REPORT

GEOTECHNICAL INVESTIGATION
PEYTON SLOUGH REMEDIAL DESIGN WITH GEOTECHNICAL RECOMMENDATIONS FOR TIDE GATE STRUCTURE

Prepared for:
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51-00070007.00

FIGURES

APPENDICES

1.1 PROJECT DESCRIPTION

The work described in this report is a part of the Peyton Slough Remedial Design Project located in Martinez, California as shown on Figure 1. The remedial design project includes the excavation of a new slough alignment, capping of the existing slough alignment, and construction of a tide gate structure along the new slough alignment where it crosses the existing levee.

1.2 PURPOSE, AUTHORIZATION, AND SCOPE OF WORK

This report summarizes the subsurface investigation completed by URS and provides our geotechnical recommendations for the design and construction of the proposed tide gate structure. The original scope of work was defined in Task 1.4 of our proposal to Rhodia Inc., dated December 3, 2001 and December 19, 2001. Partial authorization to start the field investigation was given on December 7, 2001 by Rhodia Inc. and authorization for the complete scope was given on December 21, 2001. A summary of the scope of work is provided below.

- Review existing geotechnical data developed in the vicinity of the project.

- Conduct a geotechnical investigation to characterize site soil conditions. This investigation would consist of four borings ranging from 50 to 70 feet deep. One boring was to be drilled at the mouth of Peyton Slough for cap design. One boring was to be excavated at each of two temporary trestle bridge locations, and a final boring was to be drilled in the existing levee at the proposed tide gate location.

- Conduct laboratory testing on selected samples from the borings to assist in evaluating soil characteristics and characterizing subsurface conditions.

- Perform engineering analyses and provide geotechnical recommendations in report. The analyses would include the following:

  1. Design the foundation for the tide gate along the new slough alignment.

  2. Determine settlement of the levee and tide gate after construction (The proposal assumed that the tide gate structure would be designed and constructed in a similar manner to the existing tide gate).

  3. Determine stability of the levee for construction traffic.

  4. Provide soil engineering properties for independent evaluation of existing tide gate design.

  5. Provide geotechnical data for contractor use regarding the depth through Bay

Mud for temporary trestle bridge foundation design.

- Preparation of this geotechnical design report.

During the initial geotechnical investigations the scope was modified as follows:

- Perform additional investigations at the east end of the levee for evaluation of alternative tide gate locations.

- Make geotechnical recommendations for the recommended tide gate location.

1.3 REPORT ORGANIZATION

Methods of study are presented in Section 2.0. Site conditions found during the field investigation and laboratory testing are presented in Section 3.0. Analyses and geotechnical recommendations for the new tide gate structure are presented in Section 4.0. Section 5.0 serves as the closure section of this report. Appendix A contains logs of the test borings and test pit while Appendix B contains laboratory test results.

2.1 PREVIOUS INVESTIGATIONS

The following geotechnical investigations previously conducted at the site were reviewed:

Mark Group (1989). A site investigation report of the Sulphur Products Facility for Stauffer Chemical Company, Martinez, California. This report summarizes geological conditions at the site and specific geotechnical parameters for five ponds constructed at the site from previous investigations and reports.

Harza (1996a, 1996b, 1996c, 1997, 1998a, 1998b, 1999a, and 1999b). This series of reports and letters present explorations, laboratory testing, analyses, and recommendations for the design and construction of the existing tide gate structure and for raising the levee east of the tide gate.

2.2 FIELD STUDIES

Field studies were conducted in two phases:

- Phase 1 – The Phase 1 investigation, conducted in December 2001, was performed to (1) assess the engineering properties of soils at the proposed tide gate location along the conceptual new slough alignment, (2) provide information for engineering properties of soils where existing Peyton Slough enters the Carquinez Strait, (3) provide information for engineering properties of soils along existing Peyton Slough for geotechnical cap design.

- Phase 2 – The Phase 2 investigation, conducted in January 2002, was performed to assess foundation conditions at the east end of the existing levee as an alternative tide gate location.

2.2.1 Drilling

Six exploratory boreholes, URS-B1, URS-B2, URS-B3, URS-B4, URS-B5, and URS-B6 were drilled for this geotechnical study at the locations indicated on the Site and Exploration Location Plan, Figure 2. Boring URS-B1 was drilled into 100.75 feet Below Ground Surface (BGS) into bedrock. Boring URS-B2 was drilled to bedrock, to a total depth of 81 feet BGS, where the conceptual new slough alignment passed through the levee. Boring URS-B3 was drilled in the mouth of the existing slough over water from a barge. It was drilled to a depth of 52.5 feet below the mudline. Boring URS-B4 was also drilled from a barge further in the slough channel as indicated on Figure 2 to a depth of 61.5 feet below mudline. Borings URS-B5 and URS-B6 were drilled at the eastern end of the existing levee. Both were drilled into bedrock. URS-B5 was drilled to a total depth of 31.5 feet and URS-B6 was drilled to a total depth of 16.5 feet.

Borings URS-B1, URS-B2, URS-B5 & URS-B6 were drilled using a track-mounted CME 850 drilling rig and borings URS-B3 and URS-B4 were drilled using a barge mounted D-25 skid rig both of which were owned and operated by PC Exploration, Inc. of Rocklin, California. Hollow stem auger drilling methods were used to advance borings URS-B1, URS-B2, URS-B5 & URS-B6 and mud-rotary methods were used to advance borings URS-B3 and URS-B4. A URS geologist, logged the soil cuttings and samples in the field and visually classified the soils in general accordance with the Unified Soil Classification System, as the drilling proceeded.

Samples of the subsurface materials were obtained at selected depths in the borings by pushing two types of samplers; standard 3-inch O.D. Shelby tubes and smaller 2.5 inch O.D. Shelby tubes. Some samples were also collected by driving a 2-inch I.D. Modified California sampler or a 2-inch O.D. standard split-spoon sampler as shown on the logs in Appendix A.

