ENCLOSURE 1

SOIL BACKFILL CONDITIONS

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

Chapters 4 and 5

AUGUST 31, 1983

4.0 RECONCILIATION OF SOIL CONDITIONS ON SAFETY RELATED STRUCTURES

4.1 General

The effects of the soil backfill behavior during and following the DBE event on all of the safety related structures at San Onofre Unit 1 have been evaluated. The results of these evaluations are presented in this section.

The foundation outlines of all the major structures and the modified footings of the Turbine Building Extensions, are shown in Figure 4-1. As shown in Figure 4-1 all of the major structures are bearing directly on native soil except for portions of the Ventilation Equipment Building, the Control Building, and the Turbine Building extensions. The delineation and characterization of the backfills adjacent to structures are shown in Figure 2-22 (see revision to Figure 2-22 in Addendum 1). These backfills are comprised of category A, B, C and D backfill. The effects of less dense backfill on the analyses already performed (see References 8 through 14) are a change in the design lateral soil pressure and a softening of soil stiffnesses used in the dynamic analyses due to a decrease in the contribution of embedment effects to the overall soil stiffnesses.

The structures affected by this change were evaluated by comparison of previously governing design lateral pressures with the revised in-situ backfill pressures and/or by assessment of changes in the dynamic characteristics of the structures due to modified soil spring constants. For the latter method the dynamic models of the affected structures were utilized for the evaluation. For those structures without associated backfill, this supplemental evaluation was not required.

The following sections describe the geologic conditions of each structure's foundation in terms of the present backfill conditions and provide the changed soil-foundation dynamic parameters utilized in the supplemental evaluations. These sections also discuss the supplemental structural evaluation appropriate for each structure. Specifically, Section 4.2 covers the Containment Structure and the Sphere Enclosure Building; Section 4.3, the Turbine Building and the Turbine Pedestal; Section 4.4, the Ventilation Equipment Building; Section 4.5, the Reactor Auxiliary Building; Section 4.6, the Circulating Water System Intake Structure; Section 4.7, the Control Building; Section 4.8, the Seawall; Section 4.9, the Diesel Generator Building and Section 4.10, the Fuel Storage Building.

4.2 Containment Structure and Sphere Enclosure Building

4.2.1 Present Condition

The Containment Structure foundation was constructed during initial plant construction operations in 1964 and 1965, and as indicated in Figure 2-22 is founded entirely on native San Mateo sand. The Sphere

Enclosure Building foundations were constructed in mid 1976 and had native San Mateo sand exposed at the base of all footing excavations except at the southwest end of the footing where a small portion of the footing extends over the backfill placed adjacent to the Fuel Storage Building. The existing material in this area was overexcavated to native soil (at elevation +7 ft). The area was then backfilled with San Mateo sand compacted to a minimum relative compaction of 95% to the base of the footing at about elevation +11.5 feet. Likewise, all backfill placed adjacent to the structure was San Mateo sand compacted to a minimum relative compaction of 95 percent.

The responses of the Containment Structure and the Sphere Enclosure Building are not affected by backfill conditions, as all foundations are founded on native soil except the southwest end of the Sphere Enclosure footing which was founded on backfill compacted to at least 95 percent relative compaction.

4.2.2 Structural Evaluation

Additional evaluations are not required for either the Containment Structure or the Sphere Enclosure Building.

4.3. Turbine Building and Turbine Pedestal

4.3.1 Present Condition

The Turbine Building and Turbine Pedestal mat were constructed during the initial plant construction operations in 1964 and 1965. Subsequently between February of 1982 and March 1983, modifications were made to the footings of the north and south extensions and east and west heater platforms. As indicated in Figure 2-22 the Turbine Pedestal mat is founded entirely on native soil while the perimeter backfill is category B, C and D backfill soil. The existing Turbine Building footings, including modifications, are shown in Figure 4-1. The soil conditions at each of the modified footings in the north and south extensions and the east and west heater platforms are summarized in Table 4-1. It should be noted that all footing modifications have been completed except footing D in the south extension which has been partially excavated (as of June 1983). It should also be noted that footing F has been completed; however, this is not reflected on the foundation location plans in Sections 1, 2 and 3 and the appendices of this report which were forwarded by letter dated April 18, 1983.

The bearing and embedment conditions for the north and south extension and east and west heater platform foundations summarized in Table 4-1 show that all footings are supported by native soil, either directly or indirectly through native soil and/or adjacent elements of other structures including the Turbine Pedestal foundation. Since the native soil dominates the support of these foundations, seismic settlements will be negligible for the Turbine Building and Turbine Pedestal mat.

4.3.2 Structural Evaluation

The evaluation of the in-situ backfill effects on the Turbine Building was accomplished by determining the significance of the change in the soil stiffness parameters. The revised soil stiffness values were obtained by multiplying a "Reduction Factor" times the original (based on 95% backfill compaction) stiffness terms to determine the new values. A range of values was developed to account for any variations which might exist in the in-situ backfill soil. The reduction factors for the spent fuel pool structure, which is structurally connected to the north extension at one footing location, and for the pedestal mat foundation are given in Table 4-2. The initial stiffness parameters and the correction factors for the Turbine Building extension footings (including the east and west heater platforms and the north and south extensions) are presented in Tables 4-3 and 4-4, respectively.

The turbine mat stiffness parameters previously calculated assuming backfill compacted to 95% have been reduced because portions of the perimeter backfill were compacted to less than 95%. The changes in the stiffness parameters are tabulated in Table 4-2 and were obtained by applying the procedures described in Section 3 to the backfill conditions defined on Figure 2-22.

For the east heater platform and the south extension the reduction factors to original soil stiffness values were based on a conservative assumption that the backfill conditions in those areas would be 85 percent compacted material (the lower bound condition for soil stiffness values). At the time of the analysis, the footing excavations were not yet performed. Subsequent investigations indicated that the lower bounds of the backfill and foundation support conditions in these areas, as summarized in Table 4-1, were adequately defined by the soil parameters which were used.

The modified stiffness parameters for the soil-foundation system were included in the composite Turbine Building Complex numerical model and an eigenvalue-extraction was performed using the subspace iteration algorithm. The mode-shapes for various structural frequencies were then plotted. The results obtained from this analysis were compared with those from the previous analyses and are summarized below for the west and east heater platforms, the north and south extensions, and the turbine pedestal.

The masonry walls associated with the Turbine Building are either integral with the modified Turbine Building foundations or are founded on native San Mateo sand. Therefore, supplemental reevaluation of the masonry walls was not required.

4.3.2.1 West Heater Platform Structural Evaluation

The comparisons of mode shapes made for the west heater platform are typified by the values listed below.

FREQUENCY OF FUNDAMENTAL MODES
WEST HEATER PLATFORM

Mode No.	Mode Shape Description	With Orig Soil Springs	With Low Bound Soil Springs
1	E-W Translation	4.14 CPS	4.27 CPS
2	N-S Translation	5.48 CPS	5.65 CPS

The results of the eigenvalue extraction show that for the fundamental structural modes, east-west and north-south translation, the frequency has increased by 3.1% when compared with the original analysis. This is a result of an increase in the overall stiffness of the soil-foundation system due to the changes in the foundation configurations. As discussed earlier, the analysis with the lower bound soil-springs takes into account the modified foundation configurations (support provided by the box culverts to the continuous foundation along line 13, the outrigger foundations, etc.). If the overall stiffness of the modified foundation system is considered and compared with the original analysis it can be seen that, for the vertical translation and north-south rocking, the overall stiffness has increased, while the horizontal translational stiffness and east-west rocking have decreased (see Table 4-5).

If a similar comparison is made for the column line 13 footings and the outrigger footing alone (see Table 4-6) it can be seen that the vertical translational stiffness and north-south rocking stiffness have increased while the horizontal stiffness and east-west rocking have decreased. The combined effect of these changes in the low bound analysis is a 3.1% increase in the frequency of the fundamental modes for east-west and north-south translation. This is attributed to the general increase in the foundation system stiffness.

The mode shapes themselves are essentially identical, and therefore it is concluded that no significant change in the lateral loads or displacements would occur. Since the lateral loads applied on the west heater platform framing remain essentially the same, the structural integrity evaluation and the resulting conclusions for the columns, beams, connections, and column base anchorages remain valid.

4.3.2.2 North Extension Structural Evaluation

The response of the north extension is governed by the response of the T/G Pedestal mat and the spent fuel pool. This is because two columns, D-6 and D-8, at the southern end of the extension are founded on the T/G

Pedestal mat and because the footing for column B-8 at the western edge of the extension is integrally cast with the massive fuel pool walls. Any translation or rocking mode of the fuel pool or the T/G Pedestal mat induces motion in the north extension framing.

Hence, if the soil structure interaction parameters for the fuel pool or the mat change by a significant margin, the structural response could be affected. Since the higher bound spring stiffnesses for the T/G Pedestal mat are 92% to 98% of the original springs, they are considered essentially unchanged. Thus only the lower bound soil stiffness results are considered.

A comparison of the mode shape plots show that the major structural modes of the north extension are unchanged. The frequencies of the modes associated with the fuel pool and the T/G Pedestal mat have changed as indicated in Table 4-7. The variation in frequencies range from a 4% reduction for the fundamental vertical mode of the T/G Pedestal mat to a 22% reduction in the E-W rocking mode of the fuel pool. Based on the Housner free field spectra and the shift in frequencies of the fundamental structural modes, it is concluded that the lateral loads on the north extension will increase by approximately 15 percent.

The structural modifications recently installed in the north extension convert the framing from moment carrying frames to braced moment carrying frames. Thus, the primary lateral load resistance mechanism is the bracing system. The bracing system was designed with more than a 30% margin over what was required for the original seismic reevaluation loads. Thus, the installed bracing system is more than adequate to resist the possible increase in the lateral loads. Since the primary lateral load carrying system is diagonal braces, most of the increased lateral loads will be resisted by the bracing members.

4.3.2.3 East Heater Platform Structural Evaluation

The comparisons of the fundamental frequencies between original and low bound springs made for the east heater platform are shown in the table listed below.

FREQUENCY OF FUNDAMENTAL MODES
EAST HEATER PLATFORM

Mode No.	Mode Shape Description	With Orig. Soil Spring	With Low Bound Soil Spring
1	E-W Translation	4.59	4.32
2	N-S Translation	5.82	4.77

The results of the eigenvalue extraction show that for the fundamental structural mode, the east-west translation, the frequency has reduced by 6 percent while for the north-south translational mode the frequency has decreased by 18 percent when compared with the original analyses. This is a result of a decrease in the overall stiffness of the soil-foundation system. The mode shapes themselves are essentially identical when compared with the original analysis. This reduction in the frequency will result in approximately 10 percent increase in the lateral loads.

The bracing system and associated foundation modifications have been designed with a 30 percent margin over what was required for the original seismic reevaluation loads. Since the primary lateral load carrying mechanism is diagonal braces most of the lateral loads or any increase in applied lateral loads will be resisted by the bracing system. Thus the installed bracing system and associated foundation modifications are more than adequate to resist a possible increase in the lateral loads.

4.3.2.4 South Extension Structural Evaluation

The comparisons of mode shapes for the south extension are typified by the values listed below.

Mode No.	Mode Shape Description	With Orig. Soil Spring	With Low Bound Soil Spring
1	E-W Translation	8.89	8.50
2	N-S Translation	8.27	7.61

The comparison of the mode shape plots shows that the mode shapes are essentially unchanged.

The frequency of the fundamental modes is reduced by a maximum of 8 percent in the present analysis when compared with the original evaluation. This reduction in the frequency will result in approximately 10 percent increase in the lateral loads.

The structural modifications being installed in the south extension convert the framing from moment carrying frames to braced moment carrying frames. This bracing system was designed with more than 30 percent margin over what was required for the original seismic loads. Since the primary lateral load carrying system is diagonal braces, most of the increase in loads will be resisted by bracing members. Thus the installed bracing system will be more than adequate to resist a possible increase in lateral loads when completed.

4.3.2.5 Turbine Pedestal Structural Evaluation

The comparisons of the mode shapes and the fundamental frequencies between original and low bound soil springs for the Turbine Pedestal are shown in Table 4-7. The results of the eigenvalue extraction show that for the fundamental structural modes, the frequency for the east-west rocking mode is reduced by 12 percent while for the north-south translational mode and the vertical mode the frequencies decreased by 9 and 4 percent respectively. The lower frequencies result from a decrease in the overall stiffness of the soil-foundation system.

Comparing the mode shape plots with the original analysis shows that the two are essentially identical. For the reductions in frequencies stated above, an approximate increase in lateral loads of 2 percent will result.

No structural modifications were required as a result of the seismic reevaluation of the Turbine Pedestal. Since lateral loads increase approximately 2 percent, the results and conclusions of the reevaluation remain valid.

4.3.2.6 Instructure Response Spectra For The Turbine Building

The instructure response spectra for the Turbine Building complex were originally calculated based on assuming 95 percent soil compaction in all areas. The spectra were subsequently modified with additional analyses to reconcile the effects of in-situ soil.

The reduction factors to the soil stiffness values reflecting the actual soil condition were used for the north extension and west heater platform. The excavation and foundation modifications in the east heater platform and south extension were in progress at the time of this analysis, therefore all the data for in-situ soil were not available. The soil stiffness values in the east and south extension were modified by conservatively assuming that the backfill conditions in those areas would be 85 percent compacted material. A time history analysis was performed using the modified soil-structure interaction parameters and a new set of instructure spectra was developed. The instructure spectra based on the reduced soil stiffnesses were termed lower bound spectra. Then the instructure spectra for the Turbine Building were widened using +15 percent peak broadening from the upper bound frequency (95 percent compaction, original analysis) and -15 percent peak broadening from the lower bound frequency (compaction corresponding to the backfill soil conditions). The evaluations of systems and components were based on these final instructure response spectra which were obtained by enveloping the broadened spectra due to upper bound and lower bound cases.

4.4 Ventilation Equipment Building

4.4.1 Present Condition

The Ventilation Equipment Building foundation was constructed during initial plant construction in 1964 and 1965. As shown in Figure 2-22, 19 ft of the north wall and about 10 feet in the northern portion of the east wall are founded on native soil. A cross-section of the backfill beneath the Ventilation Equipment Building is shown in Figure 4-2a and discussed below. The cross-section in Figure 4-2a shows less backfill beneath the west wall than was documented in Section 2 of this report. This change was made based on observations which have been located regarding the inspection of foundations for the dog house structure north of the Ventilation Building in December 1980. The western 3 feet of the north wall is founded on about 4 feet of backfill, designated as category D. The center 24 feet of the east wall is founded on up to about 8 feet of category D backfill and the southern 10 feet of the east wall is founded on backfill with depth varying from 8 feet to a maximum of 21 feet near the Fuel Storage Building. The west wall is founded on about 4 feet of category D backfill with the southern 10 feet of this wall founded on from 4 feet to a maximum of 21 feet of category D backfill. The south wall of the structure is founded on category D backfill with a thickness of about 21 feet near the Fuel Storage Building. As indicated in Figure 3-4, the category D backfill on the south side of this structure could develop high pore water pressures when subjected to the 0.67g Housner DBE.

The estimated maximum seismically induced settlement of the soil below this structure is summarized in Figure 4-2b. The proportion of settlement above the water table is designated by (a), and the proportion of settlement below the water table is designated by (b). It is expected that the settlement of soil above the water table, shown as a maximum of one and one-half inches of settlement in the settlement profiles in Figure 4-2b, would occur during seismic shaking. The settlement caused by liquefaction of soil below the water table would occur as pore water pressures dissipate after seismic shaking. The total soil settlement beneath the south end of the building is three inches, including post-seismic settlement.

The fill soil may affect the seismic response of the structure along the west, south, and portions of the east sides of the structure. During seismic shaking a reduction of the foundation stiffnesses (which are based on 95% relative compaction) has been considered as discussed in Section 3. The range of reduced soil stiffnesses which will be used for this structure are provided in Table 4-8. The stiffnesses consider the affects of backfill as well as potential soil separation beneath the south wall footing and 10 feet of the adjoining east and west wall footing.

4.4.2 Structural Evaluation

The structural evaluation of the Ventilation Equipment Building will be provided by September 9, 1983.

4.5 Reactor Auxiliary Building

4.5.1 Present Condition

The Reactor Auxiliary Building foundation was constructed during initial plant construction in 1964 and 1965 and as indicated in Figure 2-22 is founded entirely on native soil. The perimeter backfill, however, has been classified as a category D soil fill.

As indicated in Figure 3-4, the category D soil backfill around the perimeter of the Reactor Auxiliary Building could develop high pore water pressure when subjected to the 0.67g Housner DBE. These excess pore water pressures in the backfill below the water table could cause the structure to be subjected to lateral pressures as shown in Figure 3-6.

4.5.2 Structural Evaluation

The only effect of the in-situ backfill condition on this structure is the lateral soil pressure distribution as shown schematically in Figure 3-6. The previously performed seismic reevaluation of this structure considered the following soil pressure combinations for the below grade exterior walls.

a) Static Active + Surcharge + Hydrostatic
b) Static Active + Dynamic Passive + Surcharge + Hydrostatic + Hydrodynamic

A comparison of the soil pressure of Figure 3-6 with the pressures obtained from the load combinations listed above indicates that load combination "b" governs. Comparisons showed that the combined soil pressures are higher than the pressure distribution of Figure 3-6. Since load combination "b" governs, and is greater than Figure 3-6, there is no need for further analysis and it is concluded that the Reactor Auxiliary Building meets the seismic reevaluation criteria.

4.6 Circulating Water System Intake Structure

4.6.1 Present Conditions

The Intake Structure was constructed during initial plant construction in 1964 and 1965. As indicated on Figure 2-22, this structure is founded entirely on native soil. However the perimeter backfill is classified as category B and C soil fill.

As indicated in Figures 3-2 and 3-3, the category B and C soil backfills around the perimeter of the Intake Structure could develop high pore water pressure throughout the deeper portions of these fills when subjected to the 0.67g Housner DBE. These excess pore water pressures in the backfill below the water table could cause the structure to be subjected to lateral pressures as shown in Figure 3-6.

4.6.2 Structural Evaluation

Similar to the Reactor Auxiliary Building the effect of in-situ backfill on this structure is the lateral soil pressure distribution as described in Figure 3-6. The below grade exterior walls of the Intake Structure were previously evaluated with the same load combinations as those discussed in Section 4.5 for the Reactor Auxiliary Building. Comparisons showed that the combined soil pressures are higher than the pressure distribution of Figure 3-6. Therefore, there is no need for further analysis.

4.7 Control Building

4.7.1 Present Condition

The Control Building foundation was constructed during initial plant construction in 1964 and 1965 and as indicated in Figure 2-22 is founded on native soil with the exception of five small areas. These small areas, which are only 8.5 percent of the total foundation, are interpreted to be founded in category D soil fill, which implies 85 percent relative compaction at all depths. The soil fill is entirely above the water table, therefore it is not susceptible to liquefaction.

Only a modest reduction in overall stiffness parameters is expected as a result of the localized footing areas affected. The reduction in stiffness was calculated using the procedure described in Section 3 for these small footing areas. The resulting reduction factor to be applied to the overall stiffness of the structure as summarized in Table 4-2, is 0.97 to 1.0. In the localized areas where the footings are founded on category D fill (Figure 2-22), the fill varies from 0 to 3 or 4 feet thick beneath these footings. Because this fill is above the water table, fill settlements would be expected to occur during seismic shaking and would be less than 1/2 inch in the localized areas.

4.7.2 Structural Evaluation

The modified soil-foundation stiffness parameters for the Control Building have an overall change of about 3 percent as shown in Table 4-2. This change is insignificant and will not alter the results of the original reevaluation. Because the settlements occurring during shaking are slight and are localized, there will be negligible effect on the overall structure's performance. Therefore, it is concluded that the original seismic reevaluation of the Control Building remains valid, including the computed instructure response spectra, and that the structure meets the seismic reevaluation criteria.

4.8 Seawall

4.8.1 Present Condition

The Seawall was constructed during the initial plant construction in 1964 and 1965. The wall is constructed of sheet piles driven to elevation - 8.0 ft. For most of its 1,500 foot length, the sheet piles were driven into the native San Mateo sand, which exists at an average elevation of about +2.0 ft. except for about 300 feet in the southern section where the wall was constructed in the backfill placed in the excavation for the intake structure. Soil conditions on the sea side of the wall (outside of the main plant area) were investigated for construction of the beach walkway in April and May of 1980 by drilling 34 borings between 2 and 25 feet in depth within 15 feet of the wall. The location of these borings in plan view and a stick log summary of the materials

The factor of safety for the tsunami analysis is greater than the factor of safety for the seismic analysis because the soil parameters used for each of the analyses are quite different in that the pore pressure buildup below the water table will be partially relieved by the time the maximum tsunami/storm buildup occurs as a result of the gravel drains on the ocean side of the wall. In addition, the soil passive pressure resistance will be higher when the tsunami occurs because ground shaking is not considered to act simultaneously with maximum tsunami/storm wave buildup.

The analyses indicate that the Seawall will be stable and that stresses will be low for all possible loading conditions assuming worst case soil conditions. Therefore, it is concluded that the Seawall meets the seismic reevaluation criteria for both the DBE and the tsunami events.

4.9 Diesel Generator Building

4.9.1 Present Condition

The Diesel Generator Building foundation was constructed in 1978. The location of the foundation is shown in Figure 2-22. All footings were carefully inspected during construction and the bearing and embedment soil conditions were found to be native soil and backfill compacted to a minimum relative compaction of 95 percent. This observation is in agreement with the delineation of major backfills shown in Figure 2-22.

The response of the Diesel Generator Building will not be affected by backfill conditions as all foundations are founded on native soil and perimeter backfills have been compacted to a minimum relative compaction of 95 percent.

4.9.2 Structural Evaluation

Further evaluation is not required for the Diesel Generator Building foundations.

4.10 Fuel Storage Building

4.10.1 Present Condition

The Fuel Storage Building foundation was constructed during initial plant construction in 1964 and 1965 and as indicated in Figure 2-22 is entirely founded on native soil. The perimeter backfill on the north, south, and west sides of the structure has been classified as a category D soil fill. The backfill along the east side of the structure has been overexcavated and replaced with lean concrete fill during the recent foundation modifications to the adjacent Turbine Building. This was discussed in Sections 2 and 4.3.

