Feasibility Report
Appendix C – Engineering Designs and Costs

Attachment 1
Conveyance Facilities
Geotechnical and Seismic Evaluations

Attachment 1A – Geotechnical and Seismic Evaluations TM (March 2007)

Attachment 1B – Geotechnical Considerations Addendum (October 2015, Addendum to Attachment 1A)

Los Vaqueros Reservoir Expansion Investigation
Final Feasibility Report

February 2020
DRAFT Technical Memorandum: Geotechnical and Seismic Evaluations

Los Vaqueros Expansion Investigation
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Los Vaqueros Expansion Investigation

prepared by

Mid-Pacific Region

with

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PROFESSIONAL CERTIFICATION

Draft Technical Memorandum
Geotechnical and Seismic Evaluations
Los Vaqueros Expansion Investigation
U.S. Bureau of Reclamation
March 02, 2007

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The findings of this report are based on the readily available data and information obtained from public and private sources. Additional studies (at greater cost) may or may not disclose information which may significantly modify the findings of this report. In the event that there are any changes in the nature, design or location of the project, or if additional subsurface data are obtained or any future additions are planned, the conclusions and recommendations contained in the report will need to be reevaluated by MWH in light of the proposed changes or additional information obtained.

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(To be stamped and signed on the final copy)

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1 INTRODUCTION

1.1 PURPOSE OF DOCUMENT

This document presents the geotechnical and seismic evaluations to be used in the development of feasibility-level designs of conveyance elements related to the Los Vaqueros Expansion Investigation (LVE). It is one of several technical memoranda that provide elements of the feasibility-level concepts for review prior to incorporation into a feasibility report. Review comments will be addressed and incorporated, as appropriate, into the Draft and Final Feasibility Reports.

1.2 BACKGROUND

Los Vaqueros Reservoir watershed and associated dam and facilities are located in the California coastal foothills west of the Delta and east of the San Francisco Bay Area, the central and south Delta, and service areas of Bay Area water agencies. Contra Costa Water District (CCWD) owns and operates the existing Los Vaqueros Reservoir, 100,000-acre-foot off-stream surface storage facility.

The potential expansion of Los Vaqueros Reservoir is one of five surface storage projects recommended for additional study by the CALFED Bay-Delta Program. The U.S. Department of Interior, Bureau of Reclamation, Mid-Pacific Region (Reclamation) was authorized to conduct feasibility-level investigation for potential expansion of Los Vaqueros Reservoir, and is the lead agency under the National Environmental Policy Act (NEPA). The California Department of Water Resources (DWR) is the State agency funding the Environmental Impact Statement/Environmental Impact Report (EIS/EIR). Contra Costa Water District (CCWD) is the responsible agency under the California Environmental Quality Act (CEQA). The ongoing Los Vaqueros Expansion (LVE) feasibility studies are being conducted by Reclamation, DWR, and CCWD in close coordination.

Currently, four pipeline elements have been identified for the LVE: Delta-Transfer Pipeline, Transfer-LV Pipeline, South Bay Aqueduct Pipeline (LV-SBA) Pipeline, and Transfer-Bethany Pipeline (Figure 1). The Old River Pipeline and Transfer Pipeline are existing pipelines and currently in operation, but alternative plans under consideration may involve installation of additional pipelines (i.e., Delta-Transfer Pipeline and Transfer-LV Pipeline) parallel to the existing pipelines. The LV-SBA Pipeline and the Transfer-Bethany Pipeline would be completely new pipelines used to deliver project supplies to beneficiaries.
1.3 OBJECTIVE

The feasibility-level geotechnical and seismic evaluations will provide basic information necessary for the feasibility-level engineering design work to proceed and develop. The feasibility-level evaluation is based upon existing and available data, and industry practice and professional judgment; no new field investigations were conducted.

1.4 SCOPE OF WORK

The feasibility-level geotechnical evaluations are performed for the identified pipeline routes and pumping plant, storage and other project facility locations only. Geotechnical and seismic evaluations for the Los Vaqueros Dam and associated inlet and outlet facilities shall be completed by others. The feasibility-level evaluations are based upon existing and available data; no new field investigations were conducted.

The following activities are part of the geotechnical and seismic evaluations:

1) Data collection and review - Collect and review available regional and site geotechnical investigation data and seismic investigation data, geologic maps, stereo-aerial photographs and hazardous materials data along and adjacent to the planned conveyance routes

2) Preliminary evaluation of geologic conditions based on existing data; identification of data gaps and the potential for such gaps to affect feasibility

3) Provisions of preliminary design and construction considerations based on the preliminary geotechnical evaluations

4) Field exploration and laboratory testing plan - An exploration program is developed that fills current geotechnical/seismic data gaps identified during data collection and review; and describes additional information required in areas of known unfavorable conditions

5) Preparation of this draft Geotechnical Technical Memorandum based on the findings from this work.

1.5 INFORMATION REVIEWED

To prepare this technical memorandum, the following project and technical reports were reviewed:


1.6 ORGANIZATION OF DOCUMENT

Section 2 of this memorandum discusses the regional geologic setting and seismicity in the vicinity of the project location. Section 3 reviews the previous geotechnical data relevant to the project site. Section 4 summarizes the geotechnical and geological findings and concerns associated with the project. Section 5 provides preliminary design and construction considerations. Section 6 provides recommendations for further geotechnical investigations and study. Finally, references cited in the memorandum are listed in Section 7, followed by the table and figures for the memorandum.
2 GEOLOGIC AND SEISMIC SETTING

2.1 REGIONAL GEOLOGY

The proposed project study area is located just west of the boundary between the Great Valley and the Coast Ranges, near the southern portion of the Sacramento-San Joaquin Delta (Figure 2) (Dibblee 1980). The Great Valley is a major northwest-trending depositional basin located west of the Sierra Nevada province. The proposed project is located on the eastern portion of the Diablo Range region, consisting of relatively steep northwest-trending ridges and valleys.

The Great Valley is mainly composed of Mesozoic and early Tertiary marine sedimentary rocks overlain by non-marine sedimentary rocks (varying in thickness up to 3,000 feet) deposited during the late Tertiary and Quaternary periods (MARK 1992). The proposed site is located at the confluence and outlet of the Sacramento and San Joaquin rivers, which is underlain primarily with alluvial and Holocene and Pleistocene aged river deposits.

The Diablo Range is that of a generally west-northwest-trending up-arched fold, obliquely sliced by major northwest trending faults creating several structural blocks. The geology of the northern Diablo range consists of Cretaceous and Eocene marine sandstone, conglomerate and shale, buried by flat lying sediments of the delta. The bedrock–offsetting faults mapped in the foothills do not exhibit geomorphic evidence of Holocene or late Pleistocene fault displacement (WCC 1989a).

2.2 ANTICIPATED SITE GEOLOGIC UNITS

According to the mappings by Graymer et al. (1994) and Helley and Graymer (1997) (Figure 3), geologic units of bedrock and soil significant to the areas of planned pipelines consist of the following, from the oldest to youngest:

- **Unit C Sandstone (Kcu and Kcm)**: bedrock of Late Cretaceous age, consists of sandstone and contains interbeds of siltstone, shale, and conglomerate. The sandstone is medium grained, brown to gray, biotite rich wacke with some mudstone rip-up clasts.

- **Unit D Sandstone (Kd)**: bedrock of Late Cretaceous age, medium to coarse grained, light gray, clean sandstone. Grains include quartz, feldspar, and biotite. Spherical weathering is common. In places interbedded with fine to medium grained, biotite and muscovite bearing wacke with mudstone rip-up clasts. Sandstone beds form packages up to 30 feet thick with 3 to 6 feet of interbedded siltstone and mudstone.

- **Domengene Sandstone (Td)**: the bedrock of Eocene age, consists predominantly of weakly cemented quartz sandstone and shale with minor mudstone and conglomerate. This formation is quarried locally for high-quality sand at the Kellogg Creek Sand Quarry.
Geologic and Seismic Setting

- **Kreyenhagen Shale (Tk)**: the bedrock of Oligocene age, consists of well-consolidated brown claystone that has minor interbeds of diatomaceous siltstone and sandstone.

- **Neroly Sandstone (Tn)**: the bedrock of Miocene age, consists blue, volcanic-rich, shallow marine sandstone, with minor shale, siltstone, tuff, and andesitic conglomerate.

- **Alluvial Fans and Fluvial Deposits (Qpaf)** (Pleistocene): brown dense gravelly and clayey sand or clayey gravel that fines upward to sandy clay. These deposits display various sorting and are located along most stream channels. They are overlain by Holocene deposits on lower parts of the alluvial plain, and incised by channels that are partly filled with Holocene alluvium on higher parts of the alluvial plain. Maximum thickness is unknown but at least 150 feet.

- **Alluvial Fan and Fluvial Deposits (Qhaf)** (Holocene): alluvial fan deposits are brown or tan, medium dense to dense, gravelly sand or sandy gravel that generally grades upward to sandy or silty clay. Near the distal fan edges, the fluvial deposits are typically brown, never reddish, medium dense sand that fines upward to sandy or silty clay.

- **Basin Deposits (Qhb)** (Holocene): very fine silty clay to clay deposits occupying flat-floored basins at the distal edge of alluvial fans adjacent to the bay mud.

- **Peat and Peaty Mud (Qhpm)** (Holocene): water saturated peat and mud deposited in tidal wetlands in response to the post-Wisconsin rise in sea level. The unit consists in large part of the decomposed remains of roots and rhizome, particularly tule, common reed, salt grass and cattail.

- **Natural Levee Deposits (Qhl)** (Holocene): loose, moderately to well-sorted sandy or clayey silt grading to sandy or silty clay. These deposits are porous and permeable and provide conduits for transport of ground water. Levee deposits border stream channels, usually both banks, and slope away to flatter floodplains and basins.

- **Topsoil** (Late Quaternary age): all of the geologic soil formations listed above are covered with 3 to 10 feet of clay-rich, moderately to highly expansive topsoil developed from weathering and alteration of the underlying earth materials deposited in last few thousand years. The topsoil developed on underlying claystone is more highly expansive than that developed on diatomaceous siltstone.

- **Artificial Fill (af)** (Historic): man made deposits of various materials and ages. Some are compacted and quite firm, but fills made before 1965 are nearly everywhere not compacted and consist simply of dumped materials.

### 2.3 SEISMIC HAZARDS

#### 2.3.1 Faulting and Historic Seismicity

The Los Vaqueros Expansion project is located within the seismically active boundary between the Pacific and North American tectonic plates, hence strong ground motion is highly likely during the lifetime of the project. The proposed pipelines are located on the eastern margin of the bay area seismicity as shown in Figure 4. This figure was produced based on the data contained...
in the computer program EZ-FRISK (Version 7.1). Along the boundary of the Great Valley and Coast, contemporary faulting is concentrated along a buried system of thrust and reverse faults. These faults include the San Andreas, Hayward, Calaveras, Concord-Green Valley, Greenville, and Great Valley. Active northwest trending faults, west of the Diablo Range include the San Andreas (principal component of the Pacific-North American plate boundary), Hayward, and Rogers Creek Fault. Located within the northern Diablo Range, active right-lateral strike-slip fault zones include Calaveras, Concord-Green Valley and Greenville. Some of these faults are also summarized in Table 1.

