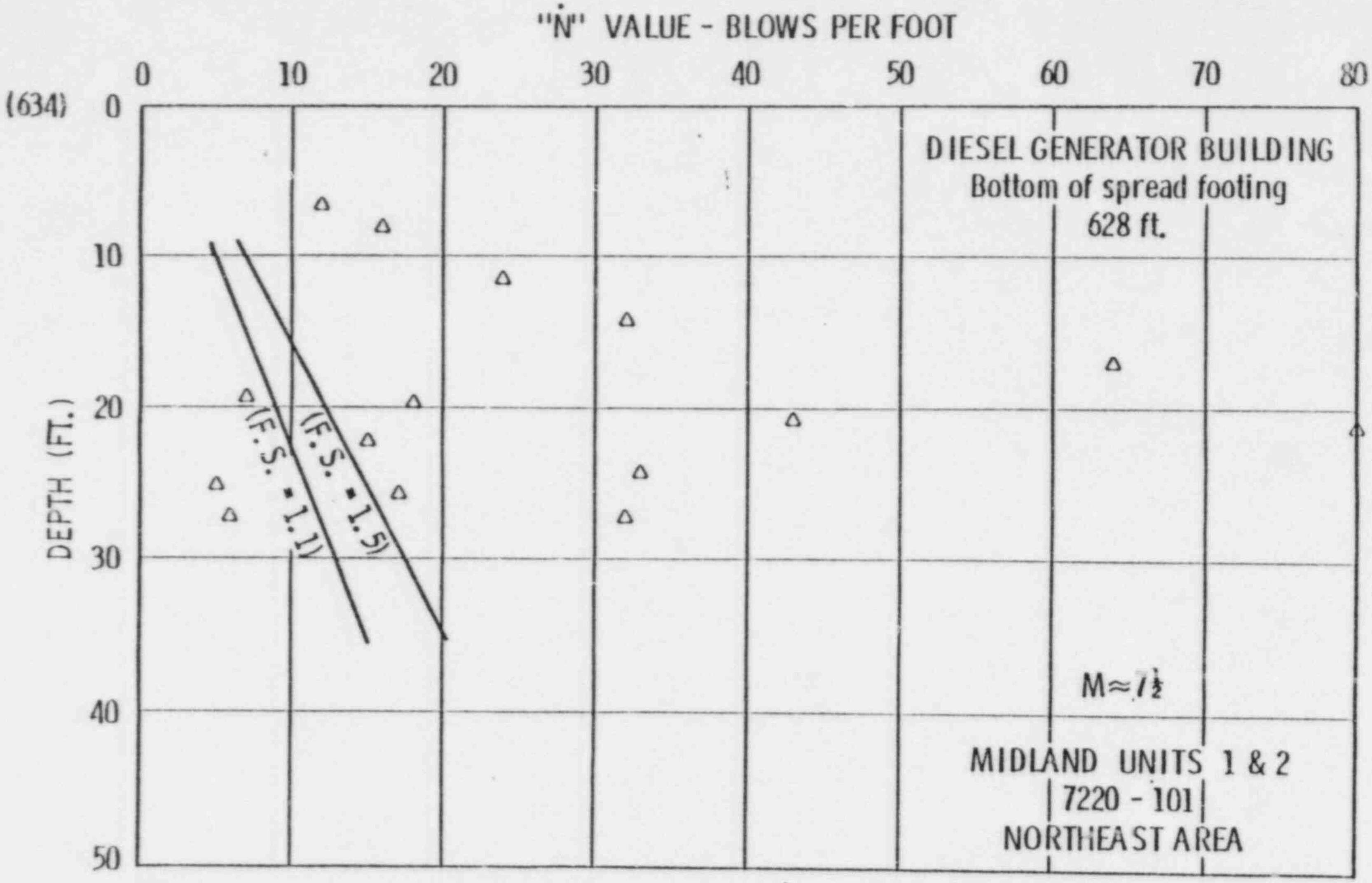


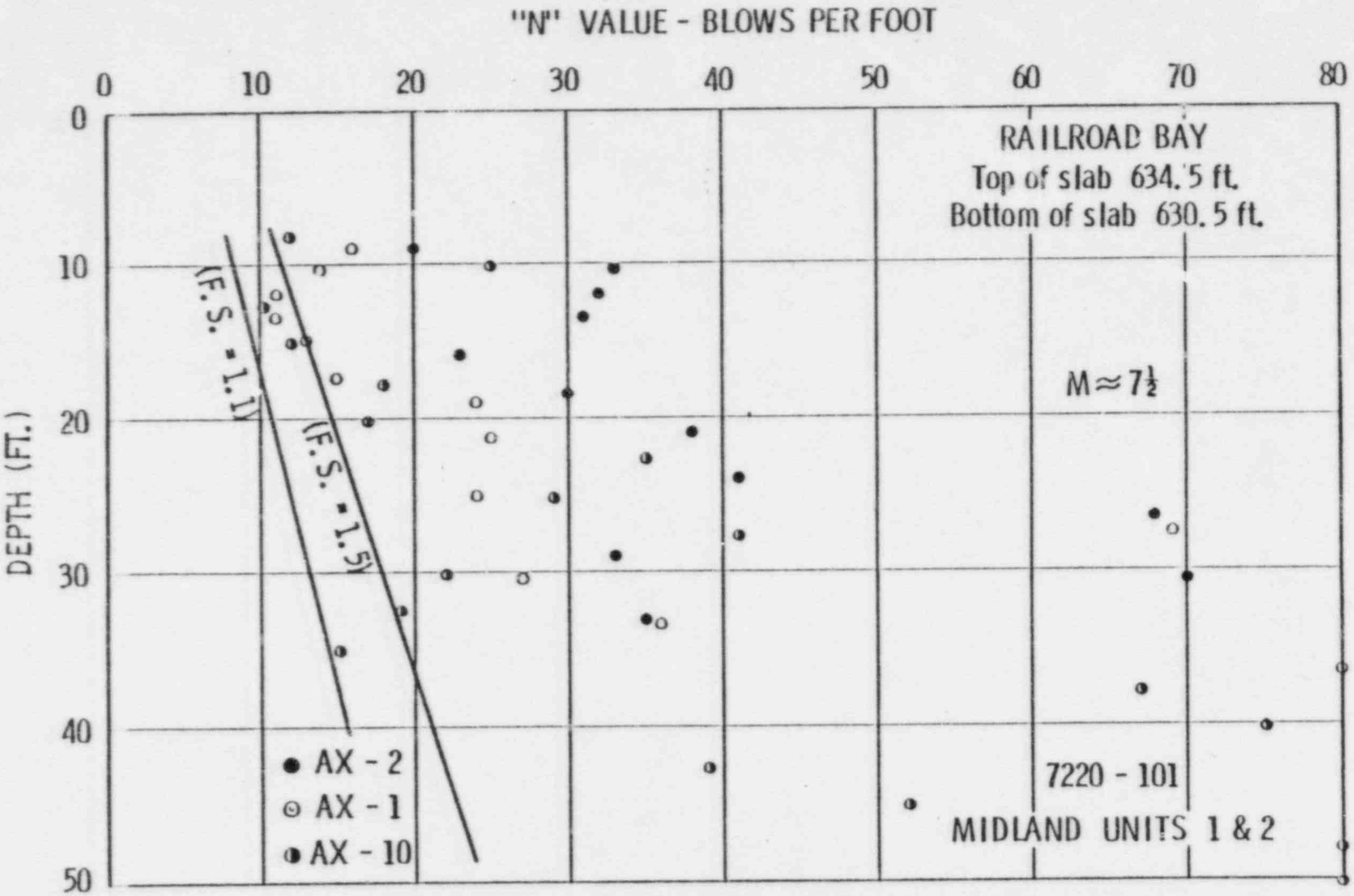
FIGURE 36

STANDARD PENETRATION RESISTANCE VERSUS
DEPTH FOR THE NORTHWEST AREA OF THE
DIESEL GENERATOR BUILDING



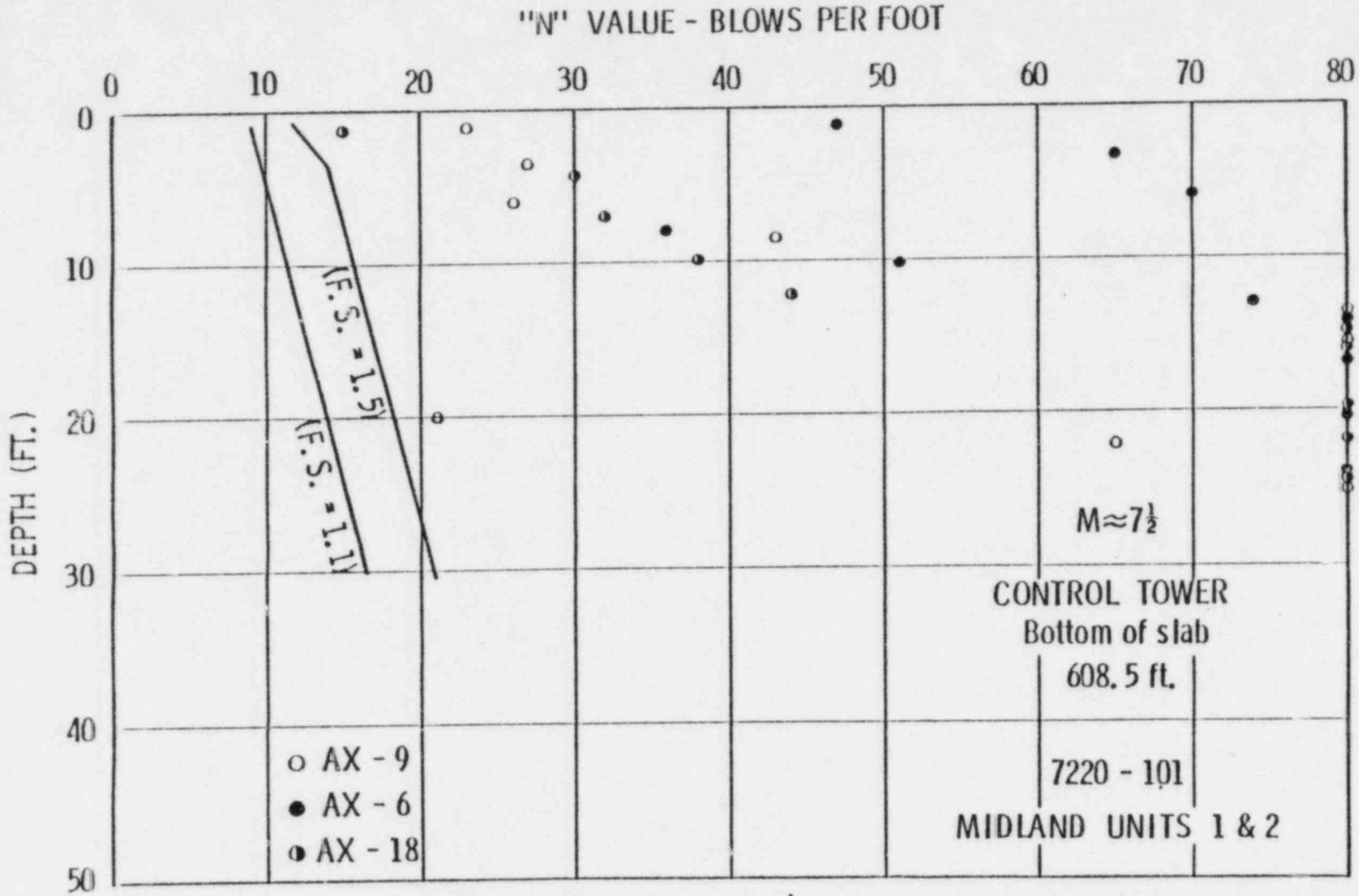
STANDARD PENETRATION RESISTANCE VERSUS DEPTH FOR THE NORTHEAST AREA OF THE DIESEL GENERATOR BUILDING

FIGURE 37



STANDARD PENETRATION RESISTANCE VERSUS
DEPTH FOR THE AUXILIARY BUILDING
RAILROAD BAY

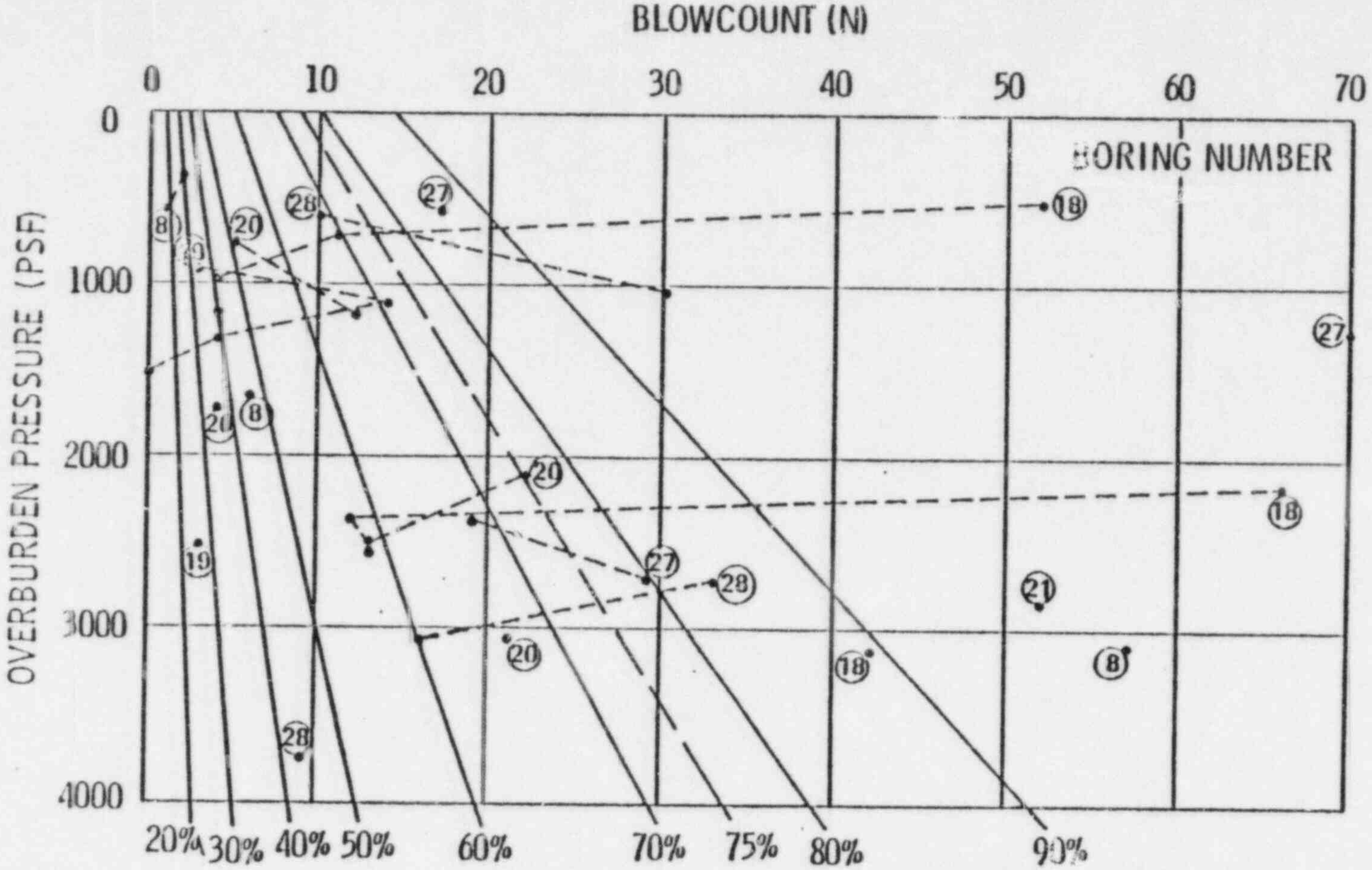
FIGURE 38



STANDARD PENETRATION BLOWCOUNT VERSUS
DEPTH FOR AUXILIARY BUILDING CONTROL TOWER

FIGURE 39A

5(T)