The samples were delivered to the URS geotechnical laboratory in Pleasant Hill, California, for further visual examination and testing. Logs of Borings were prepared based on the field logs, the visual examination in the laboratory, and the laboratory testing results, and are presented in Appendix A.

A detailed description of the procedures used to drill the borings and to obtain soil samples is given in Appendix A (Geotechnical Drilling and Sampling Program).

2.2.2 Test Pit

One test pit, URS-TP-1, was excavated just east of the levee as shown on Figure 2. The test pit was excavated using a John Deere 310D Backhoe. The excavated depth was approximately 13.2 feet to an elevation of approximately -5 feet NGVD 29. The test pit was excavated to determine the depth to bedrock as well as to assess degree of difficulty in excavating the rock to the depth of the base of the proposed tide gate. A log of test pit URS-TP-1 is presented in Appendix A.

A detailed description of the procedures used to excavate the test pit is given in Appendix A (Geotechnical Drilling and Sampling Program)

2.3 LABORATORY TESTING

Representative soil samples obtained from the exploratory borings were tested in our Pleasant Hill geotechnical laboratory in order to evaluate their engineering properties for use in the analyses. The following laboratory tests were performed on selected soil samples:

- Water content (ASTM D2216)
- Dry density (ASTM D2850)
- Unconfined compressive strength (ASTM D2166)
- Atterberg limits (ASTM D4318)
- Grain size and hydrometer analyses (ASTM D422)
- Consolidation Tests (ASTM D2435)
- Unconsolidated Undrained Triaxial Tests (ASTM D2850)

The results of the laboratory tests except the results of the consolidation tests are summarized in the logs of borings at the corresponding sample depths as shown in Appendix A and on Table B-1 in Appendix B. The results of the consolidation testing are summarized in Table B-2. Detailed laboratory test results are presented in Appendix B (Geotechnical Laboratory Test Results).

3.1 SITE DESCRIPTION

The Site is located between Waterfront Road and the Carquinez Strait and is comprised of an approximately 5,500 feet long segment of the north-flowing Slough (see Figure 1). The Slough continues south under Waterfront Road to McNabney Marsh. The Slough collects discharged treated wastewater from the Mt. View Sanitary District. The Site is surrounded by marsh lands on the east from the Carquinez Strait to Waterfront Road. The entire Slough has been dredged repeatedly in the past. Dredge spoils were placed along both banks of the Slough in linear piles of unknown thickness. Currently, portions of the piles rise to +5 feet NGVD-29, and some remain without vegetation. The unvegetated portions range in size and are located directly adjacent and parallel to the Slough. . The Slough bottom elevation resides at approximately –3.5 feet NGVD-29, and the slough embankments rise to +5 feet NGVD-29.

The Site has been subdivided into the “north slough” and the “south slough,” which are separated by a tide gate and levee located approximately 2,400 feet south of the Carquinez Strait. The following describes the north slough and south slough.

The north slough is approximately 2,400 feet long, generally 30 to 40 feet wide and extends from Carquinez Strait to the tide gate. The tide fluctuates approximately 6 feet from mean high to mean low tide. At low tide, minimum water depth is approximately 2 feet and most of the north slough embankments have a vertical face approximately 3 to 5 feet high. The east and west embankment of the north slough are densely vegetated. The embankments become inundated due to their low elevation, and the water line extends up to approximately 20 feet into the vegetation. Several tributary sloughs intersect the eastern embankment of the Existing Slough. A large marsh occupies the area east of the north slough, which is which is virtually inaccessible by vehicle. Only a small portion is accessible by vehicle near the Rhodia polishing pond on the eastern side of the tide gate.

The south slough is approximately 3,100 feet long, and extends from the tide gate to Waterfront Rd. The south slough width averages approximately 50 feet, ranging from less than 10 feet wide at the southern end near the culvert under the railroad embankment to approximately 60 feet near the tide gate. Under the current tide gate function, the influence by tides in the south slough is minor. The water level varies by approximately ½ foot. A fresh-water seasonal marsh lies adjacent to the west bank of the south Slough to the south of the property fence line. Slightly higher, grassy areas adjacent to and on Zinc Hill to the east of the south Slough are grazed by cattle. The slopes of Zinc Hill lie within 150 feet of the east bank. An overgrown vehicle track exists along the base of Zinc Hill, on Shore Terminal’s property.

The tide gate is located approximately 2,400 feet from Carquinez Strait. The approximately 60-foot-long, concrete tide gate, which was reconstructed in 1998, is configured to allow fresh water from the south side to flow through the gate during low tidal periods. The gate prevents the southerly flow of marine water to the south Slough.

The embankments in the immediate vicinity of the tide gate are up to 5 to 10 feet high and are protected with rip rap up to 2 to 3 feet in diameter. The existing levee extends generally eastward 900 feet from the tide gate to the base of Zinc Hill. The crest elevation of the existing levee ranges from 4 feet NVGD to 7 feet NVGD. The marsh on the south side of the levee has subsided to a level approximately 3.5 feet lower than the marsh on the north side of the levee.

3.2 GEOLOGICAL SETTING AND SEISMICITY

The Rhodia, Inc facility is located within the Coast Range Geomorphic Province. The Coast Ranges are characterized by north to northwest trending elongated mountain ranges and intervening valleys. The site lies on undifferentiated Cretaceous siltstones and shales that strike north to northwest and dip to the west or southwest respectively. Dibblee (1981) interpreted these marine sediments to form the eastern limb of the northwest-southeast trending Martinez (or Pacheco) syncline. This syncline is the eastern-most fold of a fold belt bounded on the east by the steeply dipping strike-slip, right-lateral Concord Fault and on the west by the Southhampton and Franklin thrust faults. These rocks are exposed to the east on Zinc Hill and to west on Rhodia's property. Measurements in outcrops on Zinc Hill indicate that the bedrock is striking approximately northwest and dipping 53° to 65° towards the southwest (Sims et al., 1968 and Dibblee, 1981). There are no outcrops exposed where the levee abuts against Zinc Hill.

