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March 14, 1994

SF94002

Attention: Dr. John Anderson / Mr. Mike Iverson

Subject: Geotechnical Technical Memorandum
EBMUD Pump Station C Storage Basin

Gentlemen:

In this technical memorandum we present limited geotechnical recommendations for the East Bay Municipal Utility District (EBMUD) Pump Station C Storage Basin Project. The recommendations are based on our geotechnical field investigation and laboratory tests performed for the project and described in Geotechnical Data Report for EBMUD Pump Station C Storage Basin Project, Geotechnical Consultants, Inc., February 1994. The project site is in Krusi Park, Alameda, California.

PROPOSED PROJECT

EBMUD intends to construct an underground reinforced concrete storage basin and a new pump station near the existing Pump Station C in Krusi Park. As proposed, the storage basin will be designed as a below-grade tank, approximately 60 feet by 110 feet in plan and embedded about 20 feet to 25 feet deep. The top of the basin will be about 1.5 feet below the finish grade. The new pump station will be approximately 20 feet by 20 feet in plan and will extend to about 10 feet below the bottom of the storage basin. The proposed basin site is within a baseball field in the park. The baseball field will be restored after the project is completed. The location of the basin, along with the locations of exploratory drill holes from our field exploration program are shown on Plate 1 - Site Plan.

We understand that the maximum depth of excavation for construction of the storage basin will be about 30 feet deep. Driven sheet piles braced at one or two levels are expected to be used to support the excavation sidewalls. The excavation and construction activities should require groundwater control. Groundwater control schemes that are currently being considered should result in dewatering within the excavation only. In preparing this technical memorandum, we have assumed that the groundwater level outside the excavation will not be lowered due to dewatering activities.

SITE CONDITIONS

All proposed improvements addressed by this investigation are located within the boundaries of the existing Krusi Park. Krusi Park, which is in a residential area in southwestern Alameda, includes four baseball fields and three floodlit tennis courts. The park is bounded by Mound Street and High Street on the northwest and southeast sides respectively and by Calhoun Street and Otis Drive on the northeast and southwest sides respectively. The park is essentially level with irrigated and maintained grass lawns. Surface elevation varies between 102 feet and 103 feet. These elevations are with respect to EBMUD datum. (EBMUD datum is National Geodetic Vertical Datum (NGVD) plus 100 feet. Unless otherwise specified, all elevations mentioned in this technical memorandum are with respect to EBMUD datum.) Presently, the project site is about 1,000 feet from the San Francisco Bay.

WORK PERFORMED

The scope of work for this technical memorandum was developed through discussions with Dr. John Anderson of CH2M HILL. The work performed consists of developing a generalized subsurface profile at the project site, developing limited geotechnical recommendations, and preparing this technical memorandum. The geotechnical recommendations are limited to: (1) seismic considerations at the site, (2) parameters for foundation design, (3) lateral earth pressures for cantilever walls, rigid walls, and braced flexible walls, (2) additional active earth pressures during seismic loading, and (3) length of cut-off wall to provide excavation bottom stability.

EARTH MATERIALS

Four geotechnical drill holes, B-1 through B-4, three environmental drill holes, E-1 through E-3, and two monitoring wells, W-1 and W-2, were drilled as part of our field exploration program. Details of the field exploration program are given in the Geotechnical Data Report. We encountered the following earth materials within the 100-foot maximum depth of our subsurface exploration.

Artificial Fill (af). A 4-foot to 8-foot thick surface layer of artificial fill was present at all our drill hole locations. The fill material encountered during drilling consisted mainly of poorly graded loose sand. The upper 2 feet to 4 feet of sand was rich in organics, presumably top soil imported for the park lawns. Below this, we encountered assorted debris within the sand matrix. This debris included cobbles, decaying pieces of tree limbs, wood chips, pieces of concrete, asphalt, brick, and glass. In drill holes B-1 and B-3 at a depth of about 3 feet, we encountered an approximately 1-foot layer of sand mixed with a dark, oily substance. No other oily soils were found in any of the other borings, including the monitoring wells and the environmental borings.

Younger Bay Mud (Qyb). Younger Bay Mud, a soft, moderately to highly plastic, compressible marine clay with abundant shells, is present beneath the artificial fill in all our drill holes. The thickness of the Younger Bay Mud varied from 5 feet to 8 feet. In drill holes B-2, B-3, and B-4, the bottom of the Younger Bay Mud is about 12 feet to 12.5 feet below the ground surface. The Younger Bay Mud extends to a depth of 16 feet in B-1.