MIDLAND - DIESEL GENERATOR BUILDING

FIGURE 39B

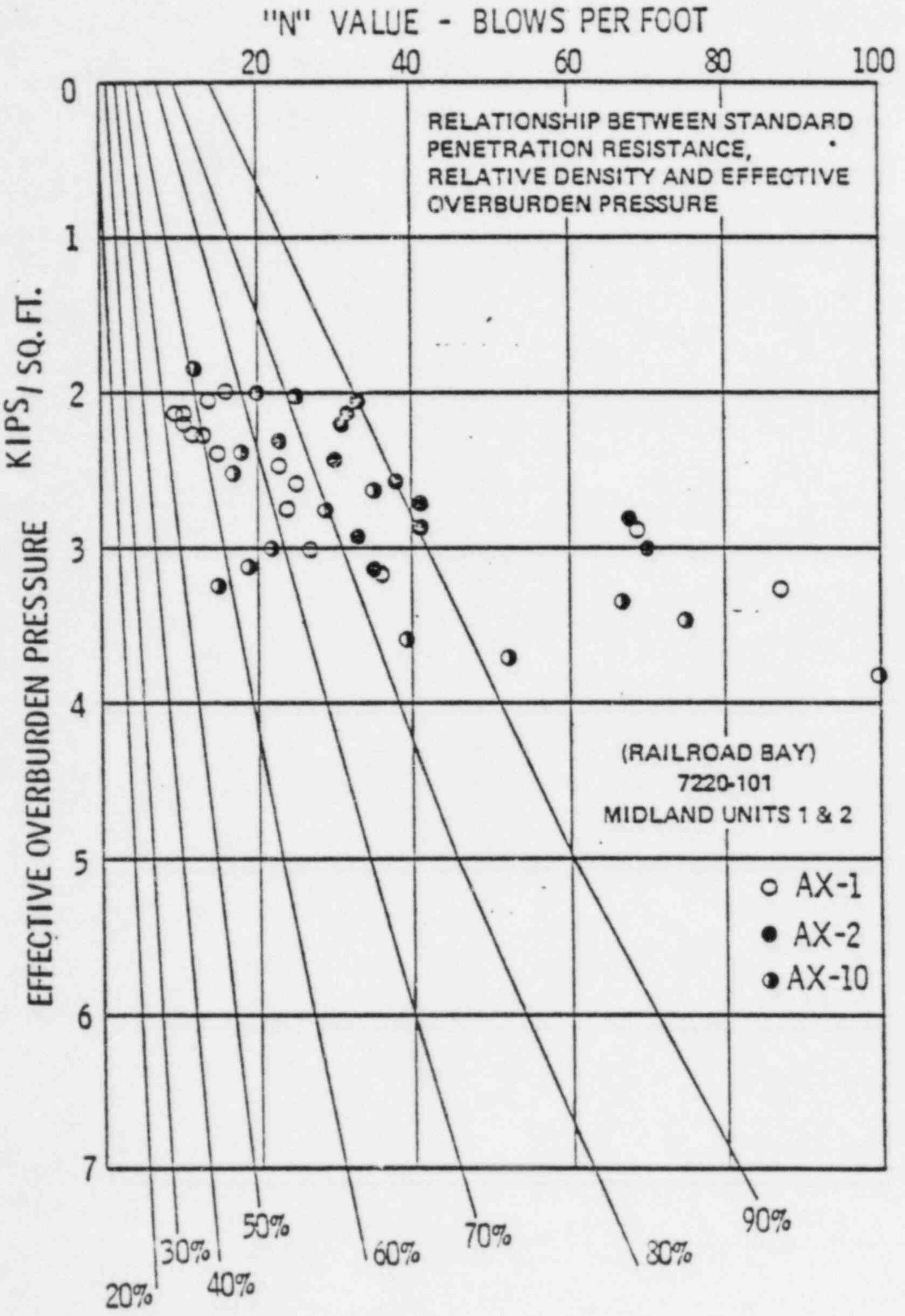

"N" VALUE - BLOWS PER FOOT
20
40
60
80
100
0
1
2
3
4
5
6
7
RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE, RELATIVE DENSITY AND EFFECTIVE OVERBURDEN PRESSURE
EFFECTIVE OVERBURDEN PRESSURE KIPS/SQ.FT.
(RAILROAD BAY)
7220-101
MIDLAND UNITS 1 & 2
AX-1
AX-2
AX-10
20%
30%
40%
50%
60%
70%
80%
90%

FIGURE 39C

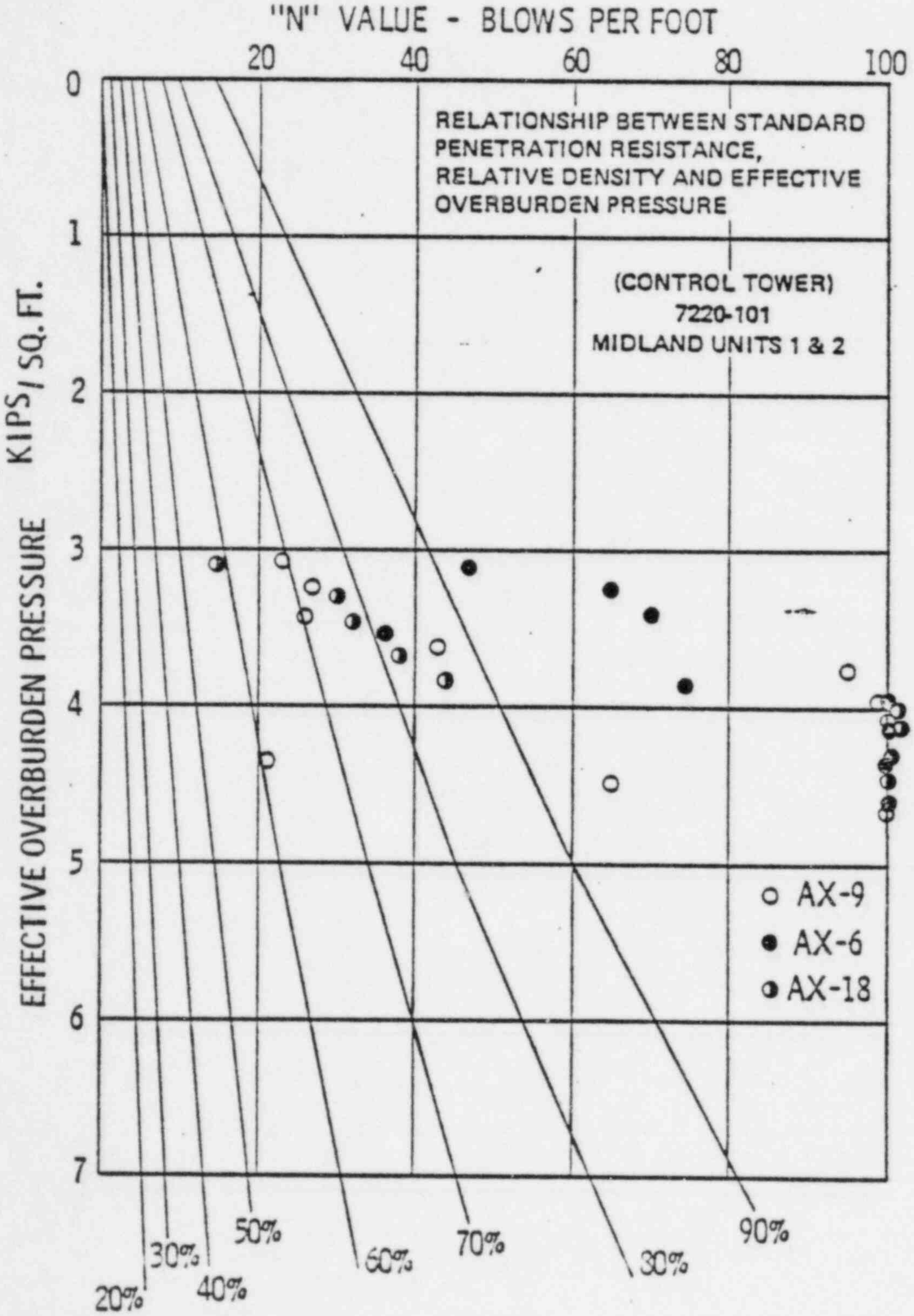
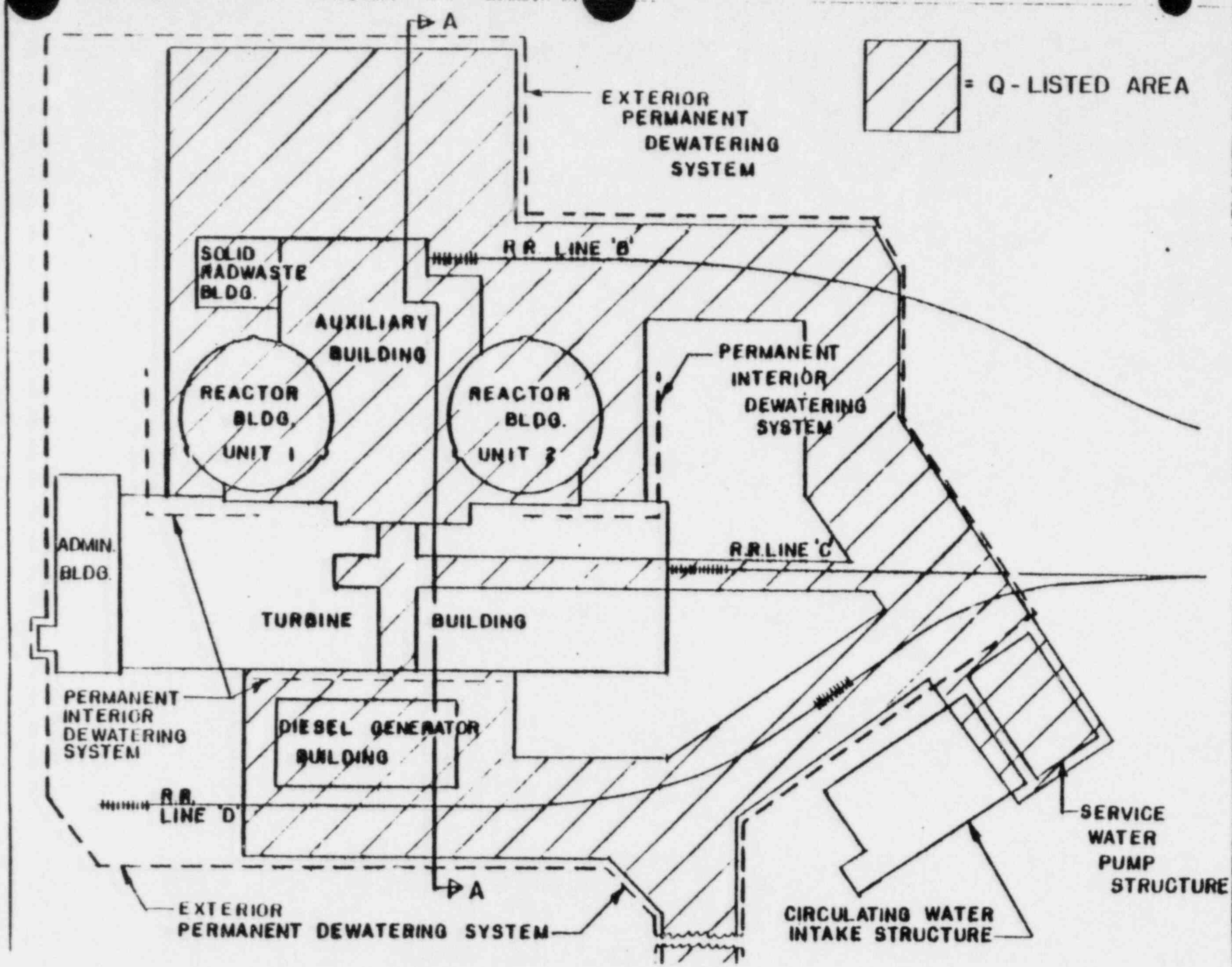
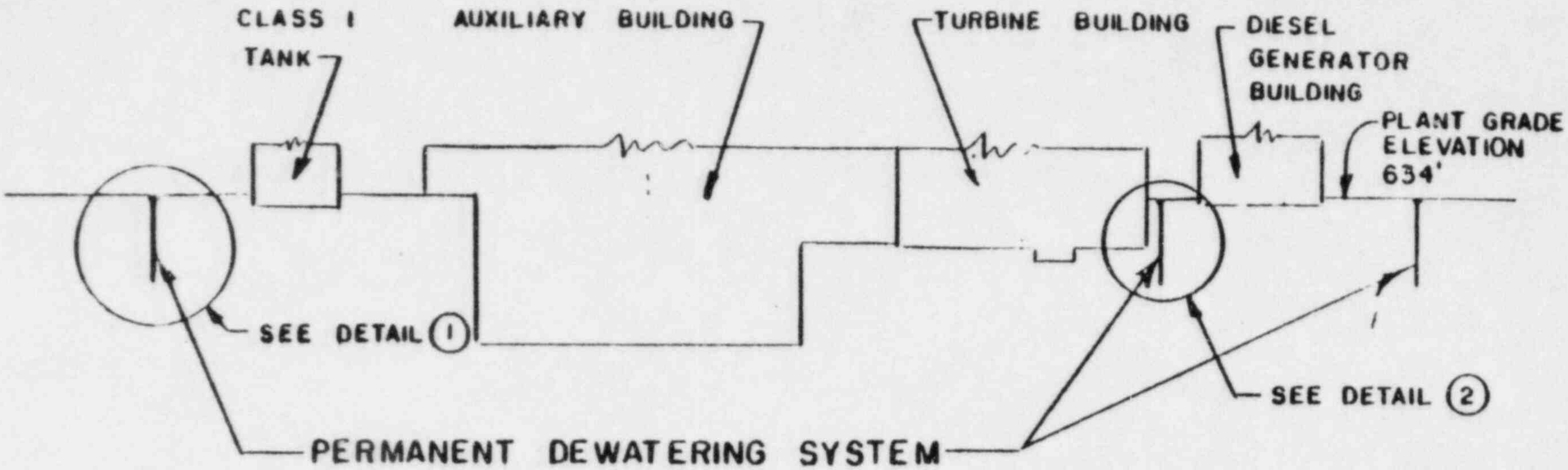