Surficial deposits cover the majority of the site. These deposits include Quaternary Bay Muds and interbedded peats overlying older Bay Muds and isolated sand lenses. The Older Bay Mud overlies approximately 10 feet of alluvium, which overlie bedrock.

The closest known active fault to the project site is the Concord fault, which is a northwest-striking right-lateral strike-slip fault of the San Andreas system. It extends for 18 km along the eastern margin of Ygnacio Valley, from the northern slopes of Mount Diablo to Suisun Bay. The Concord fault is the closest mapped Holocene fault to the project site and is zoned under the Alquist-Priolo Special Studies Zone Act. The project site is located approximately one mile outside of the Special Studies Zone. The Maximum Credible Earthquake (MCE) for the Concord fault is Mw 6.9 (maximum magnitude).

3.3 SUBSURFACE CONDITIONS

Subsurface conditions were characterized using the Unified Soil Classification System (USCS) to provide a basis for geotechnical design parameters and recommendations presented in this report. Characterization criteria included: (1) conditions found in the drill holes and test pit; (2) conditions reported during previous field studies conducted in the area by others; (3) results of laboratory testing performed on selected samples obtained during exploratory drilling; and (4) our understanding of the geological setting of the site.

3.3.1 Subsurface Stratigraphy

The project site can be divided into three areas for the purpose of describing subsurface conditions; the areas north and south of boring 52 (approximately Station 40+50 of the existing alignment) as shown on Figure 3 and the area where the existing levee abuts Zinc Hill. Soil and rock profiles adjacent to the existing slough and along the existing levee shown on Figure 3 and Figure 4 were developed by adding the borings drilled by URS and Harza (1996a) to a profile reported in Mark Group (1989). The interpolated profiles are only approximations of the stratigraphy between boring locations. Only at the boring locations should the profiles be considered to be accurate to the extent implied on the boring logs.

Index properties for the soils described below are based on laboratory results on samples from URS-1, URS-2, URS-3, and URS-4 and are summarized with respect to depth below the ground surface on Figures 5 to Figure 12.

3.3.1.1 North Area

North of boring 52 (Figure 3), the site is characterized by a depositional interface between Peyton Slough and the Carquinez Strait where organic clays and silts are interbedded with sandy clays, silts, and lean to fat clay. One boring, URS-3 located at the mouth of Peyton Slough, was drilled by URS in the north area as shown on Figure 2. The subsurface conditions found at this location are very soft silty clay (Bay Mud) interbedded with silty to clayey sands. A 5-foot thick layer of organic silty clay with woody peat fragments was encountered 45 feet below mudline. Bedrock was not found in the boring. A previous investigation, encountered 115 feet of sediment over bedrock at boring 7 (Figure 3).

Results of laboratory tests of samples from URS-3 show that the Bay Mud near the surface has a total unit weight of 87 pcf with a moisture content of 107 percent and a dry unit weight of 42 pcf. Below the near surface clay, the total unit weight varies between 110 pcf and 119 pcf and the dry unit weight varies between 72 pcf and 90 pcf, except in the organic silty clay layer where the total unit weight is 97 pcf and the dry unit weight is 53 pcf. The moisture content varies between 31 and 53 percent with the exception of the organic silty clay layer, which has a moisture content of 84 percent. The liquid limit varies from 29 to 55 and the plasticity index varies from 7 to 37. The lower values are for the sandy clays and silts. The lower bound undrained shear strength increases with depth from 20 psf to 250 psf and the upper bound undrained shear strength increases from a range of 100 psf to 440 psf at a depth of 17.5 feet to approximately 700 psf at a depth of 50 feet.

Consolidation tests in URS-3 do not show a definitive trend and may be affected by the presence of abundant sea shells in some of the samples. For the three samples tested, the overconsolidation ratio (OCR) ranges between 1.9 to greater than 10. The compression ratio (CR) ranges from 0.13 to 0.28 apparently decreasing with depth. Similarly, the recompression ratio (RR) ranges from 0.015 to 0.028. The coefficient of consolidation

($c_{v\ (virgin)}$) ranges between 0.0096 ft^2/day to 0.0218 ft^2/day and the coefficient of consolidation ($c_{v\ (recompression)}$) ranges between 0.0173 ft^2/day to 0.891 ft^2/day.

3.3.1.2 South Area

South of boring 52 (Figure 3), the subsurface conditions are characterized by a layer of very soft organic silt and clay (young Bay Mud) interbedded with a layer of very soft organic clay and silt with peat. Underlying the young Bay Mud is a layer of interbedded soft to medium stiff, medium to high plasticity clays and silts (older Bay Mud). Underlying the older Bay Mud is a layer of alluvium that is made up of medium stiff to stiff lean clays and silty sands. Bedrock underlies the alluvium.

Borings drilled in the southern area by URS included URS-1, URS-2, URS-4, URS-5, and URS-6 as shown on Figure 2. Two of the borings, URS-1 and URS-4, were drilled along the general alignment of the existing slough at proposed temporary trestle bridge locations. Boring URS-1 was drilled approximately 1400 feet south of the tide gate and URS-4 was drilled approximately 350 feet north of the tide gate. A third proposed temporary trestle bridge location was not explored due to the proximity of Harza (1996) boring EB-5 to that location. URS-2, URS-5, and URS-6, were drilled 140 feet, 45 feet, and 5 feet east, respectively, of the fence at the east end of the levee

Based on the field investigation and laboratory testing conducted by URS and information from previous investigations, the young Bay Mud varies in thickness from 25 to 32 feet below the existing slough and is up to 38 feet thick below the existing levee. The older Bay Mud varies between 10 to 50 feet in thickness below the existing slough and is up to 22 feet thick below the existing levee. The alluvium overlying bedrock varies between 5 and 15 feet in thickness. Bedrock was encountered in URS-1 at 96 feet bgs and the depth to bedrock increased from 13 feet bgs at URS-6 to 77 feet bgs at URS-2. Based on past investigations (Figure 3), the bedrock surface below the slough rises from approximately 110 feet bgs near boring 52 to 30 feet bgs at boring 4 and then drops back down to 100 feet bgs at boring 3.