According to Earth Science Associates (ESA 1992), the Antioch, Davis, Brentwood, Kellogg, Vaqueros, and Camino Diablo faults (Figure 5) are older, minor faults, which were generally truncated and offset by the younger northwest-trending major faults mentioned previously. These older, smaller faults are associated with the uplift of Mount Diablo. The following discussions for a few nearby faults as shown in Figure 5 are provided based on the research conducted by Woodward Clyde Consultants (WCC 1989a) and Geomatrix (1992).

**Antioch Fault:** The Antioch Fault was evaluated by the California Division of Mines and Geology and is no longer considered active (WCC 1989a). However, the activity of this fault is believed to be linked to the Davis Fault to the south (Geomatrix 1992).

**Davis Fault:** Although the Davis Fault may have had displacement in the middle and possibly late Quaternary time, none has occurred in the latest Pleistocene or Holocene time (Geomatrix 1992). This is based on trenching investigations completed by California Department of Water Resources and a reassessment of the activity of the Alquist-Priolo zoned Antioch Fault by the California Division of Mines and Geology (Geomatrix 1992).

**Brentwood Fault:** No evidence of faulting was observed in 11 trenches conducted by California Department of Water and Resources in the late 1970s nor in the one completed by Woodward Clyde Consultants in 1989. And no geomorphic features indicative of Holocene or late Quaternary fault displacements were observed along the Brentwood fault in either the analysis of aerial photography or field mapping.

**Kellogg Fault:** Although the Kellogg Fault has strong geomorphic expression in the vicinity of Kellogg Creek, analysis of aerial photographs or ground inspection performed by Woodward Clyde Consultants indicate that no significant fault displacements have occurred in Holocene or Pleistocene time. Several trenches were excavated across the Kellogg Fault by California Department of Water Resources in 1978 and three were conducted by Woodward Clyde Consultants (WCC 1989a); no evidence of faulting was revealed within colluvial or surficial soil units.

**Vaqueros Fault:** Woodward Clyde Consultants exposed the Vaqueros Fault in two trenches where it was characterized by a nearly vertical band of gouge and crushed rock about two feet thick (WWC 1989a). As with the other faults in the vicinity of the proposed project site Woodward Clyde Consultants did not observe geomorphic features indicating Holocene or late Quaternary fault displacements during analysis of field mapping or aerial photographs (WWC 1989a). No evidence of faulting was observed in the colluvial units and soils exposed above the Vaqueros Fault in two trenches completed by Woodward Clyde Consultants.
Camino Diablo Fault: The Camino Diablo Fault is the closest source to the planned project site. It passes along the eastern margin of the Transfer Facility and transverses the Old River Pipeline, and is approximately 3.5 miles long. This fault has been evaluated on several aerial photographs taken between 1957 and 1991 and no geologically recent fault displacements are believed to have occurred. However, this fault may affect the bedrock units and their structural orientation due to the fact that it may pass beneath a portion of the proposed project.

Midland Fault: This major, north trending subsurface structure is approximately 60 miles long. There is no indication of mid Tertiary or younger deposits having been displaced by the Midland Fault (Geomatrix 1992). However, it is believed that the Midland Fault is part of the active Coast Ranges-Sierra Nevada Boundary Zone and was included in the seismic analyses completed by Geomatrix.

The historical seismicity in the vicinity of Los Vaqueros Expansion Project is also shown in Figure 4 based on the data contained in the computer program EZ-FRISK. It appears that there were no major historic earthquakes within about 10 miles of the project site (Transfer Facility). The nearest major historic earthquake occurred about 15 miles west of the Transfer Facility with a magnitude of Richter scale 5.5 to 6.5.

2.3.2 Future Seismic Ground Motions

A preliminary probabilistic seismic hazard assessment was performed to determine peak ground acceleration (PGA) at different return periods. The calculations were performed using the computer software EZ-FRISK (Version 7.1) developed by RISK Engineering, Inc. The location used for the seismic hazard assessment is the Transfer Facility, with approximate coordinates of 37.883 (lateral) and -121.688 (longitudinal). Several faults that were included in the preliminary analysis have been listed in Table 1. The table was created based on the 2002 national Seismic Hazard Maps Fault Parameters. It should be noted that this list includes faults that were not included in previous hazard studies completed by Woodward Clyde Consulting (WCC 1989a) and Geomatrix (1992).

Three different attenuation relationships were selected in the calculations, which were Abrahamson and Silva (1997), Sadigh et al. (1997), and Boore-Joyner-Fumal (1997) methods. The results are presented in the form of a Uniform Hazard Spectra as shown in Figure 6. The table below also summarizes the mean peak ground acceleration for three different return periods. The results are mainly applicable to bedrock materials, as defined by the three calculation methods. For soil materials, additional information such as soil layer thickness, soil density, and water table, etc., would be required to estimate the seismic ground motions. Note that the results are preliminary and may be subject to review. It is recommended that further analyses be performed to determine which faults should be included in the analyses, and a deterministic seismic hazard assessment be completed to develop a maximum credible earthquake (MCE).
2.3.3 Liquefaction and Dynamic Settlement

Liquefaction describes a condition in which saturated soil loses shear strength and mobilizes as a result of increased pore pressure induced by strong ground shaking during an earthquake. The principal effects of liquefaction on structures are settlements (both total and differential), loss of foundation support, and for buried structures to rise buoyantly. Soils most susceptible to liquefaction are saturated, loose sandy soils and some granular silts. The potential for an earthquake with intensity and duration characteristic capable of promoting liquefaction is a possibility during the design life of the project.

Liquefaction susceptibility for the area of the Los Vaqueros Expansion Project is shown in Figure 7, which was based on the Liquefaction Maps and Information, a digital database produced by the Association of Bay Area Governments (ABAG 2006). It can be seen from the figure that the liquefaction susceptibility for the east portion of the Delta-Transfer Pipeline alignment is “High”, while that for the west portion of the Delta-Transfer Pipeline alignment and most of the Transfer-LV Pipeline alignment are “Moderate”. The liquefaction susceptibility for the alignment of the LV-SBA Pipeline and Transfer-Bethany Pipeline are essentially “Low” to “Very low”.

The liquefaction susceptibility shown in Figure 7 was defined based on peak ground accelerations (PGA) necessary to trigger liquefaction and estimated ground-water levels (ABAG 2006; Knudsen et al. 2000). Typical PGAs necessary to induce liquefaction of sandy or silty materials range from as low as 0.1g for saturated artificial fill and modern stream channel deposits, to much greater than 0.5g to 0.6g for saturated latest Pleistocene alluvial deposits. On the basis of the liquefaction failures that occurred during past earthquakes, it is anticipated that at least 80 percent of future liquefaction failures will take place in areas judged to have High or Very High susceptibilities; 20 percent or less of future liquefaction will take place in areas judged to be Moderate and Low, and that less than 1 percent will take place in areas judged Very Low (Knudsen et al. 2000).

The soils susceptible to liquefaction (which is generally below ground water table) may settle considerably during or after a ground shaking cause by an earthquake. In addition, loose, sandy soils or non-plastic silts above groundwater may consolidate or settle when subjected to cyclic application of loads from ground shaking. The liquefaction potential of soils and magnitude of the settlements can be evaluated once more site-specific soil and geotechnical data become available.

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>Mean Peak Ground Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>475</td>
<td>0.35</td>
</tr>
<tr>
<td>975</td>
<td>0.45</td>
</tr>
<tr>
<td>2,475</td>
<td>0.57</td>
</tr>
</tbody>
</table>
2.4 REGIONAL GROUNDWATER

The depth to ground water in areas underlain by Holocene alluvial, estuarine, and beach sediment is generally less than 10 to 20 feet throughout most of the area (Knudsen et al. 2000). Ground water is deeper beneath topographically higher parts of the landscape (for example, uplifted and dissected Pleistocene alluvial fans), and closer to the surface of topographically lower parts of the landscape (for example, Holocene basins and terraces). Pronounced seasonal changes in ground-water levels occur in the region, with variations as large as tens of feet. Small, isolated alluviated valleys and pockets within the bedrock hills may have fairly shallow ground water, generally less than 10 to 15 feet. Soils characteristic of wet environments have indicated shallow ground-water levels. Ground water beneath uplifted marine terraces can be deeper than 40 feet, except where water is perched. Ground water beneath coastal dunes that forms or mantles hills can be as deep as 50 to 100 feet, equivalent to the elevation of the hills.

In general, it is estimated that the ground water table is relatively shallow for the area of the Delta-Transfer Pipeline alignment (may be as less as 5 feet below ground surface); and relatively deep for the most area of the Transfer-LV Pipeline alignment, and the whole areas of LV-SBA Pipeline and Transfer-Bethany Pipeline alignments (larger than 20 feet below ground surface).
3 PREVIOUS GEOTECHNICAL DATA

Previous geotechnical investigations have been conducted only for the areas along the alignments of the existing Transfer Pipeline and Transfer Facility, and the existing Older River Pipeline and Older River Intake Facility. In this section, the available geotechnical data was reviewed and briefly summarized.

3.1 TRANSFER PIPELINE AND FACILITY

ESA performed extensive geotechnical investigations for the entire Transfer Pipeline alignment at design level in 1992 (ESA 1992a and 1992b); and performed some additional investigations for two cross sections (ESA 1994). WCC performed geotechnical investigations for the Los Vaqueros Dam at design level, which contained some data applicable to part of the Transfer Pipeline near the dam (WCC 1989b and 1989c). In addition, Geomatrix also performed geotechnical subsurface investigations for the Transfer Facility at design level in 1992 (Geomatrix 1992). These investigations involved soil boring, rock coring and test pit excavation in the field, and various tests in the laboratory. In this section, the available geotechnical data associated with the Transfer Pipeline and Transfer Facility was reviewed and summarized following the geotechnical terrain segments that have been identified and described by ESA (1992a and 199b). The pipeline routes were divided into series of segments, each with similar geologic, geomorphic and geographic characteristics.

3.1.1 Upper Kellogg Creek Alluvium

The Transfer Pipeline originates from Los Vaqueros Dam in the Kellogg Creek canyon, extends to the north in general, and terminates at the Transfer Facility (Figures 1 and 7). Much of the first geotechnical segment, the Upper Kellogg Creek Alluvium segment, is covered by grasses and the entire area is used as pastureland by Vaquero Farms.

A parallel series of Great Valley sequence sandstone and shale strata form the hill slopes and underlie all younger deposits of this terrain segment. Strata strikes consistently west-northwest and dips moderately to the northeast at angles measuring 23° to 47°. The rock encountered in the exploratory borings and test pits consisted of sandstone, siltstone, and claystone or shale. Hardness of the sandstone ranged from friable to hard, its strength ranged from friable to strong, and weathering ranged from deep to fresh. Siltstone encountered had friable to low hardness, friable strength, and was moderately weathered. Claystone or shale was described as having a soft to low hardness, being friable to weak in strength, and deeply to moderately weathered.