"N" VALUE - BLOWS PER FOOT
20
40
60
80
100
0
1
2
3
4
5
6
7
EFFECTIVE OVERBURDEN PRESSURE KIPS/SQ.FT.
RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE, RELATIVE DENSITY AND EFFECTIVE OVERBURDEN PRESSURE
(CONTROL TOWER)
7220-101
MIDLAND UNITS 1 & 2
AX-9
AX-6
AX-18
20%
30%
40%
50%
60%
70%
30%
90%

FIGURE 39D



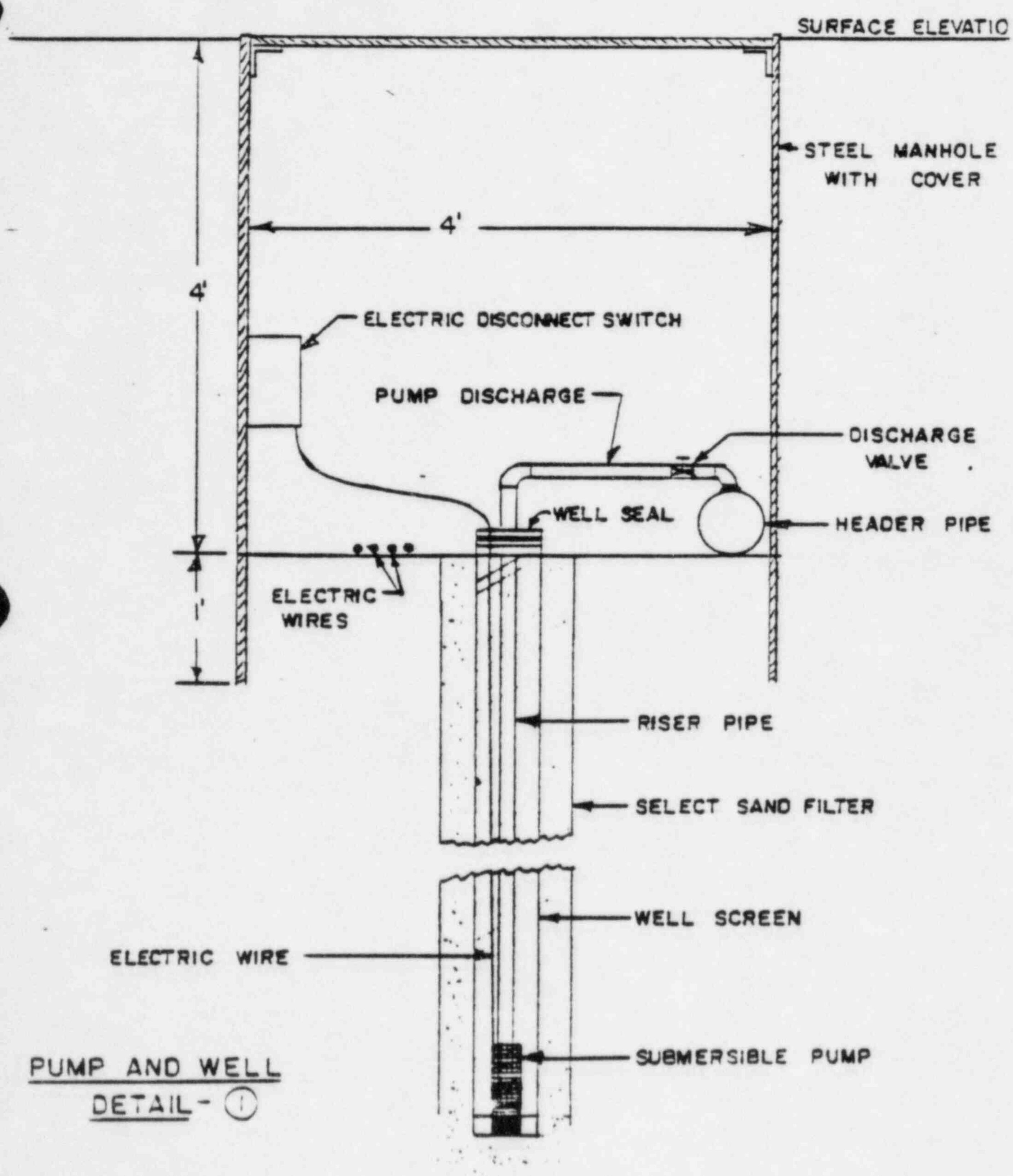
= Q-LISTED AREA
EXTERIOR PERMANENT DEWATERING SYSTEM
A
A
SOLID RADWASTE BLDG.
R.R. LINE 'B'
AUXILIARY BUILDING
PERMANENT INTERIOR DEWATERING SYSTEM
REACTOR BLDG. UNIT 1
REACTOR BLDG. UNIT 2
ADMIN. BLDG.
R.R. LINE 'C'
TURBINE
BUILDING
PERMANENT INTERIOR DEWATERING SYSTEM
DIESEL GENERATOR BUILDING
R.R. LINE 'D'
SERVICE WATER PUMP STRUCTURE
CIRCULATING WATER INTAKE STRUCTURE
EXTERIOR PERMANENT DEWATERING SYSTEM

FIGURE 40



SECTION A-A

FIGURE 41



PUMP AND WELL DETAIL- 1

FIGURE 42-

Monitor SNAPPY®

PITLESS ADAPTERS FOR SUBMERSIBLE PUMPS—4" & LARGER WELLS

In a Snappy submersible pump installation, the well casing is extended above ground, an excavation is made around the casing and a hole is cut in the casing below the frostline. The Snappy *casing fitting* is then attached to the casing around the hole to provide a delivery pipe. The pump, suspended from the Snappy *drop pipe fitting*, is lowered into the well with the neck of the drop pipe fitting pointed toward the casing fitting. When the neck reaches the level of the casing fitting, the Snappy *actuator* automatically inserts the neck with an O-ring seal into a *socket* in the casing fitting and locks it there thus providing both a support for drop pipe and pump within the well and a fluid tight conduit between the drop pipe and the discharge pipe. To remove the pump, the drop pipe fitting is first supported with a hoist. Then the neck of the drop pipe fitting is unlocked and withdrawn from the socket by a manual pull on the *control cable* thus releasing the drop pipe fitting from the casing fitting so that the pump can be lifted out with the hoist.

Snappy pitless adapters with weld-on casing fitting are approved by the Boards of Health of Michigan and Wisconsin. However, Wisconsin approval requires factory welding of the casing fitting to the well casing except for residential water systems serving no more than three families.

Snappy pitless adapters are certified water-tight under the standards of the Pitless Adapter Division of the Water Systems Council (PAS-1).

Snappy pitless adapters are available for well sizes from 4 to 8 inches I.D. and for drop and delivery pipe sizes of 1 and 1-1/4 inches I.D. with either clamp-on or weld-on casing fittings.

FEATURES

FROSTPROOF---No heating required. All water conduits are buried below frostline.

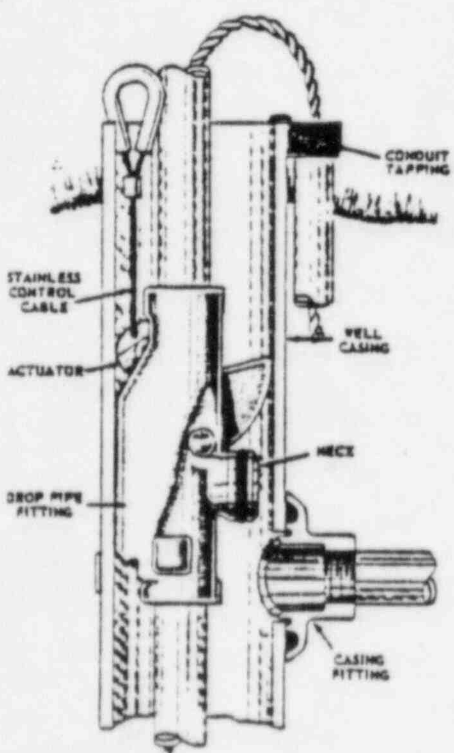
PUMP IS EASILY SET --- by simply lowering pump into well suspended from drop pipe fitting with neck of the latter pointed in the casing fitting direction.

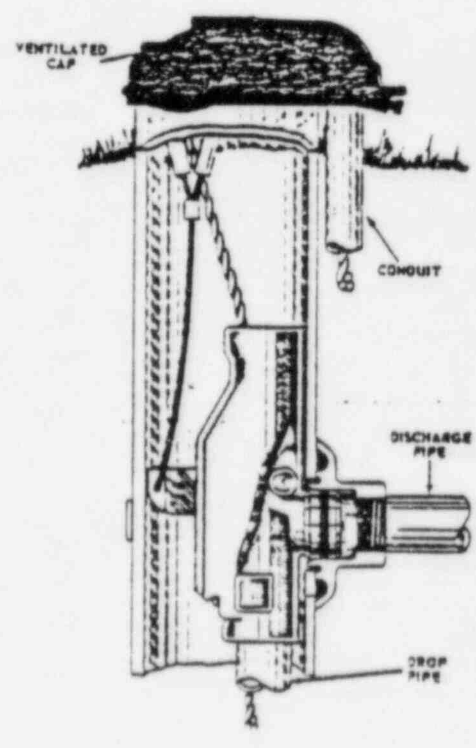
PUMP IS EASILY PULLED --- by first supporting drop pipe with hoist, and then manually pulling control cable to free pump.

LOW COST --- Regular well casing is used all the way. Extra cost of larger upper well casing used with spool-type units and expensive pit or well house construction are eliminated.

CORROSION PROTECTION --- Clamp-on and weld-on casing fittings are galvanized gray iron and stainless steel respectively. All parts within the well casing are either hot-dipped galvanized or constructed of corrosion resistant materials.

Continued

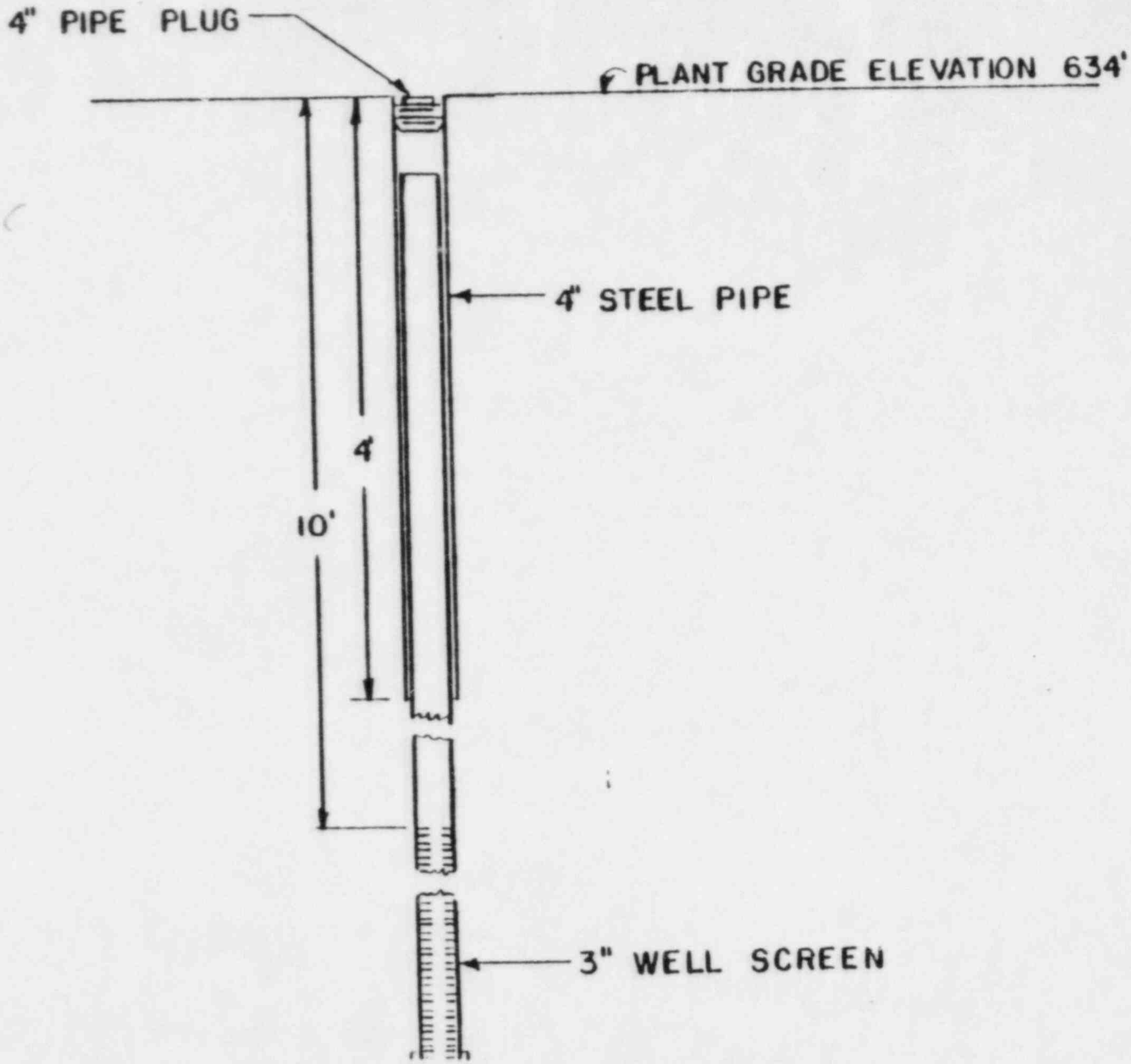




RELATED U.S. PATENTS: 3,035,732 3,054,022 3,123,589 3,136,362 3,165,370 3,239,007 3,473,573 3,722,586 3,902,532

BAKER
monitor division
EVANSVILLE, WISCONSIN 53536
PHONE: (608) 882-5100

FIGURE 43



DETAIL (2)