Merritt Sand (Qm). An approximately 24-foot to 30-foot layer of Merritt Sand was present below the Younger Bay Mud in all our geotechnical drill holes. This sand, as encountered in our drill holes, is a medium dense to dense, poorly graded silty sand with some shell fragments. Blow counts in this layer ranged from 17 to 47 blows per foot. Grain size tests on selected samples indicate that the sand from this unit contains about 10 percent to 20 percent fines. The Merritt Sand was locally clayey and locally poorly graded.

San Antonio Formation (Qs). The San Antonio Formation underlies the Merritt Sand. This formation is 35 feet thick as encountered in B-1 and B-2. This formation is predominantly stiff to very stiff lean clay, dense poorly graded sand, dense clayey sand, and gravel. Except in B-3, stiff to very stiff lean clay was found immediately beneath the Merritt Sand layer, at a depth of 40 to 45 feet below existing ground. This layer of lean clay was 5 feet thick in B-2 and B-4 and 25 feet thick in B-1. In B-3, the Merritt Sand was underlain by 5-foot layers of dense clayey sand, very dense poorly graded sand, and very dense well graded gravel. Beneath the layer of gravel, at a depth of 50 feet below existing ground, we encountered very stiff to hard lean clay. Blow counts of 79 blows per foot were recorded in the gravel layer in B-3.

Older Bay Mud (Qob). Older Bay Mud, also known as Yerba Buena Mud, was found beneath the San Antonio Formation, at a depth of 80 feet and 75 feet in B-1 and B-2 respectively. The Older Bay Mud encountered consisted of stiff lean clay with fine gravel in B-1 and clayey sand in B-2. Occasional shell fragments were present in the Older Bay Mud samples from both drill holes.

A generalized subsurface profile at the project site is shown on Plate 2 -Generalized Subsurface Profile. The idealized strength properties used in our analyses are presented in Table 1 - Idealized Strength Properties.

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TABLE 1

IDEALIZED STRENGTH PROPERTIES

Material	Approx. Depth Limits of layer	Total Unit Weight (pcf)	Undrained Shear Strength (psf)	Friction Angle
Artificial Fill	0 - 7 feet	105	---	28 degrees
Younger Bay Mud (Qyb)	7 - 14 feet	96	350	---
Merritt Sand (Qm)	14 - 43 feet	130	---	34 degrees
San Antonio Formation (Qs)	> 43 feet	130	2500[1]	42 degrees[2]

[1]For the lean clay units of the San Antonio Formation
[2]For the very dense poorly graded sand and gravel units of the San Antonio Formation

CONCLUSIONS AND RECOMMENDATIONS

1.0 FEASIBILITY

The project is geotechnically feasible as proposed, provided the recommendations presented herein are incorporated in the design and construction.

2.0 SEISMIC CONSIDERATIONS

2.1 **Ground Shaking.** The San Francisco Bay Area is one of the most seismically active areas in the continental United States. The Hayward and San Andreas faults are approximately 4 miles and 15 miles from the site respectively. The project area is likely to be subjected to strong ground shaking from an earthquake occurring along one of these faults during the service life of the storage basin.

We performed a probabilistic seismic hazard analysis for the project site using the computer program FRISK89 (Blake, 1991). Our analysis indicates that within a fifty year exposure period there is a 10 percent probability that the project site would experience ground shaking exceeding peak horizontal ground accelerations of 0.4 g, where 'g' is the acceleration due to gravity.

2.2 Liquefaction. Liquefaction is a phenomenon in which saturated, loose to medium dense cohesionless soils experience a partial to total loss of shear strength during the cyclic loading accompanying an earthquake. Consequences of liquefaction include ground settlements, bearing failure, and lateral spreading. The potential for liquefaction decreases with increase in relative density and with increase in clay content.

We evaluated the liquefaction potential of the subsurface materials using a peak horizontal ground acceleration of 0.4 g. Our analysis indicates that the medium dense Merritt Sand encountered between depths of about 20 feet and 30 feet can potentially liquefy for this level of shaking. Subsurface materials present at depths greater than 30 feet are not expected to liquefy.

Since the foundation of the proposed storage basin is expected to be within the potentially liquefiable zone, liquefaction mitigation measures should be adopted to minimize hazards associated with liquefaction. Such mitigation measures can include schemes such as (1) densification of sands prior to foundation construction and (2) excavation of the sands within this zone and replacing either with compacted fill, cement slurry backfill, or drainrock underlain by filter fabric.