Alluvial terrace deposits cover the valley floor crossed by this terrain segment and alluvial fans formed at the mouths of several of the tributary canyons. The alluvial sediments are generally clayey but contain lenses and discontinuous layers of sand and gravel. Silty, sandy, and gravelly clay have been identified in the exploratory excavations. These clays are of low to medium plasticity and have consistencies ranging from firm to hard. The silty and clayey sands in the
alluvium were found to be loose to medium dense. Boulders and cobbles were encountered extending to a depth of 7 feet. Thickness of the alluvium along the pipeline segment was identified ranging from 2 feet to 22 feet.

Two large landslides have developed on opposite sides of the canyon within a shale unit about 1,000 feet downstream of the dam. The landslide on the west side of the canyon appears to be dormant while parts of the other landslide seem to be more recently active. Renewed movement of either landslide would create new deposits above the pipeline but should not disturb the pipeline itself unless Kellogg Creek was diverted from its present channel and exposed the pipe by erosion elsewhere.

About 2 to 4 feet of sandy clay to silty silt topsoil covers the upper Kellogg Creek segment. Locally, this has accumulated into thick deposits of colluviums found to depths as great as 6 to 11 feet in some test pits. The soil locally was moderately expansive.

Ground water was encountered in some of the exploratory excavations. Water flowed into excavation from a shallowest depth of about 6 feet in one site. Water was also encountered in three borings at depths of 15 to 19 feet.

### 3.1.2 Middle Kellogg Creek Alluvium

Kellogg Creek flows through a fairly broad, north- to northwest trending valley before passing through a mountain gap and onto the floor of San Joaquin Valley. The valley floor is flat and grass-covered except for a band of riparian vegetation following the incised stream.

Cretaceous-age bedrock lies at depth beneath this alluvial valley but was not encountered in the exploration. The overlying alluvium was found to consist of clay, silt and sand. Clay was identified in each of the auger borings and found to have a consistency ranging from soft to very stiff. Loose to medium dense silt was encountered in one boring and test pit. Loose to dense sand and clayey sand was encountered at depth in two borings.

Ground water was measured in about half of the exploration sites excavated along this segment and was found in the depth range of 13 to 19 feet. A marshy area indicating near surface ground water was observed in one test site. The bed of Kellogg Creek contains tules and standing water in some locations where it is crossed by the pipeline.

At about mid of this segment, depth of scour was found to be 2 feet based on the lowest occurrence of recent alluvial materials exposed in the test pit. The Creek is incised 8 to 10 feet in a narrow, steep-walled notch. Farther north, the stream bed is about 15-feet wide and incised about 6 feet at the northern bank and about 16 feet on the south.

### 3.1.3 Camino Diablo Hillfront

The hillslopes bordering the Lower Kellogg Creek alluvial valley have been given the name “Camino Diablo Hillfront.” The hills here are of low relief, generally less than 200 feet, and as they form the headward end of a rather broad, flat valley, are inclined east, north and west. Slopes are generally covered with grass and a few scattered trees. Several sand quarries have been excavated into these hills.
Eocene-age sediments of the Meganos Formation, Nortonville Shale, and Markley Formation underlie the area. Strata are inclined to the northeast at about 30° to 40° and strike northwestward. The topographic grain follows this orientation with northwest aligned sandstone ridges separated by parallel, less resistant, shale- or claystone-floored valleys. A fault crosses the hillfront at the Kellogg Creek gap, but it is not active and is concealed beneath the alluvial valley fill.

Two borings were drilled in the segment and encountered deeply weathered bedrock beneath 4 to 5 feet of sandy clay soil. The bedrock had characteristics similar to those of a soil. It was composed mostly of medium dense to very dense clayey or silty sandstone with lesser amounts of very stiff to hard claystone. Another boring drilled in the eastern part of the hillfront encountered clayey sandstone and sandy claystone. This material also had soil-like characteristics with the sandstone being loose to medium dense and claystone being firm. Ground water probably lied below a depth of 20 feet in this area.

3.1.4 Lower Kellogg Creek Alluvium

This geotechnical segment covers the north part of the Transfer Pipeline and the Transfer Facility. The segment consists of a broad, flat, valley floor formed where the outwash plain of Kellogg Creek merges with San Joaquin Valley floor. The smooth, grass-covered hillslopes bordering this segment were composed of Tertiary-age sediments of the Meganos, Domengine, and Northville formations. Several terrace levels representing different episodes of erosion or deposition were identified on the valley floor.

In this segment, the uppermost layer of alluvium was found to be silty or sandy clay of low to medium plasticity. The clay was brown in color and had a firm to very stiff consistency. The base of the clay layer was found ranging from 5 to more than 30 feet in depth. Generally, a silty, clayey or gravelly sand was found beneath the overlying clay horizon. The sand varied from loose to very dense conditions, generally increasing with depth. The sandstone bedrock was weathered to a friable, soil-like character.

One test pit was excavated in Kellogg Creek and encountered ground water flowing at about 1 gallon per minute (gpm) at a depth of 10 feet. The active creek deposits at this site, consisting of sand, gravel and rubble, were found to be about 2.3-feet thick.

3.2 OLD RIVER PIPELINE AND FACILITY

The MARK Group performed geotechnical investigations for the Old River Pipeline (MARK 1993, 1994a and 1994b) and Old River Intake Facility (MARK 1992). The investigation involved various field methods including boring, cone penetrometer testing, vane shear testing and test pit excavation. The available geotechnical data was reviewed and is summarized in this section.

The predominant topographic feature of the Old River Pipeline alignment is the very flat, cultivated fields traversed and paralleled by irrigation ditches and dirt roads. The pipeline
alignment crosses two major irrigation ditches, one of which is approximately 15 feet wide and 8 feet deep.

Based on the geotechnical investigation data, the soils encountered along the alignment of Old River Pipeline and Facility can be generally divided into three distinct strata, i.e. the levee fill, the peaty or topsoil materials, and the alluvial soils. The levee fills were mainly encountered near the Older River Intake Facility. The soils consisted of a one-foot thick sandy gravel road surface underlain by interbedded silty sands, clayey silts and silty clays which varied from loose/soft conditions to dense/stiff conditions.

Peaty soils were found in many areas of the pipeline alignment, while top soils were generally seen along the pipeline alignment. These soils were encountered within 15 feet below ground surface. The organic material was generally moist to wet, low plasticity, very soft or loose, with occasional clay and silt lenses.

The alluvial soils which underlay the peaty material or topsoil consisted of soft to medium stiff silty clay or clayey silt with intermittent thin layers of silty or clayey sand. The sand layers were generally of very loose to medium dense conditions. The alluvial soils were at least 50 feet thick along the pipeline and more than 100 feet thick near the Old River Facility, as revealed by the depths of the drillings. The soils generally became stiffer or denser with increasing depth. Ground water was relatively shallow in many areas of the pipeline alignment, which would require dewatering during construction.
4 FINDINGS AND CONCLUSIONS

4.1 GEOTECHNICAL AND GEOLOGIC CONCERNS

In summary, the main geotechnical and geological conditions expected to influence planning, design, and construction include the following,

- The Los Vaqueros Expansion Project is located within the seismically active boundary between the Pacific and North American tectonic plates, hence strong ground motion is highly likely during the lifetime of the facility. The planned pipeline alignments will also cross several minor faults which are believed to be relatively no-active. Project features should be designed for the anticipated ground motions.

- The majority of the Delta-Transfer Pipeline will be located in soils with high liquefaction potential. Design of the pipeline and facilities should encompass such ground conditions.

- The proposed pipeline alignments will span several different geologic units. The Delta-Transfer Pipeline will be mainly located in the alluvial soils that are relatively weak and with shallow ground water table. Other proposed pipelines, depending on the depth of the pipes, will mostly encounter relatively firm soil or bedrock capable of providing direct support for the proposed pipelines and structures.

- The majority of the soils expected within the proposed pipeline depths are silts, sands and clays. Coarse gravel and cobble deposits are may exist near the riverbank or where a pipeline crosses a small watercourse.

- The corrosion potential of the soils typically ranges moderately corrosive to corrosive to almost all types of buried steel, iron, and concrete for areas of the alignments of the Transfer-LV Pipeline and the Older River Pipeline, based on the previous data. Corrosion protection measures are typically required for the buried pipes and facilities. However, soil corrosivity for the LV-SBA Pipeline and Transfer-Bethany Pipeline alignments are unknown and subject to investigation and evaluation.

- Trenchless construction method such as microtunneling and jack and bore may be used in the project area. Cohesionless soils, cemented soils, perched groundwater, bore pit shoring, bore length, and alignment/profile tolerance are the most common issues that affect the selection of an appropriate trenchless method.

- The presence of relatively hard cohesive soils throughout the project area (except for the Delta-Transfer Pipeline and northern part of the Transfer-LV Pipeline) may affect or preclude installation of driven sheet or soldier pile temporary shoring systems. Predrilling to install driven shoring systems in cemented materials may be performed near the project area.
• Cohesionless sand deposits throughout the project area are prone to caving in open excavations. Crossings of existing utilities and construction of additional pipeline near the existing one (such as the case for the Delta-Transfer Pipeline and the Transfer-LV Pipeline) may also expose granular bedding and loose trench backfill materials, and possibly trapped subsurface water. These conditions are expected to affect the selection and design of temporary shoring systems for the project, as positive restraint shoring systems and top down installation will be required where these soils exist within the excavation depths.

• The surficial soils throughout the project area (particularly for the Delta-Transfer Pipeline and part of the Transfer-LV Pipeline) may have moderate to high expansion potential and are not suitable for structure backfill or structure support. The underlying silts and sands are typically non-expansive and suitable for use as non-expansive engineering fill or structure backfill.

• It is likely for isolated or perched groundwater conditions to develop for extended periods above the finer grained or cemented, less permeable soil layers, especially in the vicinity of irrigated farmlands, landscaped areas, drainage ditches, canal crossings, and nearby creeks. In addition, the isolated layers or relatively clean and slightly silty sands and gravels encountered at various locations may provide a conduit for perched groundwater. Such flows may adversely affect excavation stability and result in wet/unstable working conditions in the bottom of the trench.
5 PRELIMINARY DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 ANTICIPATED EXCAVATION CONDITIONS

The native soils within the project area consist primarily of clays, silts, and sands. Shallow excavations within these materials for most of the areas (except for the Delta-Transfer Pipeline alignment) may stand near vertical for short periods of time. Excavations extending deeper than 5 feet where access by construction personnel is required should be sloped back or shored in accordance with applicable regulations. Flatter slopes and/or shoring may be required where are encountered along the slope face. Layers of relatively clean sands are expected throughout the site, especially at depths in excess of about 15 feet. These materials may be subject to caving in open excavations.

Cemented soils may exist throughout the area and represent moderately excavation difficulty. Excavation for the proposed pipelines and subsurface structures are anticipated to be possible using conventional, moderate- to large-sized excavators. Light rubber-tired backhoes generally have difficulty excavating cemented materials in the area.