As indicated in Figure 3-4, the category D soil backfill on the north, south and west sides of the Fuel Storage Building is subject to development of high pore water pressures when subjected to the 0.67g Housner DBE. These excess pore water pressures in the backfill below the water table could cause the structure to be subjected to lateral pressures as shown in Figure 3-6. In addition, the stiffness parameters used in the analysis of this structure could be significantly affected below the water table and to a lesser extent above the water table as discussed in Section 3. Using the procedures presented in Section 3, modifications were made to the stiffness parameters which were originally developed assuming 95 percent relative compaction. The revised parameters are summarized in Table 4-2.

4.10.2 Structural Evaluation

The original evaluation of the Fuel Storage Building was based on 95 percent compacted soil backfill conditions. The analysis revealed that the structural responses of some of the elements, namely, the masonry walls were nonlinear. The results were provided in Reference 9.

Since it is difficult to predict the effects of changing the soil-structure interaction parameters in a non-linear analysis, the Fuel Storage Building was reanalyzed, utilizing the revised interaction parameters summarized in Table 4-2. Results from the analysis demonstrated that the structure will meet the seismic re-evaluation criteria provided that modifications described therein are implemented (these modifications have subsequently been installed). The details of the analysis procedure, criteria, and the re-evaluation results were provided in Reference 12.

The evaluation of systems, components, and equipment within the Fuel Storage Building are based on final instructure response spectra which were obtained by enveloping the spectra from the initial analysis (95 percent compacted backfill condition) and the spectra from the analysis which reflects the modified soil interaction parameters for the existing backfill soil conditions. The spectra were widened +15 percent from the upper bound frequency and -15 percent from the lower bound frequency.

TABLE 4-1
SUMMARY OF SOIL CONDITIONS FOR TURBINE BUILDING
(Sheet 1 of 4)

Foundation	Soil Characterization
1. North Footing	As shown in Figure B-1, most of the footing is founded on native soil and the backfill or native soil exposed against the footing sides is dense (minimum 95 percent relative compaction) except for the western portion. In this area the foundation is founded on backfill with relative compaction varying from 80 to 93 percent. Backfill encountered in the west end of the excavation was removed and replaced with concrete extending down to about elevation +1 foot. Backfill below elevation +1 ft was compacted by vibration and probed and was left in place. Native soil was encountered, based on probing and evaluation of construction photos and excavation plans, at about elevation - 2.8 feet.
2. Northwest Footings	
o Footing E- 11	As shown in Figure B-1, most of the footing is founded on native soil except a small portion along the east wall. The backfill against the walls is dense, equivalent to a relative compaction of 95 percent except for small portions of the east and south walls where backfill having a relative compaction of 80 to 82 percent was left in place.
o Footing C- 9	As shown in Figure B-1, most of the footing is founded on native soil except for a small width near the north wall. The density of backfill against the north end ranges from 85 to 87 percent compaction. The density of backfill against the other footing sides also varies from 85 to 87 percent compaction.
3. West Footing	Footing north of column line F is founded on the native soil while the remaining footing is founded on backfill placed in the intake-culverts area. This fill varies in relative compaction from 90 to 95 percent. The density of backfill against the sides of the footing has an average relative compaction of about 90 percent. The west footing was placed as one continuous foundation from column C13 to

NOTE: See Appendix B for figures referenced.

TABLE 4-1 (continued)
SUMMARY OF SOIL CONDITIONS FOR TURBINE BUILDING
(Sheet 2 of 4)

Foundation	Soil Characterization
	column K13. The depth of excavation south of column F13 and north of J13 was extended to the top of the intake culverts. The depth of the footing in that area was increased such that the footing rested on both intake culverts. The footing was designed such that the building loads will be transferred to the intake culvert walls. The footing structurally spans over the backfill between the two culverts.
4. Southwest Footing	Northern portion of the footing along column line 13 is founded on backfill with a relative compaction of 98 to 100 percent. Approximately the western half of the remaining footing is founded on backfill with relative compaction of 95 to 99 percent. In the remainder of the footing backfill having a density ranging between 83 and 85 percent was encountered at elevation +7 ft, the planned footing base elevation. The excavation was deeped in this area to approximately elevation +3 where native soil was encountered in most of the area except in a small area (about 4 ft x 6 ft) in the northeast corner. Backfill in that area was left in place. The overexcavated area was backfilled with concrete. The density of backfill against the east side of the footing and half of the north side was found to have a relative compaction of about 90 percent. For the remaining walls the backfill varied in compaction from 90 to 95 percent.
5. Outrigger Footing	This footing, as originally planned was founded on the backfill above the intake culverts. Tests in the excavation showed the backfill to have a relative compaction varying from 87 percent to 93 percent, down to elevation +3. As a result of this observation, the footing design was changed and was modified to be supported on both ends on the intake culverts. The overexcavation below the base of the footing, at elevation +5 was backfilled with concrete. The loads from the footing are transferred directly to the culvert walls and do not rely on any subgrade support between the two culverts.

TABLE 4-1 (continued)
SUMMARY OF SOIL CONDITIONS FOR TURBINE BUILDING
(Sheet 3 of 4)

Foundation	Soil Characterization
6. Northeast Footing	
o Footing E- 3	As shown in Figure B-1, most of the footing is founded on native soil at the design base elevation, +6.0 ft, except for a small portion in the western end. In this area backfill with a relative compaction of 80 to 85 percent was removed and replaced with concrete extending down to about elevation +3 ft, at which elevation the native soil was encountered based on probing and field density test data.
7. East Footings	
o Footing A	As shown in Figure B-1, most of the footing is founded on native soil at the design base elevation +6 feet with the exception that there is a one foot wide strip along the south wall extending from the southwest corner to approximately 15 feet east. Backfill having a relative compaction of 80 to 88 percent is present in this area. The 15 square feet of backfill area is approximately 2 percent of the total area of the foundation and therefore, will have a negligible affect on the support of the foundation. Because of a design change, a partial excavation made at column location G-2 was excluded from the main footing and backfilled with concrete.
o Footing B	As shown in Figure B-1, most of the footing is founded on native soil except in a 10 ft wide area at the west end. This area was excavated to elevation -1.0 ft, and the soil exposed at the base of the excavation was found to be backfill with a relative compaction of about 81 to 89 percent, down to approximately elevation -5.0 feet. The overexcavation below the base of the footing, at elevation +5.5 ft, was backfilled with concrete. The eastern part of the footing rests on native soil and the west end is structurally connected to the anchor block. Therefore, the loads from the footing will be transferred to the native soil and to the anchor block without having to rely on the support of the backfill.

TABLE 4-1 (continued)
SUMMARY OF SOIL CONDITIONS FOR TURBINE BUILDING
(Sheet 4 of 4)

Foundation	Soil Characterization
8. Southeast Footings	
o Footing C	As shown in Figure B-1, approximately two thirds of the northern portion of the footing is founded on the existing anchor block at elevation +8.5 feet. In the south end, the footing is founded on backfill with a relative compaction of about 88 percent or lower. Results of probing indicate that backfill exists to elevation +5.0 ft underlain by native soil in this area. All the loads are transferred directly to the anchor block without having to rely on the upport of the backfill.
o Footing E	As shown in Figure B-1, the southern portion of the footing is founded on native soil at elevation +14.5 feet. In the northern portion the backfill with a relative compaction of 80 to 91 percent was excavated to elevation +12.0 ft except for an approximate 5 ft wide area at the north end. In that area the excavation was made to the turbine mat at elevation +8.5 feet. These overexcavated areas were backfilled with concrete. Therefore, the loads from the footing are transferred to the native soil and to the turbine mat without having to rely on the support of the backfill.
9. South Footing	
o Footing F	The southern portion of the footing is founded on native soils at elevation +14.5 feet. In the northern portion the footing is founded on backfill with a relative compaction of 81 to 87 percent at elevation +14.5 feet except for an approximately 5 feet wide area at the north end. In that area the excavation was made to the turbine mat at elevation +8.5 feet. These overexcavated areas were backfilled with concrete. Therefore, the loads from the footing are transferred to the native soil and to the turbine mat without having to rely on the support of backfill.

TABLE 4-2
CHANGES IN STIFFNESS PARAMETERS
BASED ON SOIL CHARACTERIZATION IN FIGURE 2-22

Structure	Ratio of Stiffness Parameter Using Figure 2-22 Characterization to that Calculated for 95 Percent Relative Compaction				
			Rocking		
	Vert. Trans.	Horiz. Trans.	About N-S	About E-W	Twisting
Fuel Storage Building*	0.80 to 0.82	0.49 to 0.54	0.57 to 0.62	0.52 to 0.57	0.46 to 0.52
Turbine Mat**	0.91 to 0.98	0.71 to 0.93	0.74 to 0.94	0.88 to 0.97	0.65 to 0.92
Control Building***	0.98 to 1.0	0.97 to 1.0	0.97 to 1.0	0.97 to 1.0	-

* Range of values represents the effect of relative compaction of backfill to be 85 percent.

** Range of values represents the effect of liquefaction of backfill, with relative compaction of 85 to 90 percent, below elevation +5 to no liquefaction of that fill.

*** Range of values represents the effect of backfill against the walls of footings having a relative compaction of 85 percent.

TABLE 4-3
SUMMARY OF STIFFNESS AND DAMPING VALUES
TURBINE BUILDING EXTENSION FOOTINGS
(BASED ON 95% BACKFILL COMPACTION)[1]

Foundation (previous designation)[2]	Stiffness				Damping % of Critical				Remark
	K_v x10^3 k/ft	K_h x10^3 k/ft	K_r N-S x10^4 k-ft/rad.	K_r E-W x10^4 k-ft/rad.	D_v	D_h	D_r N-S	D_r E-W	
North Extension ([1,2, 4,] A)	59.7* 52.2	158.6* 138.7	2803.6* 2450.0	1528.5* 1335.8	26.0* 26.0	16.0* 16.0	21.0* 21.0	6.0* 6.0	Line A & 6-8
North Extension ([2, 3, 4] B)	76.6* 69.7	211.5* 192.3	4523.9* 3869.4	1043.7* 949.0	19.0* 19.0	12.0* 12.0	11.0* 11.0	2.0* 2.0	Line B & B-8
Northwest - (C-9 North)	16.1	61.0	60.4	60.4	23.0	20.0	2.0	2.0	
Northwest - (C-9 South)	19.0	44.6	41.0	34.1	23.0	21.0			
Northwest - (E-11)	30.9	101.0	164.0	228.0	24.0	15.0	6.0	2.0	
West - Column	53.3	118.2	261.6	2314.0	32.0	19.0	4.0	16.0	
(C-13)	7.0	15.5							
(E-13)	22.7	50.3							
(F-13)	23.6	52.4							

TABLE 4-3 (continued)
SUMMARY OF STIFFNESS AND DAMPING VALUES
TURBINE BUILDING EXTENSION FOOTINGS
(BASED ON 95% BACKFILL COMPACTION)[1]

	Stiffness				Damping % of Critical				Remark
Foundation (previous designation)[2]	K_v $x10^3$ k/ft	K_h $x10^3$ k/ft	K_r N-S $x10^4$ k-ft/rad.	K_r E-W $x10^4$ k-ft/rad.	D_v	D_h	D_r N-S	D_r E-W	
West and Outrigger - Column	43.8	131.4	807.0	1310.0	34.0	21.0	11.0	13.0	
West - (G13)	21.7	65.1							
(H13)	12.9	38.7							
Outrigger -									
(G_{Br} 13)	4.6	13.80							
(J_{Br} 13)									
(H_{Br} 13)	4.6	13.80							
Southwest - Column	61.3	168.0	3450.0	1060.0	32.0	19.0	14.0	12.0	
J13	12.5	34.3							
K12	27.4	75.1							
K13	18.8	50.2							
K_{Br} 13	2.6	8.4							
M-9	10.2	9.1	38.0	40.2	31.0	19.0	1.0	23.0	
Northeast - E-3	32.3	11.7	319.0	139.0	23.0	14.0	7.0	1.0	
East - A	72.8	199.8	2493.4	4704.6	34.0	21.0	14.0	21.0	

TABLE 4-[illegible]ntinued)
SUMMARY OF STIFFNESS AND DAMPING VALUES
TURBINE BUILDING EXTENSION FOOTINGS
(BASED ON 95% BACKFILL COMPACTION)[1]

Foundation (previous designation)[2]	Stiffness K_v x10^3 k/ft	Stiffness K_h x10^3 k/ft	Stiffness K_r N-S x10^4 k-ft/rad.	Stiffness K_r E-W x10^4 k-ft/rad.	Damping % of Critical D_v	Damping % of Critical D_h	Damping % of Critical D_r N-S	Damping % of Critical D_r E-W	Remark
East - B	65.0	189.0	1820.0	2230.0	29.0	18.0	9.0	14.0	
Southeast - C	37.2	11.4	307.0	647.0	28.0	17.0	3.0	8.0	
South - E	45.3*	156.0*	363.0*	660.0*	26.0*	16.0*	1.0*	9.0*	
	42.8	147.5	343.0	623.0	26.0	16.0	1.0	9.0	
South - F	51.5*	177.0*	413.0*	750.0*	23.0*	14.0*	1.0*	8.0*	
	46.5	160.0	377.0	685.0	23.0	14.0	1.0	8.0	
South - D	70.0*	149.0	5360.0*	1310.0*	39.0*	24.0	23.0	7.0*	
	64.2	137.0	4910.0	1200.0	39.0	24.0	23.0	7.0	
North Extension of South Footing - D	26.6	67.0	73.4	375.0	34.0	21.0	4.0	16.0	Line N-P & 7
East Extension of South Footing - D	23.0	78.1	157.0	157.0	26.0	16.0	4.0	4.0	Line P & 4-5
L-9	10.8	31.8	15.0	45.7	31.0	19.0	1.0	23.0	

NOTES:

(1) The modified stiffness parameters for the soil-foundation system were obtained by multiplying these stiffness values by the reduction factors in Table 4-4.

(2) For footing locations, see Figure 4-1. The previous designations which appeared in Tables 4-3 and 4-4 of the August 12, 1982 report are given in parenthesis.

* Values are with crane loads.

TABLE 4-4
SUMMARY OF CORRECTION FACTORS
TURBINE BUILDING EXTENSION FOOTINGS[a]

Foundation (previous designation)[1]	K_v	K_h	K_r N-S	K_r E-W	Remark
North Extension ([1, 2, 4,] A)	0.95	0.86	0.89	0.87	Along Line A & 6-8
North Extension ([2, 3, 4] B)	0.95	0.87	0.89	0.88	Along Line B & 6-8
Northwest (C-9 North)	0.92	0.75	0.77	0.77	
Northwest - (C-9 South)	0.90	0.73	0.77	0.77	
Northwest - (E-11)	0.96	0.92	1.31 (0.93)[2]	1.53 (0.92)[2]	
West			2.47-3.89		
- Column					
C13*	0.73-0.76	0.45-0.53			
E13*	0.73-0.75	0.45-0.53			
F13*	0.86-1.00	0.53-0.93			
West and Outrigger			1.05-2.72		
- Column					
West - G13#	1.19-1.07	0.52-0.92			
H13#	0.00-1.76	0.00-1.51			
Outrigger -					
G_{Br} 13#*	6.51-5.89	2.29-5.23			
J_{Br} 13#	K culvert				
H_{Br} 13#*	0.00-5.89	0.00-5.23			

TABLE 4-4 (continued)
SUMMARY OF CORRECTION FACTORS
TURBINE BUILDING EXTENSION FOOTINGS[a]

Foundation (previous designation)[1]	K_v	K_h	K_r N-S	K_r E-W	Remark
Southwest - Column			1.11-1.27	0.45-0.50	
J13#	1.98-1.69	0.98-1.65			
K12+	0.74-0.77	0.55-0.73			
K13+	1.03-1.06	0.90-1.01			
K_{Br} 13+	3.84-3.99	3.39-3.81			
M-9	-	-	-	-	Attached to Turbine Generator Pedestal
Northeast - E-3	0.72-0.75	0.43-0.49	0.48-0.54	0.45-0.51	
East - A	0.90	0.70	0.73	0.77	
East - B	0.61-0.80	0.43-0.52	0.46-0.57	0.47-0.59	Supported on anchor block and native soil
Southeast - C	-	-	-	-	Supported on anchor block
South - E	0.56 (0.77)[3]	0.34 (0.50)[3]	0.60 (0.56)[3]	0.20 (0.54)[3]	Supported on T/G Pedestal and native soil.
South-F	0.53 (0.77)[3]	0.35 (0.50)[3]	0.55 (0.56)[3]	0.19 (0.54)[3]	Supported on T/G Pedestal and native soil.

TABLE 4-4 (continued)
SUMMARY OF CORRECTION FACTORS
TURBINE BUILDING EXTENSION FOOTINGS[(a)]

Foundation (previous designation)[1]	K_v	K_h	K_r N-S	K_r E-W	Remark
South - D	1.0 (0.83)[3]	1.0 (0.58)[3]	1.0 (0.70)[3]	1.0 (0.59)[3]	Partially supported on native soil
North Extension of South Footing - D	1.0 (0.83)[3]	1.0 (0.58)[3]	1.0 (0.70)[3]	1.0 (0.59)[3]	Along line N-P & 7. Supported on native soil
East Extension of South Footing - D	0.83 (0.83)[3]	0.58 (0.58)[3]	0.70 (0.70)[3]	0.59 (0.59)[3]	Along line P & 4-5 Estimated based on characterization; to be verified during excavation/ construction
L-9	-	-	-	-	Attached to Turbine Generator Pedestal

NOTES:

(1) For footing locations, see Figure 4-1. The previous designation which appeared in Tables 4-3 and 4-4 of the August 12, 1982 report is given in parentheses.

(2) Values in parenthesis were used in the present analysis of the turbine building; revised values, shown above, are the correct values. The difference between the two will have negligible effect on the overall conclusions for the turbine building.

(3) Values in parenthesis were used in the present analysis of the Turbine Building and represent an assumed backfill compaction of 85 percent.

* Range of stiffness values represents conditions of liquefaction and noliquefaction of soil between culvert.

\+ Range of stiffness values based on assuming that portions of the backfill against the footing varies in relative compaction from 90 to 95 percent.

\# Culvert support has been incorporated in evaluating stiffness values for these nodal points.

a Values greater than one reflect the effects of the structural modifications implemented.

TABLE 4-5
SUMMATION OF SOIL STIFFNESSES - WEST HEATER PLATFORM

	K_v(K/in)	K_h(K/in)	K_{rN-S}(k-in/rad)	K_{rE-W}(K-in/rad)
ORIGINAL	39,608	113,393	9.89×10^8	2.05×10^9
LOWER BOUND	42,959	101,849	1.11×10^9	1.75×10^9

TABLE 4-6
SUMMATION OF SOIL STIFFNESSES
COLUMN LINE 13 AND OUTRIGGER FOUNDATIONS

	K_v(K/in)	K_h(K/in)	K_{rN-S}(k-in/rad)	K_{rE-W}(K-in/rad)
ORIGINAL	13,200	34,361	3.99×10^8	3.37×10^8
LOWER BOUND	16,816	25,092	5.26×10^8	4.20×10^7

TABLE 4-7
FREQUENCY OF STRUCTURAL MODES
NORTH EXTENSION
(Considering Turbine Pedestal, Fuel Pool and North Extension Interaction)

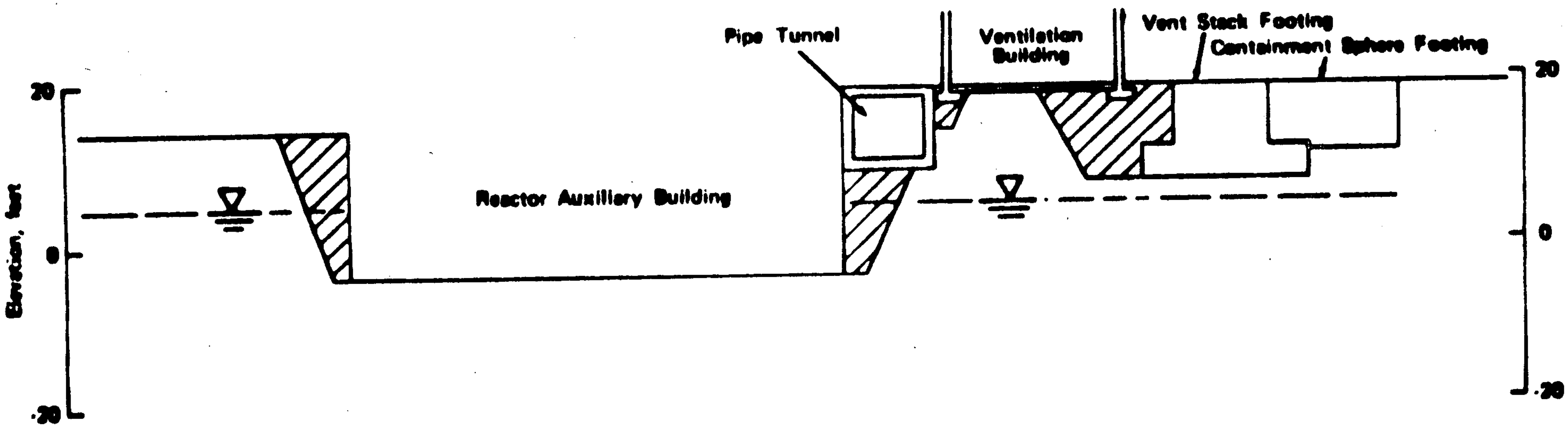
Mode Shape Description	With Orig. Soil Springs	With Low Bound Soil Springs	% Reduction in Freq.
Ped. E-W Rocking	3.29 CPS	2.91 CPS	12% Red.
Ped. Vert.	3.36 CPS	3.24 CPS	4% Red.
Ped. N-S Trans.	3.65 CPS	3.31 CPS	9% Red.
North Ext. F. Pool N-S Trans. Mode	4.80 CPS	3.84 CPS	20% Red.
Fuel Pool Vert. Trans. Mode	4.61 CPS	4.16 CPS	10% Red.
Fuel Pool E-W Rocking Mode	6.80 CPS	5.32 CPS	22% Red.