Due to their temporary loss of shear strength, liquefied soils adjacent to the walls of the storage basin are expected to impose temporary additional lateral loads on the walls. Such liquefaction induced temporary lateral loads can be reduced by reducing the liquefaction potential of these soils. One approach to reduce liquefaction potential is to facilitate the dissipation of excess pore water pressure that is generated during seismic shaking. For this approach, we recommend that an 18-inch thick layer of well-graded aggregate filter (similar to Class 2 permeable material described in Section 68, Caltrans Standard Specifications) be introduced between the native Merritt Sand and the walls of the storage basin. Alternatively, an 18-inch thick layer of open-graded coarse drain rock wrapped in durable filter fabric (such as Mirafi 700X) may also be used. If sheet pilings are to be permanently left in place, vertical gravel drains constructed outside the boundary of the storage basin should be effective in dissipating excess pore pressures.

2.3 **Fault Rupture.** No known active or potentially active faults underlie the project site. Consequently, the potential for fault rupture at the project site is considered negligible.

3.0 GROUNDWATER

Groundwater levels measured during our exploration ranged from 3 feet to 7 feet. The groundwater level could be affected by seasonal variations and tidal fluctuations in the nearby San Francisco Bay. We recommend that the design groundwater level be assumed at a depth of 3 feet below existing ground level, corresponding to about elevation 100 feet.

The layer of Younger Bay Mud encountered beneath the fill will probably act as an aquitard, hydraulically separating the upper fill from the Merritt Sand. For short-term changes in the groundwater level, for example due to tidal fluctuations, pumping, and rainfall, it is reasonable to consider the groundwater within the upper fill as a perched aquifer.

Construction operations should be conducted under dry conditions and provisions must be made for groundwater control. Groundwater control can be facilitated by extending cut-off walls through the Merritt Sand and into the predominantly clayey San Antonio Formation and dewatering within the excavation. We recommend that the design and implementation of a suitable groundwater control scheme be made the responsibility of the contractor. The groundwater control scheme should require that the groundwater level be lowered and maintained at least two feet beneath the lowest excavation elevation until all construction activities within the excavation are completed.

4.0 FOUNDATION SUPPORT

4.1 **General.** It is our understanding that the walls of the storage basin will be supported on footing foundations. The bottom slab of the storage basin will be designed as a mat foundation. The foundations are expected to be at a depth of 25 feet to 30 feet. Potentially liquefiable soils underlying the foundation should be treated or removed and replaced as discussed in Section 2.2. It is important that adequate care be exercised during excavation to minimize loosening or disturbance of the sandy foundation soils.

4.2 **Foundation Bearing Capacity.** For design of the wall footings and the bottom slab, the allowable bearing capacity of competent subgrade may be assumed to be 3,000 pounds per square foot (psf) for dead plus normal duration live loads. The allowable bearing capacity may be increased by one-third when considering additional short-term seismic loading.

The bottom slab may also be designed using the modulus of subgrade reaction concept. The modulus of vertical subgrade reaction for a one-foot square bearing plate, k_1, may be assumed as 80 tons per cubic foot (tcf). The k_1 value should be modified to yield k_B, the modulus of vertical subgrade reaction for a mat foundation of effective width B, using the equation $k_B = k_1 \, [(B+1)/(2B)]^2$.

4.3 **Settlements.** Total long-term settlements of foundations designed using the allowable bearing capacity given above should be less than 1 inch. The majority of this settlement should occur immediately after the dead loads are imposed.

5.0 LATERAL EARTH PRESSURES

5.1 **Active Earth Pressures.** Active earth pressures develop behind retaining walls that are unrestrained. To develop active earth pressures, walls should be capable of lateral movements of at least 0.4 percent of the height of the retained soil. The lateral earth pressure coefficients for active condition, K_A, are presented in Table 2 - Lateral Earth Pressure Coefficients. The active earth pressure at any depth is obtained by multiplying the effective overburden stress, using buoyant unit weight below the groundwater table, at that depth by the active earth pressure coefficient. A vertical surcharge load should be provided to account for vehicles, structures, or stockpiled material near the boundaries of the storage basin. As a minimum, we recommend using a 200 psf vertical surcharge pressure at the ground surface when calculating effective overburden stress. The full hydrostatic pressure should be added to the lateral earth pressure for calculating the total lateral pressure.