5.2 SHORING

Trench shields, speed shoring, driven sheet piles, soldier piles, or other forms of shoring may be used where needed throughout the project as appropriate for the soil conditions. Driven sheet pile or similar systems may not be suitable where cemented materials are present along the alignment. Pre-drilling could be used to help facilitate installation. If soldier piles are to be used, continuous lagging will be required to retain any potentially caving cohesionless materials. Shoring systems should be designed to provide positive restraint of trench walls in pavement areas, adjacent to parallel utilities, existing structures, and in other areas where trench wall movement are objectionable. Internally braced or similar shoring systems may be required in these areas to limit deformations of trench walls and the surrounding soils. Discontinuous shoring systems will not be suitable in cohesionless soils such as silty and poorly graded sands, and where groundwater is encountered. In areas where groundwater is not encountered, shoring systems such as hydraulic shores with plywood, trench boxes, and/or other shoring systems may also be utilized provided California Occupational Safety and Health Administration (Cal/OSHA) regulations are met.

5.3 TEMPORARY DEWATERING

Excavations that extend below perched groundwater levels may need to be dewatered. Dewatering of narrow trench excavations may be possible by directing inflow to a sump where
water can be removed by a pump. In some instances, soft trench bottom conditions may result from the introduction of perched groundwater into excavation.

Depending on the depth of excavation below groundwater, soil conditions encountered along the excavation face and slope inclination, caving or sloughing of excavation slopes is likely within the vicinity of seepage areas. Sloughing or caving of excavation slopes could endanger personnel working within or adjacent to the excavation as well as nearby equipment, structures, or other existing improvements. The Contractor should be aware of the potential for caving and take appropriate precautions to protect the safety of site personnel as well as the integrity of the excavation slopes and any existing nearby structures or other improvements.

Since temporary dewatering will impact and be dependent on construction methods and scheduling, we recommend the Contractor be solely responsible for the design, installation, maintenance and performance of all temporary dewatering systems. Where shoring is employed, the dewatering system should be compatible with the type of shoring to be used.

### 5.4 TRENCH BOTTOM CONDITIONS

In general, the materials expected at the proposed pipeline trench bottom depths should be suitable for pipe and subsurface structure support. If loose or disturbed materials are present at trench bottoms, the material should be over-excavated to firm material and replaced with engineered fill or additional bedding material.

### 5.5 TRENCH BACKFILL

Trench backfill depths for the project are anticipated to be about 10 to 15 feet. The materials anticipated to be encountered within the proposed trench areas consist of clayey fill materials and native clays, silts and sand. The majority of these materials are expected to be suitable for trench backfill throughout the project except for the near-surface soils in the area of the Delta-Transfer Pipeline alignment.

Consideration should be given to construction staging where the excavated materials can be stockpiled and processed to meet requirements for engineered fill and trench backfill. The near-surface soils in the area are typically very wet and unstable during the winter, spring and early summer since infiltration water often becomes perched above a shallow impermeable layer. Significant processing to dry these materials for use as trench backfill will be required. If processing areas are not available near the excavation sites, the excavated materials may need to be hauled off and imported or otherwise suitable soils brought in for backfill.

### 5.6 PIPE BEDDING AND INITIAL BACKFILL

It is anticipated that pipe bedding and initial backfill materials will consist of clean crushed rock and/or Controlled Density Fill material that meets the relevant requirements. Crushed rock bedding and initial backfill should be placed and compacted in a manner to eliminate voids.
beneath the pipe. In general, the lift thickness prior to compaction should not exceed one foot. Shovel slicing of initial backfill materials in pipe haunch areas is recommended.

5.7 TRENCH DAMS

Perched groundwater tends to accumulate within the relatively permeable crushed rock bedding and initial backfill materials within pipeline installation in the area. It may be prudent to consider placement of lean concrete or Controlled Density Fill slurry collars within the pipe zone to limit the migration of water along the pipeline alignment through the porous bedding and shading materials. The trench dams should be about one foot or more in thickness and engage the trench wall soils and overlying intermediate backfill materials. Trench dams may be placed at regular intervals along the pipelines. We generally recommend placing trench dams at intervals of no more than 1,000 feet or near each manhole or junction structure location.

5.8 FILTER FABRIC ENVELOPE

To reduce the potential for migration of the fine-grained native soils and intermediate backfill into crushed rock bedding and initial backfill, a filter fabric should be placed between native or fill soils and the crushed rock creating an envelope around the rock. Filter fabric should be laid-out and overlapped according to the manufacturer’s recommendations.

5.9 UBC SEISMIC DESIGN PARAMETERS

Structures should be designed for lateral force requirements as set forth in the International Building Code (2006). Parameters for input to seismic modeling should be provided on the basis of information contained in the geotechnical design report.

5.10 SOIL CORROSION

Corrosion Engineering and Research Company (CERC) conducted soil corrosivity investigation and analyses for several previous projects in the Los Vaqueros Expansion Project site area (CERC 1988 and 1992). The projects investigated included the existing Old River Pipeline and Intake Facility, and Transfer Pipeline and Facility. The information was reviewed to provide general information regarding soil corrosivity. Soil samples from these projects were subjected to chemical analysis for the purpose of corrosion assessment. The samples were tested for pH, resistivity, soluble sulfates, and soluble chlorides.

Based on the field testing results, CERC concluded that the soils were considered to be “severely corrosive” to dielectric coated steel pipe, “corrosive” to mortar coated welded steel pipe and “moderately corrosive” to concrete pipe for the alignments of Delta-Transfer Pipeline and Intake Facility, and Transfer-LV Pipeline and Facility. The soils at the Old River Facility and Transfer Facility were also considered to be “corrosive” to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated iron or steel.
Generally, corrosivity by these test results may only be an indicator of potential soil corrosivity for the samples tested. Other soils found on the site may be more, less, or a similar corrosive nature.

Due to the high potential corrosivity of the soils in the project area, we recommend that a competent corrosion engineer be retained to evaluate the corrosion potential of the project site, to propose improvements, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project.

5.11 TRENCHLESS CONSTRUCTION

It is anticipated that some form of trenchless construction will be necessary if environmentally sensitive areas need to be crossed. Trenchless methods could also be considered for crossings of major intersections, thoroughfares, drainage canals, etc. Due to the interlayered soil conditions in the area, mixed face conditions are expected to be common for trenchless applications. Tunneling equipment will likely need to be capable of excavation within moderately cemented soils (SPT N values of 35 to 100) and be steerable. However, it is possible that the soil profiles in some areas will be conducive to jack and bore applications if groundwater is not present. Jack and bore applications are generally best suited for crossings that are less than about 150 feet long due to alignment and grade control issues.
6 RECOMMENDATIONS FOR FUTURE INVESTIGATIONS AND STUDY

This section describes the proposed geotechnical investigation required for detailed design. The available information is sufficient for the feasibility-level design.

6.1 FIELD RECONNAISSANCE AND EXPLORATION

For detailed design, a comprehensive field reconnaissance should be performed for the project site. This would include visualization of the ground conditions, slopes and site topography; reconnaissance-level mapping of surface geologic conditions and topography, and possible mapping of subsurface geologic conditions based on exposed surface or outcrop. Particular attention will be paid to look for evidence of potentially thick, weak soil layers. Evidence for potential slope instability, including ground surface anomalies such as cracks, bulges, etc., will also require attention. This is because it is envisioned that some slopes may exist in the valleys where parts of the LV-SBA Pipeline and Transfer-Bethany Pipeline will be located, and the slope stability may need to be evaluated.

Shallow borings and test pits should be advanced to confirm and evaluate soil and groundwater conditions along the planned pipeline alignments. In general, it is anticipated that soil borings and test pits will be advanced at approximate 1,000-foot intervals along the selected alignment. Each boring should be advanced to about 5 to 10 feet below the floor of the pipeline trench, an anticipated total depth of about 15 to 20 feet below ground surface. Boring numbers for the Delta-Transfer Pipeline and Transfer-LV Pipeline may be relatively less, depending on the configuration and distance between the new pipes and the existing pipes. Some borings along the LV-SBA Pipeline and Transfer-Bethany Pipeline alignment may also need to be advanced to depths of 30 to 50 feet to evaluate liquefaction potential, if relatively thick, loose sandy and silty soils are encountered. Some borings and test pits may need to be advanced to investigate potentially unstable slopes that would impact the pipelines, particularly for parts of the LV-SBA Pipeline and Transfer-Bethany Pipeline alignments.

Three relatively undisturbed soil samples should be collected from each boring at about 5-feet, 10-feet, and at 15-feet below grade surface (bgs). The soil samples should be collected using standard penetration test (SPT) and Modified California split spoon samplers driven 18-inches into the ground with a 140-pound slide hammer dropped 30-inches (or equivalent).

A project geologist or engineer will be responsible for coordinating field activities, geologically logging the soil cuttings and collected soil samples per Unified Soil Classification System (USCS) criteria, recording hammer blow counts, preparing and labeling relatively undisturbed and bulk soil samples, monitoring boring closures, and preparing the soil samples for submittal to the project laboratory.
If other project activities identify potential hazardous materials issues that could impact soil and/or groundwater during excavation of the pipeline, the field geologist/engineer should also be responsible for collecting soil and groundwater grab samples for analysis. The suite of analyses for the hazardous materials study is outside the scope of this report.

6.2 LABORATORY TESTING

Relatively undisturbed and bulk soil samples should be submitted to the project laboratory for geotechnical testing. Geotechnical tests would likely include, but not be limited to, the following:

- Sieve Analysis / Gradation (ASTM Test Method D422 / D1140)
- Liquid Limit / Plasticity Index (ASTM Test Method D4318)
- Maximum Density / Optimum Moisture Content (ASTM Test Method 4318)
- Unconfined Compressive Strength (ASTM Test Method D2166)
- Direct Shear (ASTM Test Method D5607)
- Triaxial Compression (ASTM Test Method D2850)

Small bulk soil samples would be submitted to the project Corrosion Engineer for testing and evaluation. Corrosive constituent analyses would likely include, but not be limited to, the following:

- Chloride (ASTM Test Method G57/ D2937)
- Sulfate (ASTM Test Method 417 modified)
- Sulfide (Qualitative by Lead Acetate paper)
- pH (ASTM Test Method G51 / Caltrans 643)
- Resistivity (ASTM Test Method G57 / Caltrans 643)

6.3 FURTHER GEOTECHNICAL STUDY

The derived geotechnical data (together with the previous geotechnical investigation reports, soil borings, and laboratory testing; and previous design and construction drawings and construction records) should be compiled and evaluated with regard to the proposed pipeline design. Recommendations should be developed with respect to potential geologic and seismic hazards, soil and rock excavation characteristics, trench support and stability evaluations, earthwork techniques, and pipeline design. The findings and recommendations should be presented in a Geotechnical Investigation Report specific to the proposed pipeline alignments of Los Vaqueros Expansion Project.
7 REFERENCES


References


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Figure 1. Site Location Maps. (1) Vicinity Map; (2) Location Map.
Figure 2. Regional Geologic Map
Figure 3. Site Geologic Map

Source of base map: Helley and Graymer (1997)
Figure 4. Faulting and Seismicity Map. (Source: EZ-FRISK 2005)
Figure 5. Subregional Geologic Map of the Project Site
Figure 6. Uniform Hazard Spectra
Figure 7. Liquefaction Susceptibility Map. (Source of base map: ABAG 2006)
Draft Addendum to Technical Memorandum: Geotechnical Considerations

Los Vaqueros Expansion Investigation
Contra Costa County, California
Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation’s natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.
Limitations and Disclaimer

The findings, interpretations of data, recommendations, specifications or professional opinions are presented within the limits prescribed by available information at the time this addendum was prepared, in accordance with generally accepted professional engineering and geologic practice. There is no other warranty, either expressed or implied.