FIGURE 44

STRUCTURAL INVESTIGATION

(1) ORIGINAL DESIGN

(2) SEISMIC RESPONSE

(3) NEW ANALYSES

FIGURE 45

SEISMIC ANALYSIS

GENERAL

(1) RESPONSE SPECTRA PRESENTED IN FSAR

(2) STICK MASS MODELS WITH FOUNDATION SPRINGS

(3) MATERIAL DAMPING VALUES PRESENTED IN FSAR (MODAL DAMPING LIMITED TO 10% EXCEPT RIGID BODY MODES)

(4) SPECTRUM RESPONSE AND TIME HISTORY MODAL ANALYSES

DIESEL GENERATOR BUILDING

(1) ORIGINAL (V_s = 1360 FPS) - ONE ANALYSIS EQUIPMENT SPECTRA WIDENED BY ± 15%

(2) NEW (V_s = 500 FPS) - NEW SPECTRA WILL ENVELOP BOTH V_s = 500 FPS AND 1360 FPS

FIGURE 46

SEISMIC ANALYSIS

SERVICE WATER BUILDING

(1) ORIGINAL (V_s = 1360 FPS BASE CASE) THEN G VARIED BY ± 50% - EQUIPMENT SPECTRA ENVELOP

(2) NEW (V_s = 1360 FPS) - PILING IS MODELED FOR VERTICAL DIRECTION AND TORSION IS CONSIDERED

AUXILIARY BUILDING (INCLUDE CONTROL TOWER AND ELECTRICAL PENETRATION AREAS

(1) ORIGINAL - ONE ANALYSIS USING COMPOSITE FOUNDATION SPRINGS WITH EQUIPMENT RESPONSE SPECTRA WIDENED BY ± 15%

(2) NEW - ONE ANALYSIS INCLUDING CAISSONS UNDER ELECTRICAL PENETRATION AREAS, EQUIPMENT RESPONSE SPECTRA WIDENED BY ± 15%

FIGURE 47

TYPES OF LOADS

PRIMARY

1. MECHANICAL (DEADLOAD, PRESSURE, WIND, ETC.)
2. SEISMIC INERTIA (BUT SHORT DURATION)
3. MISSILE IMPACT & PIPE RUPTURE (LIMITED ENERGY)

SECONDARY

1. INTERNAL SELF CONTRAINT
   - (A) SEISMIC DISPLACEMENT (CYCLIC)
   - (B) THERMAL (CYCLIC)
2. SETTLEMENT (1/2 CYCLE)
3. FORMING (1/2 CYCLE)