TABLE 4-8
VENTILATION EQUIPMENT BUILDING
SOIL STIFFNESS VALUES

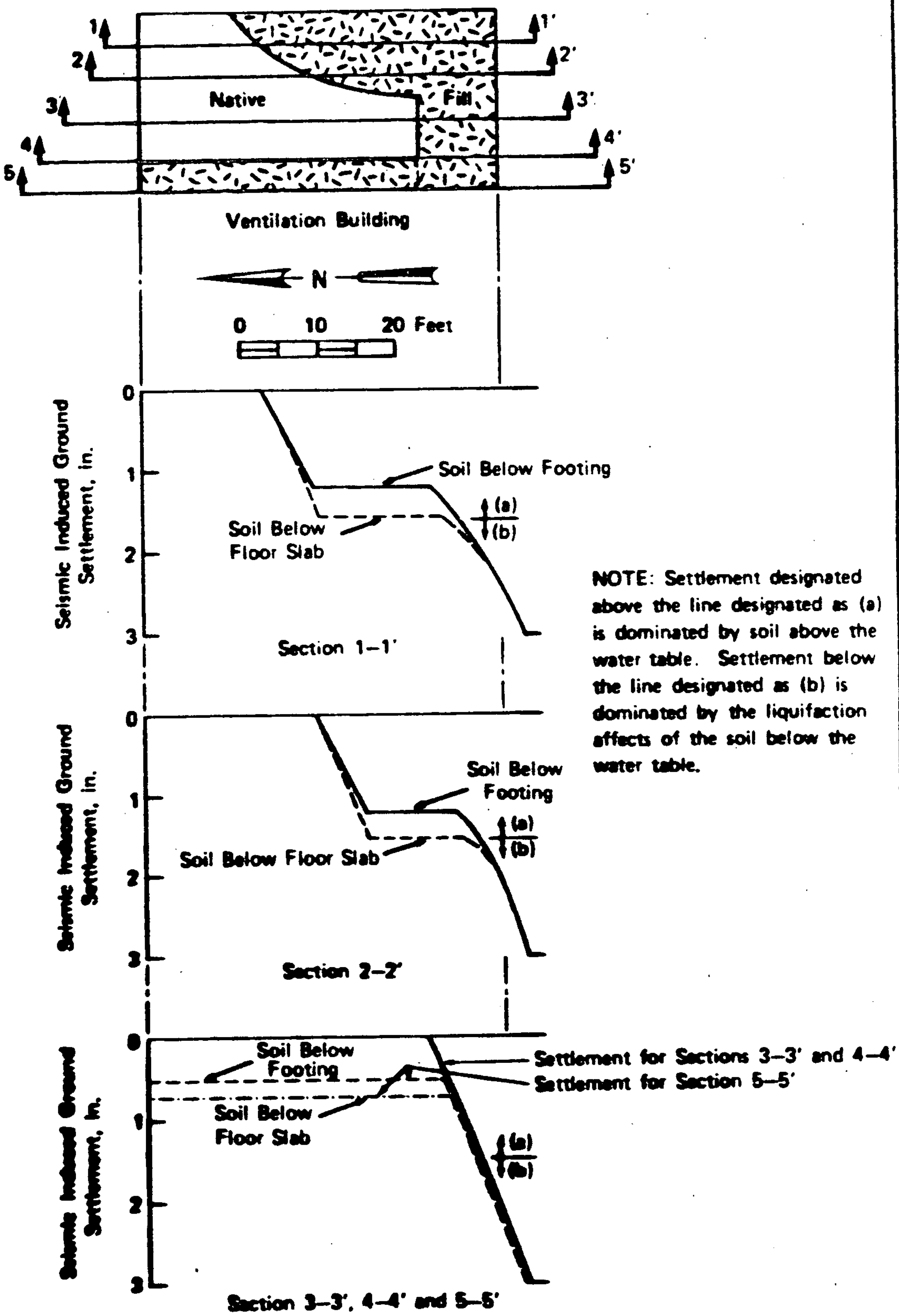
Soil Spring Orientation	Original Soil Springs	Modified Soil Springs[a] Upper Bound	Lower Bound
Vertical	10740 K/FT	7625 K/FT	6551 K/FT
Horizontal	11350 K/FT	7151 K/FT	6129 K/FT
Rotational	28.52×10^5 $\frac{\text{K-FT}}{\text{RAD}}$	18.54×10^5 $\frac{\text{K-FT}}{\text{RAD}}$	15.69×10^5 $\frac{\text{K-FT}}{\text{RAD}}$

NOTES:

(a) Upper bound values assume continuous contact; Lower bound values assume loss of contact at south wall footing and 10 feet of adjoining east and west walls.



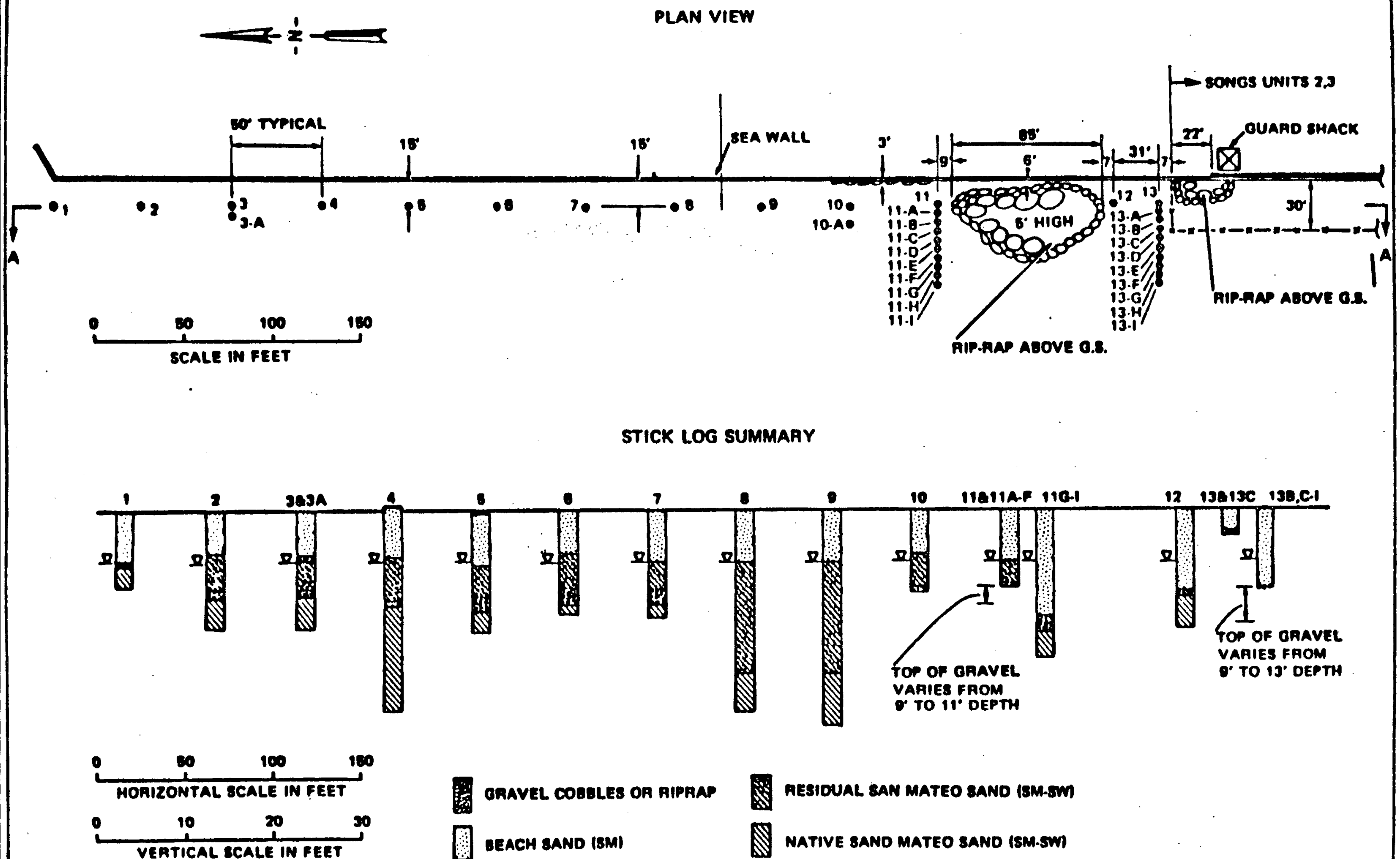
CROSS SECTION
BACKFILL CHARACTERIZATION
FIGURE 4-2a



NOTE: Settlement designated above the line designated as (a) is dominated by soil above the water table. Settlement below the line designated as (b) is dominated by the liquifaction affects of the soil below the water table.

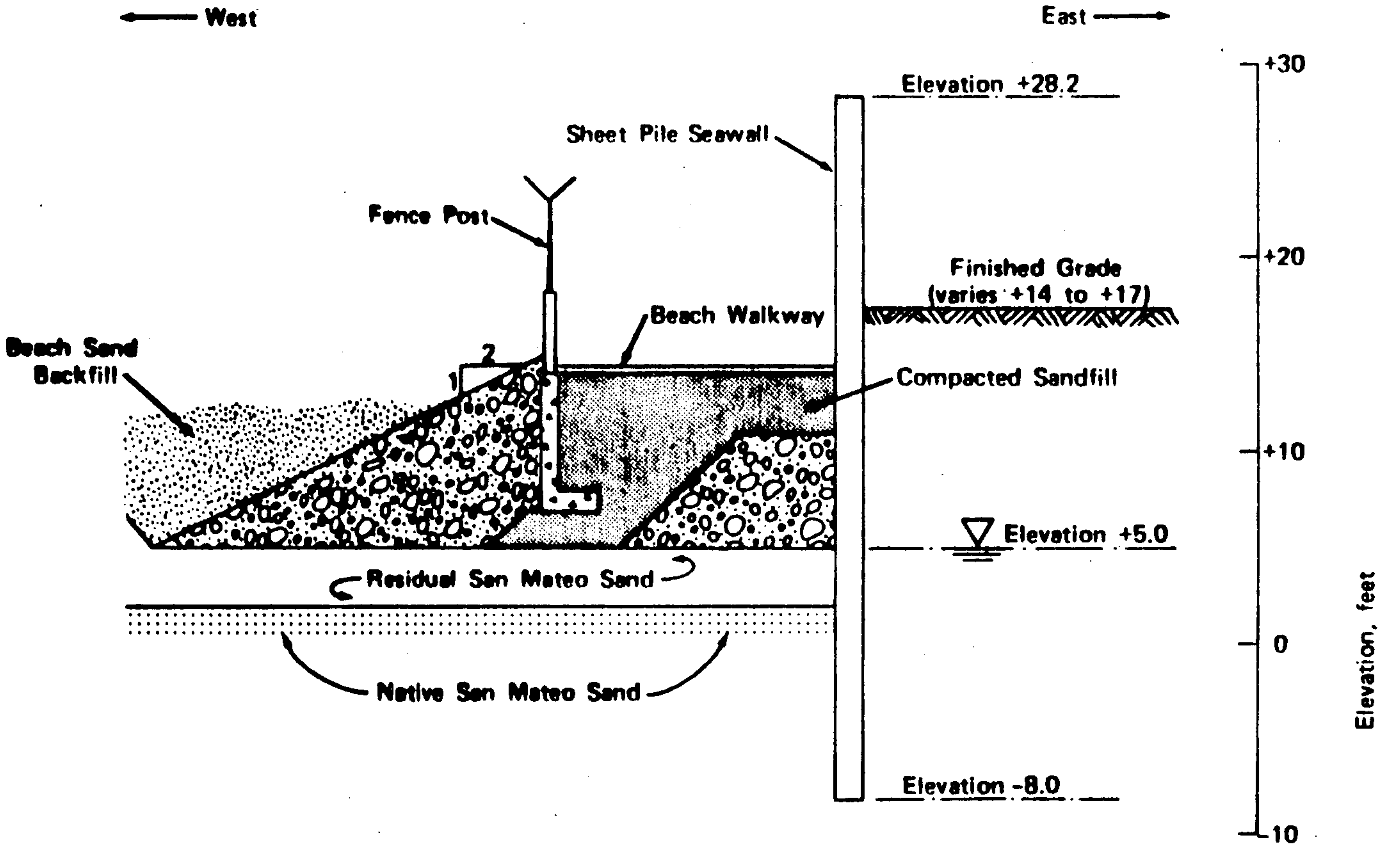
GROUND SETTLEMENT PROFILES
FOR VENTILATION BUILDING

FIGURE 4-2b



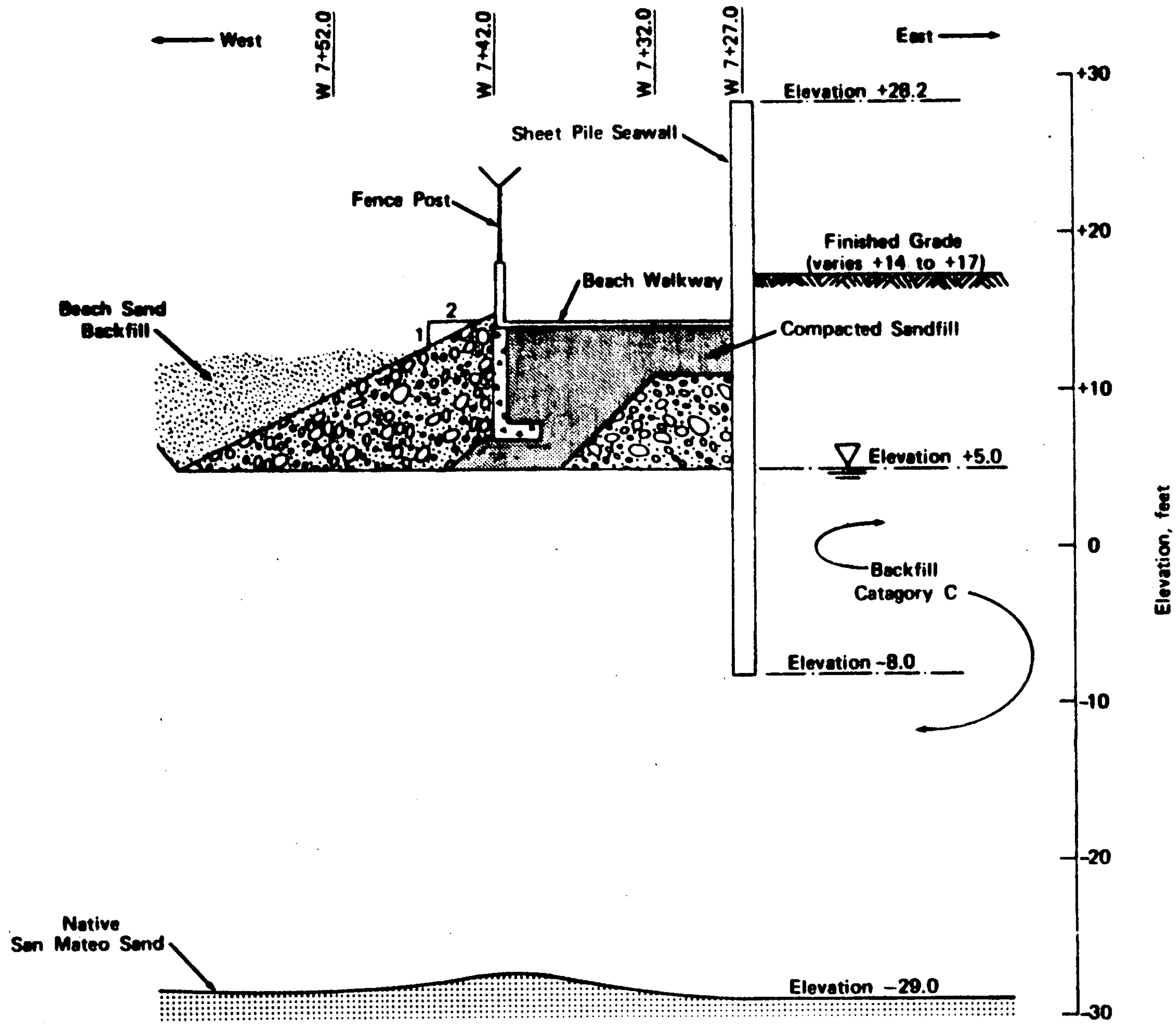
BORING LOCATION PLAN
AND STICK LOG SUMMARY

FIGURE 4-3



CROSS SECTION SHOWING TYPICAL SOIL CONDITIONS AT THE SEA WALL

FIGURE 4-4



CROSS SECTION SHOWING SOIL CONDITIONS USED IN THE EVALUATION OF THE SEAWALL

FIGURE 4-5

5.0 RECONCILIATION OF SOIL CONDITIONS FOR EQUIPMENT FOUNDATIONS AND STRUCTURAL COMPONENTS

5.1 General

The effects, during and after a DBE event, of the soil backfill behavior on safety related equipment and structural components have been evaluated. Summaries of these evaluations, and the results thereof, are presented in this section.

In order to determine which specific systems and components could be affected by the postulated in-situ backfill conditions a thorough investigation was instituted. A group was established consisting of the site soil consultants (Woodward-Clyde Consultants) and personnel from the various disciplines (nuclear, mechanical, electrical, civil/structural and plant design) who are familiar with the plant. The areas where backfill material exists were established and illustrated in Figure 2-22 (see revision to Figure 2-22 in Addendum 1). With the areas of interest established, all pertinent drawings were reviewed and several field walkdowns conducted in order to compile a complete list of safety related systems or components which are supported by, routed through or reside directly upon any of the backfill soils.

Twenty-seven equipment foundations and structural components that are located over soil backfills at the San Onofre Unit 1 site were identified for consideration. They are located as shown in Figure 5-1, which also provides the backfill characterization from Figure 2-22. The response of fill soils beneath these structures was evaluated in accordance with the procedures delineated in Section 3.

For these evaluations cross sections were drawn through individual foundations to depict fills underlying them and for estimating depths of fills above and below the water table. These cross sections are presented within this section in Figures 5-2 through 5-22. Estimates of settlements for fills above and below the water table were evaluated based on procedures suggested by Silver and Seed (1972) (Reference 4) and Lee and Albaisa (1974) (Reference 5), respectively. For the purposes of the present evaluation, the estimates were based on the backfill characterization at the location of the individual foundation shown in Figure 5-1.

The estimates for settlement of fills, both above and below the water table, were adjusted to account for factors such as variations in the depth of fill below the foundations and the proximity of adjacent boundaries which constrain the development of shear strains in the fill. These adjustments were made based on engineering judgement and the experience of Woodward-Clyde Consultants and were reviewed by a consulting review board consisting of Drs. I. M. Idriss, H. B. Seed, and R. L. McNeill. The estimated settlement responses of fill under equipment foundations are summarized in Table 5-1 and reflect the best estimates arrived at in this manner. The potential settlement responses of backfill soil beneath the various foundations were postulated on the basis of considerations which included: configuration of underlying fill, proximity of the water table to the foundation, size of foundation, and interfaces with the walls of adjacent structures. The postulated responses were characterized in terms of the estimated fill settlement and the possibility of foundation tilting. It is noted that based on actual observations of settlements made in the field and on the results of mechanistic analyses of pore pressure induced settlements, (Seed, Martin, and Lysmer; 1975, Reference 6) all settlement of soil below the water table would occur after the seismic shaking had ceased and therefore are characterized as "post-seismic settlements." Settlements of soil above the water table would be expected to occur during seismic shaking.

The following sections describe the geotechnical conditions that affect each item in terms of the postulated backfill conditions. These sections also present the structural evaluation of each item, and conceptual modifications, if required, to restore design margins consistent with the seismic reevaluation criteria.

In the evaluation of components the following two loading combinations were considered:

I) Dead load + component seismic inertia loads + settlement during seismic shaking.

II) Dead load + total settlement.

Combination I addresses the load condition during the design basis earthquake and combination II addresses the load condition after the design basis earthquake. In most cases the components were evaluated assuming total loss of support in the backfill areas, so that, loading combination I usually governs. In cases where the assumption was made that there will be no support due to the settlement of the soil during and after the earthquake the magnitude of the expected settlements are of no consequence.

For cases where the components do not satisfy the acceptance criteria, modifications are identified to restore their design margins. The modifications will generally consist of (1) additional foundation supports which will span the backfill areas or (2) relocating the components so that they are supported on native soil.

5.2 Auxiliary Feedwater Pumps (Items 1 & 2)

5.2.1 Present Condition

As indicated in Figure 5-2, the No. 1 Auxiliary Feedwater Pump is founded on about 16 feet of category D fill of which 9 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, it is estimated that the potential seismically induced settlement of the backfill soil beneath this foundation is in the range of 2 to 3 inches. Up to one inch of this settlement could occur during seismic shaking.

Figure 5-3 shows that the north side of the No. 2 Auxiliary Feedwater Pump is founded on about 16 feet of category D fill of which 9 feet is below the water table. The south side of the pump is located directly above the west anchor block on about 3 feet of fill. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, it is estimated that the potential seismically induced settlement of the backfill soil beneath this foundation could be about 1-1/2 inches with a tendency for the foundation to tilt toward the foundation of Auxiliary Feedwater Pump No. 1. Up to one inch of this settlement may occur during seismic shaking.

5.2.2 <u>Structural Evaluation</u>

The piping and the related supports connected to the pumps were evaluated for differential settlements. It was found that support modifications would be needed in order for the piping and supports to meet the seismic reevaluation criteria.

Conceptual modifications have been identified which, when installed, will preclude the estimated settlements and thus restore design margins consistent with the seismic reevaluation criteria. The conceptual modifications are shown in Figures 5-3A and 5-3B. Two grade beams will be constructed to support both pump footings by spanning the backfill area. The grade beams will be supported by a pier that is founded on native soil on the northern end and by a pier that rests on the anchor block at the southern end. Since this structural system is to be entirely supported on native soil, seismically induced settlement of the Auxiliary Feedwater Pumps will be precluded.

5.3 <u>East-West Electrical Duct Bank, East Of The Intake Structure</u> (Item #3)

5.3.1 <u>Present Condition</u>

The east-west duct bank No. 3 consists of two duct banks running parallel to each other. As indicated in Figure 5-4 the east end of the duct bank is founded on 11 feet of category B fill of which 8 feet is below the water table. The fill depth increases to a maximum of 38 feet of category C fill at the west end near the intake structure with 35 feet of the fill being below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath this structure is estimated to be about 3 to 5 inches with the maximum settlement occuring near the intake structure. Less than 1/2" of this settlement would occur during seismic shaking.

5.3.2 <u>Structural Evaluation</u>

Both north and south duct banks run from the west extension's foundation, for column line #13, west to the east wall of the intake structure. In addition these two duct banks are supported near their midspan by the new west heater platform outrigger footing, and the concrete backfill associated with its construction (see Section 4.3, Figure 4-1 and Figure 5-4).

These two duct banks were analyzed to determine their adequacy to resist soil overburden, dead load and seismic loads (Case I), assuming total loss of support from soil in the backfill areas. The maximum moments that would be developed in the north and south banks under this loading are 32 k-ft and 79 k-ft, respectively. The corresponding ultimate moment capacities, considering the conduits as reinforcement, are 57 k-ft and 738 k-ft respectively.