The findings of this addendum are based on the readily available data and information obtained from public and private sources. Additional studies (at greater cost) may or may not disclose information, which may significantly modify the findings of this addendum. In the event that there are any changes in the nature, design or location of the project, or if additional subsurface data are obtained or any future additions are planned, the conclusions and recommendations contained in this addendum will need to be reevaluated by MWH in light of the proposed changes or additional information obtained.

This addendum was prepared solely for the benefit of U.S. Bureau of Reclamation – Mid-Pacific Region. No other entity or person shall use or rely upon this addendum or any of MWH's work products unless expressly authorized by U.S. Bureau of Reclamation – Mid-Pacific Region. Any use of or reliance upon MWH's work product by any party, other than U.S. Bureau of Reclamation – Mid-Pacific Region, shall be solely at the risk of such party.
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## Abbreviations and Acronyms

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<th>Definition</th>
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<tr>
<td>CPT</td>
<td>Cone Penetration Tests</td>
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Chapter 1
Introduction

The purpose of this Draft Addendum is to supplement the Technical Memorandum dated March 2007 prepared by MWH (Document 1 referenced below). The Addendum provides preliminary geotechnical characteristics of the predominant soil types anticipated at the proposed new and modified facilities for the Los Vaqueros Expansion Project in San Joaquin Delta, California, and addresses potential design and construction issues at preliminary and qualitative levels. The recommendations presented in this Addendum for use in the feasibility study should not be used in the final design.

This Addendum is based on limited data available from previous investigations conducted in the project area in the early 1990s; no specific investigation was performed at the proposed facilities for the preparation of this Addendum. A more detailed evaluation based on supplementary geotechnical investigation program along the proposed pipelines, intake and pump stations will be required in the feasibility study.
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Chapter 2
Project Documents Used in Review

The following four project documents were reviewed:


- **Document 2** – Los Vaqueros – Conveyance Facilities, Pumping Plants (Bid Package 1) – Old River and Transfer Facilities – Record Drawings (191 drawings), May 1998;

- **Document 3** – Los Vaqueros – Conveyance Facilities, Pipelines (Bid Package 2) – Old River, Transfer & Los Vaqueros Pipelines – Record Drawings (204 drawings), May 1998;


- **Document 6** – Morrison Knudsen Corporation, 1998. Los Vaqueros Project – Conveyance Facilities – Old River Facility – Construction Completion Report prepared for Contra Costa Water District, January; and

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Chapter 3
Facilities Evaluated

Foundation conditions were evaluated for the following new facilities:

- Delta-Transfer Pipeline (to be constructed parallel to the existing Old River Pipeline);
- Transfer-LV Pipeline (to be constructed parallel to the existing Transfer Pipeline);
- Mokelumne-Transfer Pipeline (to be constructed parallel to the existing Los Vaqueros Pipeline); and
- Expanded Transfer Pump Station (to be constructed near the existing Transfer Pump Station).

Foundation conditions were evaluated for the following modified facilities:

- Expanded Old River Intake and Pump Station; and
- Transfer Pump Station.

Figure 3-1 presents the general project location map including the facilities listed above. Attachment A shows the overview plan and profile drawings for the existing Delta-Transfer, Transfer-LV, and Los Vaqueros Pipelines.

Table 3-1 summarizes the new and modified facilities grouped in three areas as functions of general site subsurface conditions and anticipated performances, as follows:

- Area 1 – Delta-Transfer Pipeline (Sta. 0+00 to Sta. 100+00) and Expanded Old River Intake and Pump Station;
- Area 2 – Delta-Transfer Pipeline (Sta. 100+00 to Sta. 250+00); and
- Area 3 – Delta-Transfer Pipeline (Sta. 250+00 to Sta. 375+00), Transfer-LV Pipeline, Mokelumne-Transfer Pipeline, Expanded Transfer Pump Station and Transfer Pump Station.

Surface faulting is not a hazard at the new and modified facilities because no active faults that would rupture the surface are identified in the project area (DWR, 2009).
Chapter 3
Facilities Evaluated

It is assumed that the new and modified facilities could not be struck by a tsunami because the project area is located within the San Joaquin Delta at least 50 miles from the Pacific Ocean. Although no maps of tsunami hazards in the Delta were found in the publications reviewed, potential risks of catastrophic inundation are assumed to be small. This is because the tsunami hazards map west of the Delta area indicates a maximum inundation of 3 feet above mean sea level and tsunami effects would be attenuated in Suisun and Grizzly bays before reaching the Delta area (DWR, 2013b).
Figure 3-1. General Project Location Plan
### Table 3-1. Geotechnical Characteristics and Potential Design and Construction Issues

<table>
<thead>
<tr>
<th>Subject</th>
<th>Area 1</th>
<th>Area 2</th>
<th>Area 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Predominant Geotechnical Characteristics</td>
<td>Saturated medium stiff silty clay / clayey silt, loose silty sand / sandy silt, and muck</td>
<td>Partially saturated medium stiff to stiff silty clay / clayey silt, and loose silty sand / sandy silt</td>
<td>Dry stiff to hard silty clay / clayey silt, and dense silty sand / sandy silt</td>
</tr>
<tr>
<td>Design and Construction Issues:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pipe Buoyancy</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Delta Subsidence</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Bearing Capacity</td>
<td>Yes</td>
<td>Possible ¹</td>
<td>No ⁴</td>
</tr>
<tr>
<td>Settlement</td>
<td>Yes</td>
<td>Possible ¹, ²</td>
<td>Yes ⁵, ⁶</td>
</tr>
<tr>
<td>Trenching and Temporary Excavation</td>
<td>Yes</td>
<td>Yes ²</td>
<td>Yes, localized ⁵, ⁶</td>
</tr>
<tr>
<td>Seismic Shaking</td>
<td>Yes</td>
<td>Yes ²</td>
<td>Yes ⁵, ⁶</td>
</tr>
<tr>
<td>Irrigation and Drainage Crossings</td>
<td>Yes</td>
<td>Yes ²</td>
<td>No</td>
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<tr>
<td>Road Crossings</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
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<tr>
<td>Creek Crossings</td>
<td>No</td>
<td>Yes ²</td>
<td>Yes ²</td>
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<tr>
<td>Canal Crossings</td>
<td>No</td>
<td>Yes ²</td>
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<tr>
<td>Borrow Sources</td>
<td>Yes</td>
<td>Possible ³</td>
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<tr>
<td>Corrosion Potential</td>
<td>Yes</td>
<td>Yes ²</td>
<td>Yes</td>
</tr>
<tr>
<td>Expansive Soils</td>
<td>Yes</td>
<td>Yes ²</td>
<td>Yes ²</td>
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<td>Levee Failure</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
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<tr>
<td>Scour</td>
<td>No</td>
<td>No</td>
<td>Possible ³</td>
</tr>
<tr>
<td>Rock Excavation</td>
<td>No</td>
<td>No</td>
<td>Possible ³</td>
</tr>
</tbody>
</table>

Notes:
- Yes = issues are expected; No = issues are not expected; Possible = potential issues to be confirmed in next design phase.
- ¹ Possible issues for structures that might be built in Area 2; no issues are expected for the pipeline.
- ² Issues similar to those described in Area 1 (see Area 1 for descriptions of issues).
- ³ Possible issues that cannot be currently confirmed due to insufficient information.
- ⁴ No bearing capacity issues are expected at Expanded Transfer Pump Station and Transfer Pump Station, but allowable bearing pressure is provided in Area 3.
- ⁵ Issues are expected to be localized adjacent to Kellogg Creek where the soils could be soft / loose and saturated.
- ⁶ Localized issues adjacent to Kellogg Creek similar to those described in Area 1 (see Area 1 for descriptions of issues).
Chapter 4
Detailed Review

Following are detailed reviews of Project Area 1, Area 2, and Area 3. The reviews include preliminary geotechnical characterizations and potential design and construction issues.

Area 1 – Delta-Transfer Pipeline (Station 0+00 to Station 100+00) and Expanded Old River Intake and Pump Station

The ground surface along this segment of Delta-Transfer Pipeline in Area 1 is approximately located between elevation (El.) -10 feet and El. 10 feet (i.e., the area is as low as 10 feet below mean sea level). This pipeline segment is primarily characterized by shallow groundwater level. The area below mean sea level is assumed to be protected from sea water by a levee system although no information was found in the documents reviewed confirming this assumption.

Preliminary Geotechnical Characterization
The buried pipeline, anticipated to be up to 11 feet in diameter, will be for the most part constructed in medium stiff silty-clay/clayey-silt, described as low to medium plasticity with measured N-values from Standard Penetration Tests (SPT) of 7 to 8 blows per foot, and loose sandy-silt/silty-sand with SPT N-values of 6 to 10 blows per foot. The pipeline centerline at spring-line level will be located at an approximate average depth of 10 feet. Boring logs ORP RB-1 to ORP RB-9 (Sheets LV30 OP-B-01 and LV30 OP-B-02 in Document 3) indicate that these soil deposits could extend deeper than 46.5 feet (end of boring) below ground surface. The water level measured during drilling was 2 to 3 feet below ground surface and was affected by the irrigation and drainage systems located in this area.

Approximately the upper half of the pipeline could be locally constructed in olive gray soft clay with some decomposed vegetation (no blow counts are available in this soil). This addendum assumes this soil to be the muck (URS, 2003). The boring logs show that the muck is present within the top 3 to 10 feet below ground surface, but it is reported to extend as deep as 40 feet in nearby Bacon Island (URS, 2003). The muck can be described as “peat deposits which have advanced stage of decomposition to such extent that the botanical character is no longer evident” (NAVFAC, 1982a). The principal engineering characteristic of the muck is its high compressibility which makes it unsuitable for supporting building foundations. Typical unit weight of muck is about 100 pounds per cubic foot.
(pcf), and the undrained shear strength is approximately 100 pounds per square foot (psf) (USACE, 2006).

**Potential Design and Construction Issues**

**Pipe Buoyancy**

The pipeline alignment is expected to cross areas with high water table, flood plains subjected to potential inundation, and existing irrigation and drainage canals. Therefore, pipe buoyancy is likely to be a major design issue.