5.2 **At-Rest Earth Pressure.** At-rest earth pressures develop behind rigid walls that are restrained from undergoing any displacement. The lateral earth pressure coefficients for at-rest condition, K_O, are presented in Table 2. The at-rest earth pressure at any depth is obtained by multiplying the effective overburden stress at that depth by the at-rest earth pressure coefficient.

5.3 **Passive Earth Pressure.** Sheet piles used during excavations will derive their stabilizing passive earth pressure below the bottom of excavation. The lateral earth pressure coefficient for passive condition, K_P, is given in Table 2. The passive earth pressure at any depth is obtained by multiplying the effective overburden stress at that depth by the passive earth pressure coefficient. The value of K_P given in Table 2 is an ultimate value, and appropriate factors of safety should be used in design.

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5.4 **Active Earth Pressure During Seismic Loading.** Active earth pressures can increase during earthquakes due to the additional lateral dynamic loading accompanying an earthquake. The increase in active earth pressure on permanent walls during seismic loading can be evaluated using the additional dynamic active earth pressure coefficient, K_{EQ}, given in Table 2. The additional dynamic active earth pressure at any depth is obtained by multiplying the effective overburden stress at that depth by the additional dynamic active earth pressure coefficient. The resultant of the additional dynamic active earth pressure should be assumed to act at 0.6H from the base of the soil being retained, where H is the height of the retained soil. Permanent walls designed for at-rest earth pressure conditions should be checked for active plus additional dynamic active earth pressure conditions.

TABLE 2

LATERAL EARTH PRESSURE COEFFICIENTS

Earth Material	Unit Weight (pcf) Above GWT	Unit Weight (pcf) Below GWT[1]	K_A	K_O	K_P	K_{EQ}
Artificial Fill	105	42	0.33	0.53	---	0.17
Younger Bay Mud	100	38	0.50	0.70	---	0.21
Merritt Sand	130	68	0.25	0.45	4.5	0.15

[1]Hydrostatic pressure should be considered in addition to lateral earth pressure.

5.5 **Braced Temporary Walls.** The design of bracing will depend on the depth of excavation and the excavation and bracing sequence. The type of bracing system to be used and the design of the bracing system should be the responsibility of the contractor. For preliminary design of temporary sheet piles braced at two levels, at 0.25H from the top and bottom of the total depth of excavation where H is the total depth of excavation, the earth pressure diagram presented in Plate 3 - Earth Pressures on Temporary Braced Walls may be used. Below the groundwater table, hydrostatic pressure should be considered in addition to the earth pressure diagram shown on Plate 3. If the sheet piles are braced at the top only, or are designed as tied-back walls, the earth pressure coefficients presented in Table 2 may be used.

5.6 **Embedment of Sheet Piles.** To prevent bottom heave of the excavation due to seepage forces, temporary sheet piles supporting the excavation should extend at least 15 feet below the excavation bottom, and should be embedded within the lean clay layer of the San Antonio Formation encountered below the Merritt Sand. Extending the sheet pile embedment into the clay unit of the San Antonio Formation should provide a cut-off to facilitate dewatering of the excavation. The recommended 15-foot minimum embedment below the excavation bottom should also provide adequate stabilizing passive resistance for temporary sheet piles braced at the top only.

6.0 HYDROSTATIC AND HYDRODYNAMIC LOADS

In addition to earth pressures, hydrostatic pressures due to the groundwater table should be considered in the design. During the lateral dynamic loading accompanying an earthquake, additional hydrodynamic forces may be induced on the walls of the storage basin, especially if the basin is empty when the seismic event occurs. These hydrodynamic forces should be considered in the design as short-duration loads. For design, a hydrodynamic force resultant equal to $8h^2$ pounds per linear foot of wall, where h is the difference in level between the groundwater table and the water level within the storage basin, may be assumed to act at h/2.

CLOSURE

The conclusions and recommendations presented in this technical memorandum are professional opinions for the project as described in this memorandum. A review by this office of any foundation and excavation plans, together with the opportunity to make supplemental recommendations, is considered an integral part of this memorandum and a condition of the recommendations presented herein. Should the project change from that described in the proposed project, we should be given the opportunity to review our recommendations in light of those changes.

Subsurface exploration of any site is necessarily confined to selected locations and conditions may, and often do, vary between and around these locations. Should varied conditions become known during project development, additional exploration, testing, and recommendations may be required.