MIDLAND DESIGN CRITERIA

FSAR

(A) $1.4D + 1.7L$

(B) $1.4 (D + L + E_O) + \ldots$

(C) $1.25 (D + L + W) + \ldots$

(D) $1.0D + 1.0L + 1.0E_{SS} + \ldots$

(E) $1.0D + 1.0L + 1.0W_T + \ldots$

ADDITIONAL CRITERIA

(A) $1.05D + 1.28L + 1.05\ SET$

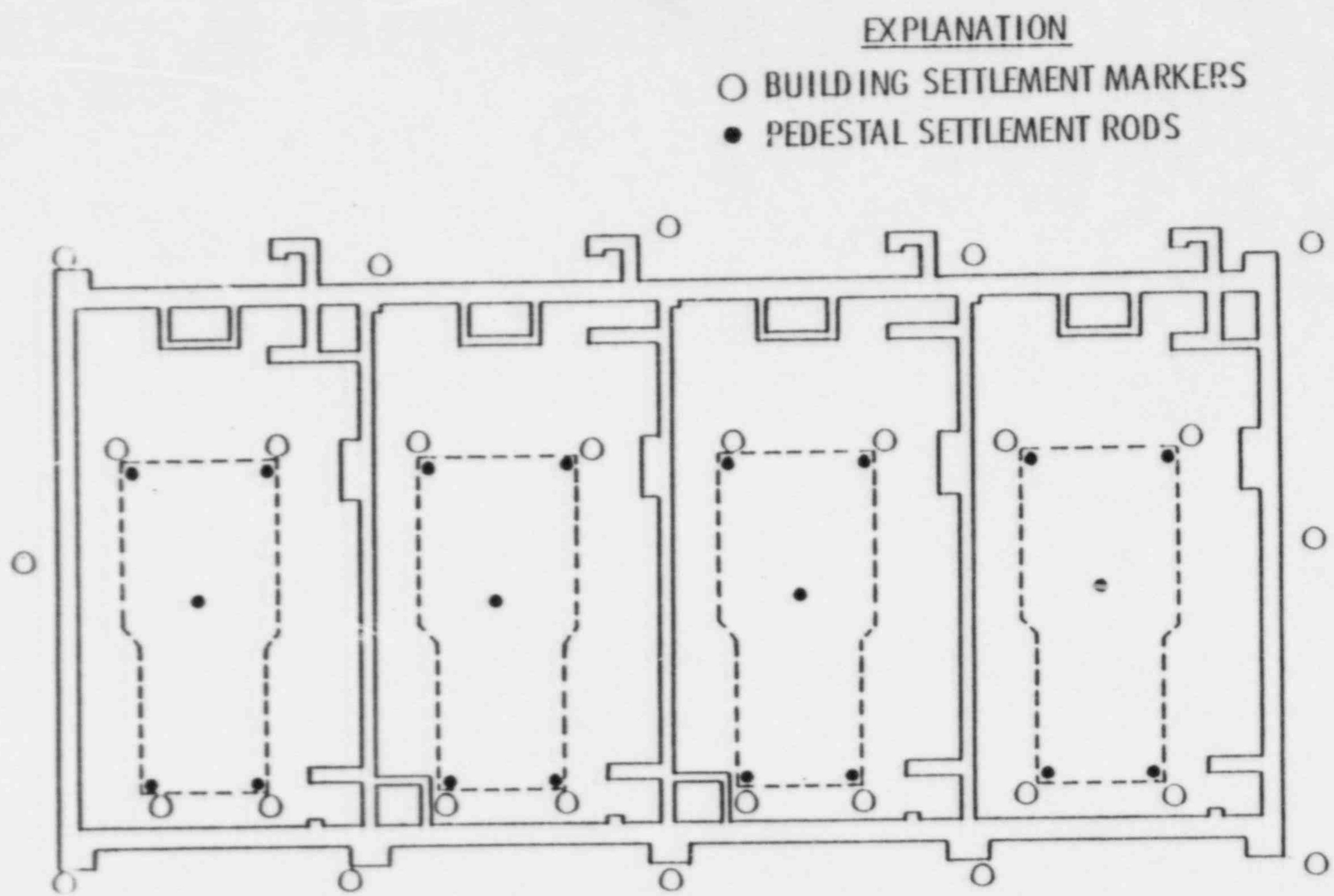
(B) $1.4D + 1.4\ SET$

(C) $1.0D + 1.0L + 1.0W + 1.0\ SET$

(D) $1.0D + 1.0L + 1.0E_O + 1.0\ SET$

D:	DEAD LOAD	E_{SS}:	(SSE) EARTHQUAKE
L:	LIVE LOAD	W_T:	TORNADO
E_O:	(OBE) EARTHQUAKE	SET:	SETTLEMENT
W:	DESIGN WIND		

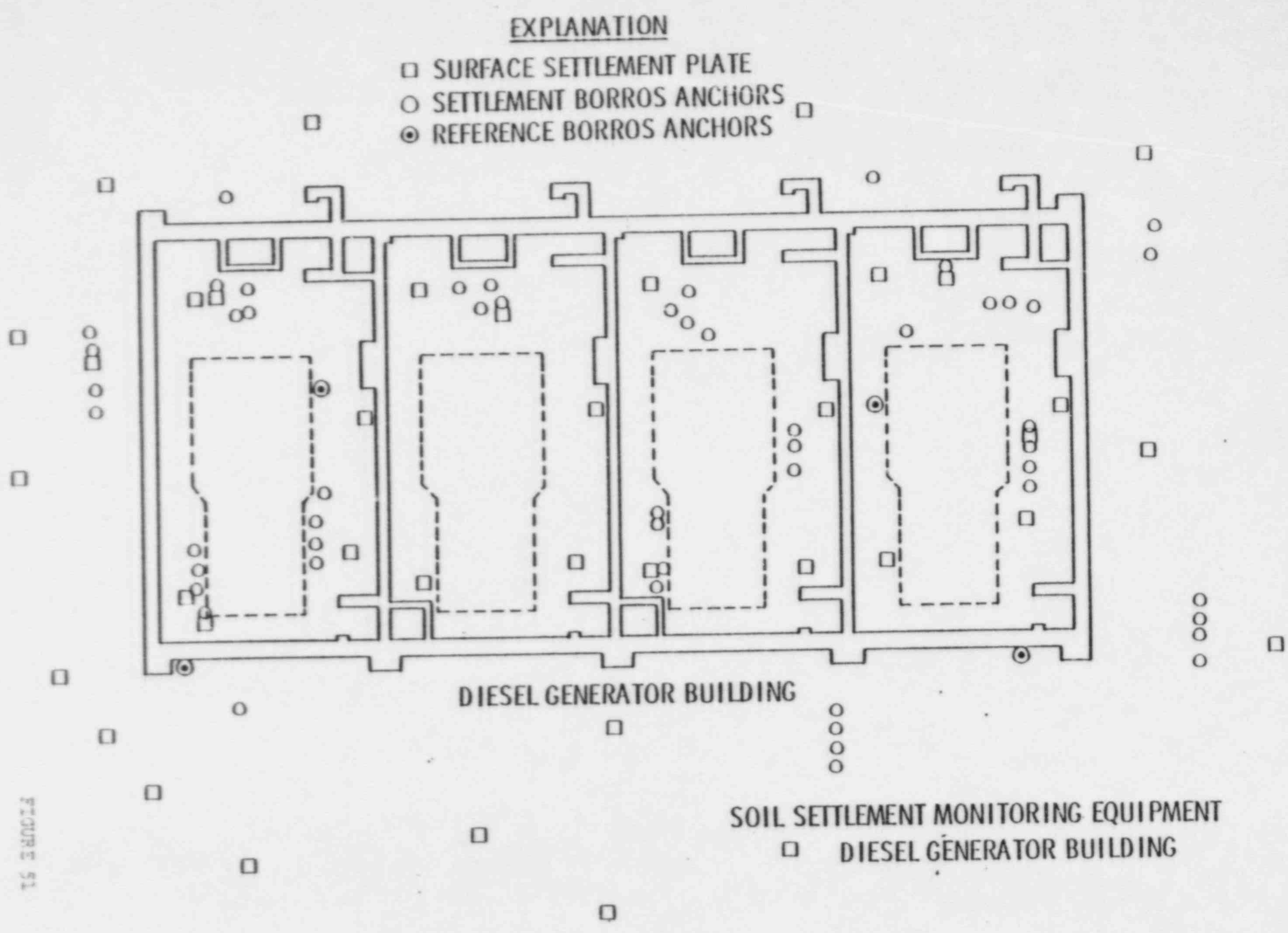
FIGURE 49



DIESEL GENERATOR BUILDING

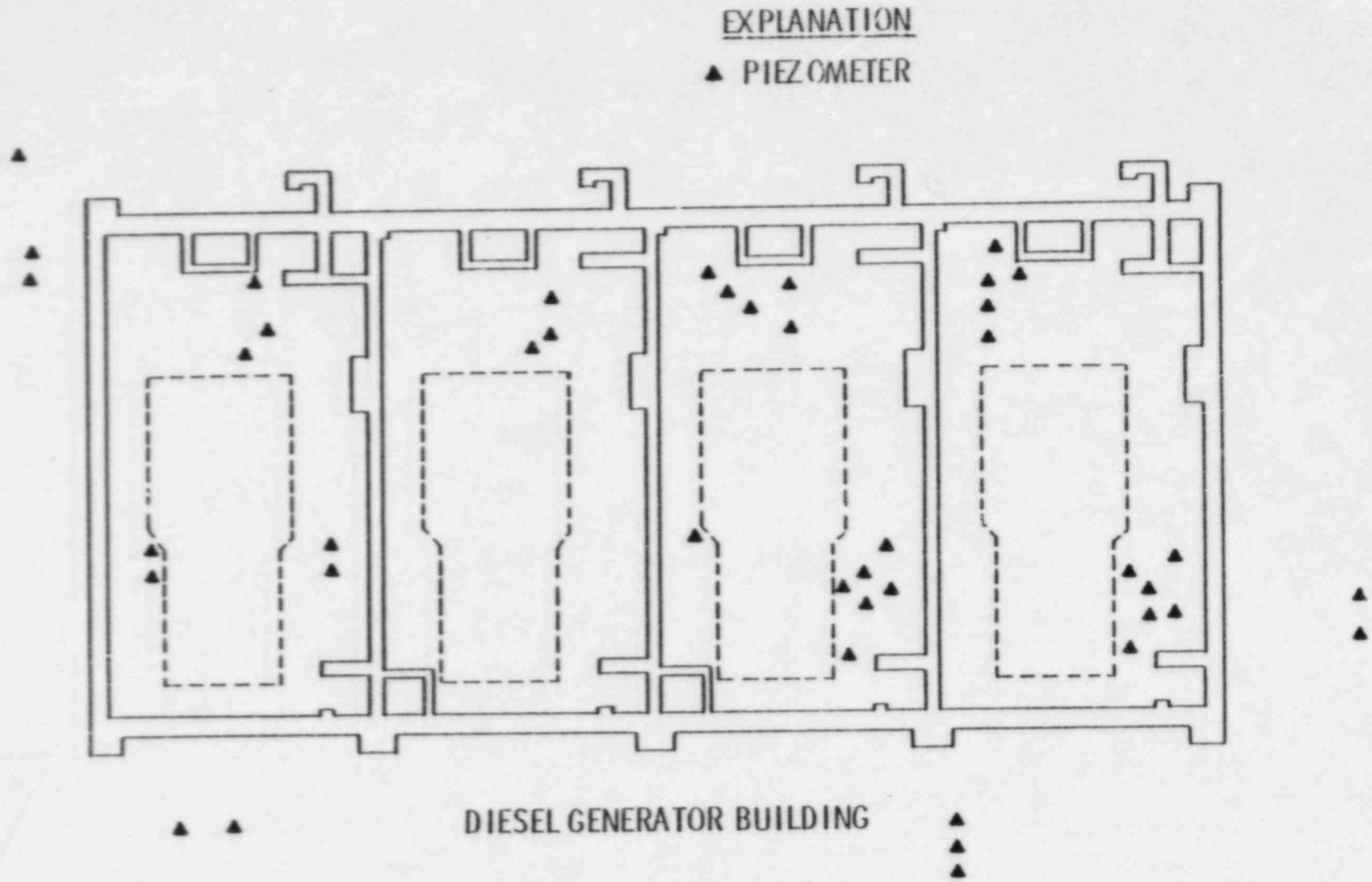
BUILDING AND PEDESTAL MOVEMENT
MONITORING EQUIPMENT
DIESEL GENERATOR BUILDING

FIGURE 50



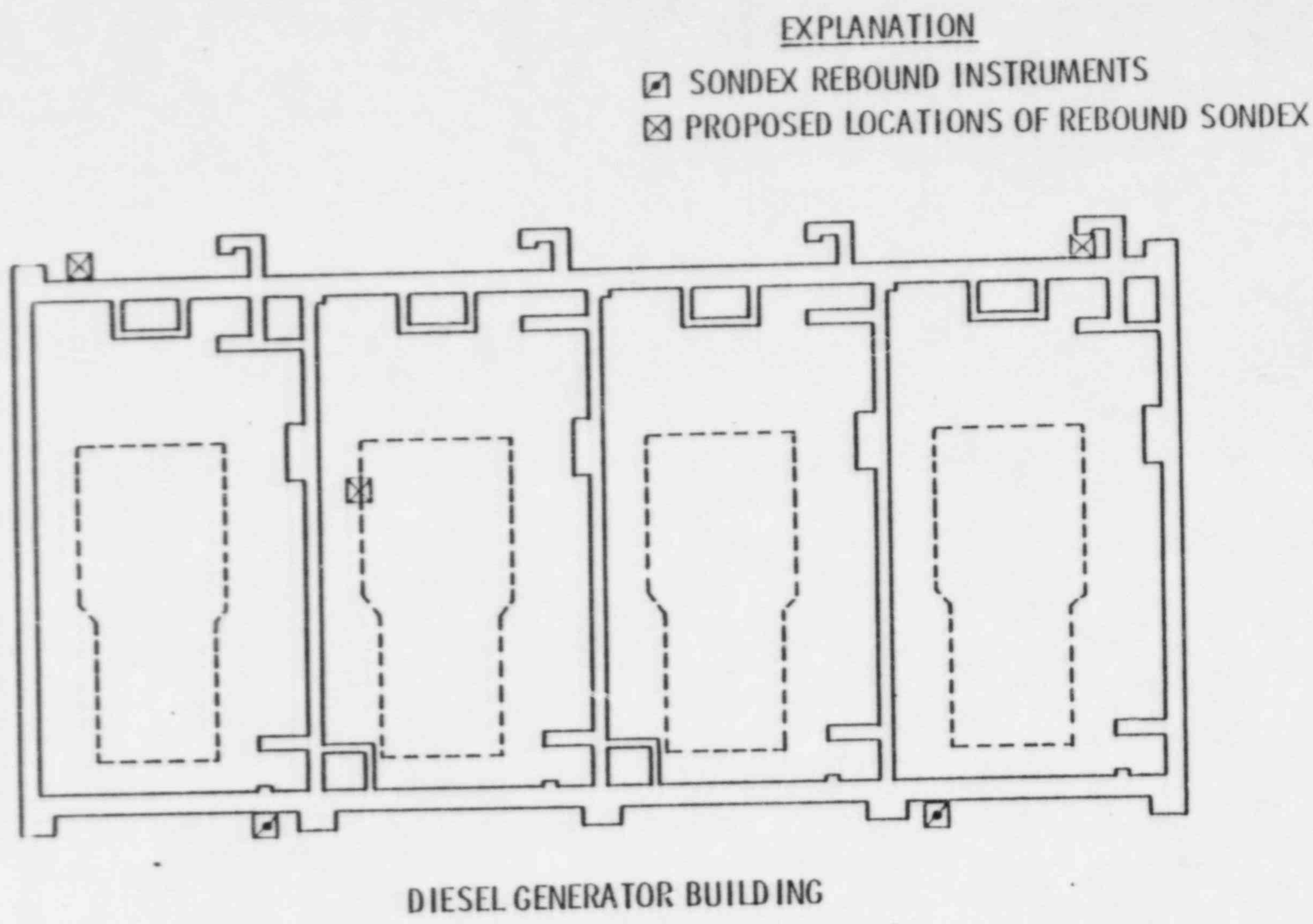
SOIL SETTLEMENT MONITORING EQUIPMENT
□ DIESEL GENERATOR BUILDING

FIGURE 51



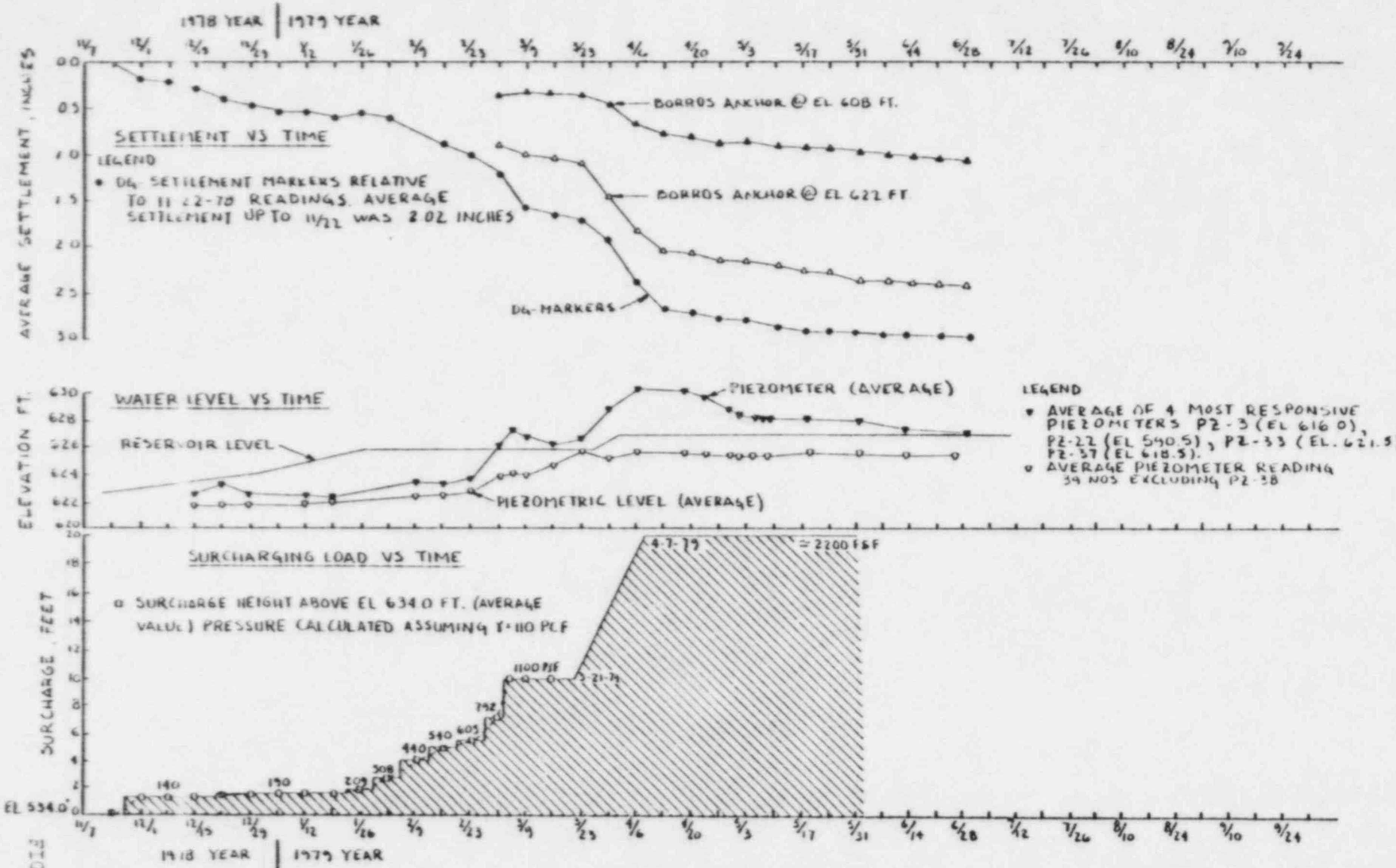
PORE WATER PRESSURE MONITORING
EQUIPMENT - DIESEL GENERATOR BUILDING

FIGURE 52



SOIL REBOUND MONITORING EQUIPMENT
DIESEL GENERATOR BUILDING

FIGURE 53



DIESEL GENERATOR BUILDING: AVERAGE VALUE OF SETTLEMENT, SURCHARGE, AND WATER LEVEL VS. TIME

FIGURE 54

STRUCTURE	NO. of BORINGS	SUPPORTING FILL TYPE	PLANNED REMEDIAL MEASURES
A. AUXILIARY BUILDING			
1). CONTROL TOWER	3	SAND	NONE *
2). UNIT 1 ELECTRICAL PENETRATION AREA	2	SAND & CLAY	UNDERPINNING
3). UNIT 2 ELECTRICAL PENETRATION AREA	2	SAND & CLAY	UNDERPINNING
4). RAILROAD BAY	3	SAND	NONE
B. FEEDWATER ISOLATION VALVE PITS			
1). UNIT 1	2	SAND & CLAY	UNDERPINNING
2). UNIT 2	3	SAND & CLAY	UNDERPINNING
C. SERVICE WATER PUMP STRUCTURE - PORTION ON FILL	9	CLAY & SAND	UNDERPINNING

* GROUTING IS PLANNED BELOW MUD NOT AT AX - 9.

FIGURE 55

SUMMARY OF FILL TYPE AND PLANNED REMEDIAL ACTION

STRUCTURE	NO. of BORINGS	SUPPORTING FILL TYPE	PLANNED REMEDIAL MEASURES
D. TANKS			
1). DIESEL FUEL OIL STORAGE TANKS	7	CLAY	NONE
2). BORATED WATER STORAGE TANKS	6	CLAY	NONE
E. DIESEL GENERATOR BUILDING	32	SAND & CLAY	SURCHARGE
F. UTILITIES			
1). PIPING	50	SAND & CLAY	NONE
2). DUCT BANKS	38	SAND & CLAY	NONE
3). VALVE PITS	2	SAND & CLAY	NONE