The state-of-the-practice calls for a conservative evaluation of the potential buoyancy by assuming an empty pipe. The magnitude of the buoyant (upward) force is equal to the weight of the water displaced (Rinker, 1994). If the buoyant force is larger than the weight of the pipe displacing the water, flotation will take place. The weight of water displaced per linear foot of a completely submerged pipe is computed using the expression below.

\[
W_w = \pi \cdot (OD)^2 \cdot \gamma_w / 4 \text{ (pounds/ft)}
\]

\[
W_w = \pi \cdot (OD)^2 \cdot 62.4 / 4
\]

\[
W_w = 49.01 \cdot (OD)^2
\]

Where: \(W_w = \) weight of displaced water; \(\gamma_w = \) unit weight of water.

Table 4-1 summarizes the estimated vertical forces and the resulting factors of safety against uplift on fully submerged 8-foot through 11-foot diameter concrete and steel-concrete composite pipes when they are empty.

**Table 4-1. Summary of Vertical Forces on Fully Submerged Empty Pipes**

| Pipe Material | ID (in) | OD (in) | \(W_p\) (lbs/ft) | \(W_B\)^2 (lbs/ft) | \(W_p + W_B\) (lbs/ft) | \(W_w\) (lbs/ft) | FS  
|---------------|---------|---------|-----------------|-----------------|-----------------|----------------|--------
| Concrete      | 96      | 112     | 2,710           | 2,106           | 4,816           | 4,266          | 1.13   
|               | 108     | 126     | 3,446           | 2,369           | 5,815           | 5,403          | 1.08   
|               | 120     | 140     | 4,263           | 2,632           | 6,895           | 6,674          | 1.03   
|               | 132     | 154     | 5,148           | 2,895           | 8,043           | 8,067          | 1.00   
| Steel 1       | 96      | 109.25  | 2,691           | 2,054           | 4,745           | 4,062          | 1.17   

Notes:

1. Composite pipe of concrete lining (2")-steel pipe (0.625")-concrete coating (4") totaling 6.625" thickness.
2. Backfill cover assumed to be equal to 6 feet; unit weight of backfill material = 100 pcf.
3. \(FS = (W_p + W_b) / W_w\).
4. Concrete pipe dimensions per American Concrete Pipe Association (ACPA, 2007).

Key:

- FS = factor of safety against uplift force
- ID = inside diameter
- \(W_p\) = pipe weight per foot of pipe
- \(W_w\) = weight of displaced water per foot of pipe
- \(W_B\) = weight of backfill
- pcf = pounds per cubic foot
- lbs/ft = pounds per foot
- OD = outside diameter

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Geotechnical Considerations

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In the examples presented in Table 4-1, the 8-foot through 11-foot diameter empty concrete pipes would be stable against the estimated uplift forces, provided that a backfill cover of minimum 6 feet is placed. Use of soil anchors is unlikely to be needed.

**Delta Subsidence**

The primary cause of ongoing land subsidence in the Delta is the decomposition of organic carbon in the peat soils. Other two key processes that are contributing to subsidence are soil compaction caused by consolidation and farm equipment, and wind erosion which is deemed to remove 0.25 to 0.5 inch of topsoil per year (DWR, 2013a).

Historical subsidence rates in the Delta have been found to strongly correlate with the organic matter content of the soil and the age of the reclaimed island and have ranged from 1.8 to 4.6 inches per year, with higher rates in the central Delta. Long-term average rates of subsidence are 1 to 3 inches per year (DWR, 2013). The subsidence rate between Holt and Stockton is estimated to be 1 to 3 inches/year and the differential subsidence over short distances (100 feet) could be approximately 1 to 2 inches/year which could damage levees, pipelines and other infrastructure improvements (MWH, 2014).

Areas that are at elevations lower than -5 feet can be assumed to have subsided (DWR, 2013a). This condition includes the pipeline segment between Sta. 0+00 and Sta. 30+00 and the Expanded Old River Intake and Pump Station site. Possible facilities from Sta. 30+00 to Sta. 85+00 (Attachment A) founded within the upper 10 feet of subsoil where muck is present could be subject to potential damaging effects of ongoing subsidence.

**Bearing Capacity**

The muck which is assumed to possess an undrained shear strength ($s_u$) of about 100 psf (USACE, 2006) would allow work and movement of low ground pressure equipment with an assumed bearing pressure up to 200 psf ($= N_c s_u / FS = 5.71 \times 100 / 3 \sim 200$ psf, where $FS = \text{factor of safety}$). This assumption is subject to confirmation in the next design phase when the results of laboratory tests are available. Use of ordinary equipment would require movable plank footing, temporary access roads, or similar improvement, to reduce the bearing pressure to 200 psf or less.

The allowable bearing pressure of the underlying medium stiff silty-clay/clayey-silt and loose sandy-silt/silty-sand to support building foundations is assumed to be 1,000 psf, provided that this value meets the allowable settlement criteria (NAVFAC, 1982b – Chapter 4, Table 1). This value also needs to be confirmed by calculations in the next project phase when the allowable settlement criteria are developed, using laboratory test results and taking into account the loaded area and foundation depth.
Settlement
In general, pipe settlement is not anticipated because the weight of full pipe would be smaller than the weight of displaced soil, as summarized in Table 4-2.

Table 4-2. Summary of Vertical Forces on Full Pipes

<table>
<thead>
<tr>
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<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>96</td>
<td>112 (4)</td>
<td>2,710</td>
<td>3,137</td>
<td>5,847</td>
<td>6,842</td>
<td>-995</td>
</tr>
<tr>
<td></td>
<td>108</td>
<td>126 (4)</td>
<td>3,446</td>
<td>3,970</td>
<td>7,416</td>
<td>8,659</td>
<td>-1,243</td>
</tr>
<tr>
<td></td>
<td>120</td>
<td>140 (4)</td>
<td>4,263</td>
<td>4,901</td>
<td>9,164</td>
<td>10,690</td>
<td>-1,526</td>
</tr>
<tr>
<td></td>
<td>132</td>
<td>154 (4)</td>
<td>5,148</td>
<td>5,930</td>
<td>11,078</td>
<td>12,935</td>
<td>-1,657</td>
</tr>
<tr>
<td>Steel</td>
<td>96</td>
<td>109.25</td>
<td>2,691</td>
<td>3,137</td>
<td>5,828</td>
<td>6,510</td>
<td>-682</td>
</tr>
</tbody>
</table>

Notes:
1 Without compacted engineering fill on top of trench backfill material.
2 Composite pipe of concrete lining (2")-steel pipe (0.625")-concrete coating (4") totaling 6.625" thickness.
3 The weight of displaced soil, $W_s$, was computed assuming a unit weight of muck equal to 100 pcf, to be confirmed by performing laboratory tests.
4 Concrete pipe dimensions per American Concrete Pipe Association (ACPA, 2007).

Key:
ID = inside diameter
in = inch
lbs/ft = pounds per foot
OD = outside diameter
pcf = pounds per cubic foot
$W_p$ = pipe weight per foot of pipe
$W_{wi}$ = weight of water in pipe per foot of pipe
$W_s$ = weight of displaced soil per foot of pipe

Placement of compacted engineering fill on top of the trench backfill would, however, generate some degree of pipe settlement. The optimum condition is provided by placement of minimum fill that minimizes settlements, but prevents potential pipe buoyancy.

For the purpose of this preliminary geotechnical evaluation, the muck and underlying medium stiff silty-clay/clayey-silt below water level are assumed to undergo some degree of consolidation settlement where additional loading, such as fill, is placed. The consolidation settlement could potentially occur due to the placement of access road embankment fills and due to road construction and site development in the future.

Dewatering to keep the trench and other construction excavations free of water is also expected to cause consolidation settlement outside the trench and excavation footprints. Potential adverse effects of dewatering on existing facilities adjacent to the trench and other excavations need to be analyzed before the start of construction.

Buildings, including the Expanded Old River Intake and Pump Station, will likely require foundations to be placed on top of more competent soil strata underlying the compressible muck. The depth of the shallow foundation base will depend on the load types (vertical, horizontal, compressive, tensile, permanent, temporary,
etc), magnitude of applied loads, dimensions of the loaded area, allowable total and differential settlements, and deformation parameters of soil. Settlement analysis will be necessary to identify possible construction issues, constraints and requirements, and potential need for mitigation before construction including excavation and replacement of muck and other compressible soils using structural backfill and ground improvements using stone columns of as deep as 35 feet (Document 6). Use of deep foundations is not anticipated at preliminary level, but this assumption is subject to confirmation in the next design phase.

**Trenching and Temporary Excavation**

Muck and underlying soft/loose soils are Type C soils per OSHA excavation standards (Subchapter 4 – Construction Safety Orders, Article 6 – Excavations of the Cal/OSHA). Trenching in these soils would require slopes not steeper than 1.5H:1V for maximum excavation depth of 20 feet, or a temporary vertical shoring with internal bracing to resist lateral earth pressures during construction. Given its low shear strength, the muck could be approximately assumed as an equivalent fluid with a unit weight of 100 pcf which applies a hydrostatic pressure on the bracing system.

If the muck and soft/loose soils extend beyond the bottom of the trench excavation, potential instability of the trench bottom could be a design issue. In this case, use of sheet piles embedded into the underlying soil deposits could be necessary to provide adequate trench stability.

Significant water could be present in trenches requiring use of dewatering wells ahead of excavation and a sump system inside the trench to provide dry working base (Documents 5, 6 and 7)

Potentially unstable excavation walls adjacent to existing facilities will require the design of adequate temporary shoring support using accepted industry standards. Excavations will require investigation to identify potential weak strata (soft clays and loose sands) and groundwater levels. Lateral support systems and bottom stability required during construction will be evaluated, and potential groundwater infiltration during excavation will be estimated. Analysis of possible ground improvements will be required taking into consideration potential effect of excavation on surrounding structures.

All temporary construction trenches and excavations are required to meet Subchapter 4 – Construction Safety Orders, Article 6 – Excavations of the Cal/OSHA and OSHA Excavation Standard 29 CFR 1926 - Subpart “P.”

**Seismic Shaking**

The low shear strength of muck could potentially lead to an amplification of ground shaking during an earthquake increasing the risk of damaged pipe. Excessive ground shaking could weaken pipeline joints or laterally displace pipe segments, but complete rupture of the pipe is expected to be localized mostly at liquefied sites if they occur. Pipelines buried in saturated sand deposits could
undergo damage due to uplift pressure caused by excess pore water pressure generated in liquefied soil during earthquake with associated risk of potential floatation. The damage leading to leakage could result in temporary service disruption until the damage is identified and repaired.

Liquefaction potential assessment of saturated loose silty-sand/sandy-silt encountered below muck will be required in the next design phase to evaluate possible adverse impacts on the proposed facilities. Liquefaction analysis using empirical methods should be based on the results of SPTs, Cone Penetration Tests (CPT), and laboratory tests. Where liquefaction is expected, computation of post-liquefaction reconsolidation settlement due to the dissipation of earthquake-induced excess pore water pressure should be required.

Occurrences of potentially liquefiable soils and muck are likely to characterize this area as Site Class F which would require site-specific evaluation (CBSC, 2013). This assumption is subject to confirmation based on liquefaction potential assessment.

**Drainage Canal Crossings**

Possible methods of construction at three existing drainage canal crossings include open cut crossings or trenchless crossings depending on the rate of flow and depth to the canal bed; aerial crossings could be another option provided that they meet the design criteria. Details of canal design and details of operation and maintenance were not available for this assessment.