Since total loss of support was assumed in the backfill areas in this evaluation, load case I is the governing loading condition. Therefore, it is concluded that both duct banks meet the seismic reevaluation criteria.

5.4 Instrument Air Compressors (Item #4)

5.4.1 Present Condition

As indicated in Figure 5-5 the northern 75 percent of the foundation of the Instrument Air Compressors is located over the anchor block on about 2 feet of category B fill. The southern 25 percent of the foundation is founded on category D fill which extends a maximum of about 15 feet below the foundation. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the estimated potential seismically induced settlement of the backfill soil beneath this foundation is negligible (< 1/4").

5.4.2 Structural Evaluation

The seismically induced settlement of this foundation was determined to be negligible. Therefore, any stresses induced by these settlements would not be significant. In addition, the Instrument Air system is no longer part of the seismic reevaluation program because safety related valves will be provided with a seismically qualified nitrogen supply. Therefore, further analysis is not necessary.

5.5 Instrument Air Receivers (Item #5)

5.5.1 Present Condition

Figure 5-6 indicates that the northeastern half of foundation 5 is located on 9 feet of category D fill, 1 foot of which is below the water table. The southwestern half of the footing rests on category A fill of variable depth. The depth of this fill ranges from 9 to 25 feet with as much as 17 feet being below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath this foundation is negligible (< 1/4").

5.5.2 Structural Evaluation

As with the Instrument Air Compressors, the settlement here was determined to be negligible. In addition, the Instrument Air System is no longer part of the seismic reevaluation program. Therefore, further analysis is not necessary.

5.6 Electrical Duct Bank To North Tsunami Gate (Item #6)

5.6.1 Present Condition

As indicated in Figure 5-7, the northeast end of the duct bank is founded on native soil with the southwest end founded on up to 41 feet of category C fill of which about 34 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the estimated potential seismically induced settlement of the backfill soil beneath this component is about 3 to 5 inches along the southwest portion of the component with little or no settlement expected along the northeast portion of the component. Only 1/2 inch of the 3 to 5 inches is predicted to occur during seismic shaking.

5.6.2 Structural Evaluation

The electrical duct bank to the north tsunami gate is no longer part of the seismic reevaluation program because flooding of the site through the intake and discharge culverts is not considered to be a problem. Therefore, further analysis is not necessary.

5.7 Motor Control Center #3 (Item #7)

5.7.1 Present Condition

As indicated on Figure 5-8 the southern portion of the 8" slab foundation that supports the Motor Control Center #3 is founded on native soil. The northern end is founded on up to 16 feet of category D fill of which about 1 foot is below the water table. Based on a settlement analysis, performed in accordance with procedures documented in Section 3, the estimated potential seismically induced settlement for the backfill soil beneath the structure is about 1-1/2 inch. It is estimated that all of this settlement would occur during seismic shaking.

5.7.2 Structural Evaluation

The structural evaluation of the slab which supports the Motor Control Center #3 will be provided by September 9, 1983.

5.8 Electrical Conduit Duct - North Turbine Extension (Item #8)

5.8.1 Present Condition

As indicated in Figure 5-9 the north end of the conduit duct is founded on native soil and the central one-third of this structure is founded on up to 5 feet of category D fill above the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the estimated potential seismically induced settlement of the backfill soil beneath this structure is about 1/2 inch. It is estimated that all of this settlement would occur during seismic shaking.

5.8.2 Structural Evaluation

The duct bank was assumed to be unsupported for a span of 20 feet through the backfill. The maximum moment due to soil overburden, dead weight and seismic excitation (load case I) was calculated to be 87 k-ft. The ultimate moment capacity of the duct bank was computed to be 472 k-ft. Since total loss of support was assumed for the duct bank during seismic shaking, case I is the governing loading case. Therefore, it is concluded that the duct bank meets the seismic reevaluation criteria.

5.9 Turbine Plant Coolers (Item #9)

5.9.1 Present Condition

As indicated in Figure 5-10 the northwest one-third of the foundation for the Turbine Plant Coolers is founded on native soil with the remaining two-thirds founded on up to 41 feet of category B and C fill of which 34 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath this foundation is estimated to be 3 to 5 inches for the southeast corner with little or no settlement expected for the northwest portion of the structure. It is estimated that up to 1 inch of this settlement would occur during seismic shaking.

5.9.2 Structural Evaluation

This equipment itself is not safety related. However, due to its proximity to the Salt Water Cooling Pumps the effect of tilting of the Turbine Plant Coolers was investigated. It was found that tilting of less than 3° could occur. This is not severe enough to cause overturning of the equipment and therefore it will not adversely affect the Salt Water Cooling Pumps.

5.10 Intake Culverts (Item #10)

5.10.1 Present Condition

As indicated in Figure 5-11 the eastern two-thirds of the Intake Culverts are founded on native soil with the western one-third being founded on up to 24 feet of category C fill, all of which is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath the west end of the structure is estimated to be 3 to 5 inches. All of this settlement would occur after seismic shaking had ceased. There would be no settlement along the portion of the structure supported on native soil. Because the soil is entirely submerged it may be subject to uplift pressures that can be estimated from Figure 3-3 as a proportion of the effective overburden pressure (uplift pressure equals r_u x the effective overburden pressure).

5.10.2 Structural Evaluation

The Intake Culverts are not safety related, however they do provide support for several columns and structural braces for the west extension of the turbine building which is safety related. Therefore, the culverts between the circulating water system pumpwell and the turbine condensers were evaluated.

The Intake Culverts were assumed to be unsupported for a span of 15' through the backfill. Since there is no backfill settlement postulated during seismic shaking, loading case I is not a governing case and only case II was analyzed. The loadings considered were the sum of: dead load, soil overburden, the turbine building west footing reaction, and the weight of water with and without buoyancy effects (load case II). The maximum moment for this case was calculated to be 689 k-ft. The culverts' moment capacity was determined to be 1990 k-ft. Therefore, it is concluded that the Intake Culverts meet the seismic reevaluation criteria.

5.11 Spent Fuel Pit Pump (Item #11)

5.11.1 Present Condition

As indicated in Figure 5-12 the foundation of the Spent Fuel Pit Pump is founded on up to 6 to 7 feet of category D fill above the water table with the fill becoming deeper to the north. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath this foundation is estimated to be about 1 inch, all of which would occur during seismic shaking. The foundation may tend to tilt toward the Reactor Auxiliary Building.

5.11.2 Structural Evaluation

The spent fuel pit cooling system is no longer part of the seismic reevaluation program because sufficient time is available to provide makeup to the spent fuel pool to cool the spent fuel. Therefore, further analysis of the spent fuel pit pump is not necessary.

5.12 Refueling Water Pump (Item #12)

5.12.1 Present Condition

As indicated in Figure 5-13, the foundation of the Refueling Water Pump is founded on up to 6 to 7 feet of category D fill above the water table with the fill becoming deeper to the north. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath this foundation is estimated to be about 1 inch, all of which would occur during seismic shaking. This foundation may tend to tilt toward the Reactor Auxiliary Building.

5.12.2 Structural Evaluation

The piping and the supports connected to the pump were evaluated for differential settlements of 3 inches which is greater than the estimated value of about 1 inch. It was found that support modifications would be needed so that the piping and supports would satisfy the seismic reevaluation criteria. However, relocation of the pump to an area where native soil exists would eliminate the settlement concern and thus restore the design margins consistent with the seismic reevaluation criteria. A conceptual relocation of the pump is shown in Figure 5-13A. Alternatively, reevaluation based on the lower settlement may preclude the need for this modification.

5.13 Pipe Tunnel (Item #13)

5.13.1 Present Condition

As indicated in Figure 5-14 the northeastern corner and eastern edge of the Pipe Tunnel are founded on native soil with the western portion being founded on up to 14 feet of category D fill of which 10 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath the west side of this structure is estimated to be about 2 inches with little or no settlement expected along its eastern edge. It is estimated that up to 1/2 inch of this settlement would occur during seismic shaking.

5.13.2 Structural Evaluation

The bottom slab of the Pipe Tunnel is doweled into the Reactor Auxiliary Building on its west side and supported by native soil on its east side. The slab was modelled as a unit strip running east-west. It was assumed that the slab will be unsupported over the backfill area. The analysis considered dead plus seismic loads (load case I) and the computed moment was 8.0 k-ft/ ft. The ultimate moment of the slab was calculated to be 30.8 k-ft/ft. Since total loss of support in the backfill region was assumed in this evaluation, load case I is the governing case and case II need not be analyzed. Therefore, it is concluded that the pipe tunnel meets the seismic reevaluation criteria.

5.14 480V Switchgear Room Slab (Item #14)

5.14.1 Present Condition

As indicated in Figure 5-15 the southern portion of the floor slab of the 480V switchgear room is founded on native soil with the northern portion founded on up to 17 feet of category D fill, of which 9 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath the northern portion of the slab is estimated to be about 2 to 3 inches with little or no settlement expected beneath the southern portion. It is estimated that up to 1 inch of this settlement would occur during seismic shaking.

5.14.2 Structural Evaluation

For this evaluation a finite element model of this slab was developed. The slab was assumed to be simply supported on all four sides and supported by the underlying soil along its southern edge for a width of 8 feet. Soil springs were used to model the soil structure interaction. The sum of all vertical soil springs used to represent the support condition of the slab in the southern 8 feet width were 41,000 and 17,000 k/ft for the static and dynamic analyses, respectively. The total damping associated with each one of the springs (sum of hysteretic and geometric damping) was determined to be 25 percent. In the response spectrum analysis the composite modal damping was limited to a maximum of 20 percent. The soil underlying the northern portion of the slab was assumed to settle away from the slab, thereby providing no soil support in the backfill areas. The weight/mass of the electrical equipment that is supported by the slab was included in the model. The location of this equipment is shown in Figure 5-15A.

This model was used to perform both static and response spectrum type dynamic analyses. The results from these analyses were superimposed to obtain the system's response to combined seismic and dead loading (load case I). For this load case the analysis indicates that the most severely stressed east-west cross section experiences an average moment of 2.74 k-ft/ft. In the most critical regions of this cross section, a 4 foot length located beneath switchgear #2 and #3, the maximum moment reaches 4.21 k-ft/ft. These moments exceed the slab's moment capacity of 1.17 k-ft/ft and correspond to ductility ratios of 3.3 and 7.0. Both exceed the seismic reevaluation criteria allowable of 3.0, consequently either more detailed analysis or modifications are required. Additional analysis could include nonlinear analysis and evaluation of the consequences of settlement of the slab. Modifications would include the addition of north-south running grade beams. These grade beams would span from the fuel pool wall at the north to native soil at the south. Their addition would increase the slab's bending stiffness, thereby decreasing the bending stresses and the deflections.

5.15 Miscellaneous Piping And Supports (Items 15, 16, 17, 18, 19, 20 & 21)

5.15.1 Present Condition

As indicated in Figure 5-16, both of the footings for item Nos. 15 and 16 are founded on up to 16 feet of category D fill of which 9 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath the footings is estimated to be about 2 to 3 inches. As indicated in Table 5-1, up to 1 inch of this settlement would occur during seismic shaking.

Figure 5-17 indicates that footings 17, 18 and 19 are founded on native soil. As such, no seismically induced settlement is expected beneath these footings, consequently they require no further consideration.

As indicated in Figure 5-17 footing 20 is founded on up to 18 feet of category B and D fill of which 10 feet is below the water table. Footing 21 is founded on up to 3 feet of category B fill. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath these footings is estimated to be 1 to 2-1/2 inches for footing 20 and 1/4 inch for footing 21. As indicated in Table 5-1, up to 1 inch of the footing 20 and all of the footing 21 settlements would occur during seismic shaking.

5.15.2 Structural Evaluation

Miscellaneous pipe supports, labeled as items 15 and 16, consist of a number of supports attached to the steel framing supporting the jet impingement barrier. The foundation for this barrier framing will be supported by the same grade beam assembly that will support Auxiliary Feedwater Pumps G10 and G10S. The method of supporting this foundation is shown in Figures 5-3A and 5-3B. Since the foundations of these pipe supports will be supported directly or indirectly on native soil, no differential movement due to seismically induced settlement is expected and no further analysis is required.

Modification of piping will be required for the relocation of the Refueling Water Pump. In doing so, new piping support frames will be designed such that they are supported either on native soil or off the south wall of the Reactor Auxiliary Building. Following installation of these new frames, the framing systems that are currently supported by footings 20 and 21 will no longer be used to support safety related piping. Alternatively, if the refueling water pump can be shown to be satisfactory (see section 5.12.2) further evaluation and/or modification of these pipe supports will be performed.

5.16 North-South Electrical Duct Bank, East Of Pumpwell (Item #22)

5.16.1 Present Condition

As indicated in Figure 5-18 the east segment of the duct bank is founded on the existing west extension footing with the remaining portions founded on up to 15 feet of category B and C fills, of which up to 10 feet is below the water table.

Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, it is estimated that the potential seismically induced settlement of the backfill soil beneath the duct bank will be about 2 inches. About 1/2 inch of this settlement would be expected to occur during seismic shaking. Negligible settlement is expected for portions of the duct bank supported by the west footing.

5.16.2 Structural Evaluation

The portion of the duct bank north of column line G has an unsupported length of 20'-6" spanning through the backfill. It was assumed that the duct bank will lose support in the area of backfill. The maximum moment due to soil overburden, dead weight and seismic excitation (load case I) was calculated to be 89 k-ft. The ultimate moment capacity, considering the conduits as reinforcement, was computed to be 170 k-ft. Load case I was determined to be the governing case and hence case II was not analyzed. Therefore, it was concluded that the duct bank meets the seismic reevaluation criteria.

The portion of the duct bank south of column line G has been cast integrally with the new west footing of the Turbine Building or is supported by the concrete backfill associated with that construction. Since the foundation modifications were designed to the seismic reevaluation criteria, this portion of the duct bank will also meet the criteria.

5.17 Refueling Water Storage Tank (Item #23)

5.17.1 Present Condition:

As indicated in Figures 5-1 and 5-19, 40 percent of the tank is founded on native soil with the remaining 60 percent founded on up to 8 feet of category B fill. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the soil is about 1 1/2 inch directly under the northern and western portions of the tank with negligible settlement expected under the eastern and southern portions. As indicated in Table 5-1, all of this settlement is expected to occur during seismic shaking.

5.17.2 Structural Evaluation

It is currently intended to either provide a new refueling water storage tank as a result of the seismic reevaluation program or to perform alternative seismic analyses of the tank. In either case, appropriate consideration will be given to the in-situ backfill soil conditions under the tank foundation.

5.18 Auxiliary Feedwater Piping Trench and Tank (Items #24 and 25)

5.18.1 Present Condition

As shown in Figure 5-1, the majority of the trench is founded on native soil with the exception of the northern portion in the intake culvert area. As seen in Figure 5-20 the trench is founded on up to 32 feet of category C fill, of which 26 feet is below the water table. Based on a settlement evaluation, performed in accordance with procedures documented in Section 3, the potential seismically induced settlement of the backfill soil beneath the piping trench is estimated to be about 3 to 5 inches. Only 1/2 inch of this settlement would occur during seismic shaking.

As shown on Figure 5-1, the Auxiliary Feedwater Tank is founded on native soil or backfill compacted to a minimum of 95 percent relative compaction. Therefore, settlement under this structure will be negligible.

5.18.2 Structural Evaluation

The Auxiliary Feedwater Piping Trench and the tank foundation were recently designed and constructed during the current outage. The structural design of the trench provides for spanning the entire width of the category C fill and therefore provides for adequate support of the piping in the trench. The Auxiliary Feedwater Tank is founded on native soil or backfill compacted to a minimum of 95 percent relative compaction, and is therefore expected to undergo a negligible amount of settlement during or after a seismic event. Therefore, the trench and tank foundation meet the seismic reevaluation criteria.

5.19 Salt Water Cooling Lines (Item #26)

5.19.1 Present Condition

The salt water cooling lines exit the north wall of the intake structure and span 22 feet of category B fill before entering native soil to the north (see Figure 5-21). It is estimated that the backfill soil beneath these lines could settle 3 to 5 inches with the maximum settlement occurring adjacent to the intake structure wall. As indicated in Table 5-1, all settlement would occur after the seismic shaking had stopped. Thus, the backfill may separate from the pipe and the pipe may have to carry, in addition to its own weight, the weight of the soil column above it and additional loads from the slab and turbine plant cooling water system components which are located at the surface.

5.19.2 Structural Evaluation

Approximately 22 feet of the salt water cooling lines span backfill from the intake structure to native soil. Analysis shows that this span experiences a maximum tensile stress of 13.8 ksi, which is greater than the allowable stress of 9.0 ksi. Therefore, new salt water cooling lines will be installed above grade.

5.20 Miscellaneous Items (Item #27)

5.20.1 Present Conditions

As indicated in Figure 5-22, the Refueling Water Filter and Refueling Water Filter Pump are located on up to 5 and 10 feet of category D fill, respectively. The potential seismically induced settlements of the Refueling Water Filter and the Refueling Water Filter Pump would be less than 1/2 inch. All of this settlement would occur during seismic shaking.

5.20.2 Structural Evaluation

These items form part of the pressure boundary for accident mitigation piping. As such, settlement of this equipment could affect connected safety related piping. Either the piping will be analyzed considering the potential settlements or the foundations of these items will be modified.

TABLE 5-1

ESTIMATED SETTLEMENT RESPONSE OF FILL UNDER EQUIPMENT FOUNDATIONS AND STRUCTURAL COMPONENTS

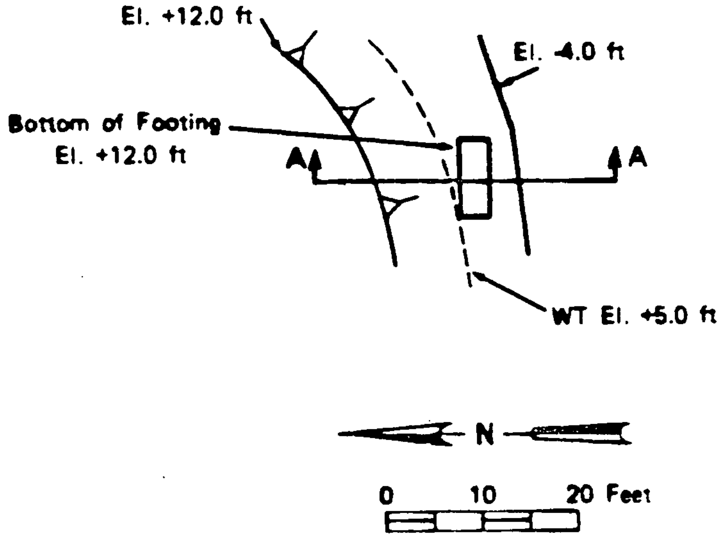
		Settlement (inches)		
Item Number	Description	During Seismic Shaking	Total	Notes
1	Aux. Feedwater Pumps	1	2-3	
2	Aux. Feedwater Pumps	1/2 - 1	1-1/2	Potential tilting toward north
3	E-W Duct Bank, East of Intake Structure	1/2	3-5	
4	Air Compressor	*	*	
5	Air Receivers	*	*	
6	Duct Bank to North Tsunami Gate	1/2	3-5	
7	Motor Control #3	1-1/2	1-1/2	
8	Conduit Duct Bank	1-1/2	1-1/2	Potential for tilting toward southwest
9	Turbine Coolers	1	3-5	
10	Intake Culverts	*	3-5	
11	Spent Fuel Pit Pump	1	1	Potential for tilting toward north
12	Refueling Water Pump	1	1	Potential for tilting toward north
13	Pipe Tunnel	1/2	2	
14	480V Switchgear Room	1	2-3	
15	Column Footing for Piping Supports	1	2-3	
16	Column Footing for Piping Supports	1	2-3	
17	Column Footing for Piping Supports	*	*	
18	Column Footing for Piping Supports	*	*	
19	Column Footing for Piping Supports	*	*	
20	Column Footing for Piping Supports	1	1 - 2-1/2	Potential for tilting north

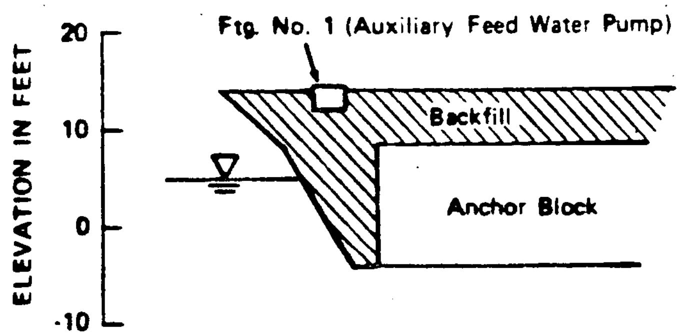
TABLE 5-1 (CONTINUED)

ESTIMATED SETTLEMENT RESPONSE OF FILL UNDER EQUIPMENT FOUNDATIONS AMD STRUCTURAL COMPONENTS

Item Number	Description	Settlement (inches) During Seismic Shaking	Total	Notes
21	Column Footing for Piping Support	1/4	1/4	
22	N-S Duct Bank, East of Intake Structure	1/2	2	
23	Refueling Water Storage Tank	1 1/2	1 1/2	
24	Aux. Feedwater Piping Trench	1/2	3-5	
25	Aux. Feedwater Tank	*	*	
26	Salt Water Cooling Line	*	3-5	
27	Refueling Water Filter Pump & Refueling Water Filter	1/2	1/2	Potential for tilting toward north.