Young Bay Mud – Samples of young Bay Mud in URS-1, URS-2, and URS-4 had a total unit weight varying between 67.5 pcf and 80 pcf with a dry unit weight varying between 15 pcf and 31 pcf. The water content generally decreases from nearly 350 percent from the top of the layer to approximately 150 percent at the bottom of the layer. Correspondingly, the void ratio decreases from greater than 7 to approximately 4 from top to bottom. The liquid limit varies from 87 to 168 and the plasticity index varies from 47 to 74. The liquidity index, a measure of soil behavior during shearing, is above one. The lower limit undrained shear strength varies between approximately 100 psf and 150 psf, except at the bottom of the layer where lower-bound strengths are as low as 20 psf. The upper limit undrained shear strength generally varied between approximately 200 psf and 300 psf. The overconsolidation ratio (OCR) varied between 7.1 and 10.7 near the surface and decreased to between 2.1 and 2.7 near the bottom. The compression ratio (CR) varied from 0.27 to 0.31 near the surface and increased to between 0.33 to 0.56 with depth with the higher values occurring in peats. Similarly, the recompression ratio (RR) varied from 0.017 to 0.046 near the surface and generally increased to 0.031 to 0.087

with depth. The coefficient of consolidation ($c_{v\ (virgin)}$) varied between 0.0075 ft^2/day to 0.0153 ft^2/day and the coefficient of consolidation ($c_{v\ (recompression)}$) varied between 0.0032 ft^2/day to 0.115 ft^2/day.

Older Bay Mud – Samples of older Bay Mud from borings URS-1, URS-2, and URS-4 had a total unit weight varying between 90 pcf and 114 pcf with a dry unit weight varying between 49 pcf and 87 pcf. The water content generally decreases from nearly 86 percent from the top of the layer to approximately 35 percent at the bottom of the layer. Correspondingly, the void ratio decreases from approximately 2.5 to approximately 1.5 from top to bottom. One Atterberg test found the liquid limit was 49 and the plasticity index was 26. The lower limit undrained shear strength increases with depth from approximately 100 psf to 280 psf. The upper limit undrained shear strength increases with depth from 250 psf to 1200 psf. The overconsolidation ratio (OCR) varied between 1.7 and 1.8 for the two consolidation tests performed on Older Bay Mud. The compression ratio (CR) varied from 0.2 to 0.22 and the recompression ratio (RR) varied from 0.027 to 0.047. The coefficient of consolidation ($c_{v\ (virgin)}$) varied between 0.040 ft^2/day to 0.046 ft^2/day and the coefficient of consolidation ($c_{v\ (recompression)}$) varied between 0.0145 ft^2/day to 1.05 ft^2/day.

Alluvium – Samples of alluvium in URS-1, URS-2, and URS-4 have a total unit weight of approximately 126 pcf with a water content of about 24 percent and a dry unit weight of about 101 pcf. One Atterberg test found the liquid limit was 42 and the plasticity index was 22. The lower limit undrained shear strength is approximately 800 psf and the upper limit is approximately 1200 psf.

3.3.1.3 Levee Abutment at Zinc Hill

At the east end of the levee where it ties into Zinc Hill, URS-TP-1, encountered weathered shale and sandstone, which was able to be excavated to the final depth of the test pit with a John Deere 310D Backhoe. The bedrock surface sloped gradually downward from east to west along the length of the test pit below 4 to 7 feet of fill and clay. At the bottom of the test pit the rock had to be broken with the end of the bucket to be excavated. Bedding of the rock was not able to be observed in the test pit. It is anticipated that the bedding orientation and dip is similar to that described in Section 3.2. Fragments of the shale excavated from the test pit and exposed on the ground surface were observed to slake.

3.4 GROUNDWATER

Based on hydrogeologic studies performed as part of the Peyton Slough Remedial Design and reported in the Remedial Design Report (URS, 2002), groundwater is generally within a few inches of the ground surface on both sides of the levee. Groundwater was not measured in the borings drilled for this investigation. During the excavation of TP-1 approximately 1-inch of seepage accumulation on the bottom of the test pit was observed in one hour. The groundwater level is above the bottom of the proposed tide gate structure and dewatering will be required during construction.

4.1 GENERAL

It is our understanding that the new tide gate structure will be approximately 40 feet wide with sufficient length to pass through the levee and will have a central headwall configuration similar to the existing tide gate structure. The base of the structure will be at approximately elevation –7 feet NVGD and the top of the structure will be at elevation 7.5 feet NVGD.

Based on the results of our geotechnical studies and engineering analyses, it is our recommendation that the new tide gate structure be located at the east end of the levee at the base of Zinc Hill. This location is generally suitable for the construction of the proposed structure, with the understanding that there is seismic risk that includes relatively high levels of ground shaking that need to be accounted for in the design of the project.

4.2 GEOLOGIC HAZARDS

4.2.1 Fault Rupture

The site is located outside of the Alquist-Priolo Special Studies Zone for the closest known active fault as described in Section 3.2. Therefore, the hazard associated with surface fault rupture is low.