Open cut crossings may involve cofferdams in conjunction with pumping, or opening of temporary channels to deviate flow. Trenchless methods may include micro-tunneling and pipe jacking with dewatering.

Drive and reception shafts will require groundwater control by dewatering or use of watertight shaft including diaphragm slurry wall and secant pile wall.

**Borrow Sources**

Potential borrow sources for trench backfill material may be available below the muck. The depth to possible borrow materials identified in boring logs is estimated to be approximately 3 to 10 feet below ground surface. Use of these borrow materials would require substantial amount of removal of the overlying muck. The borrow materials would be predominantly located below groundwater level, requiring stockpiling to drain excess water. Moisture conditioning of the borrow soils could require disking and aeration.

Use of more competent trench backfill materials from the Coastal Ranges about 4 miles southwest of this pipeline segment could be an option to be considered based on a technical and economic feasibility evaluation.

**Corrosion Potential**

The soils in this segment of pipeline and at the Expanded Old River Intake and Pump Station present high to very high corrosivity to uncoated steel and moderate
corrosivity to concrete as described in the MWH’s 2007 Technical Memorandum (Document 1).

Expansive Soils
The shrink-swell characteristics of soils present in this area are classified to be moderate to high (ICF International, 2013). The occurrences of expansive soils can be identified by direct observation, i.e., polygonal soil cracking (mudcracks) or popcorn texture in exposures is indicative of shrink-swell soil (Harris and Pearthree, 2002). Available soil survey maps are useful to delineate areas of clay-rich soils that are known to have shrink/swell potential. Where soil survey maps are not available, laboratory testing will be required to evaluate the shrink-swell potential of soil.

Mitigation of expansive soils can be accomplished by the application of hydrated lime, removal of the expansive soil and replacement with non-expansive fill, pre-wetting, drainage, and use of protection barriers (coatings, geomembranes). For large structures, use of deep piers or footings and specially reinforced or post-tensioned foundation slabs are common (Harris and Pearthree, 2002).

Levees Failure
Potential levees failure during earthquakes would cause the area below mean sea level to be completely flooded by as much as 10 feet of water. Analysis of sudden buildup of uplift pressures and potential heave of the Expanded Old River Intake and Pump Station foundations under partially submerged condition may be necessary in the next design phase.

Area 2 – Delta-Transfer Pipeline (Station 100+00 to Station 250+00)

The ground surface along this segment of Delta-Transfer Pipeline is approximately located between El. 10 feet and El. 40 feet. Presence of muck is not expected in Area 2. Pipe buoyancy, Delta subsidence, levees failure and associated uplift-pressure/heave are not issues in this project area as summarized in Table 3-1.

Preliminary Geotechnical Characterization
The buried pipeline, anticipated to be up to 11 feet in diameter, will be mostly constructed in soft to stiff, low to medium plasticity, silty-clay/clayey-silt with measured SPT N-values of 4 to 18 blows per foot, and very loose sandy-silt/silty-sand with SPT N-values as low as 2 to 4 blows per foot. The pipeline centerline at spring-line level will be located at an approximate average depth of 10 feet below ground surface. Boring logs ORP RB-10 to ORP RB-19, ORP AB-20 and ORP AB-21 (Sheets LV30 OP-B-02 through LV30 OP-B-04 in Document 3) indicate that these soil deposits could extend deeper than 57.5 feet (end of boring) below ground surface. The water level measured during drilling was generally 5 to 6 feet below ground surface.
Chapter 4
Detailed Review

Potential Design and Construction Issues
Design and construction issues related to settlement, trenching, seismic shaking, four canal crossings, one drainage ditch crossing, Frisk Creek crossing, corrosion potential and expansive soils are comparable to those issues addressed for Area 1 and are not repeated in this section.

Issues specific only to Area 2 are presented in subsections below.

Bearing Capacity
The surficial soft silty-clay/clayey-silt and loose silty-sand/sandy-silt is assumed to allow work and movement of construction equipment with a bearing pressure up to 1,000 psf (NAVFAC, 1982b – Chapter 4, Table 1). Bearing capacity calculations will be necessary in the next design phase to confirm this assumed value.

No building facilities are proposed along the Delta-Transfer Pipeline between Sta. 100+00 and Sta. 250+00 in Area 2. However, for possible structures that might be necessary in this area, the underlying medium stiff to stiff silty-clay/clayey-silt is assumed to provide an approximately allowable bearing pressure of 2,000 psf, provided that it meets the allowable settlement criteria (NAVFAC, 1982b – Chapter 4, Table 1). This assumed value would also require confirmation by calculations in the next project phase when the allowable settlement criteria are prepared.

Road Crossings
Open-cut crossings or trenchless crossings will be necessary at the intersections with road types listed below. Approximate stations at the road crossings are provided for reference.

- Bixler Road Sta. 141+00
- Kellogg Creek Road Sta. 180+50
- Byron Highway Sta. 211+00
- Southern Pacific Railroad Sta. 239+50
- Hoffman Lane Sta. 240+50

The type of crossing at each road type will depend on site specific conditions, including subsurface conditions, impact to road operation, crossing lengths, site constraints, and truck traffic volumes. Most of the trenchless crossings would be constructed in saturated loose sand and soft clay. Possible trenchless methods for these crossings may include pipe ramming, micro-tunneling and pipe jacking with dewatering.
Construction shafts at both ends of the trenchless crossings are likely to require groundwater control by dewatering or use of watertight shaft including diaphragm slurry wall and secant pile wall.

**Borrow Sources**

Excavated soils from the trench above groundwater level may be the principal borrow sources for trench backfill material. Based on available boring logs, the depth of groundwater level is assumed to be approximately 5 feet below ground surface.

Use of borrow materials located below groundwater level would require stockpiling and moisture conditioning before the placement into trench, as described under Area 1.

Another source of trench backfill material with better quality may be found in the Coastal Ranges about 1.5 miles southwest of this pipeline segment.

**Area 3 – Delta-Transfer Pipeline (Station 250+00 to Station 375+00), Transfer-LV Pipeline, Mokelumne Transfer Pipeline, Expanded Transfer Pump Station and Transfer Pump Station**

The ground surface along these pipelines and pump stations is approximately located above El. 40 feet. In general, occurrences of muck, soft clay and loose sand are not expected in Area 3, but saturated soft clay and/or loose sand soils may be locally present at Kellogg Creek. Pipe buoyancy, Delta subsidence, bearing capacity, settlement, borrow sources, levee failure and associated uplift-pressure/heave are not issues in this project area.

Although no bearing capacity issues are expected at Expanded Transfer Pump Station and Transfer Pump Station, the allowable bearing pressure is provided. Since the available borings show that the groundwater level is generally located below pipe crown, most of excavated soils from the trench could be used as borrow materials for trench backfill.

**Preliminary Geotechnical Characterization**

The buried pipeline, anticipated to be up to 11 feet in diameter along Delta-Transfer Pipeline (Sta. 250+00 to Sta. 375+00), will be mostly constructed in stiff to hard, low to medium plasticity, silty-clay/clayey-silt with measured SPT N-values of 9 blows per foot to refusal (SPT N-values of larger than 50 blows per 6 inches). The pipeline could be occasionally constructed in dense sandy-silt/silty-sand with a SPT N-value of about 40 blows per foot and in siltstone. The pipeline centerline at spring-line level will be located at an approximate average depth of 10 feet. Boring logs ORP AB-22 to ORP AB-31 (Sheets LV30 OP-B-04 and LV30 OP-B-05 in Document 3) indicate that these soil deposits could extend...
deeper than 46.5 feet (end of boring) below ground surface. The water level measured during drilling is approximately 20 feet below ground surface.

Data from the CPTs used to assess the approximate subsurface conditions along the Transfer-LV Pipeline (Sheet LV30 TP-B-01 in Document 3) and Mokelumne Transfer Pipeline (Sheets LV30 LP-B-01 and LV30 LP-B-02 in Document 3) generally confirm the ground conditions found along Delta-Transfer Pipeline (Sta. 250+00 to Sta. 375+00). However, the CPT tip resistance measured in soil deposits adjacent to Kellogg Creek appears to be about ten times smaller than the tip resistance in soil deposits located away from the creek. This could be partly attributed to the saturated nature of soils tested near the creek.

**Potential Design and Construction Issues**

Possible design and construction issues associated with trenching are anticipated to be localized and limited to the proximities of Kellogg Creek. These issues and issues related to seismic shaking in soft/loose soil deposits, five Kellogg Creek crossings, and expansive soils are comparable to those issues addressed for Area 1 and are not repeated in this section.

Issues specific only to Area 3 are presented in subsections below.

**Bearing Capacity**

Where stiff silty-clay/clayey-silt are found at ground surface, no bearing capacity issues are anticipated for work and movement of construction equipment. Where surficial soft silty-clay/clayey-silt are encountered, particularly adjacent to Kellogg Creek, the allowable bearing pressure for construction equipment is assumed to be about 1,000 psf (NAVFAC, 1982b – Chapter 4, Table 1). Bearing capacity calculations will be necessary in the next design phase to confirm this assumed value for construction equipment.

Borings ORP AB-29 and ORP AB-31 drilled around Vasco Road in the early 1990s, which are located some 1,500 feet to the east of Expanded Transfer Pump Station and Transfer Pump Station, indicate the occurrences of very dense silty-sand/sandy-silt starting 5 feet below ground surface with SPT N-values larger than 50 blows per foot. Boring ORP AB-31 also encountered siltstone at about 11 feet below ground surface (approximately El. 113 feet). The Expanded Transfer Pump Station and Transfer Pump Station are founded above El. 180 feet where the site subsurface conditions are assumed to be comparable or better than the conditions at borings ORP AB-29 and ORP AB-31, and an allowable bearing pressure of 3,000 psf is preliminarily assumed (NAVFAC, 1982b – Chapter 4, Table 1). This assumed value also requires confirmation by calculations in the next project phase.

**Trenching**

Most soils identified along the pipelines in Area 3 can be characterized as stiff to hard silty-clay/clayey-silt which can be included in the category of Type B soils per OSHA excavation standards (Subchapter 4 – Construction Safety Orders,
Article 6 – Excavations of the Cal/OSHA). Type B soils are required to have a slope not steeper than 1H:1V for maximum excavation depth of 20 feet. Stable siltstone can be excavated vertically up to an excavation depth of 20 feet.

Occasional soft silty clay / clayey silt and all silty sand / sandy silt are Type C soils which are required to have a slope not steeper than 1.5H:1V for maximum excavation depth of 20 feet.

Site specific conditions may require use of shoring with internal bracing. Trench boxes can also be used for excavations in open areas, and in combination with sloped excavations.

These OSHA requirements will direct the minimum clearance between the proposed and existing pipelines, and the alignment of the proposed pipeline within the project right-of-way.

Seismic Shaking
In general, effects of ground shaking are assumed to be more critical where occasional soft silty-clay/clayey-silt and loose silty-sand/sandy-silt are present below groundwater table adjacent to Kellogg Creek.