*negligible < 1/4 inch

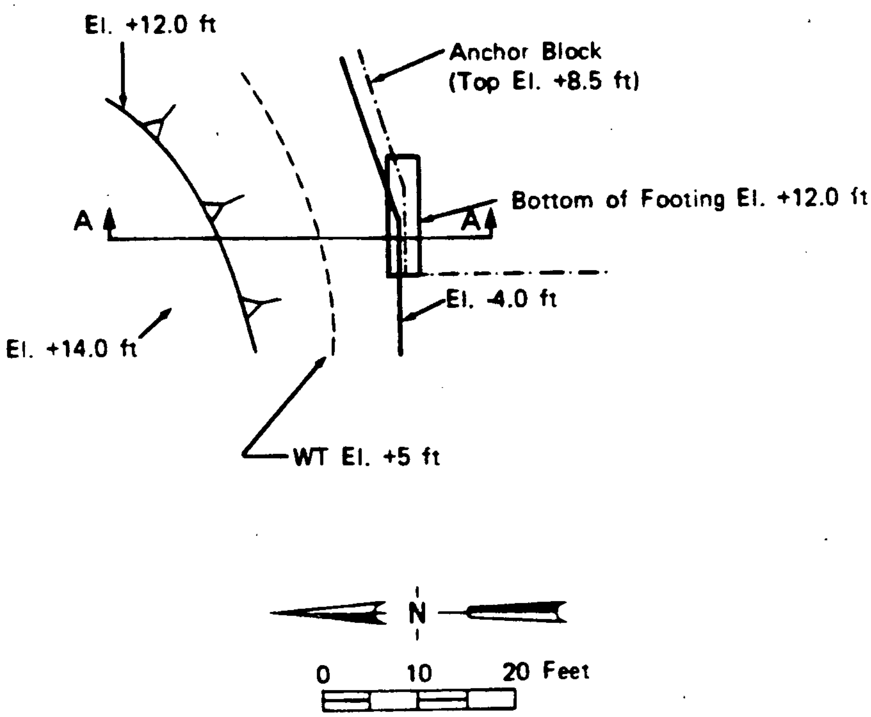


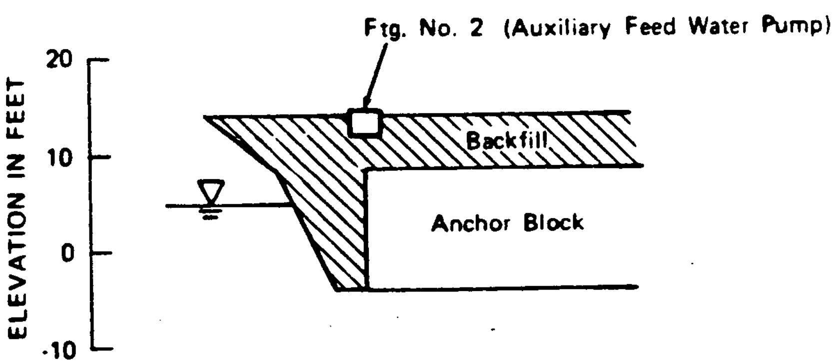


Section A–A

LOCAL SOIL CONDITIONS UNDER
FOOTING NO. 1

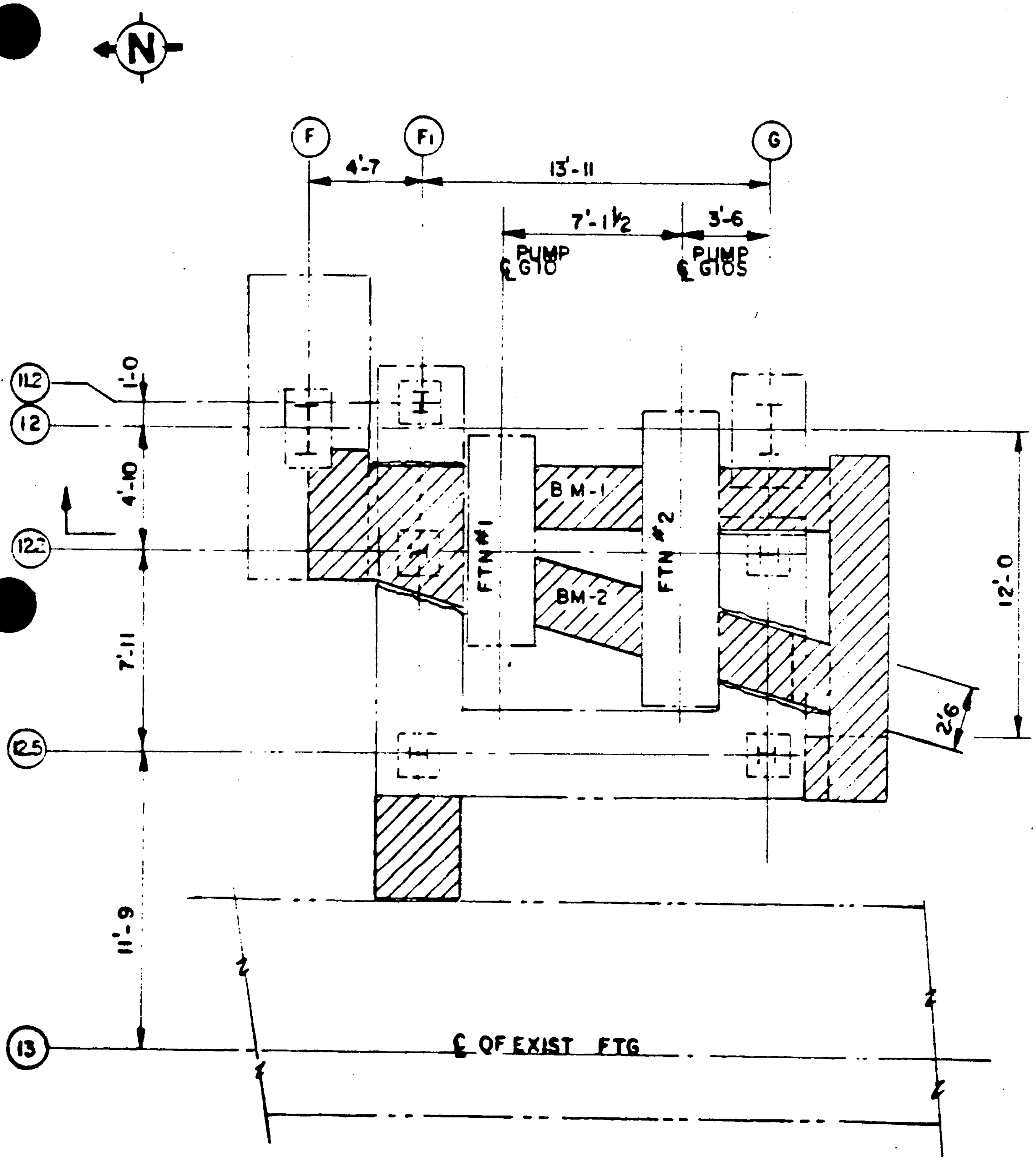
FIGURE 5-2





Section A—A

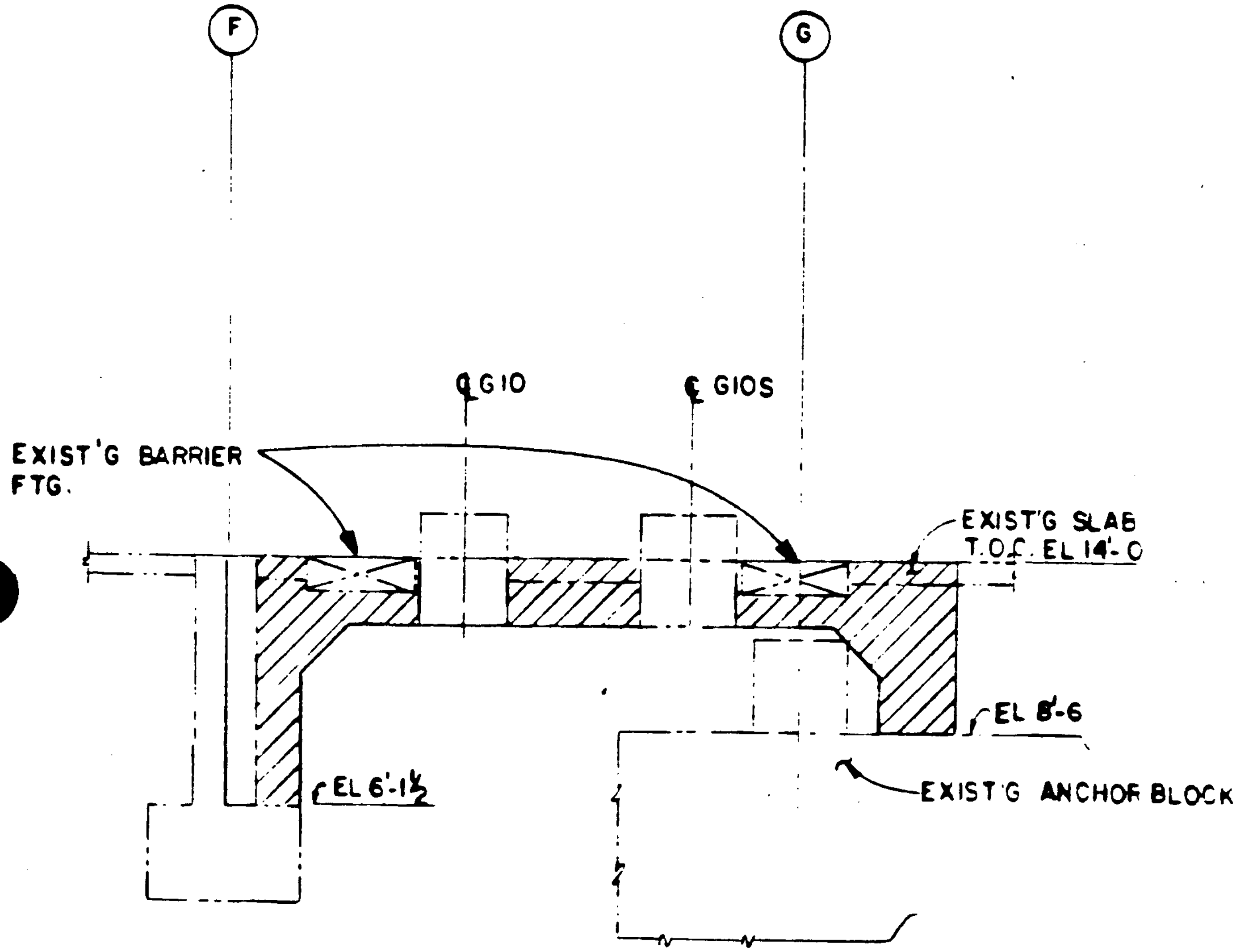
LOCAL SOIL CONDIITONS UNDER
FOOTING NO. 2
FIGURE 5-3



PLAN

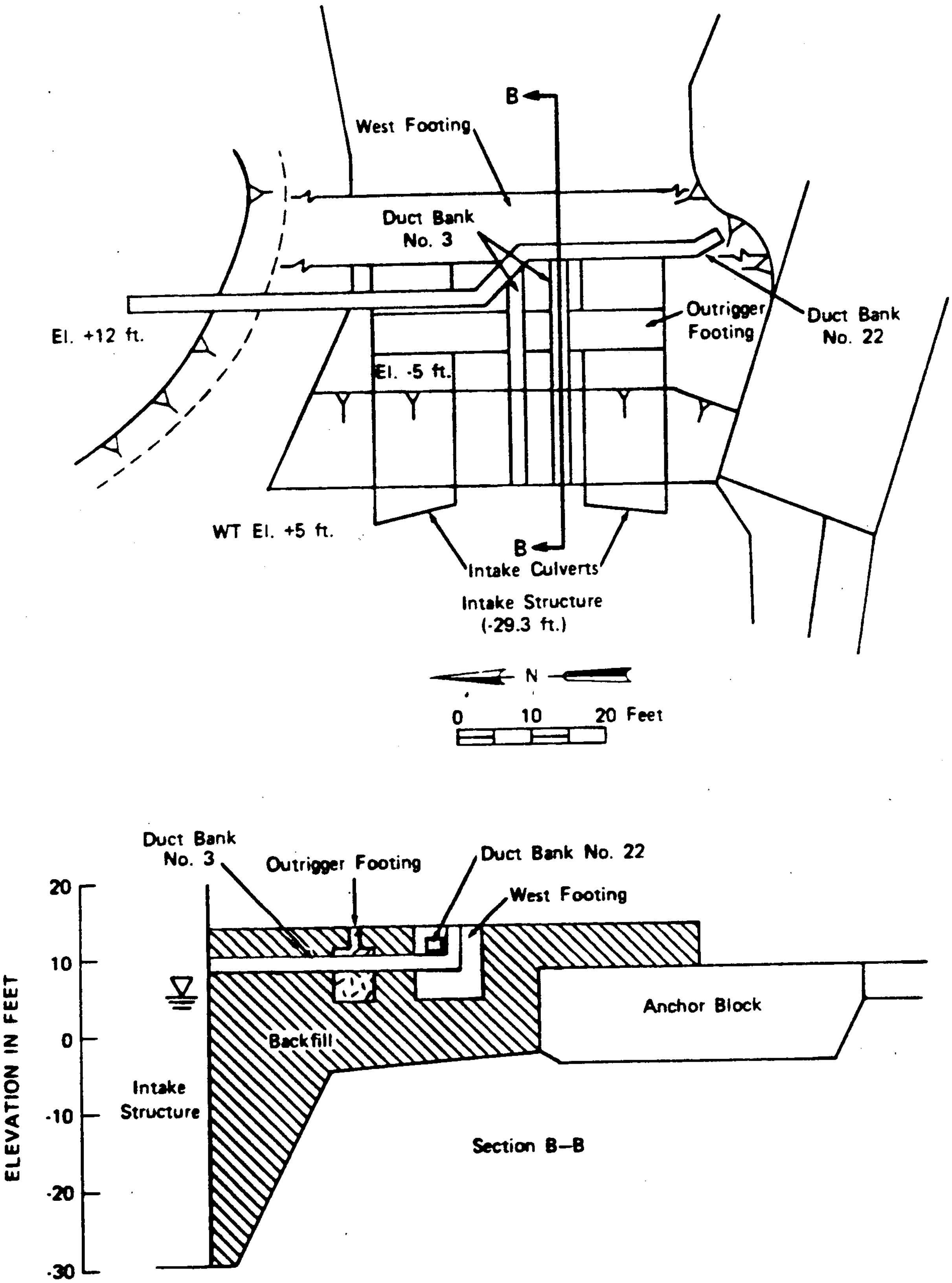
AUX FEEDWATER PUMPS: MODIFICATIONS

FIGURE 5-3A



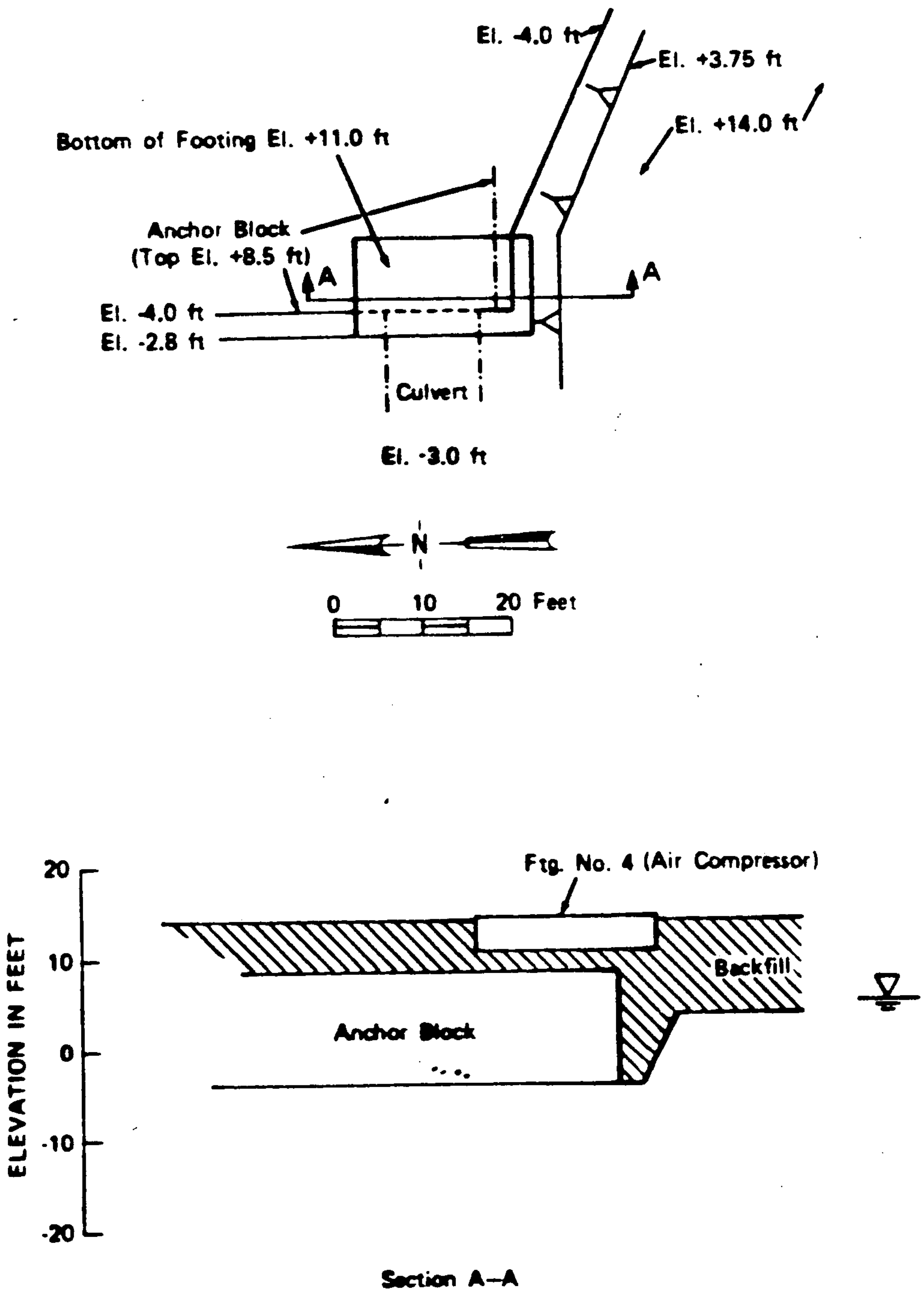
SECTION A 5-3A

FIGURE 5-3B



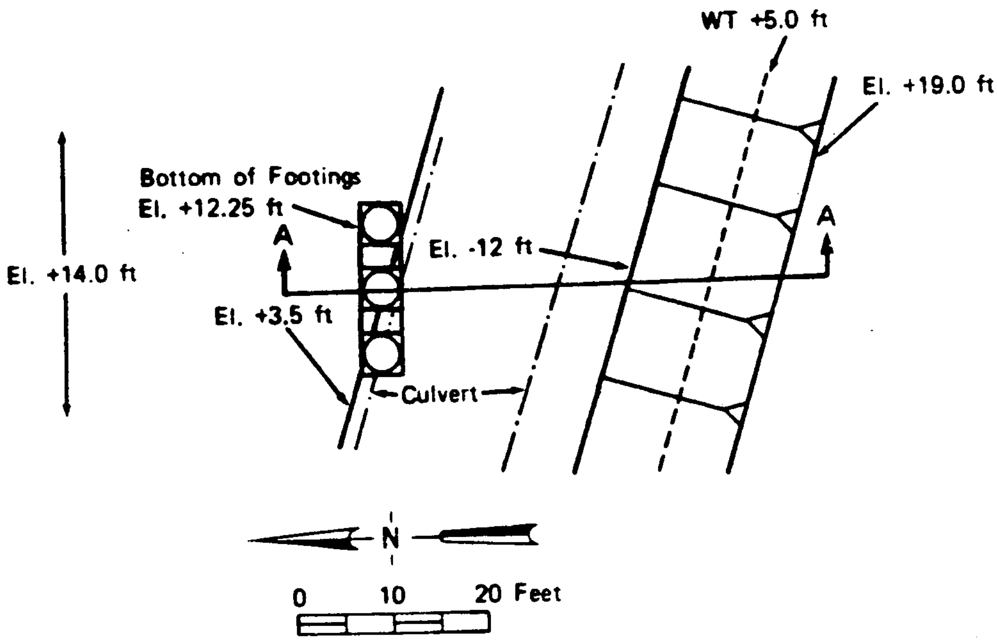
LOCAL SOIL CONDITIONS UNDER
DUCT BANK – NO. 3

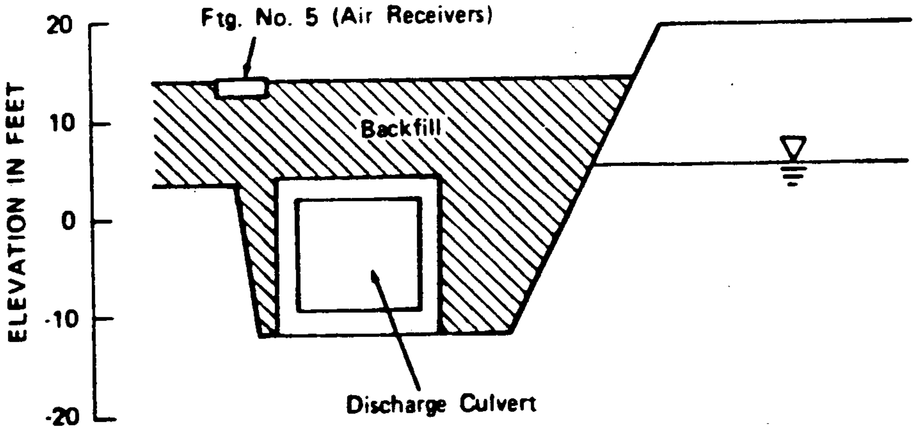
FIGURE 5-4



LOCAL SOIL CONDITIONS UNDER FOOTING NO. 4

FIGURE 5-5

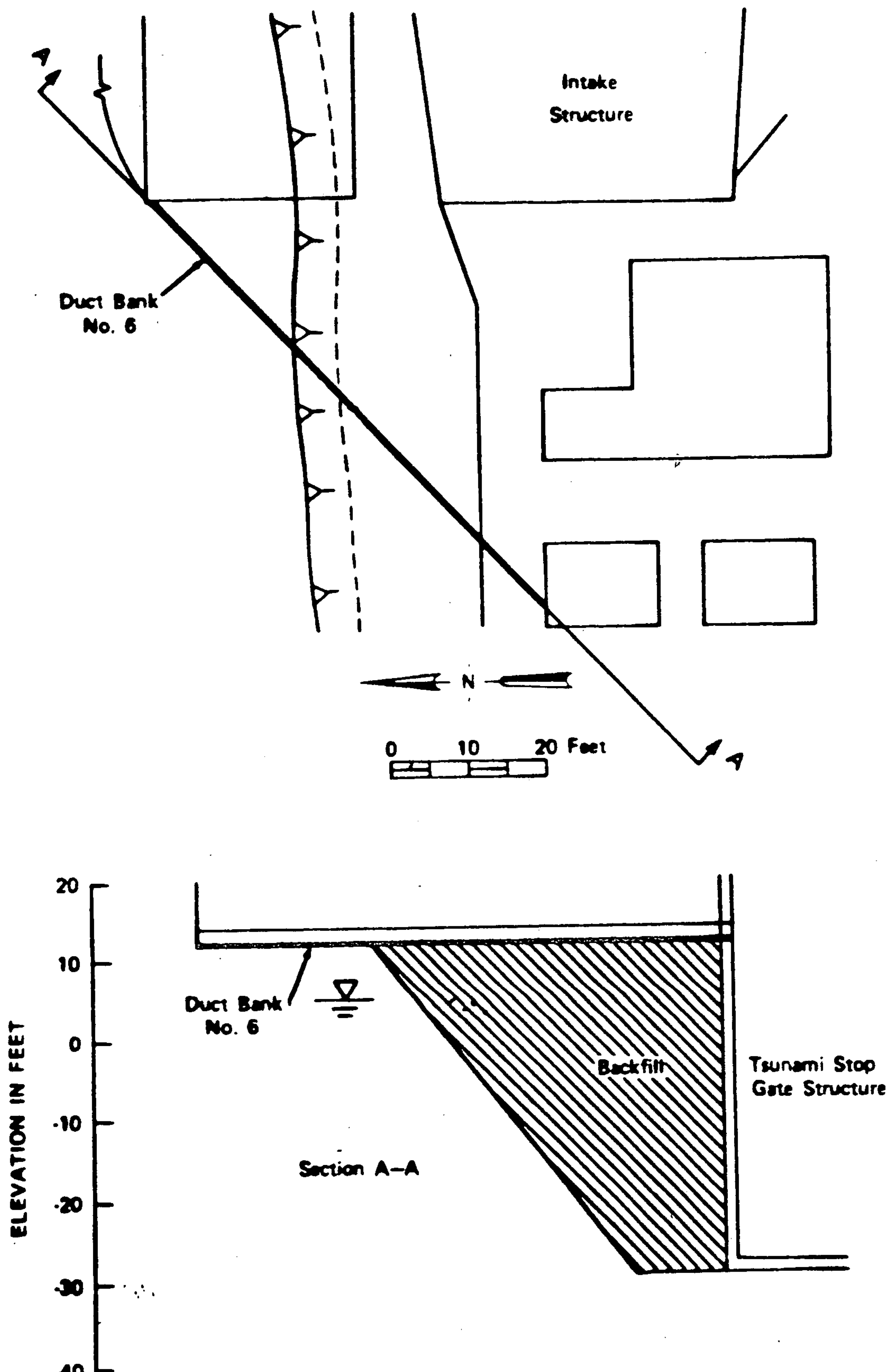




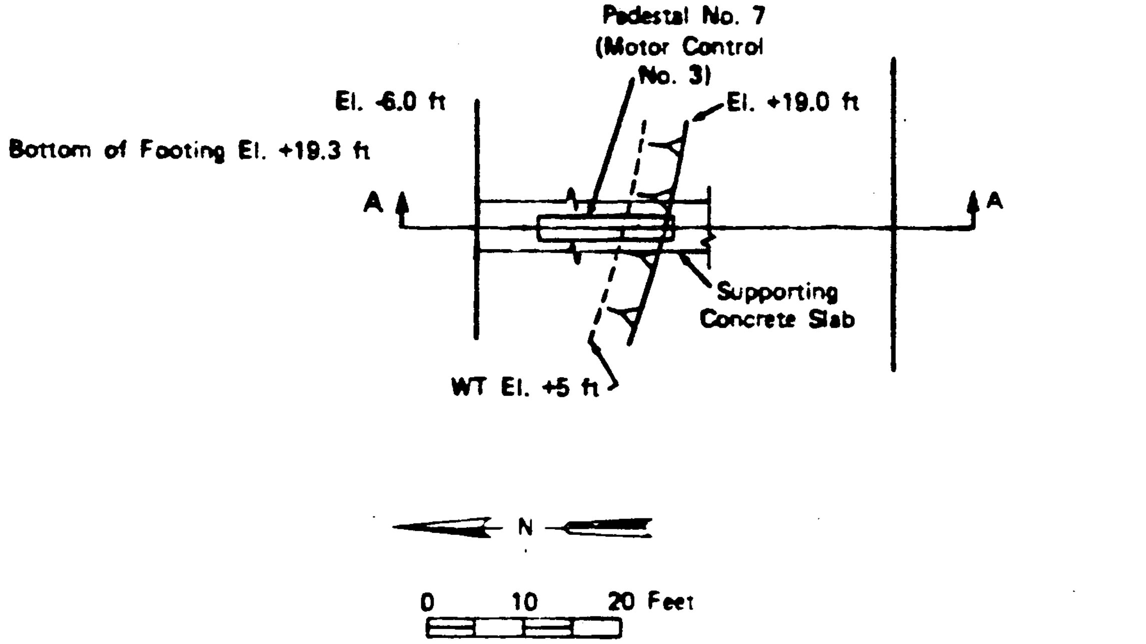
Section A–A

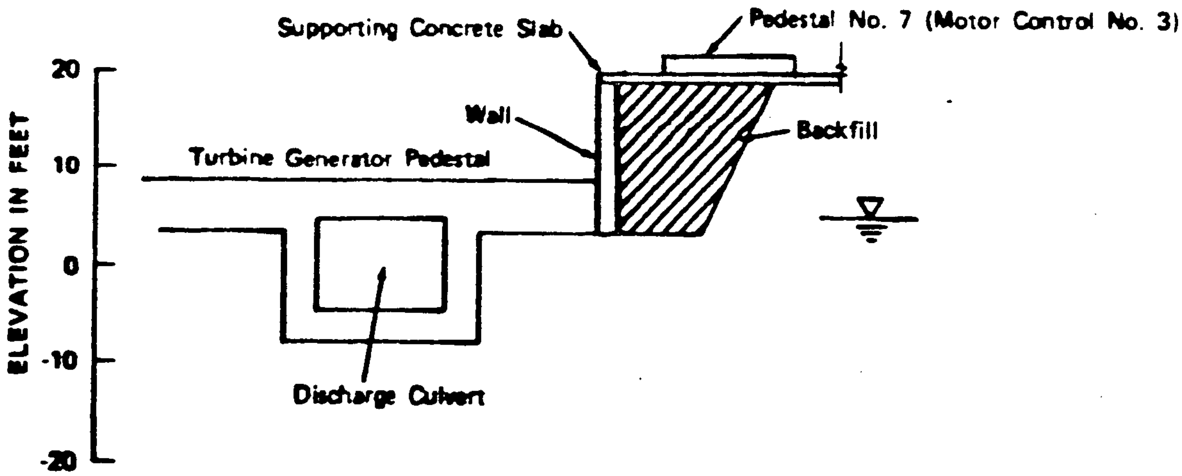