4.2.2 Ground Shaking

A design basis ground motion corresponding to the peak ground acceleration (pga) for the event with 10 percent chance of being exceeded in 50 years was selected in accordance with the procedure described in the Uniform Building Code (UBC, 1997). The United States Geological Survey National Seismic Hazard Mapping Survey (http://geohazards.cr.usgs.gov/eq/), provides the pga for the 10 percent in 50 year event on a grid with resolution to the nearest 0.1° latitude and longitude. The calculated weighted-average pga for the four nearest grid point locations to the levee, which is located at approximately 38.0307° latitude and 122.1111° longitude is 0.73g.

4.2.3 Liquefaction

Because of the classification of the material at the site, the probability of liquefaction at the site is low.

4.2.4 Landslide

There are no discernable landslide features on the west side of Zinc Hill. The landslide potential for Zinc Hill is considered to be low.

4.3 TIDE GATE FOUNDATION

Three alternative tide gate locations were considered in our analyses as follows:

Alternative 1 - 140 feet west of the east end of the levee

Alternative 2 - 40 feet west of the east end of the levee

Alternative 3 - The east end of the levee at the base of Zinc Hill

The foundation condition at Alternative 1 is soft Bay Mud varying in thickness from between 52 feet and 72 feet. At Alternative 2, the Bay Mud varies in thickness from 8 feet to 24 feet. Finally, the foundation condition at Alternative 3 is weathered sandstone and shale.

Mat and pile foundations for the tide gate structure were considered for Alternative 1 and Alternative 2. A mat on grade was considered for Alternative 3. The following assumptions were made in our evaluation.

- Construction of the tide gate structure at Alternative 1 or Alternative 2 will require widening of the levee from a 10 foot crest width to 15 feet to provide sufficient access to construct a tide gate structure similar to the existing structure. The height of the levee would be increased at the same time to a final crest elevation of 7.0 feet NVGD.

- Widening of the levee will be towards the south. It is our understanding that widening of the levee to the north will not be allowed due to permitting restrictions.

- Access for construction of the tide gate at the east end of the levee (Alternative 3) is available from the base of Zinc Hill either from Terminal Shores property or from a temporary access road along the new slough alignment.

4.3.1 Settlement Analyses

A tide gate structure constructed on a mat foundation similar to the existing tide gate at Alternative 1 or Alternative 2 will subside as soft, compressible sediments underlying the structure consolidate under additional loads placed on them. Consolidation of the underlying sediments will occur as a result of raising and widening the levee, which will occur prior to construction of the tide gate.

Additional levee fill was assumed to have a total unit weight of 125 pcf, which is similar to the materials placed during raising of the levee in 1999, (Harza, 1999a and 1999b.) Widening of the levee was assumed to be towards the south. Settlement calculations were based on one-dimensional primary consolidation theory.

For Alternative 1, primary consolidation of the underlying sediments was estimated to be approximately 60 inches at the center of the raised and widened levee and approximately 12 inches at the northern edge of the levee. Differential settlement from north to south is exaggerated due to levee widening placing additional stress eccentrically towards the south rather than symmetrically on both sides of the levee. Differential settlement from east to west was estimated to be approximately 6 inches due to the steeply sloping bedrock surface below the unconsolidated sediments.

A similar estimate of settlement was made at Alternative 2 where the soft underlying soil is not as thick. At the center of the raised and widened levee it is estimated that approximately 14 inches of settlement will occur at the center of the structure and at the northern edge of the levee approximately 6 inches of settlement will occur. Differential settlement from east to west is estimated to be approximately 15 inches due to the steeply sloping bedrock surface below the unconsolidated sediments.

The calculated primary settlements stated above do not account for the following factors:

- Reduction of vertical stress on the subsurface soils that would occur when a portion of the levee is excavated for the tide gate structure.

- Preconsolidation of the subsurface soils resulting from raising of the levee prior to the start of construction of the Peyton Slough Remedial Project. It is our understanding that Contra Costa Mosquito and Vector Control District (CCMVCD) may raise the levee to elevation 7 feet NVGD sometime in 2002 prior to such construction.

Both factors will result in some reduction of total primary consolidation. The magnitude of the reduction would require modeling of consolidation in three dimensions and an understanding of the timing of the raising of the levee by CCMVCD in relation to construction of the tide gate.

Secondary consolidation during the life of the project was not calculated. However, substantial secondary consolidation should be anticipated based on the subsurface conditions present at the site.

4.3.2 Mat Foundation

4.3.2.1 Alternative 1 and Alternative 2

Based on the estimated ultimate and differential settlement during the life of the structure we do not recommend that a mat foundation be used for the tide gate structure at these locations.

4.3.2.2 Alternative 3

Based on foundation conditions at Alternative 3, we recommend a net allowable bearing capacity of 6,000 psf.

4.3.3 Pile Foundation

If Alternative 1 or Alternative 2 are selected, piles driven to bear directly on bedrock should be used to support the structure. Based on foundation conditions at Alternative 1 it is estimated that pile lengths for a 40 foot wide gate structure will vary from 60 feet to 80 feet in length from east to west. Pile lengths for Alternative 2 are estimated to vary from 10 to 25 feet. Harder driving through approximately 10 feet of stiffer clay overlying bedrock should be anticipated for Alternative 1. Approximately five feet of harder driving through stiffer clay overlying bedrock should be anticipated for Alternative 2. Pile design should consider "downdrag" forces due to the anticipated settlement discussed in 4.3.1.

4.3.4 Recommended Location For Tide Gate Structure

Based on our analyses, the tide gate structure can be placed on piles at Alternative 1 or Alternative 2 or it can be constructed on bedrock at Alternative 3. We recommend that the tide gate structure be founded on bedrock at the east end of the existing levee for the following reasons:

- Settlement of Bay Mud below a pile supported tide gate will require careful design of a cutoff to prevent seepage below the tide gate structure. A bedrock foundation surface alleviates this concern.