FIGURE 56

SUMMARY OF FILL TYPE AND
PLANNED REMEDIAL ACTION

BEARING PILES
FOR
SERVICE WATER PUMP STRUCTURE

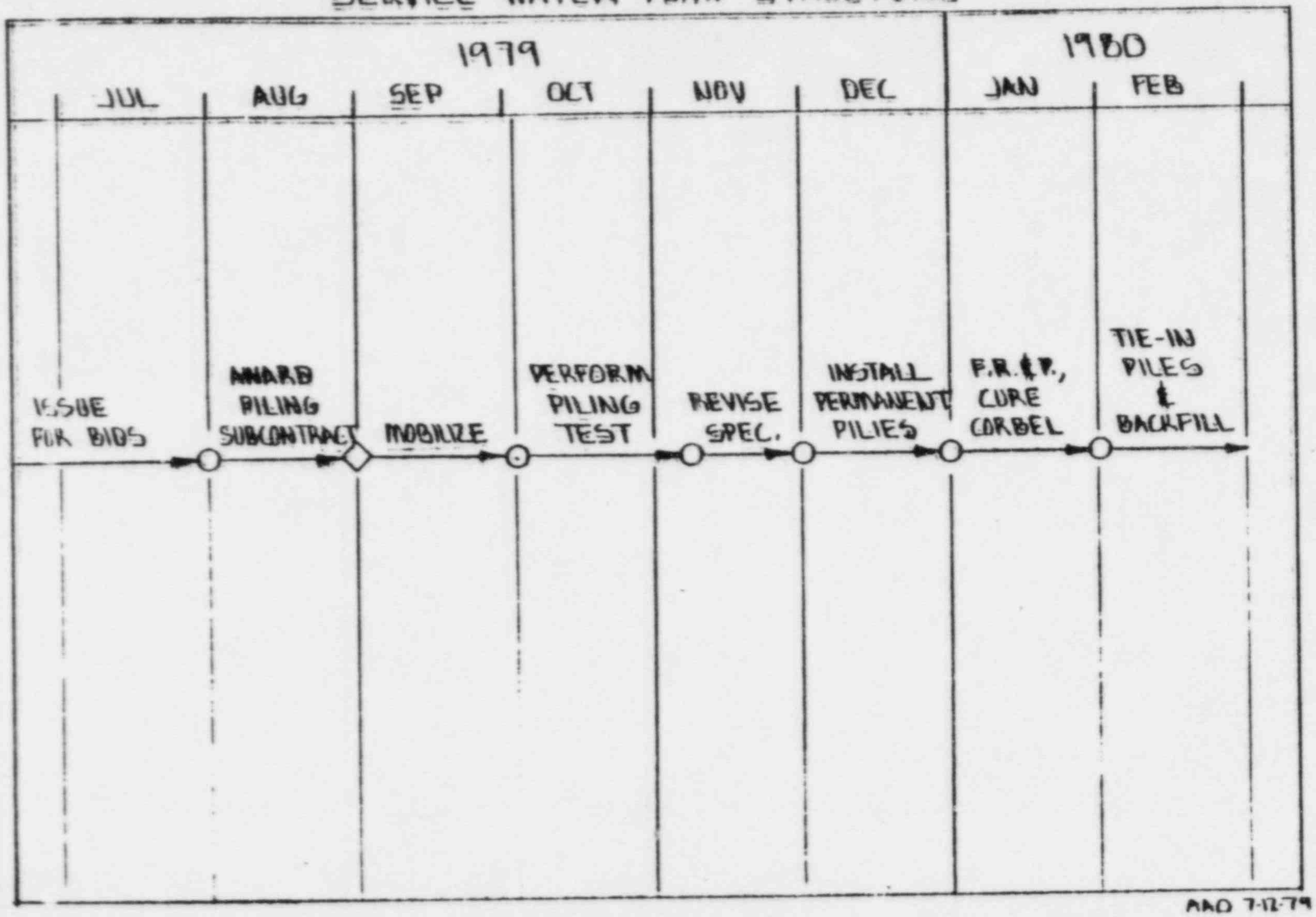


FIGURE 57

UNIT 1 & 2 AUXILIARY BLDG., ELECTRICAL PENETRATION AREAS
AND
UNIT 1 & 2 FEEDWATER ISOLATION VALVE PITS

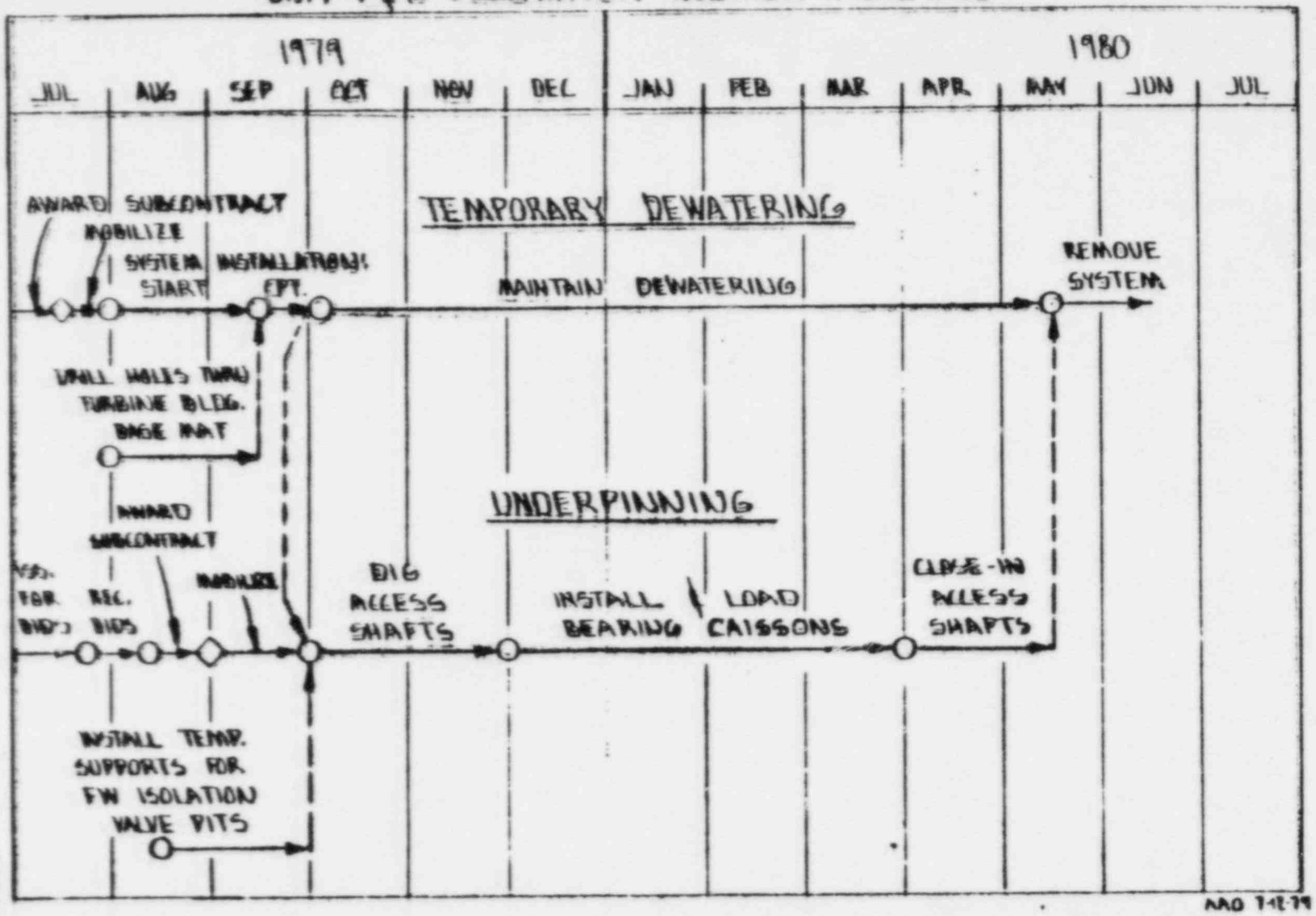


FIGURE 58

BORATED WATER STORAGE TANKS

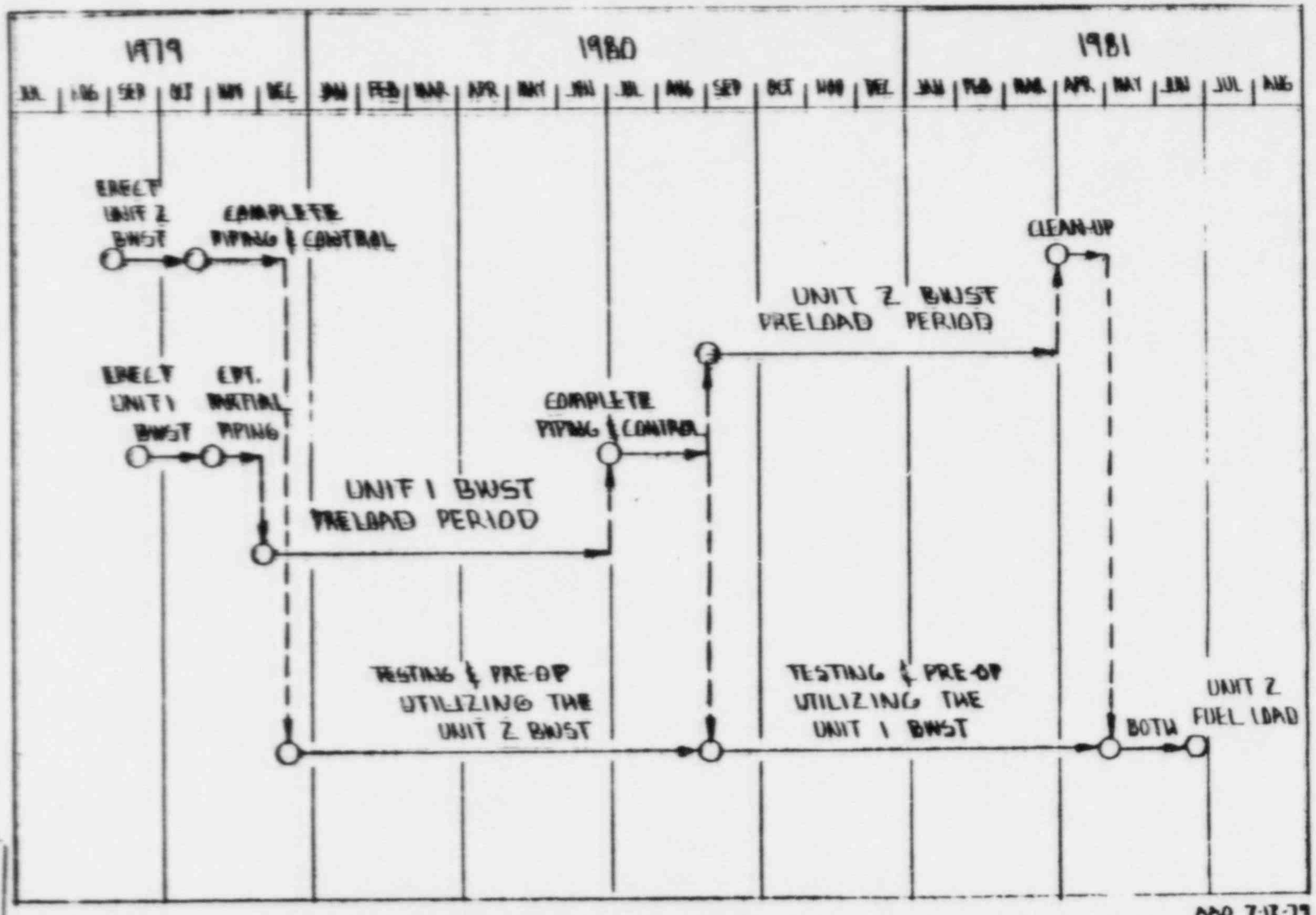
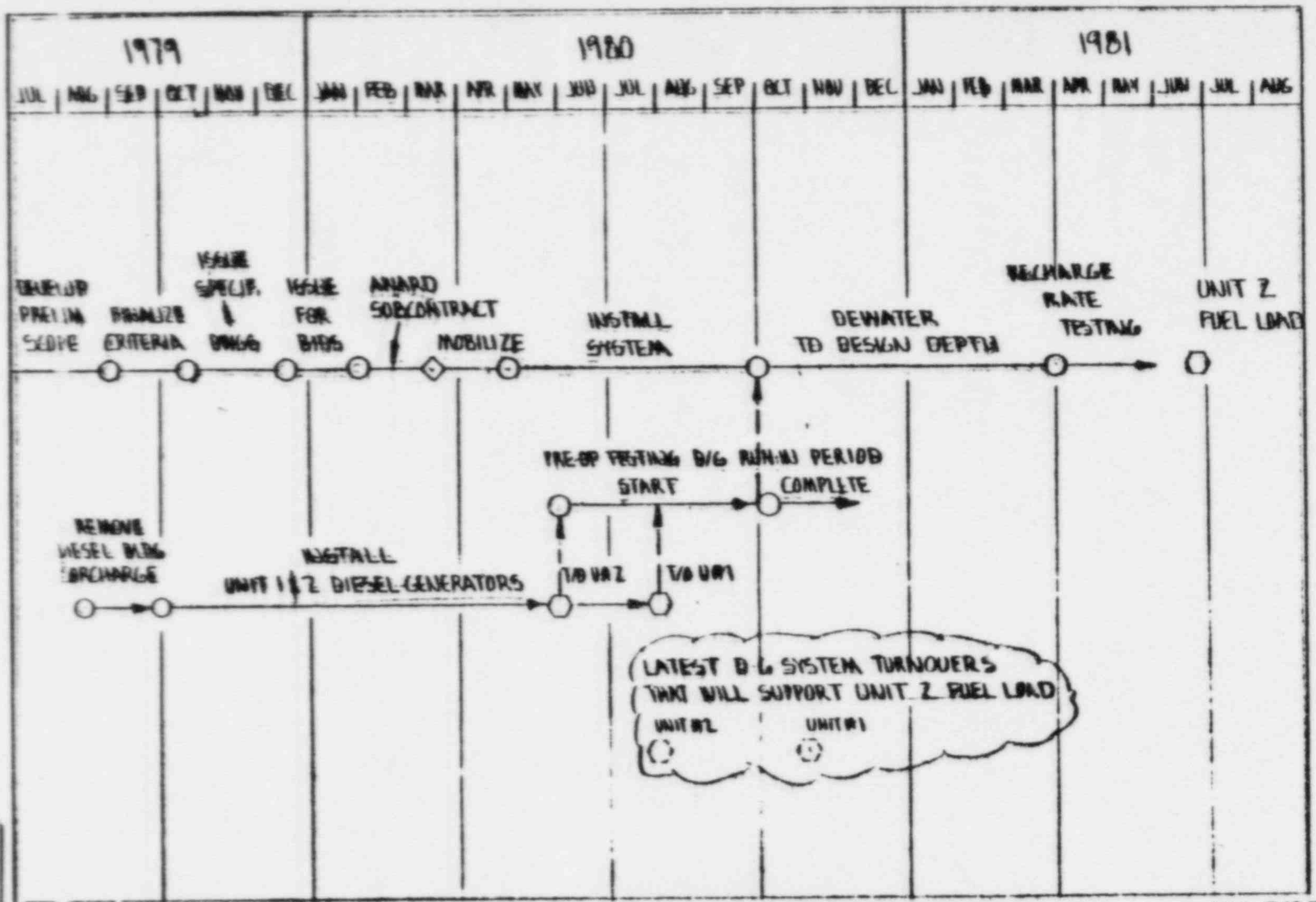