LOCAL SOIL CONDITIONS UNDER FOOTING NO. 5

FIGURE 5-6



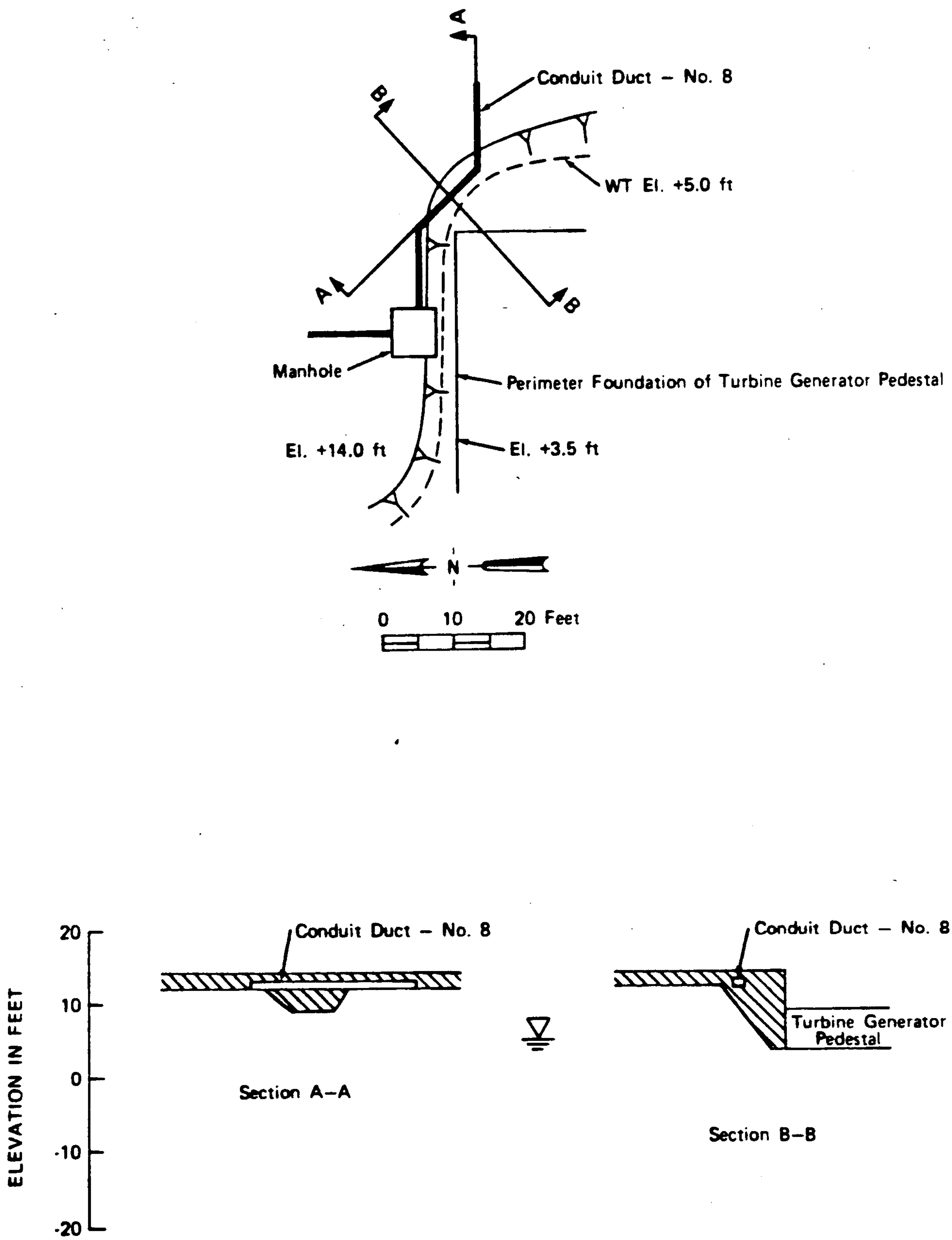
LOCAL SOIL CONDITIONS UNDER
DUCT BANK – NO. 6
FIGURE 5-7





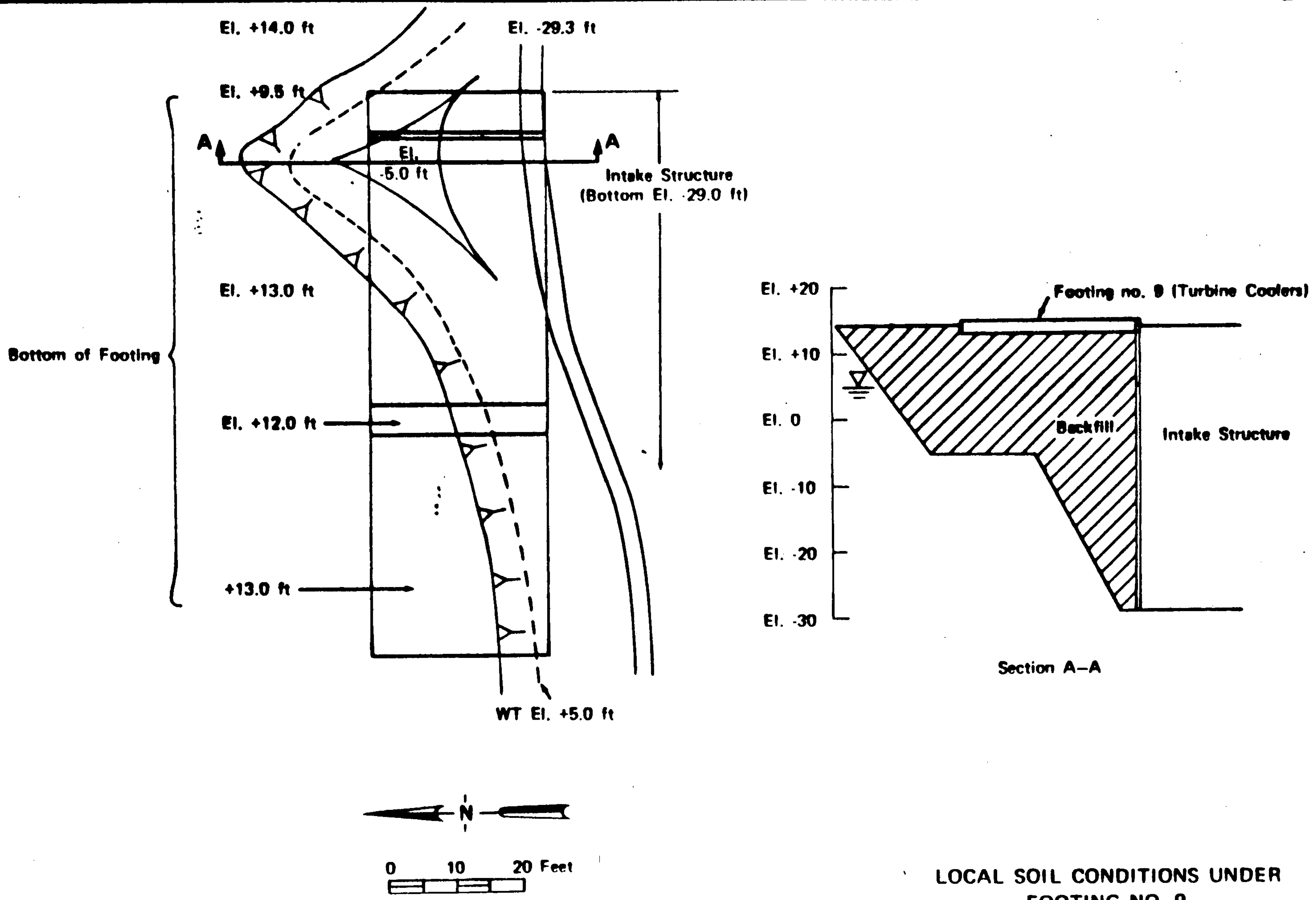
Section A–A

LOCAL SOIL CONDITIONS UNDER
FOOTING NO. 7
FIGURE 5-8



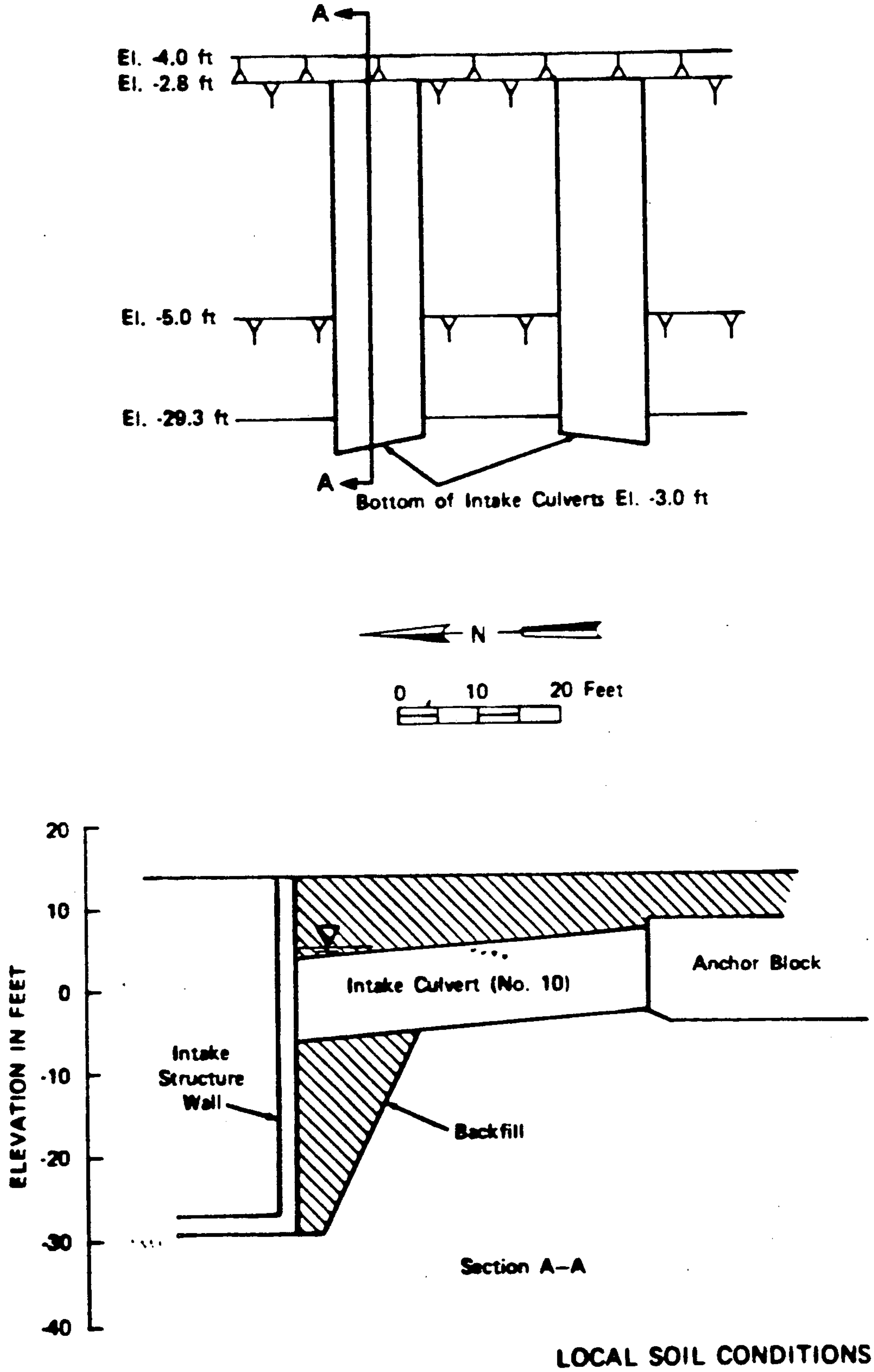
LOCAL SOIL CONDITIONS UNDER
CONDUIT DUCT – NO. 8

FIGURE 5-9



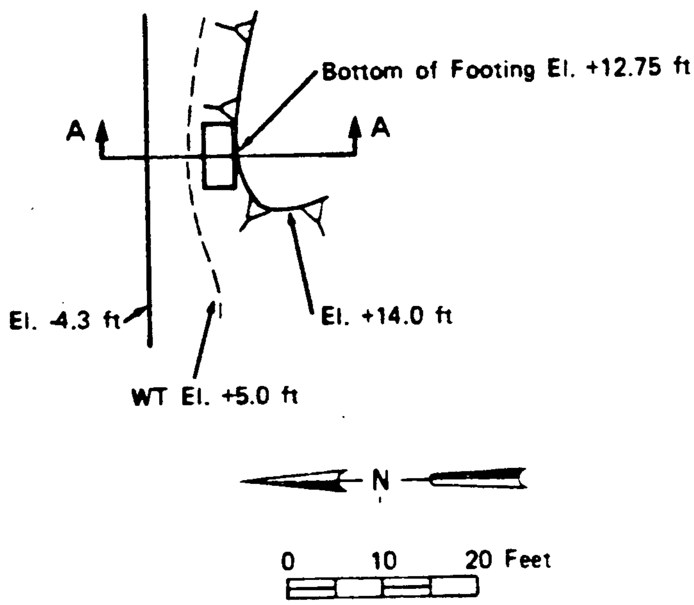
LOCAL SOIL CONDITIONS UNDER
FOOTING NO. 9

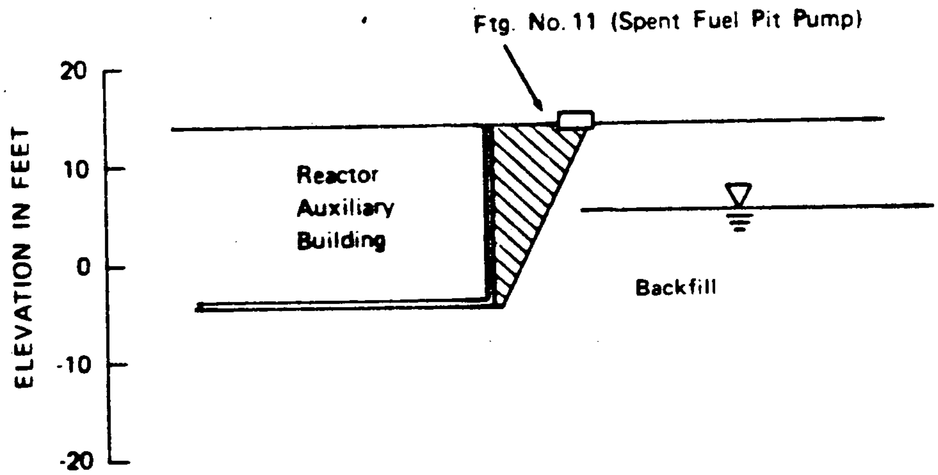
FIGURE 5-10



LOCAL SOIL CONDITIONS UNDER
INTAKE CULVERTS NO. 10

FIGURE 5-11

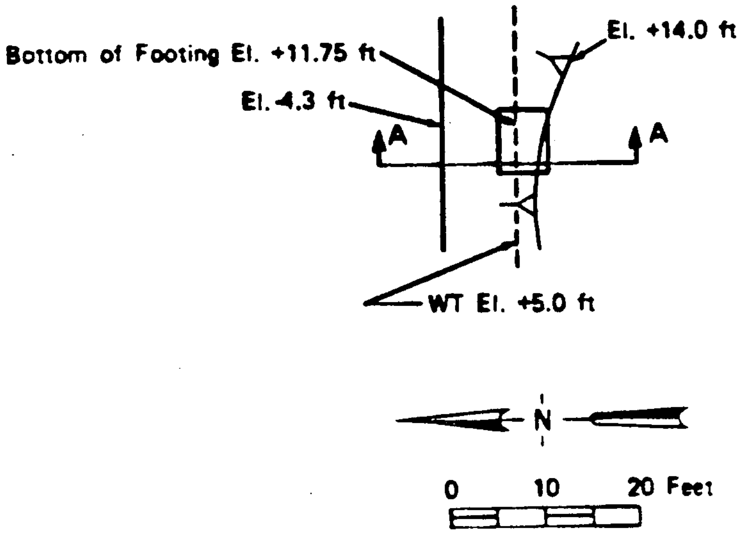


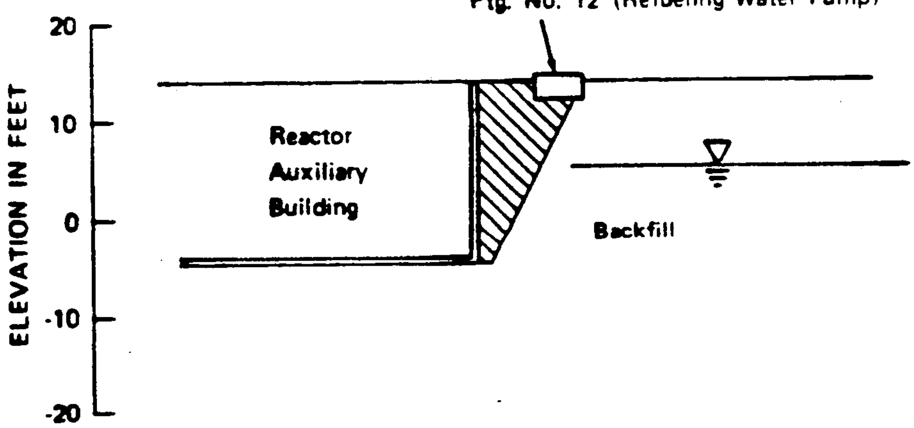


Section A–A

LOCAL SOIL CONDITIONS UNDER FOOTING NO. 11

FIGURE 5-12

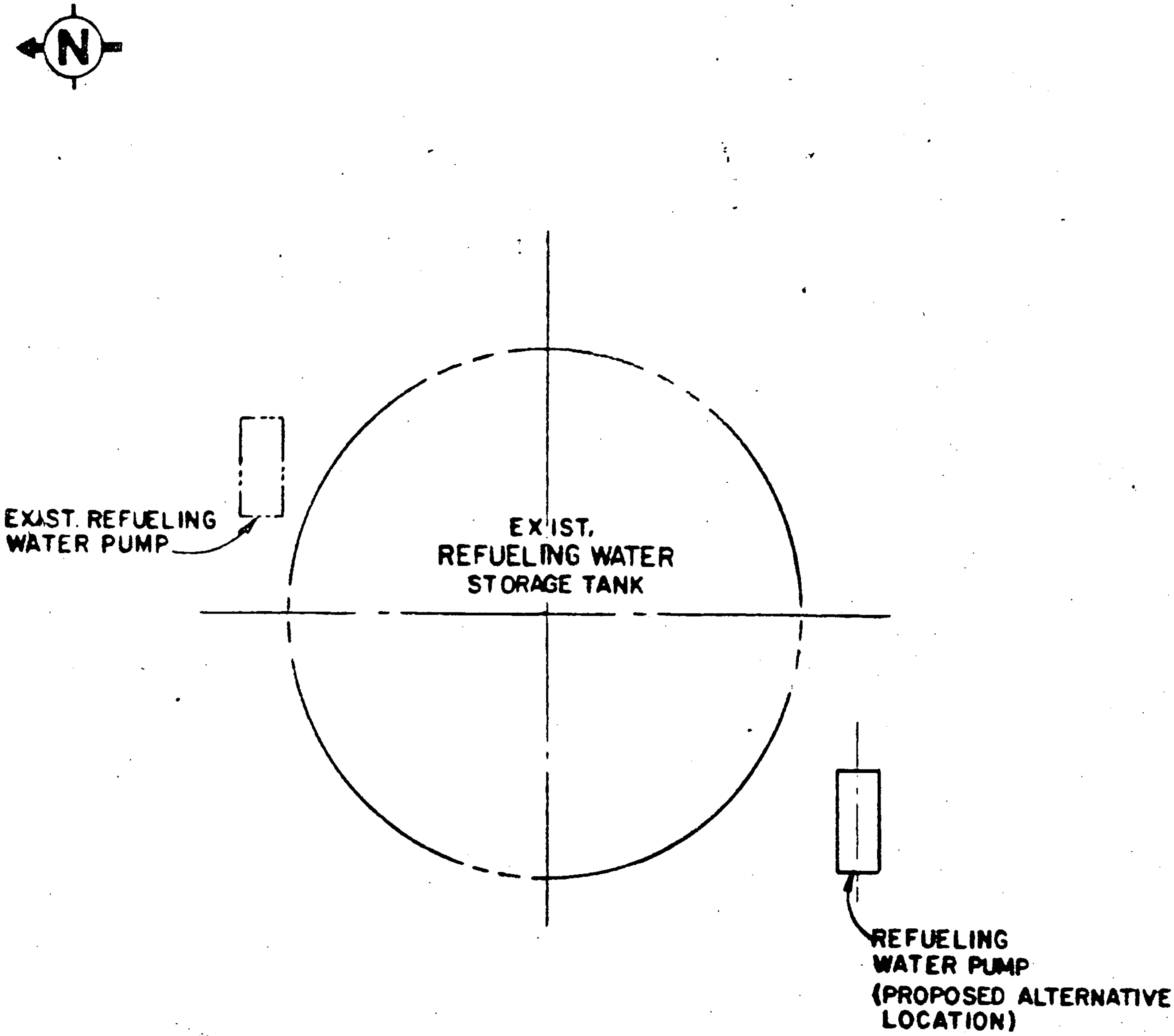




Section A–A

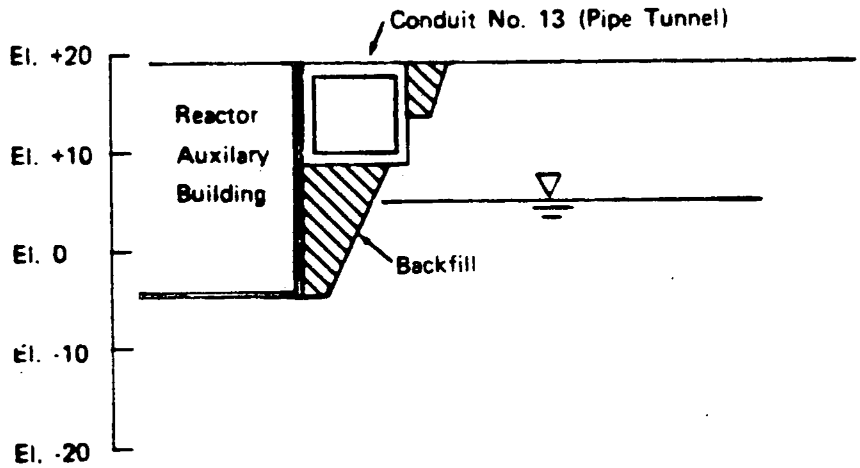
LOCAL SOIL CONDITIONS UNDER
FOOTING NO. 12