- Stability of the widened levee for Alternative 1 or Alternative 2 construction was not analyzed. However, based on the results discussed in Section 4.4, a widened levee stability may not be stable during construction.

- Construction costs should be reduced with the elimination of piles, elimination of widening of the levee, and potential simplification of the structure.

4.4 LEVEE STABILITY

Access for construction of the tide gate at Alternative 3 can occur either along the existing levee or from temporary access roads constructed south or north of the levee. The levee was analyzed to determine allowable surcharge loads during construction.

South of the existing levee the ground elevation of the marsh is approximately 0 feet N.G.V.D. A borrow pit having a bottom elevation of approximately –4 N.V.G.D. is located approximately 25 feet south of the toe of the levee. North of the levee, the ground elevation is approximately +3.5 feet N.V.G.D. Groundwater is present at the ground surface at the toe of the levee on both the upstream and downstream sides. The analyses assume that the levee has been raised to a final elevation of 7 feet N.V.G.D. 29 and that both the upstream and downstream sides are sloped at 3H:1V.

Levee fill materials were assumed to have similar properties to the existing fill materials. A total unit weight of 125 pcf was used based on results of testing of the upper lift of the levee after it was raised in 1999 (Harza, 1999). An undrained shear strength of 800 psf was assumed for the levee material based on results of direct shear testing for material from the borrow pit compacted to 87 percent (ASTM D1557) reported in Harza (1996c).

The total unit weight used in the analysis for young Bay Mud with peat on which the levee is founded was 70 pcf, corresponding to the average unit weight of samples from the upper 20 feet of Boring URS-2. An undrained shear strength of 100 psf at the surface, which linearly increases with depth at a rate of 5 psf/foot was selected, corresponding to results from undrained shear strength testing in the upper 20 feet of the young Bay Mud with peat.

Slope stability analyses were performed using UTEXAS3 (Wright, 1992), a microcomputer program that uses the limit equilibrium method of slices. The results indicate that the raised levee has a factor of safety against sliding equal to 1.2 at the end of construction without considering surcharge loading due to construction activity. Construction surcharges equivalent to 150 psf on the levee reduce the factor of safety to 1.1. The factor of safety against sliding will increase with time as the underlying soft Bay Mud consolidates.

Based on this analysis it is not recommended that the levee be used for heavy construction traffic without modifications that might include resloping the side south of the levee, removal of a portion of the levee to reduce soil loads, and filling of the borrow pit. Prior to such modifications, stability analyses should be performed using the modified geometry and the anticipated surcharge loads due to construction equipment and traffic.

4.5 DESIGN EARTH PRESSURES AND RESISTANCE

The magnitude of lateral pressure exerted against an object by soil depends on several factors. They include the effective weight of soil, the nature of the soil (friction angle and cohesion), the depth of the soil, the slope of the surface of the soil, the direction of movement (soil moves toward the object, or object moves toward the soil), the amount of movement and any additional loads imposed on the soil behind the structure. For design purposes the "earth pressure coefficient" or "equivalent fluid pressure" is usually provided for determining the lateral earth pressure.

4.5.1.1 Static Lateral Earth Pressures

Soils acting against the walls of the tide gate structure will be a combination of levee fill and Bay Mud on the west side and compacted levee fill on the east side. Where cohesive materials are acting against structures at rest lateral earth pressure conditions should be used. We recommend that the lateral earth pressure be computed as an equivalent fluid weighing 81 pcf above groundwater and 41 pcf below groundwater. This pressure is based on horizontal backfill and no back-face wall batter and does not in include groundwater pressure acting against the wall.

Additional lateral earth pressure generated during construction activities as surface live loads should be used in design of the tide gate structure walls. The surface live loads were based on a Caterpillar CP-433C roller operating 2 feet or more from the wall. The induced lateral pressure distribution can be linear approximated as an inverted trapezoid as a function of depth as having an uniform pressure of 350 psf minus an equivalent fluid weight of 25 pcf.

The pressures shown above are based on the assumptions that backfill soils will be compacted to 85 percent of maximum dry density as determined by the ASTM D-1557 test method and that compaction of soil within two feet of the wall will be performed by hand-operated or other lightweight equipment to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.

If other loading conditions are to be considered in the vicinity of the walls, URS should be contacted to provide additional recommendations.

4.5.1.2 Dynamic (Seismic) Lateral Earth Pressures

For cast-in-place reinforced concrete (CIPRC) walls in non-yielding, below grade structures, potential seismic loading should be considered. Wood (1973) developed the dynamic thrust, ΔPe_n, acting on smooth rigid non-yielding walls as:

$$\Delta Pe_n = k_h \gamma H^2$$

where k_h is horizontal ground acceleration divided by gravitational acceleration and H is the height of wall below the ground surface. For lateral earth pressure calculations, the horizontal acceleration acting against the retaining wall was reduced from 0.73g by 25 percent to 0.55 g.

The pressure diagram for this dynamic component can be approximated as an inverted trapezoid with stress decreasing with depth and the resultant acting at the distance 0.53 times the structure height above the base of the structure. The magnitude of the resultant is:

$$\Delta Pe_n = 58.4\ H^2$$

This dynamic component should be added to the at-rest static pressure for seismic loading conditions.

For cast-in-place reinforced concrete (CIPRC) walls that yield, potential seismic loading should be considered. Seed and Whitman (1970) developed the dynamic thrust, ΔPe_y, acting on yielding walls as:

$$\Delta Pe_y \sim (3/8)\ k_h\ \gamma\ H^2$$

where k_h is horizontal ground acceleration divided by gravitational acceleration and H is the height of wall. For lateral earth pressure calculations, the horizontal acceleration acting against the retaining wall was reduced from 0.73g by 25 percent to 0.55 g.