The findings and professional opinions presented in this memorandum are presented within the limits prescribed by the client, in accordance with generally accepted professional engineering and geologic practices. There is no other warranty, either express or implied.

Respectfully submitted,

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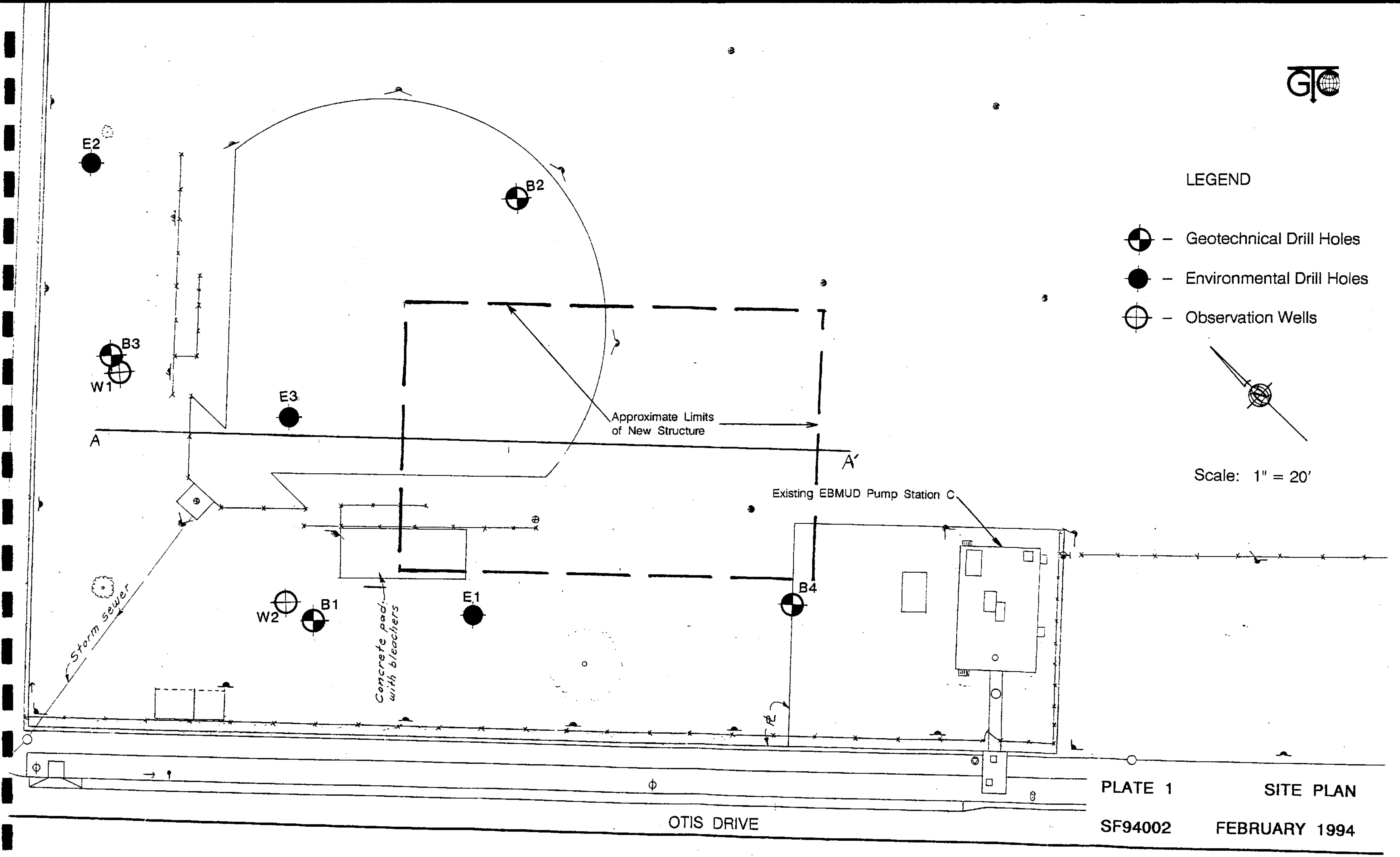
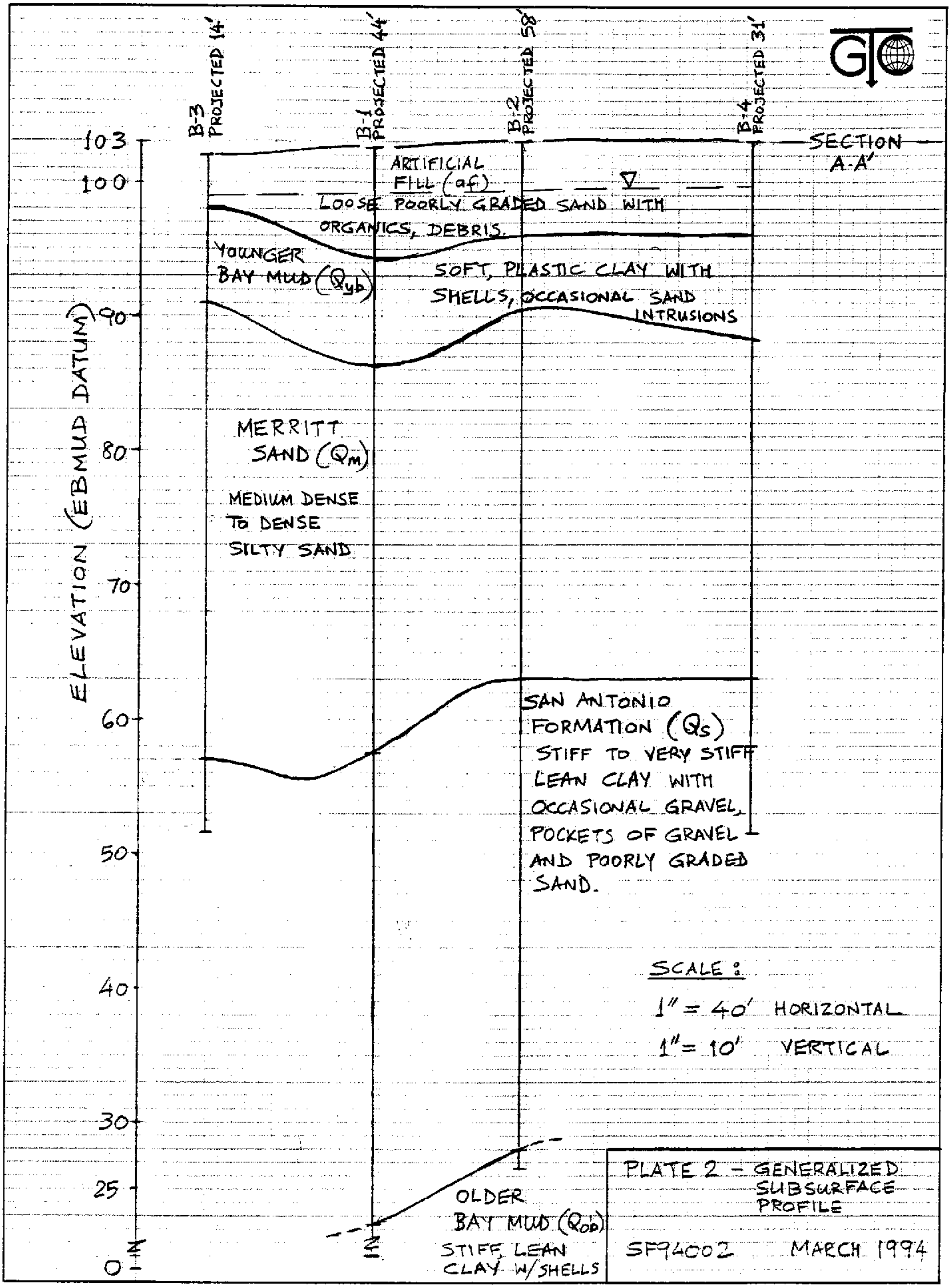