FIGURE 5-13

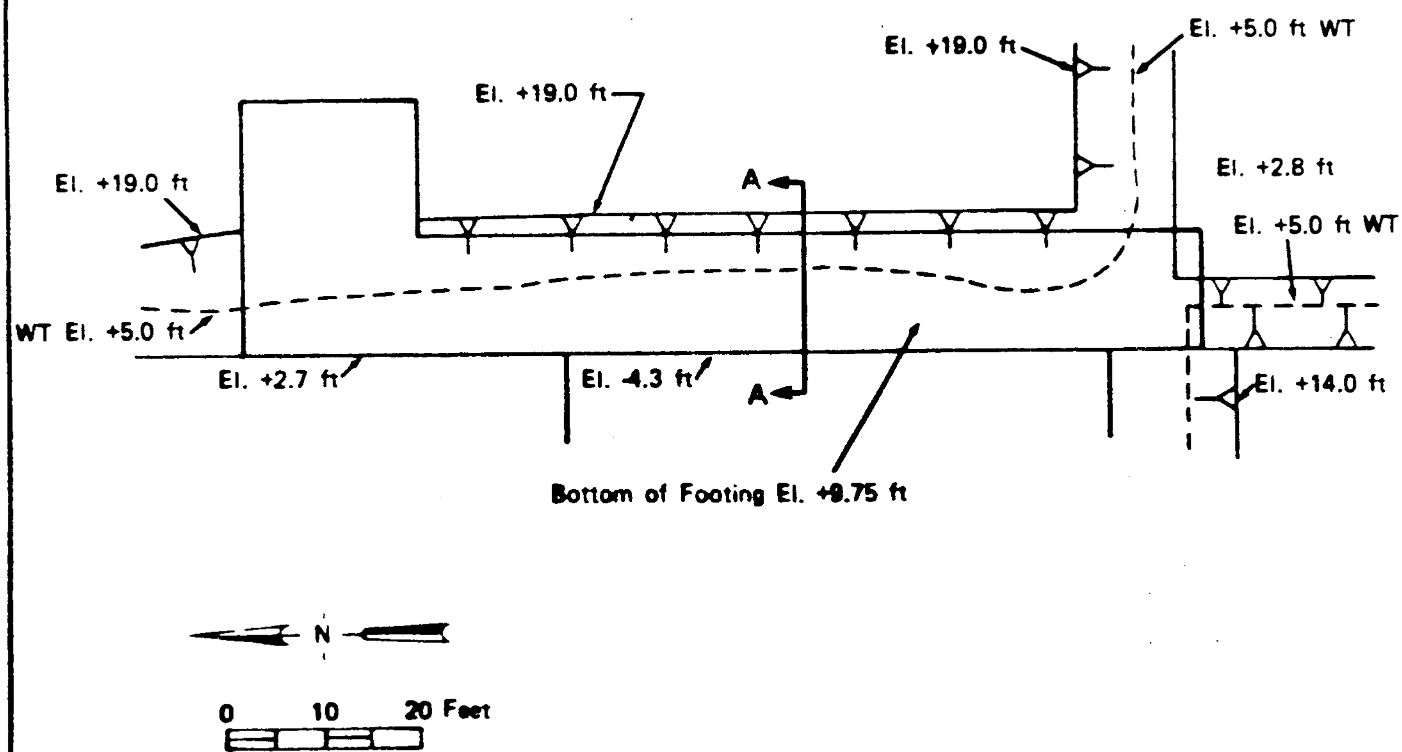


AREA 14 NEW PUMP LOCATION

FIGURE 5-13A

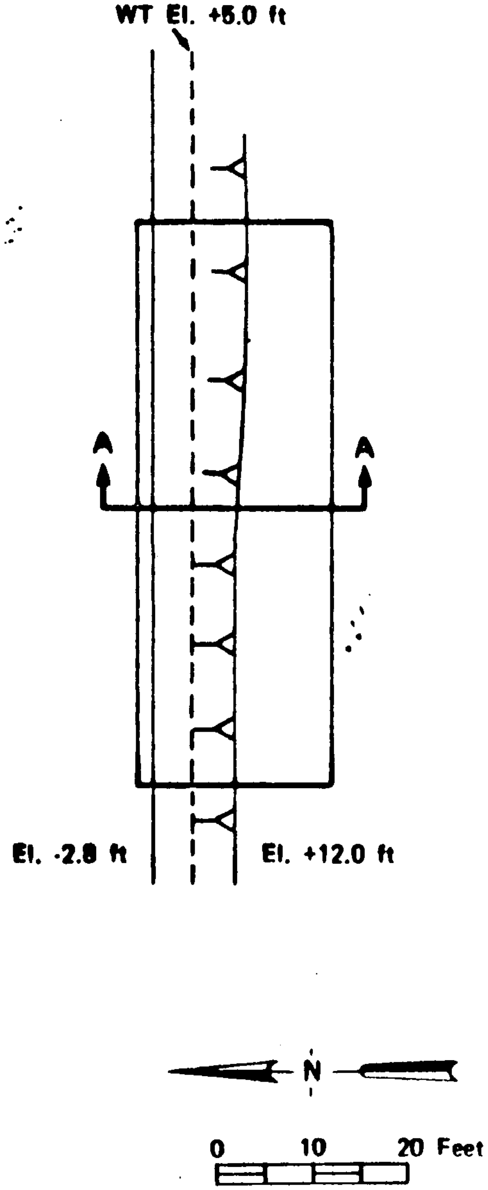


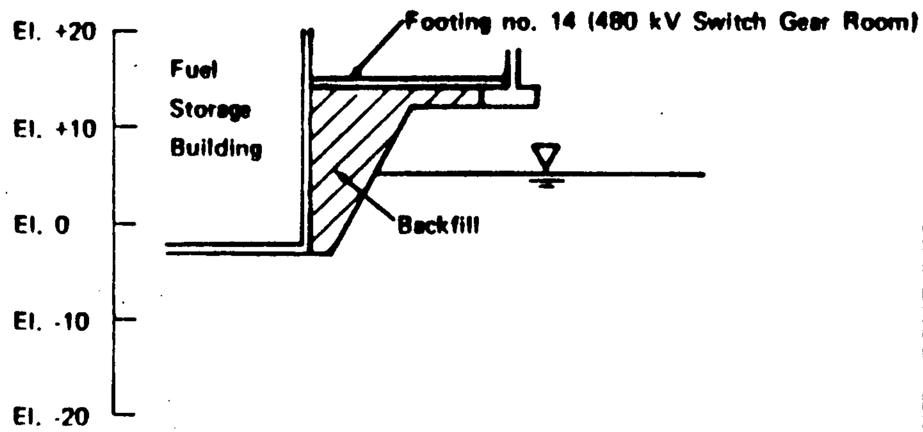
Section A–A



LOCAL SOIL CONDITIONS UNDER CONDUIT – NO. 13

FIGURE 5-14

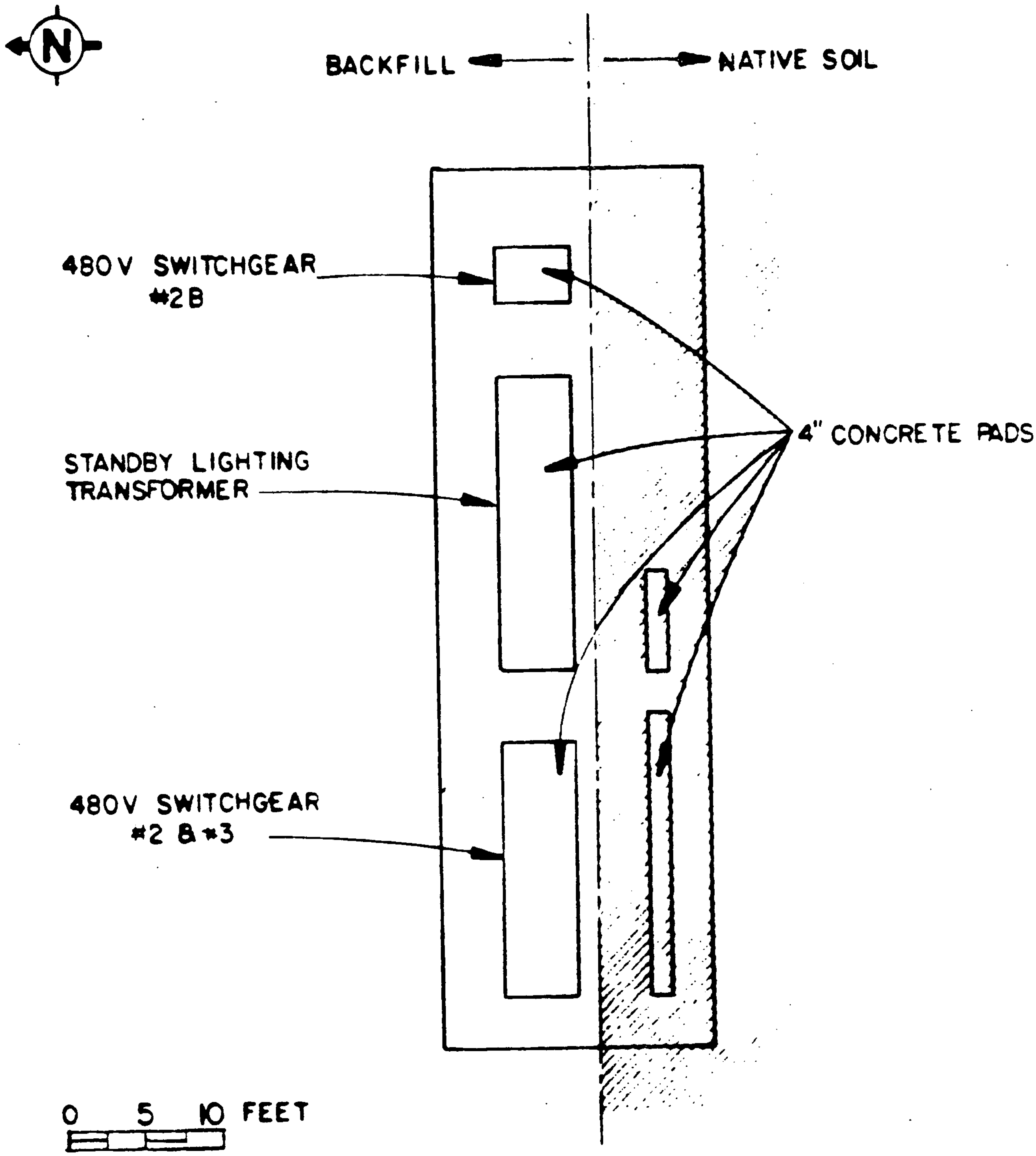




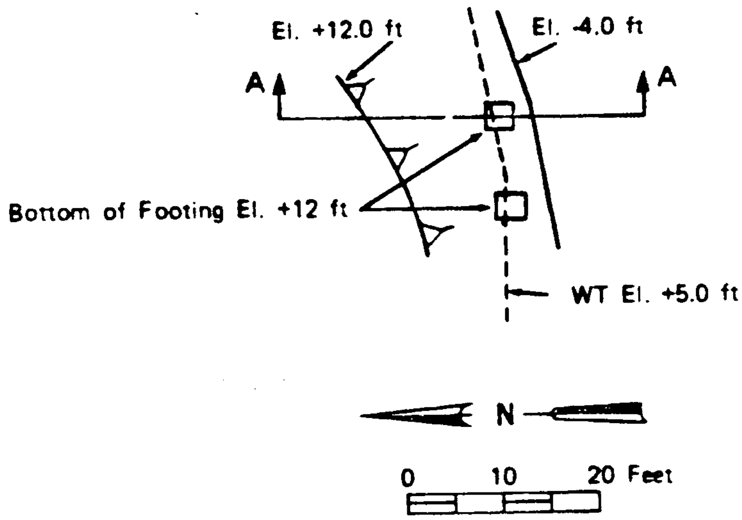
Section A–A

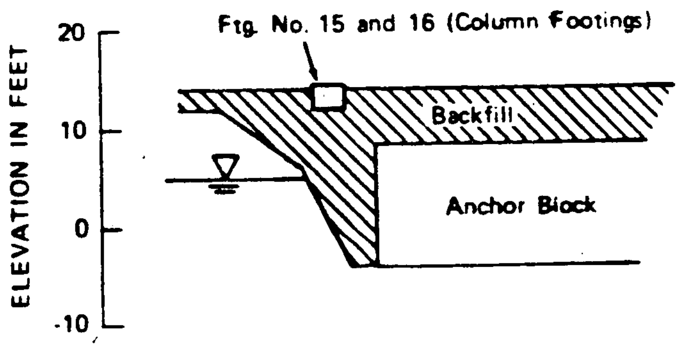
LOCAL SOIL CONDITIONS UNDER
FOOTING NO. 14

FIGURE 5-15



ELECTRICAL CABINET LOCATION
480 VOLT SWITCHGEAR ROOM SLAB
FIGURE 5-15A

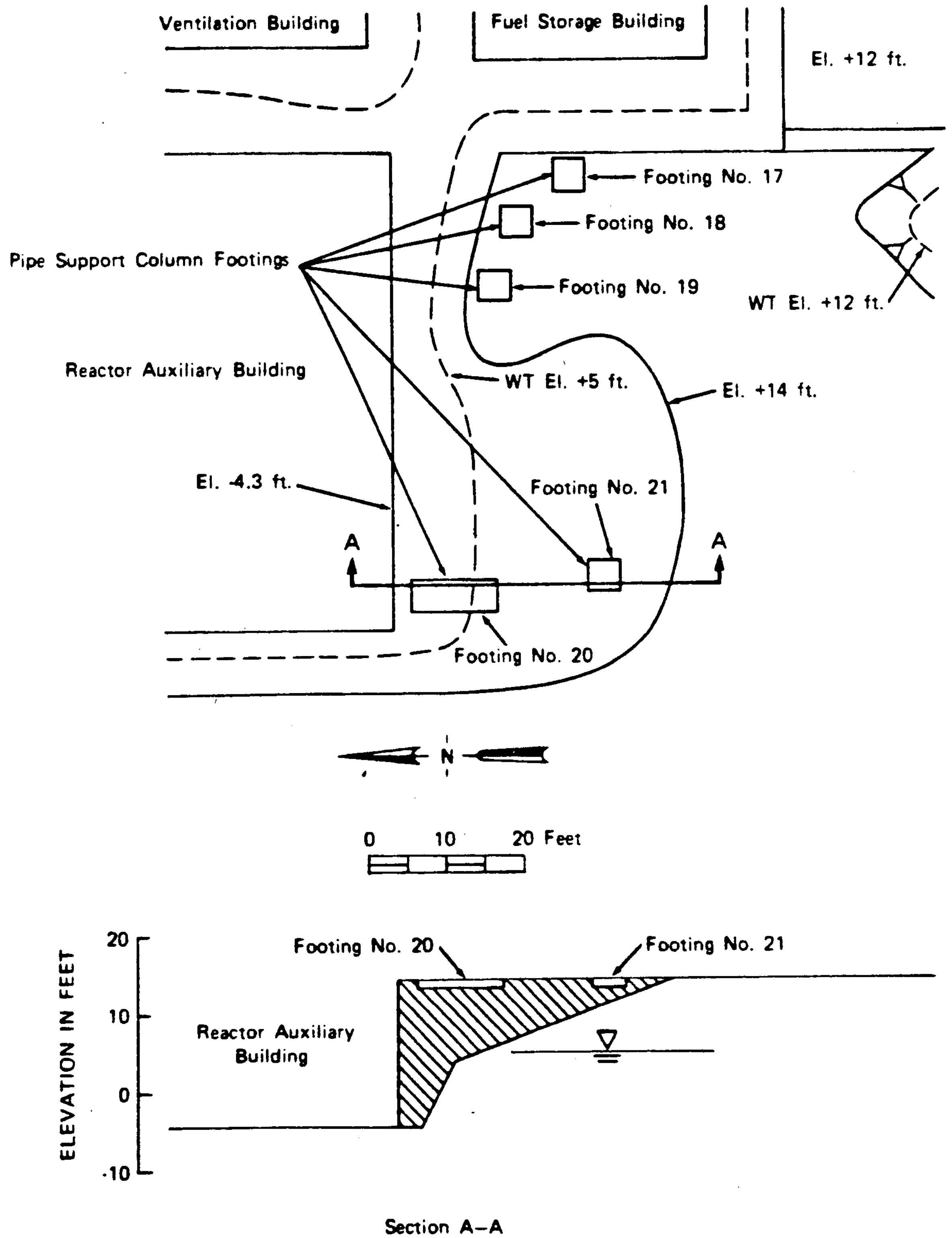




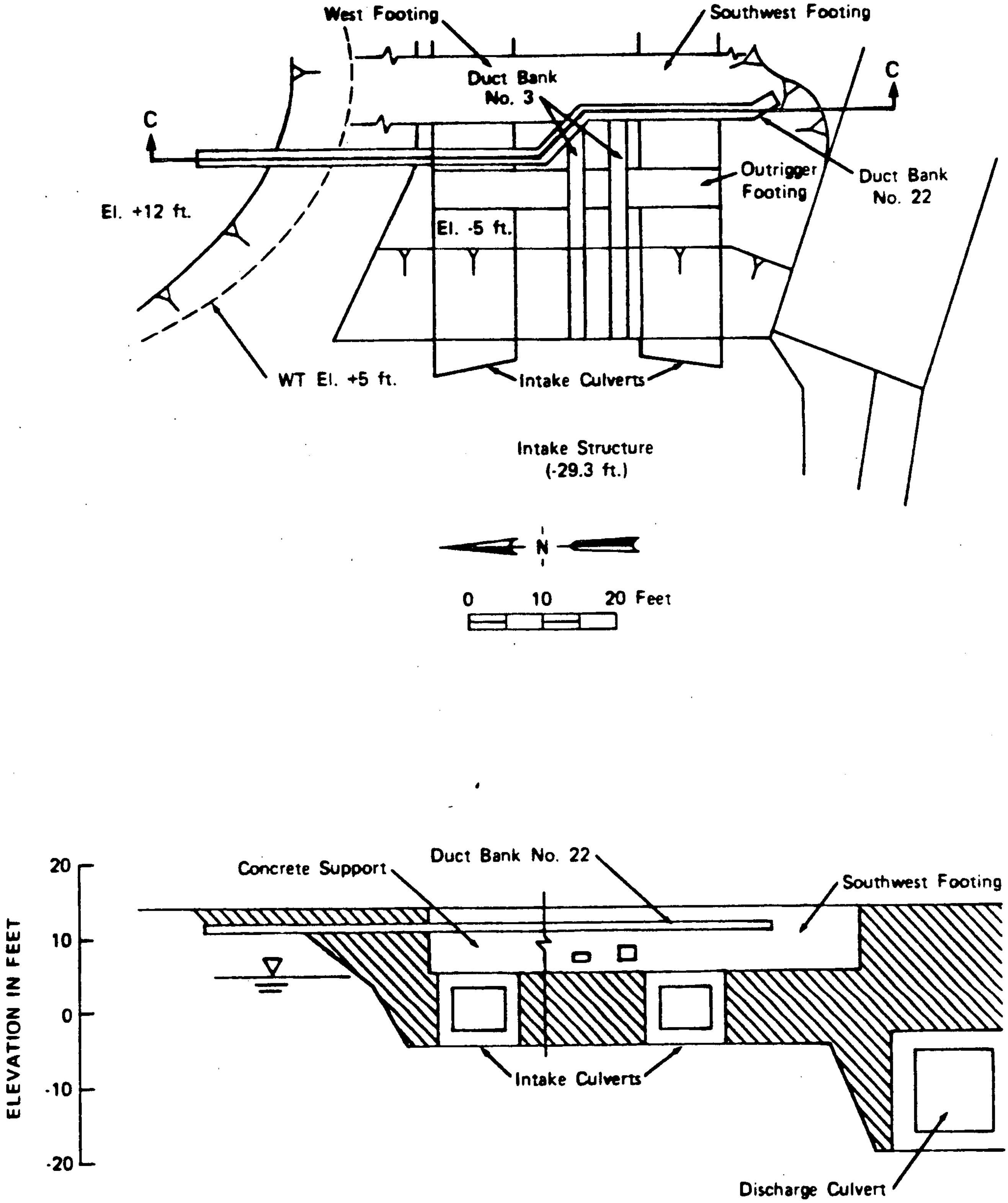
Section A–A

LOCAL SOIL CONDITIONS UNDER FOOTINGS NO. 15 AND 16

FIGURE 5-16

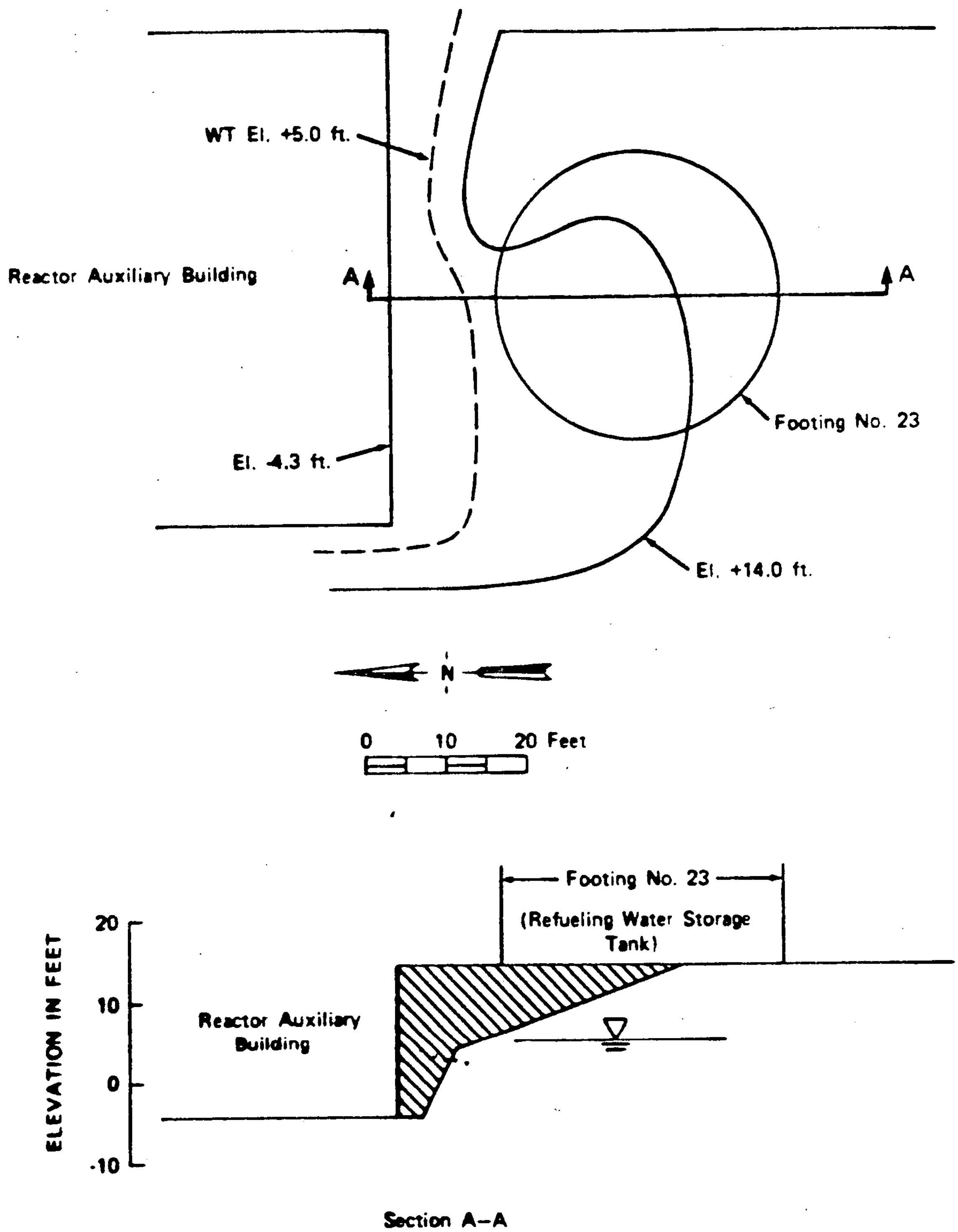


LOCAL SOIL CONDITIONS UNDER
COLUMN FOOTINGS FOR PIPING SUPPORTS
ITEMS NO. 17–21
FIGURE 5-17

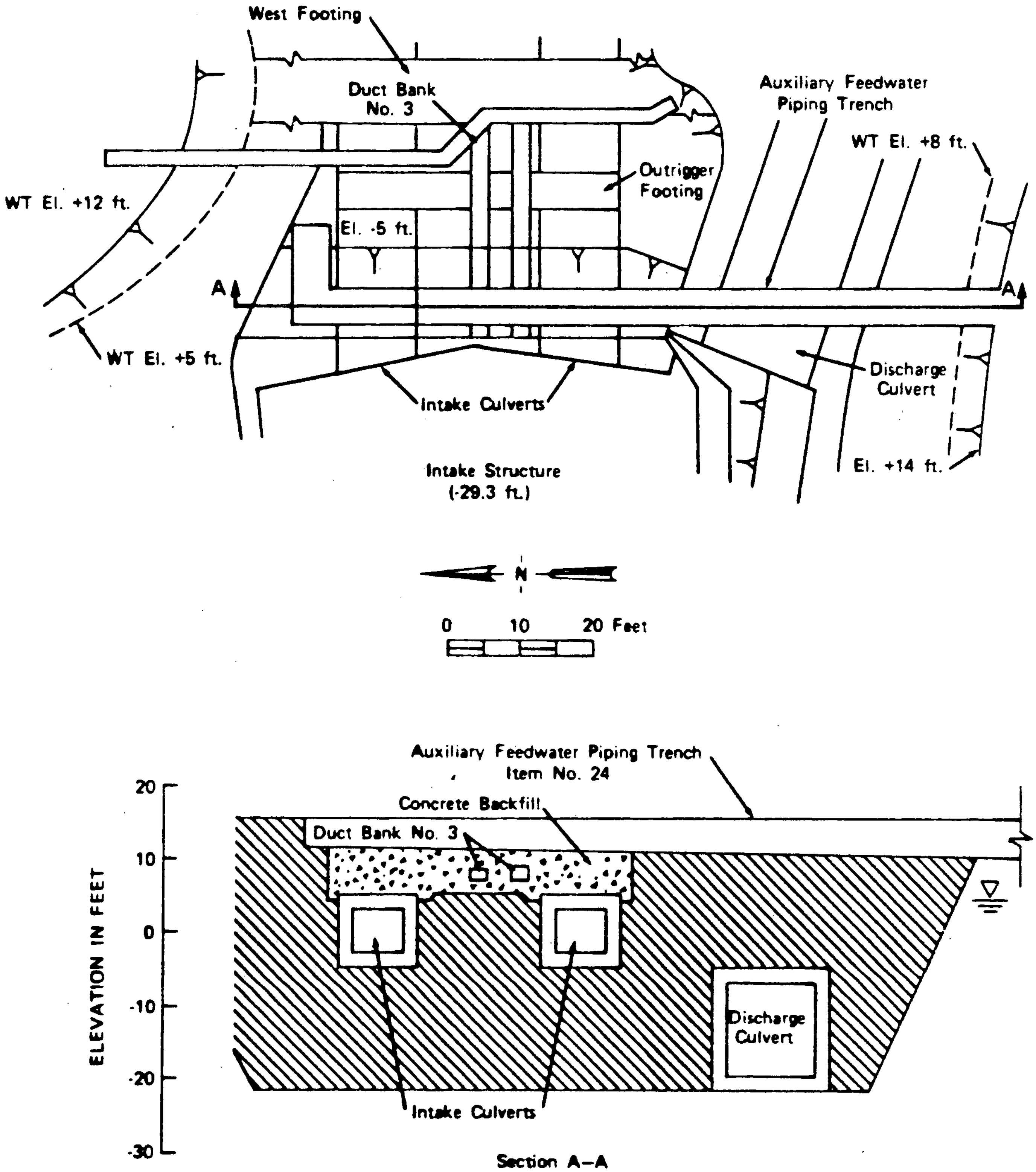


Section C–C

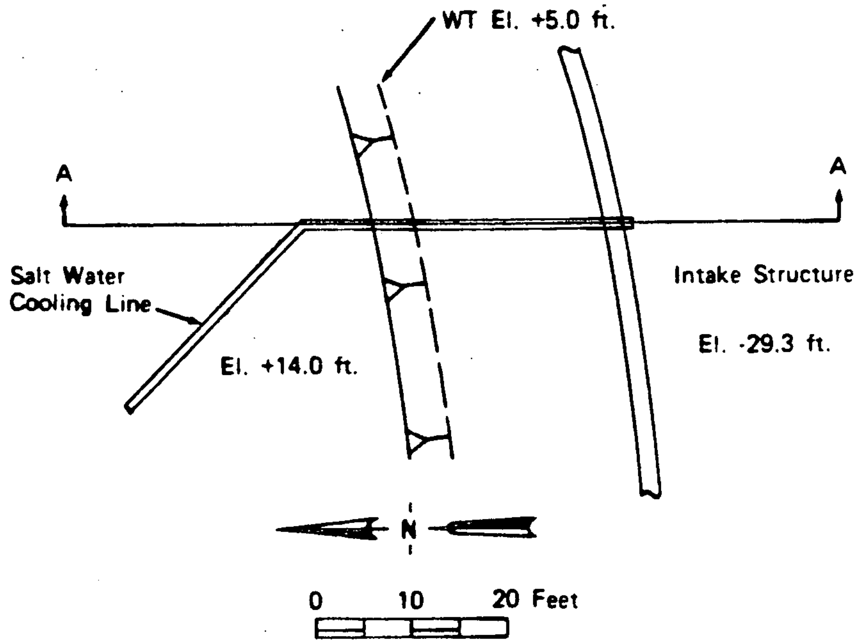
LOCAL SOIL CONDITIONS UNDER
DUCK BANK – NO. 22
FIGURE 5-18

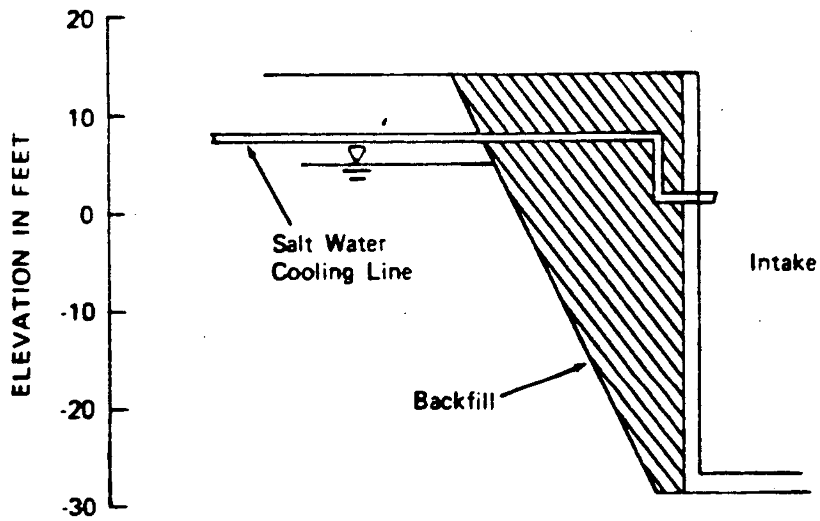


LOCAL SOIL CONDITIONS UNDER
REFUELING WATER STORAGE TANK
ITEM NO. 23
FIGURE 5-19



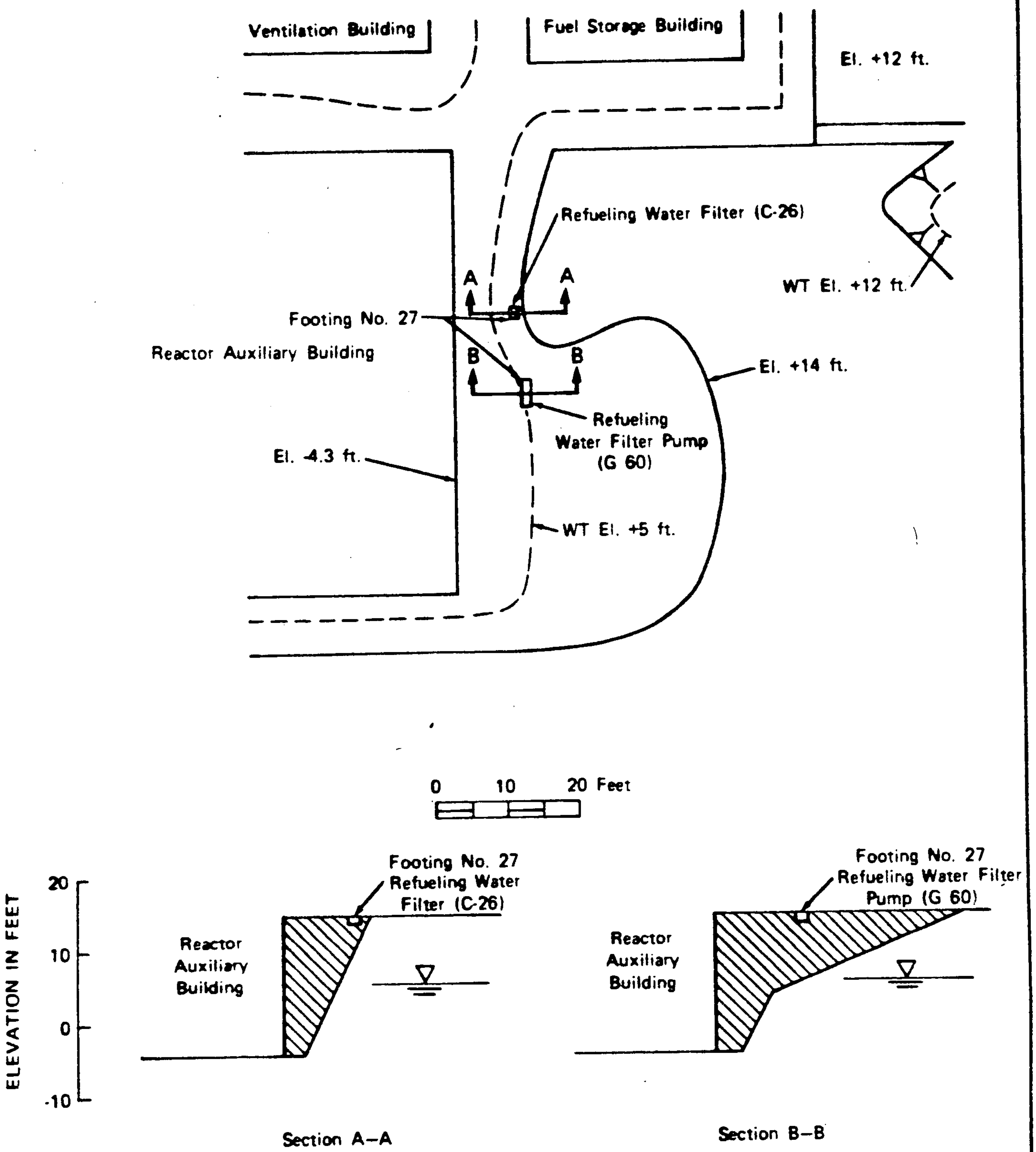
LOCAL SOIL CONDITIONS UNDER
AUXILIARY FEEDWATER PIPING TRENCH
ITEM NO. 24
FIGURE 5-20





Section A–A

LOCAL SOIL CONDITIONS UNDER
ITEM NO. 26
FIGURE 5-21



LOCAL SOIL CONDITIONS FOR
REFUELING WATER FILTER PUMP
ITEM NO. 27
FIGURE 5-22

REFERENCES

1. "Material Property Studies, San Onofre Nuclear Generating Station," San Onofre Nuclear Generating Station Units 2 and 3 PSAR, Amendment, 11 Attachment A3 to Appendix 2E, March 13, 1972.

2. Balance of Plant SONGS Unit 1. "Soil-structure Interaction Methodology Report," Revision 1, Woodward-Clyde Consultants, Orange, California, July 1978.

3. "Balance of Plant Structures Seismic Reevaluation Criteria," San Onofre Nuclear Generating Station, Unit 1, February 11, 1981.