The pressure diagram for this dynamic component can be approximated as an inverted trapezoid with stress decreasing with depth and the resultant acting at the distance 0.6 times the structure height above the base of the structure. The magnitude of the resultant is:

$$\Delta Pe_y \sim 25.8\ H^2$$

This dynamic component should be added to the at-rest static pressure for the seismic loading conditions.

4.5.2 Lateral Resistance

An ultimate friction angle of 35 degrees is recommended for sliding resistance at the interface between the structure and the rock foundation. Development of sliding resistance is dependent upon the foundation bearing directly on undisturbed bedrock that has been properly cleaned and prepared.

4.6 DESIGN FLOOR SLAB

The concrete slab for the tide gate should be placed directly on bedrock that has been properly cleaned and prepared. Weathered shale exposed in the excavation will slake with exposure to repeated wetting and drying cycles. It is recommended that the foundation surface be blown with high pressure air prior to concrete placement to remove slaked bedrock.

4.7 DRAINAGE

Positive drainage should be established by sloping surfaces away from the structure, both during and after construction.

4.8 EARTHWORK

Based on the proposed structure plan and the existing grade, we anticipate that up to 14 feet of excavation is required for tide gate structure construction. The maximum excavation is located at the crest of the levee just downstream of centerline of the levee. The fill is anticipated primarily in the excavated areas.

Based on the boring information, it is anticipated that excavation of the on-site soil and bedrock can be accomplished with conventional earth moving equipment. The existing on-site levee material and Bay Mud material excavated from the new slough alignment should generally be suitable for reuse as backfill with proper moisture conditioning. Soil and bedrock materials excavated from Zinc Hill are not suitable for backfill behind the tide gate structure walls.

4.8.1 Site Preparation

It is recommended that prior to the start of earthwork operations, the existing ground surface be initially prepared for grading by removing all vegetation, debris, other organic material, and non-complying fill (if present). Voids created by the removal of such material should be properly backfilled and compacted.

Excavation for the tide gate structure should be checked for loose, soft, deleterious or otherwise objectionable materials by the geotechnical engineer or his technical representative. Any disturbed, loose, soft, deleterious or otherwise objectionable materials encountered beneath foundation grade should be removed and replaced with mass concrete.

4.8.2 Open-cut Excavation

The slope selected for cut and fill slopes depends on many factors including: soil type, soil moisture content, acceptable factors of safety for static and seismic conditions, design life, ease of construction, susceptibility to erosion, and final landscaping treatment. *The slopes provided in this report are for design purposes only – the contractor should rely upon his own methods to determine and maintain safe and stable slopes during construction subject to his particular construction procedures and those subsurface conditions more fully exposed during construction.* Unless otherwise specified, slope recommendations provided in this report assume horizontal ground surfaces at the head and toe of the slope and do not include the effects of surcharge loadings (e.g., excavation spoil piles, or heavy equipment). *All excavations should comply, as a minimum, with the Occupational Safety and Health Administration's (OSHA) construction standards for excavations*. All excavations should be observed by qualified personnel.

We recommend the slope for temporary open-cut excavation into bedrock for tide gate construction not exceed 1:1 (horizontal to vertical). Permanent excavation slopes that will be exposed on Zinc Hill should not exceed 2:1 (horizontal to vertical).

Open cut excavation to the west of the proposed tide gate structure location is not recommended if the excavation exposes Bay Mud in the side of the cut slope. Our analyses indicates such an excavation is not stable if the levee fill on top of the bay mud cracks perpendicular to the dam axis.

Permanent excavation slopes for the new slough alignments upstream and downstream of the tide gate structure should not exceed 2:1 (horizontal to vertical). This recommendation is based on the condition that there are no construction live loads imposed along the side slopes of the new slough and does not consider seismic forces.

4.8.3 Support of Excavations

Internally braced temporary support for excavation may be required on the west side of the tide gate structure depending on its final location. Considerations for the design of such temporary support should include maintaining the integrity of the existing levee to seepage after construction. We recommend that such temporary shoring consist of internally braced sheetpile driven to bedrock. Sheetpile driven adjacent to the west wall of the structure can be used as the outer formwork for wall construction. Sheetpile driven through the levee and underlying Bay Mud should provide protection against the formation of a seepage path through the levee due to deformation of soft Bay Mud aginst the sheetpiles and an increased seepage path length across the levee formed by the sheetpile corrugations.

Lateral earth pressures previously described and lateral pressures equal to one-half of construction surcharges due to equipment, traffic, or other loads should be used in the design for the internally braced shoring. The Contractor should be responsible for designing the temporary support system. URS should review the design of the excavation support system.

4.8.4 Seepage Controls

Seepage potential along or under the structure is anticipated to be low due to small head differences from upstream to downstream, length of the seepage path along the structure, and low permeability of levee fill and Bay Mud on the west side of the structure and levee fill against the east side of the structure.

Cutoff walls should be constructed at the downstream and upstream end of the structure to prevent seepage below the structure and to provide protection against undercutting of the structure.

4.8.5 Levee Fill

Levee fill should be clean Bay Mud free from debris, vegetation, roots, other deleterious material, and excess moisture.

We anticipate that on-site material dredged from the new slough alignment is suitable for reuse as levee fill after it has been appropriately moisture conditioned.

4.8.6 Compaction

Levee fill placed against the tide gate structure should be compacted to a minimum compaction of 85 as determined by ASTM D 1557.

We recommend that earth materials be placed and compacted as near to the optimum moisture content (as described by ASTM D 1557) as feasible. This practice generally facilitates achievement of minimum compacted densities. Fill materials should be placed in 8-inch loose lifts. In confined areas, lift thickness should be reduced, not exceeding 6-inch loose lifts. Adjacent concrete work should be allowed to obtain designed 28-day strength prior to backfilling. Large compaction equipment should not be allowed within 5 feet of any concrete work.