FIGURE 59

PERMANENT PLANT DEWATERING SYSTEM



AAD 7-12-79

FIGURE 60

CRITERIA FOR INSUFFICIENTLY COMPACTED PLANT AREA FILL (On a "To Date" Basis)

- SETTLEMENT GREATER THAN EXPECTED
- RESULTS OF SOILS INVESTIGATION

G-0695-23

FIGURE 61

SEISMIC CATEGORY I STRUCTURES ON FILL

- AUXILIARY BUILDING (Part)
- SERVICE WATER PUMP STRUCTURE (Part)
- RETAINING WALL AT SERVICE WATER PUMP STRUCTURE
- BORATED WATER TANKS
- EMERGENCY DIESEL GENERATOR FUEL OIL STORAGE TANKS
- SERVICE WATER PIPE LINES AND VALVE PITS
- FW ISOLATION VALVE PITS
- DIESEL GENERATOR BUILDING
- ELECTRICAL DUCT BANKS (Part)
- EMERGENCY DIESEL FUEL OIL & BORATED WATER LINES

FIGURE 62

INSUFFICIENTLY COMPACTED PLANT AREA FILL
WHAT

Is	Is Not	Distinctions	Changes
DG Bldg	Aux Bldg Control Tower	Time Differential between Placement of Fill and Constr of Facility	
Diesel Tank Area	Plant Area Dikes	Plant Fill Not Dike	Placement Method Controlled
Borated Storage Tank Area			Compaction Results
		Specification C-211	Lift Thickness
SW Pipelines			
Aux Bldg Elec Pen Areas			Moisture Control
FW Isolation Vlv Pits			
SW Pump Structure (Part)			Frost Protection
Aux Bldg RR Bay		Materials	Structural Backfill Introduced (Spec C-211)
Emerg Diesel Fuel Lines Borated Water Lines			
Elect Duct Banks (Part)			
SW Vlv Pits		Acceptance Criteria	Relied on Testing

G-0695-03

FIGURE 63

INSUFFICIENTLY COMPACTED PLANT AREA FILL WHERE AND EXTENT

Is	Is Not	Distinctions	Changes
Plant Fill Area	Plant Dike	Small Areas	Increased Test Frequency and Location
			Different Contractor (Bechtel)
			Struct backfill Introduced
			Hand-Held Equipment
			Nonuniform Compaction Methods
		Open to Cooling Pond	Moisture Intrusion in Ground

G-0695-07

FIGURE 64

INSUFFICIENTLY COMPACTED PLANT AREA FILL WHEN

Is	Is Not	Distinctions	Changes
During Placement of Plant Fill		Pond Filled 3/78	Moisture Intrusion
		Used Stockpile for Borrow after 3/77	Weathered Material
			Initial Moisture Content
			Material In Stockpile?
		1977 Dry Year	Final Moisture Content
		Late In Backfill Operation	Own Weight Settlement (Calcs)

G-0695-08

FIGURE 65

INSUFFICIENTLY COMPACTED PLANT AREA FILL (Cont.) WHEN

Is	Is Not	Distinctions	Changes
During Placement of Plant Fill		QC Changed to Surveillance In Summer 1976	Inspection Procedures Personnel Qualifications
		Canonie QC Program Discontinued 9/77	
		Canonie Worked 8/77 - 9/77	
		Changed Moisture Control Method 8/77 - 3/73	
		1974-75 Slowdown	Personnel Mobilization Bechtel U. S. Testing
		Spec C-211 Issued & Revised to Include Clay Materials	

FIGURE 66

G-0695-06

POSSIBLE CAUSES

Distinction or Change	Possible Cause	Comments
1. TIME DIFFERENCE BETWEEN PLACEMEMT OF FILL AND CONSTRUCTION OF FACILITY	NO	Cannot Cause Insufficient Compaction
2. PLACEMENT METHOD		
Lift Thickness/Compactive Effort	YES	Equipment Capability Exceeded in Certain Areas
Compaction Equipment	YES	Equipment Capability Exceeded in Certain Areas
Type of Materials	NO	Compatibility Confirmed
Moisture Control	NO	Period of Inadequate Moisture Control Occurred after All but Top Few Feet Compacted
Compaction by Flooding	NO	Problem Occurs in Clays Also
3. ACCEPTANCE CRITERIA: THEORETICAL COMPARISON OF BMP COMPACTION VERSUS SETTLEMENT	NO	Testing to Confirm

FIGURE 67

G-0695-05

POSSIBLE CAUSES (Cont.)

	Distinction or Change	Possible Cause	Comments
4.	SPECIFICATIONS	NO	
5.	SOILS TESTING	YES	Investigation In Process
	Methods Equipment Results/Reports Retests Reviews/Evaluations Personnel		
6.	TEST FREQUENCY FOR SMALL AREAS	NO	Problem not Confined to Small Areas
7.	DIFFERENT CONTRACTORS		
	Personnel Qualifications	NO	See #16
	Different Inspection Methods	YES	See #15
	Placement Methods	YES	See #2

G-0695-09

FIGURE 68

POSSIBLE CAUSES (Cont.)

	Distinction or Change	Possible Cause	Comments
8.	EXTENSIVELY REEXCAVATED AREA	NO	Similar Problems In Areas Where Reexcavation Was Not Done
9.	MOISTURE INTRUSION IN GROUND	NO	Not a Cause for Poor Compaction Possible Increase In Settlement if Compaction was Poor
10.	LEAN CONCRETE FILL	NO	
11.	POND FILLED MARCH 1978	NO	See #9 Above
12.	STOCKPILED MATERIAL Weathering Drying Out	NO	See #13 Below

G-0695-21

FIGURE 69

POSSIBLE CAUSES (Cont.)

	Distinction or Change	Possible Cause	Comments
13.	DRY YEAR 1977	NO	1977 Not a Dry Year
14.	OWN WEIGHT SETTLEMENT (Calculations)	NO	Cannot Cause Poor Compaction
15.	INSPECTION PROCEDURES	YES	Bechtel Quality Control Method Relied on the Test Results
16.	PERSONNEL	NO	Review of Qualifications of Bechtel and U.S. Testing. Personnel Shows Sufficient Education, Experience and Training to Carry Out Tasks Assigned
17.	EFFECTS OF 1974-75 SLOWDOWN	NO	

G-0695-22

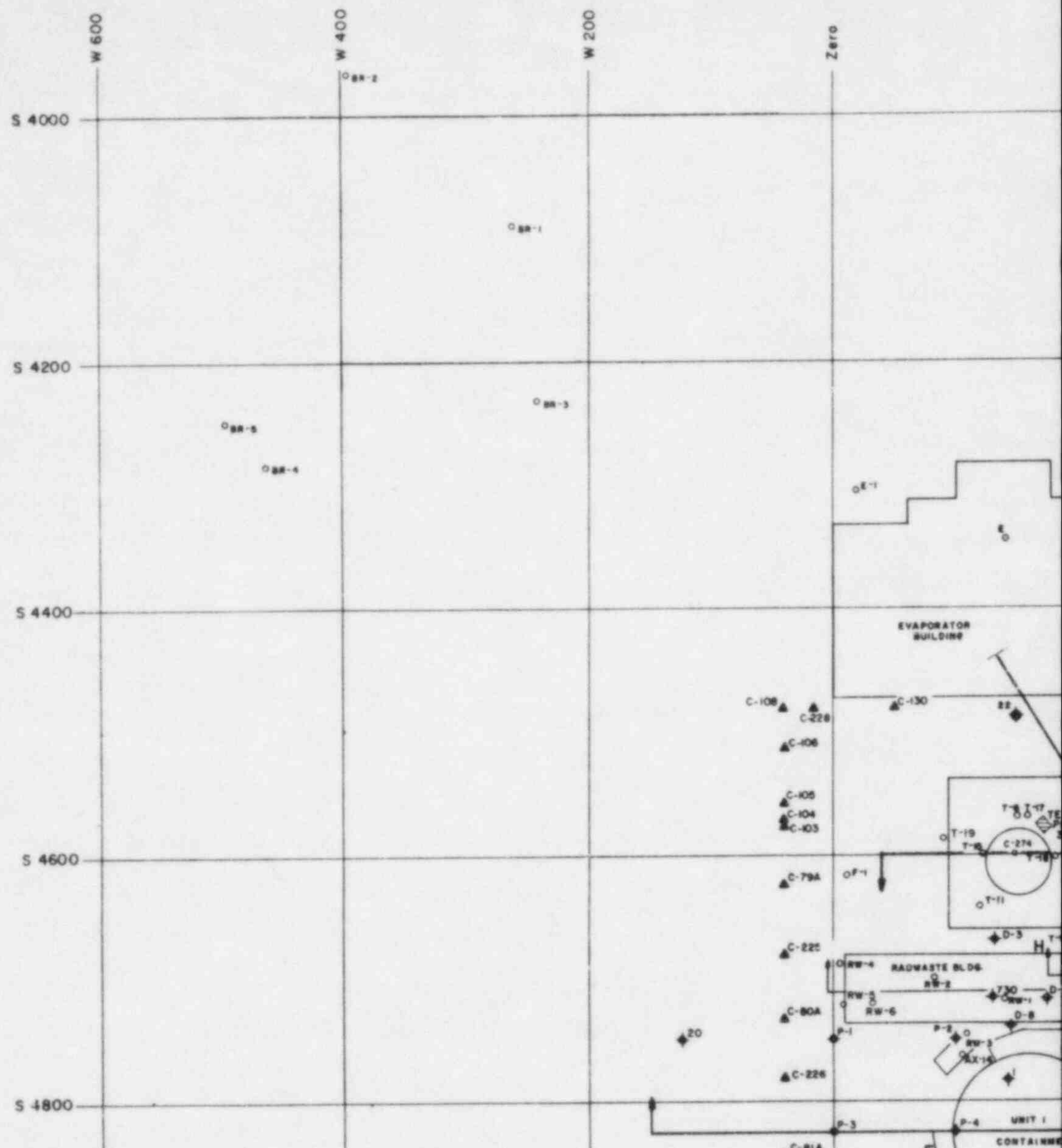
FIGURE 70 -

MOST PROBABLE CAUSES

- LIFT THICKNESS/COMPACTIVE EFFORT
- COMPACTION EQUIPMENT/QUALIFICATION
- TEST PROCEDURES AND RESULTS
- INSPECTION PROCEDURES
- RELIANCE ON TEST RESULTS

G-0695-14

FIGURE 71



W 600
W 400
W 200
Zero
S 4000
S 4200
S 4400
S 4600
S 4800
S 5000
S 5200
S 5400
EVAPORATOR BUILDING
RADWASTE BLDG.
UNIT I
ADMIN AND SERVICE BUILDING
GUARD HOUSE
FOR DIESEL GENERATOR SECTIONS, SEE INSERT A

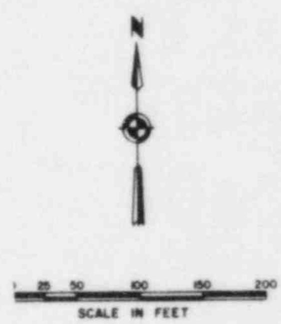
EXPLANATION

EXPLORATION PROGRAM - PRECONSTRUCTION

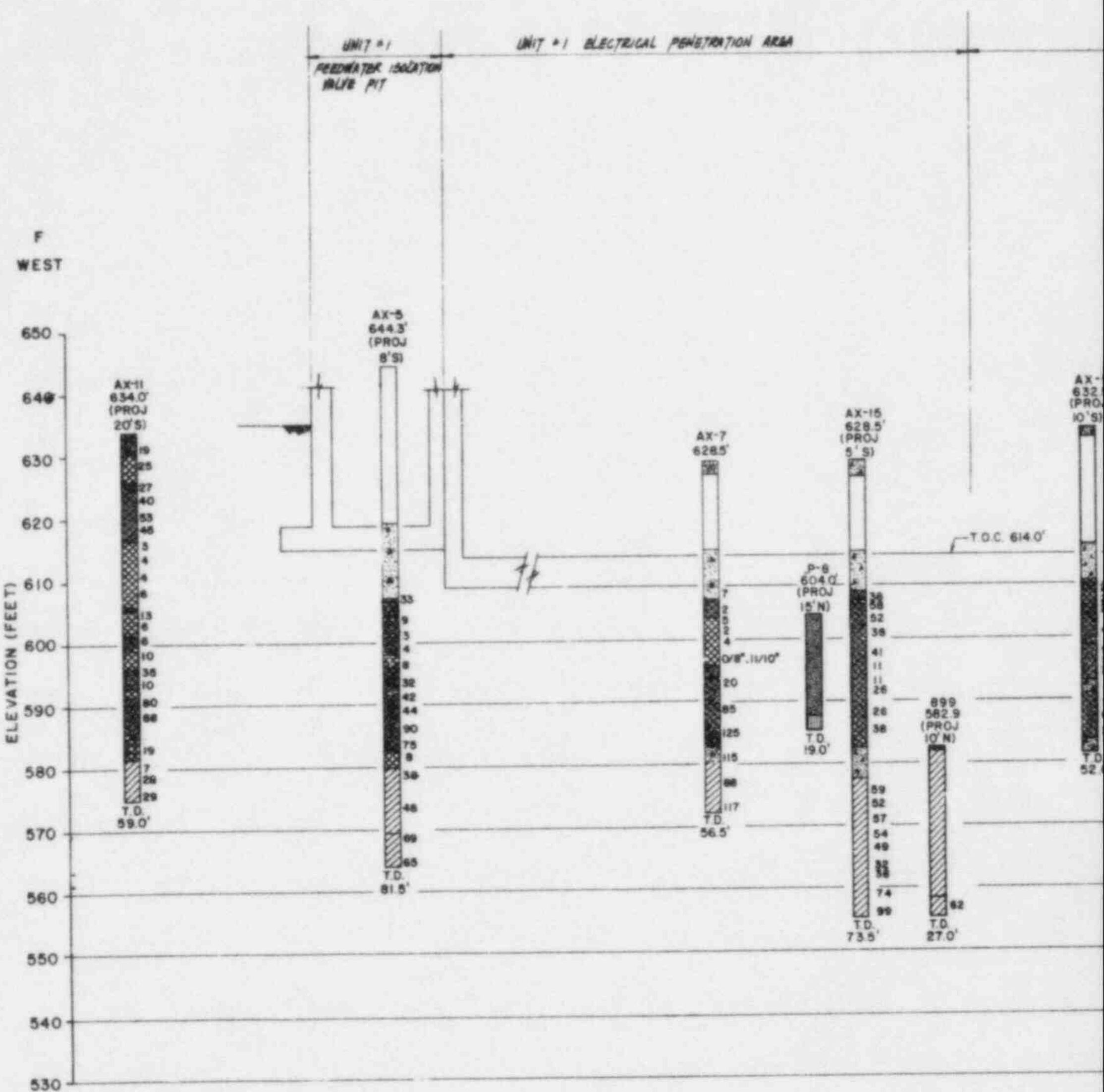
- ■ Michigan Drilling Company Borings; 1956 & 1968
- ◆ Dames & Moore Borings; 1968 & 1969