PLATE 1 SITE PLAN

SF94002 FEBRUARY 1994



CHECKED BY

DATE CHECKED

CALCULATION NO.

SUBJECT GENERALIZED SUBSURFACE PROFILE

PREPARED BY G. NEELAKANTAN

PROJECT PUMP STN. C STORAGE BASIN

JOB NO. SF94002

DATE MARCH 3 '94

PAGE 1 OF 1

PLATE NO. 2

NOTES

1. THE PRESSURE DIAGRAM DOES NOT INCLUDE ANY SURCHARGE LOADS.
2. IN ADDITION TO EARTH PRESSURE, HYDROSTATIC PRESSURE SHOULD BE CONSIDERED IN DESIGN.

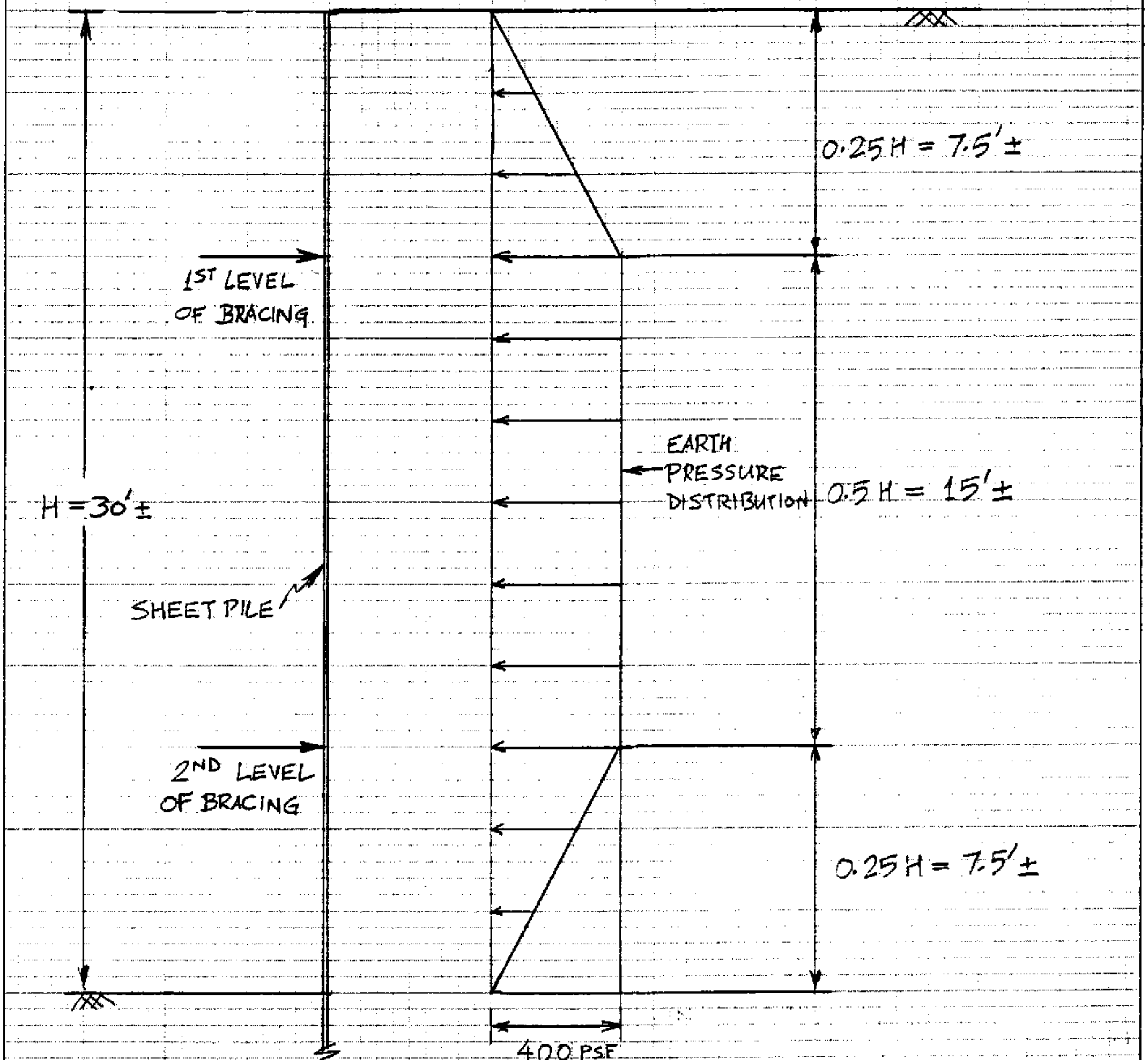


PLATE 3 - EARTH PRESSURES ON TEMPORARY BRACED WALLS

SF94002 MARCH 1994

CALCULATION NO.		SUBJECT	EARTH PRESSURES ON TEMP. BRACED WALLS
PREPARED BY	G. NEELAKANTAN	PROJECT	PUMP STN. C STORAGE BASIN JOB NO. SF94002
DATE	MARCH 3 '94	PAGE	1 OF 1 PLATE NO. 3

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