5.1 LIMITATIONS

This geotechnical study has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The foundation recommendations presented in this report are developed exclusively for the proposed tide gate structure described in this report and are not valid for other locations and construction in the project vicinity. The recommendations made in this report are based on the assumption that the subsurface soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory borings. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made.

We should be informed of any changes that are made in the assumptions mentioned in this report so that additional recommendations may be given, if necessary. We recommend that URS has the opportunity to review those portions of the project plans and specifications that are affected by the recommendations made in this report to verify that the intent of our recommendations are properly incorporated into the construction documents. We also recommend that a geotechnical engineer be retained to observe the foundation excavation, earthwork, and the foundation construction to recognize differing site conditions.

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A.1 FIELD EXPLORATION

Six exploratory borings were drilled for this study to depths of 16.5 to 100.75 feet to explore the subsurface conditions at the project site. Borings URS-B1, URS-B2, URS-B3 and URS-B4 were drilled between December 20, 2001 and December 27, 2001 and borings URS-B5 and URS-B6 were drilled between January 29, 2002 and January 30, 2002 under the supervision of an URS geologist. Borings URS-B1, URS-B2, URS-B5 and URS-B6 were drilled using a track-mounted CME 850 drilling rig and borings URS-B3 and URS-B4 were drilled using a barge mounted D-25 skid rig both of which were owned and operated by PC Exploration, Inc. of Rocklin, California.

One test pit was excavated to a depth of 13.2 feet BGS using a John Deere 310D Backhoe owned and operated by PC Exploration, Inc.

A.2 SOIL SAMPLING

Sampling Method

Soil samples were obtained at selected depths in the borings by advancing the sampler into the soils at the bottom of the borehole. The following type of sampling equipment was used:

- Standard 3-inch OD Shelby tubes and Small 2.5-inch OD Shelby tubes. The shelby tubes were attached to a drill rod and lowered through the augers to the bottom of the boring and then pushed 28 inches. The samples were left to sit in the hole for two minutes to allow the clay to swell and stick in the sampler.

- Modified California Sampler - 2-inch I.D., 2-½-inch O.D., split-barrel sampler equipped with four thin brass tube liners, three 4 inches long and one 6 inches long.

  The sampler was threaded to fit a cutting shoe on one end and a check-valve connection at the other end. After the borehole was drilled to the specified depth, the sampler was lowered down through the auger stem to the bottom, seated, and then driven into the soil with a 140-pound hammer falling 30 inches for each blow. The hammer was controlled by an autohammer system. The number of hammer blows required to advance the sampler each of the three successive 6-inch increments was counted in the field. The number of blows required to advance the sampler the last 12 inches was recorded as the penetration resistance (blows-per-foot).

After drilling and sampling, the boreholes were backfilled with lean cement grout and cuttings. Excess drill cuttings were drummed and labeled non-hazardous, cinder contaminated material and left in the Chemite building on site.

Sample Handling

Soil recovered from the Modified California sampler was retained in the thin brass liners; when the sampler was brought to the surface, the liners were taken from the sample barrel and sealed at both ends with plastic caps and vinyl tape. Shelby tubes collected in the borings were handled in the same manner.

A.3 LOGS OF BORINGS AND TEST PITS

The soil samples and cuttings were examined and classified in the field as the drilling proceeded. The samples were later taken to our geotechnical laboratory in Pleasant Hill, California, for further examination and testing. Preliminary visual soil classifications were made in accordance with the Unified Soil Classification System and verified by further inspection of the samples in the laboratory and by testing. Logs of borings were prepared from the field logs and laboratory test data.

The logs of borings show the soil classifications (according to the Unified Soil Classification System) of subsurface strata encountered, locations where soil samples were obtained, type of sampler used, sampling resistance, and the results of several of the laboratory tests.

The exploratory test pit was excavated using a John Deere 310D Backhoe. A URS geologist supervised the excavation and soil description efforts. During excavation, the exposed soil was examined and classified. Soil classifications were made in the field in accordance with the Unified soil Classification System. A log of the test pit was prepared and is presented in the Log of Test Pit, Figure A12.

A geotechnical testing program was performed in the laboratory to measure the index and engineering properties of representative soil samples obtained from the exploratory borings. The testing was performed by URS Laboratory in Pleasant Hill, California.

The following laboratory tests were performed on selected soil samples:

- Water content (ASTM D2216)
- Dry density (ASTM D2850)
- Unconfined compressive strength (ASTM D2166)
- Atterberg limits (ASTM D4318)
- Grain size and hydrometer analyses (ASTM D422)
- Consolidation tests (ASTM D2435)
- Unconsolidated Undrained Triaxial Tests (ASTM D2850)

A summary of the laboratory test results is given in Tables B-1 and B-2. The test result sheets are attached generally in order by boring and test number.

Consolidation Tests

Table B-2 presents the results of the consolidation tests. For each test, the table summarizes the moisture content, unit weight, in-situ effective stress, maximum past pressure, overconsolidation ratio (OCR), the compression ratio (CR), recompression ratio (RR), coefficient of consolidation during virgin compression ($c_{v(virgin)}$) and coefficient of consolidation during recompression ($c_{v(rec)}$).

The in-situ effective stress was calculated by assuming a unit weight based on laboratory tests. The maximum past pressure was estimated using the Casagrande construction and the end-of-primary consolidation curve. The overconsolidation ratio (OCR) was computed by dividing the maximum past pressure by the in-situ effective stress. The compressibility parameters, CR and RR, were obtained from the end-of-primary consolidation curve. The compression ratio (CR) and recompression ratio represent the vertical strain per log cycle of stress during virgin compression and recompression, respectively. Using the square root of time method, the vertical coefficient of consolidation (c_v) was calculated for virgin compression ($c_{v(virgin)}$) and for recompression ($c_{v(rec)}$).