4. Silver, M. L., and Seed, H. B., "Settlement of Dry Sands During Earthquakes," Journal of the Soil Mechanics and Foundation Division, ASCE, v. 98, no. SM4, April 1972.

5. Lee, K. L. , and Albaisa, A., "Earthquake Induced Settlements in Saturated Sands," Journal of the Geotechnical Engineeering Division, ASCE, Vol. 100, No. GT4, Proc. Paper 10496, April 1974, pp. 387-406.

6. Seed, H. B., Martin, P. P., and Lysmer, J., "The Generation and Dissipation of Pore Water Pressures During Soil Liquefaction," Report No. EERC 15-26, Earthquake Engineering Research Center, University of California, Berkeley, California, August 1975.

7. Seed, H. B., and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. EERC 71-10, Earthquake Engineering Center, University of California, Berkeley, California, 1970.

8. Letter from K. P. Baskin to A. Schwencer dated May 18, 1977, Subject: Seismic Reevaluation and Modifications, San Onofre Unit 1

9. Letter from K. P. Baskin to D. M. Crutchfield dated April 30, 1982, Subject: SEP Topic III-6, Seismic Design Considerations, San Onofre Unit 1

10. Letter from K. P. Baskin to D. M Crutchfield dated December 8, 1981, Subject: SEP Topic III-6, Seismic Design Considerations, San Onofre Unit 1

11. Letter from R. W. Krieger to D. M. Crutchfield dated February 9, 1982, Subject: SEP Topic III-6, Seismic Design Considerations, San Onofre Unit 1

12. Letter from K. P. Baskin to D. M. Crutchfield dated September 30, 1982, Subject: SEP Topic III-6, Fuel Storage Building, San Onofre Unit 1

13. Letter from K. P. Baskin to D. M. Crutchfield dated July 9, 1982, Subject: SEP Topic III-6, Seismic Design Considerations, San Onofre Unit 1

14. Letter from K. P. Baskin to D. M. Crutchfield dated August 17, 1982, Subject: In-Situ Soil Conditions, SEP Topic III-6, Seismic Design Considerations, San Onofre Unit 1