EXPLORATION PROGRAM - CONSTRUCTION PERIOD

- ▲ Walter Flood Company Borings; 1969 & 1970
- ◆ Bechtel Borings; 1970
- ■ Soil and Materials Engineering Inc. Borings; 1973
- ◆ Bechtel Borings; 1973 & 1974
- ○ Bechtel Borings; 1977 & 1978
- ○ Bechtel Borings; 1978 & 1979

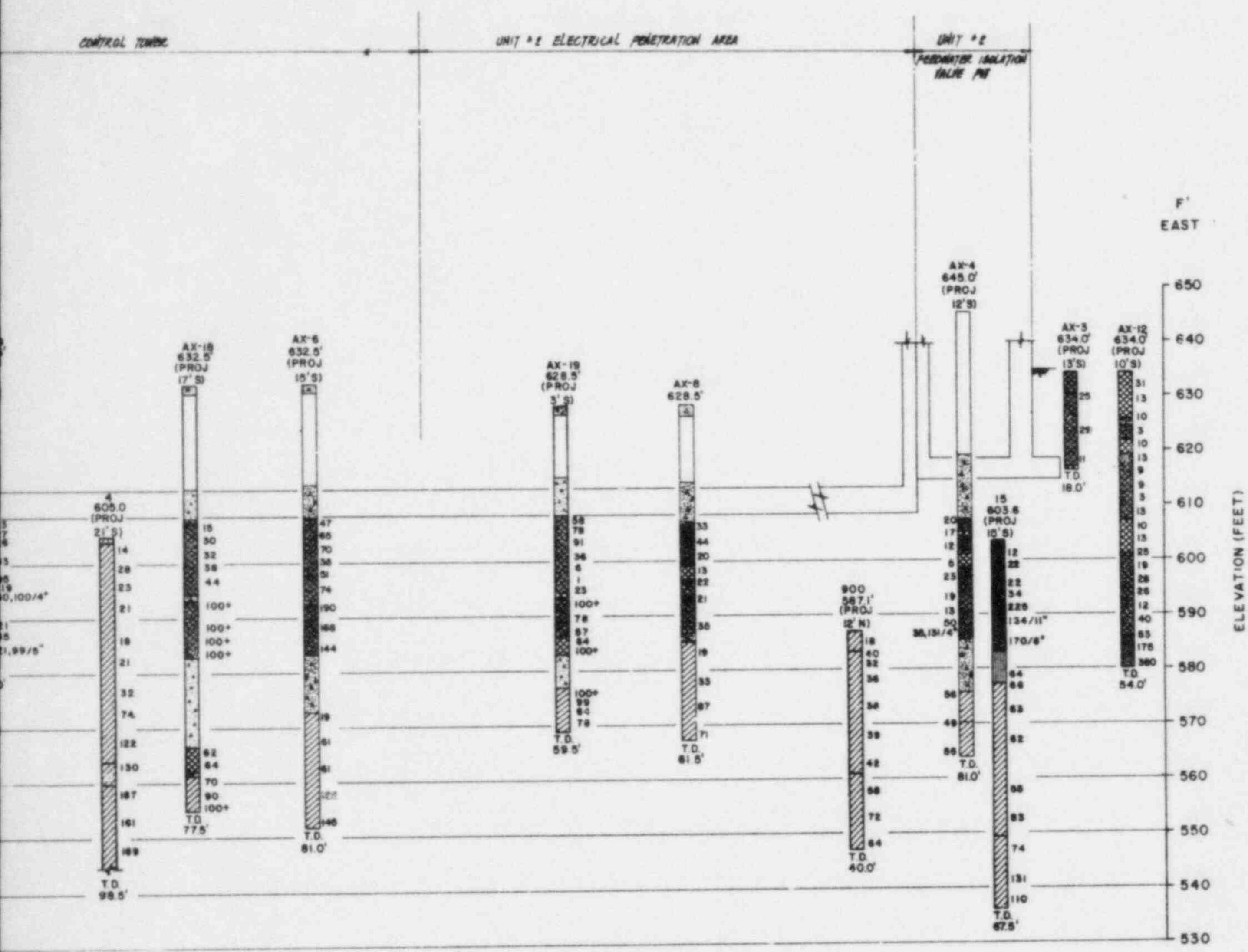


MIDLAND POWER PLANT	
PLANT AREA BORING PLAN	
FIGURE NO. 1	



LEGEND

- CLAY FILL
- SAND FILL
- SANDY GRAVEL (GW)
- CONCRETE
- CLAY (CL)
- SAND (SP)
- SILT (ML)



EXPLANATION

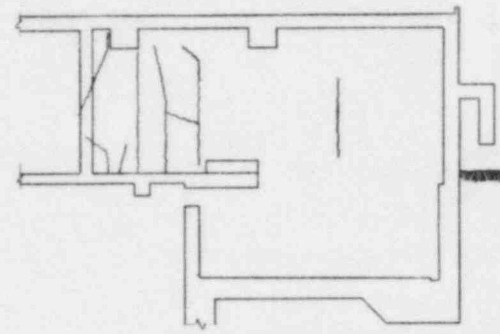
AX-6 — BORING NUMBER
632.5' — GROUND SURFACE ELEVATION
(PROJ 15'S) — DISTANCE AND DIRECTION OF BORING TO SECTION LINE
47 — STANDARD PENETRATION BLOWCOUNT BLOWS/FOOT
T.D. 81.0' — TOTAL DEPTH IN FEET

NOTES:

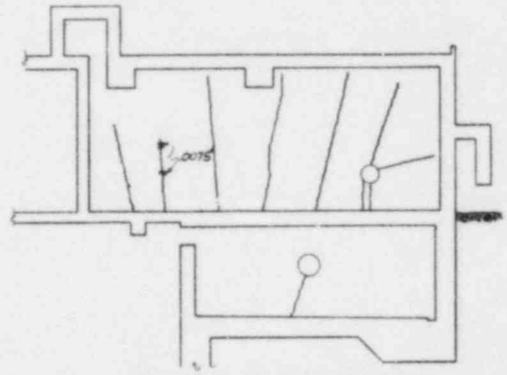
1. Borings P-8, 4, 15, 899, & 900 were made for preconstruction exploration program before 1969. The remaining borings were made specifically for backfill investigation in 1978 & 1979.

0 5 10 20 30 40
SCALE IN FEET

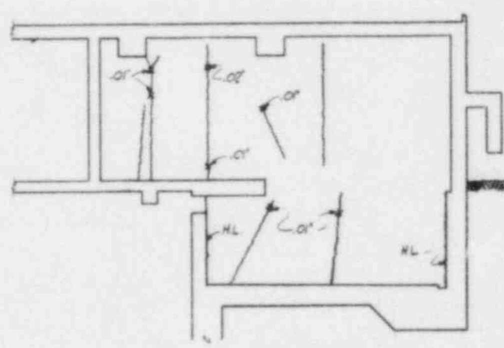
MIDLAND POWER PLANT	
CROSS-SECTION F-F' AUXILIARY BUILDING SOUTH	
FIGURE NO.	REV.
2	6



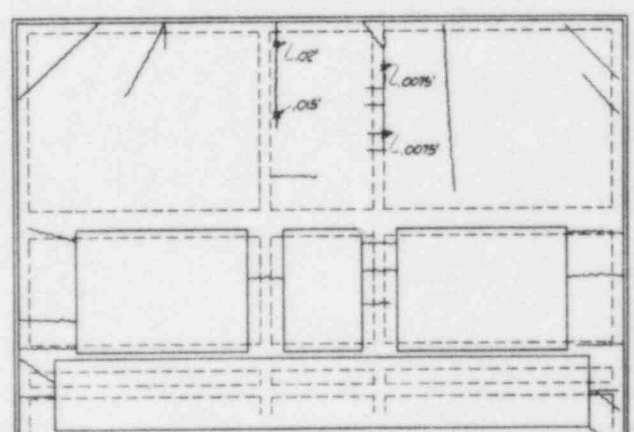
EAST WALL·EAST FACE
LOOKING WEST



CENTER EAST WALL·WEST FACE
LOOKING WEST



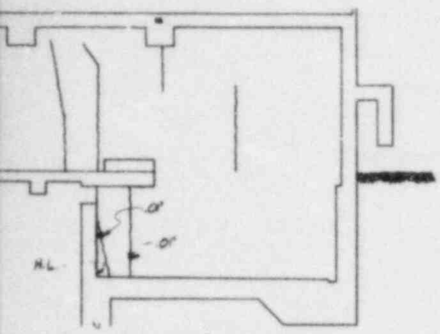
WEST WALL·EAST FACE
LOOKING WEST



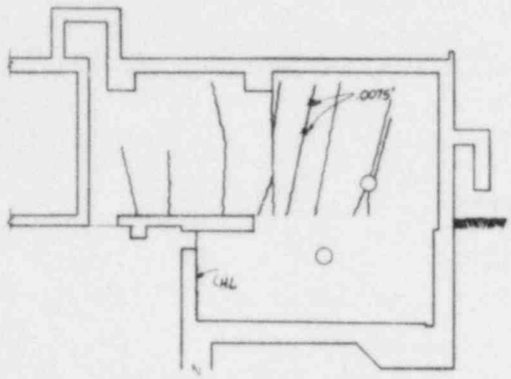
ROOF SLAB ABOVE

8 | 7 | 6

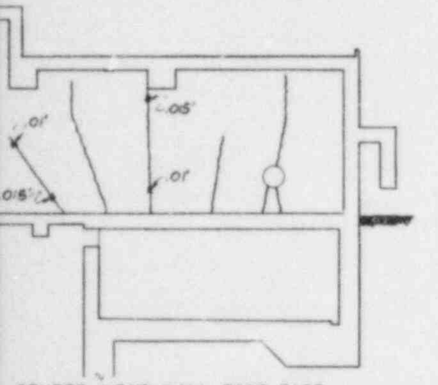
SIZE E



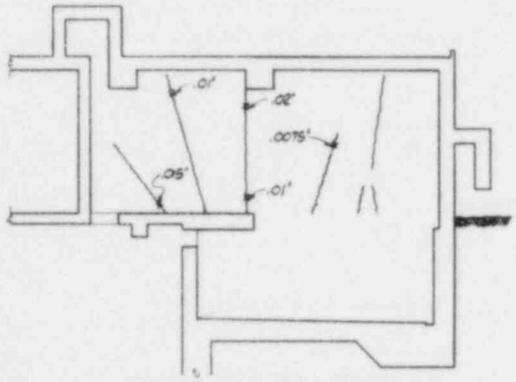
EAST WALL-WEST FACE
LOOKING WEST



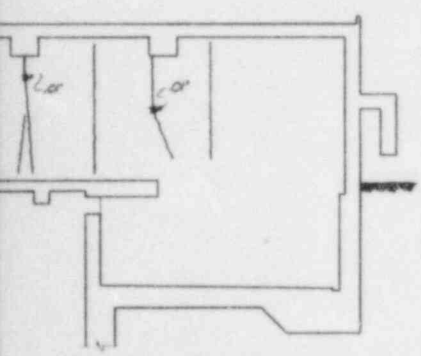
CENTER EAST WALL-EAST FACE
LOOKING WEST



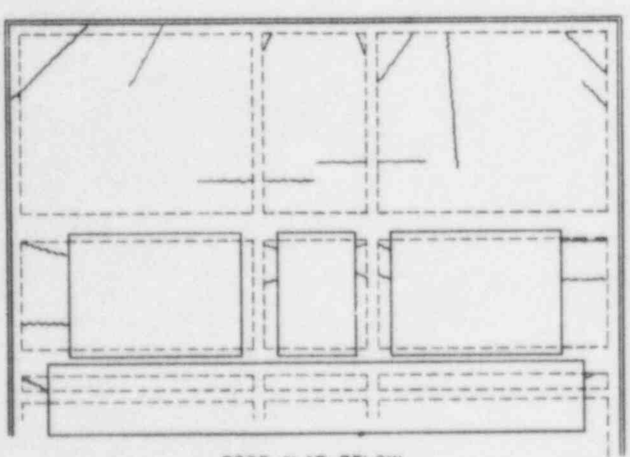
CENTER WEST WALL-EAST FACE
LOOKING WEST



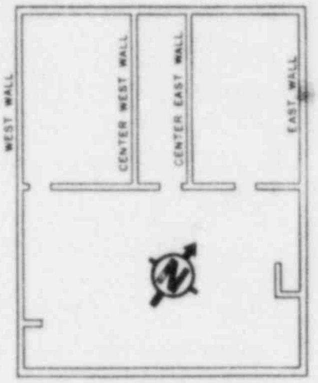
CENTER WEST WALL-WEST FACE
LOOKING WEST



WEST WALL-WEST FACE
LOOKING WEST



ROOF SLAB BELOW



KEY PLAN

NOTES

1. ALL CRACKS ARE HAIRLINE OR LESS THAN .005' THICK UNLESS NOTED.

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

SERVICE WATER PUMP STRUCTURE

CRACK MAPPING

DRAWING No.	REV.
FIGURE 3	SR