Road Crossings
Open cut crossings or trenchless crossings will be necessary at the intersections with road types listed below. Approximate stations at the road crossings are provided for reference.

- Delta-Transfer Pipeline (Station 250+00 to Station 375+00):
  - Vasco Road Sta. 340+00

- Transfer-LV Pipeline:
  - Longwell Avenue Sta. 34+00
  - Camino Diablo Road Sta. 45+50
  - Walnut Avenue Sta. 47+00
  - Vasco Road Sta. 139+00
  - Vasco Road Sta. 212+00

- Mokelumne Transfer Pipeline:
  - Sand Creek Road Sta. 170+00
  - Balfour Road Sta. 240+50
The type of crossing will depend on site specific conditions, including ground conditions, crossing lengths, site constraints and truck traffic volumes.

Most of trenchless crossings would be constructed in stiff to hard silty-clay/clayey-silt and medium-dense to dense silty-sand/sandy-silt. Possible trenchless methods for these crossings may include pipe jacking and conventional tunneling. Trenchless crossings underneath Upton Pyne Drive and Walnut Boulevard may require dewatering.

Construction shafts at Upton Pyne Drive and Walnut Boulevard crossings are likely to require groundwater control by dewatering or use of watertight shaft including diaphragm slurry wall and secant pile wall. Use of sheet piles for temporary shaft support is risky because of potential refusal to sheet pile penetration due to the presence of hard clay and dense (gravelly) sand.

**Corrosion Potential**
The 2007 Technical Memorandum (Document 1) indicates that the soils in the Delta-Transfer Pipeline (Sta. 250+00 to Sta. 375+00) and Transfer-LV Pipeline present high to very high corrosivity to uncoated steel and moderate corrosivity to concrete. The soils at the Expanded Transfer Pump Station and Transfer Pump Station are considered to be corrosive to steel. No information is currently available for the Mokelumne Transfer Pipeline.

**Scour**
Computation of maximum scour depths will be necessary in Kellogg Creek using peak discharges and mean grain size (i.e., the grain size corresponding to the 50 percent mark) on the cumulative frequency distribution curve, to evaluate potential impacts on Delta-Transfer Pipeline in Area 3. Assessment of storm duration, precipitation intensity and Kellogg Creek tributary areas would be necessary to compute reliable discharge rates for use in the scour depth calculation. Sediment supply from upstream to downstream reaches of Kellogg Creek would also be evaluated for a more accurate scour assessment (NRCS, 2007)

**Rock Excavation**
Boring ORP AB-31 drilled approximately at Sta. 340+00 of the Delta-Transfer Pipeline located in the Coastal Ranges indicated the top of siltstone about 10 feet below ground surface. Since the available drawings show that the approximate
centerline of the pipeline is located 10 feet below ground surface, the lower half of the pipeline could be constructed in bedrock. Based on limited information available, rock excavation could be required from about Sta. 340+00 to Sta. 375+00. This assumption needs to be confirmed in the next design phase.
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Chapter 5
Additional Recommendations for Future Investigations and Study

This section supplements the recommendations for future investigations and study presented in the 2007 Technical Memorandum (Document 1) which still apply to this project. The recommendations presented in the 2007 Technical Memorandum are more generically applicable to all new and modified facilities, whereas the recommendations in this Addendum are more specific to each facility.

Investigations

The following supplementary recommendations are provided:

- A sufficient number of borings will be required at trenchless crossings, comprised of at least:
  - Trenchless crossing shorter than 300 feet 1 boring
  - Trenchless crossing longer than 300 feet 2 borings

Representative soil samples will be selected for geotechnical laboratory tests recommended in the 2007 Technical Memorandum (Document 1);

- Hydrometer analysis (ASTM D422) on selected fine-grained soil samples will be added to the geotechnical tests to measure clay fraction which may affect construction of trenchless crossings;

- Sieve analysis will be required on bulk samples from the Kellogg Creek bed in Delta-Transfer Pipeline to measure mean grain size for use in scour analysis at Kellogg Creek crossings;

- Expansion index tests (ASTM D4829) on selected fine-grained soils to evaluate expansion potential at the foundation site of Expanded Old River Intake and Pump Station

- CPTs spaced 1,000 feet apart along Delta-Transfer Pipeline from Sta. 0+00 to Sta. 250+00 where most of potentially liquefiable soils are present; and
Chapter 5
Additional Recommendations for Future Investigations and Study

- Seismic refraction investigation is recommended in the Delta-Transfer Pipeline from Sta. 310+00 to Sta. 375+00 located in the Coastal Ranges where the bedrock could be shallow to evaluate ripability.

Study

The following supplementary recommendations are provided:

- A site-specific seismic hazard assessment due to the occurrences of potentially liquefiable loose silty-sand/sandy-silt located below groundwater level along Delta-Transfer Pipeline from Sta. 0+00 to Sta. 250+00 (Areas 1 and 2);

- Liquefaction potential assessment based on SPT blow counts, CPT tip resistance and results of laboratory tests along Delta-Transfer Pipeline from Sta. 0+00 to Sta. 250+00 (Areas 1 and 2);

- Development of geotechnical parameters for use in pipeline design and specifications, including lateral earth pressures and modulus of soil reaction (E’);

- Scour analysis at Kellogg Creek crossings along Delta-Transfer Pipeline in Area 3;

- Settlement calculations along trenchless crossings; and

- Foundation analysis at the Expanded Old River Intake and Pump Station site, Expanded Transfer Pump Station site and Transfer Pump Station site including bearing capacity and settlement calculations.
Chapter 6
References


URS, 2003. In-Delta Storage Program Borrow Area Geotechnical Report, prepared for DWR.
Draft Addendum to Technical Memorandum: Geotechnical Considerations

Attachment A
Overview Plan and Profile Drawings for Existing Delta-Transfer, Transfer-LV, and Los Vaqueros Pipelines

Los Vaqueros Expansion Investigation
Contra Costa County, California
BEGIN LOS VAQUEROS PIPELINE BID SCHEDULE C
AT LOS VAQUEROS PIPELINE STA. 0+31.60

NEROLY BLENDING FACILITY

NOTE:
1. FOR LOS VAQUEROS PIPELINE STEEL PIPE ALTERNATIVE
   STEEL PIPE TO STATION 9540 CONTRACTOR TO SUPPLY PIPE
   WITH MEANS TO STEEL PIPE ADAPTOR AND ISOLATING
   JUNCTION TESTING AT EACH REEL TO STEEL PIPE CONNECTION.
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Attachments

Attachment 1 – Los Vaqueros Project Drawings
Attachment 2 – USGS Design Maps
Attachment 3 – Multi-Ripper Bucket

Abbreviations and Acronyms

Delta  Sacramento-San Joaquin Rivers Delta
EL  elevation
MCC  Motor Control Center
pcf  pounds per cubic foot
PS  Pump Station
Chapter 1
Introduction

This appendix to the Neroly Pump Station Design (Site B) Technical Memorandum for the Los Vaqueros Reservoir Expansion Investigation documents the geotechnical investigations done for the feasibility-level design of the Neroly Pump Station (PS).

Neroly Pump Station Description

The Neroly PS was investigated as an option to allow water that originates from the Rock Slough Intake to be conveyed to either the expanded Los Vaqueros Reservoir or the Bethany Reservoir. The primary use of this facility will be during times when water originating from the Old River and Middle River Intakes is not available due to environmental regulatory constraints. Los Vaqueros Reservoir filling from Rock Slough Intake and deliveries to partners south of the Sacramento-San Joaquin Rivers Delta (Delta) would not be constrained when Old and Middle River flow restrictions control Central Valley Project and State Water Project south Delta export operations. Therefore, the Neroly PS will provide an alternate method for supplying water to the Los Vaqueros and Bethany Reservoirs.

The pump station will include six – 50 cfs capacity vertical turbine pumps, a new electrical substation, and a Motor Control Center (MCC). Power was assumed to be routed from the nearby 69kV towers from an existing endpoint. The initial configuration was similar to that described in the MWH Technical Memorandum for Site C, with the intake on the Contra Costa Canal and twin 78” suction pipelines, leading to a suction manifold. The suction pipes would need to be routed under the existing 60” diameter Randall-Bold Intake Pipeline. In addition, the 84” discharge pipeline would need to be routed under three existing pipelines. The discharge pipe would be routed to a tie in with the Los Vaqueros Pipeline past Station 3+50.

Project Background

Los Vaqueros Reservoir watershed and associated dam and facilities are located in the California coastal foothills west of the Delta and east of the San Francisco Bay Area, the central and south Delta, and service areas of San Francisco Bay Area water agencies. Contra Costa Water District owns and operates the existing Los Vaqueros Reservoir, an existing 160,000-acre-foot off-stream surface storage facility.

The potential expansion of Los Vaqueros Reservoir is one of five surface storage projects recommended for additional study by the CALFED Bay-Delta Program. The U.S. Department of Interior, Bureau of Reclamation, Mid-Pacific Region was authorized to conduct pre-feasibility-level investigations for potential expansion of Los Vaqueros Project, and is the lead agency under the National Environmental Policy Act.
Chapter 1
Introduction

Purpose of Document

The foundation conditions of the Neroly Pump Station Site B were preliminarily evaluated using very limited geotechnical investigation data available from the adjacent Los Vaqueros Pipeline Project and readily available literature.

No investigation data (boring logs, Cone Penetration Test data, shear wave velocity measurements and/or laboratory test results) were made available within the footprints of the proposed structures during the course of this evaluation. As such, this preliminary geotechnical assessment at conceptual level is subjected to possible revisions based on representative geotechnical investigation data that should be collected at the foundation sites of the proposed structures.

Proposed Structures

The approximate foundation conditions of the following structures were evaluated at a conceptual level:

- Pump station
- Substation
- MCC building
- Surge tank
- 84” suction pipeline
- 84” discharge pipeline

This evaluation requires confirmatory geotechnical investigation data that should be obtained within the footprints of the largest structures, including the pump station, substation and MCC building. Geotechnical investigation along the pipelines is also essential for the project.

Figure 1-1 presents the approximate locations of the structures above.
Figure 1-1. Preliminary Project Site Layout for Site B (Sheet C-2)
Appendix Organization

This appendix is organized as follows:

Chapter 1, Introduction, provides an overview of the Neroly PS.

Chapter 2, Site Subsurface Conditions, summarizes surface geology and the preliminary site subsurface assessment conducted.

Chapter 3, Approximate Foundation Conditions, presents the site class, liquefaction potential, and other geotechnical hazards at the site.

Chapter 4, Preliminary Geotechnical Recommendations, summarizes the geotechnical recommendations for use in the feasibility-level design.

Chapter 5, Recommendations for the Next Design Phase, lists several recommendations for gathering geotechnical data to support future design work.

Chapter 6, Limitations, presents the limitations of the geotechnical analysis presented in this appendix.

Chapter 7, References, lists the sources used in preparing this technical appendix.
Chapter 2
Site Subsurface Conditions

Surface Geology

The quaternary geological map of the Contra Costa County (Figure 2-1, Helley and Graymer, 1997) shows that the project site is located on Holocene Era alluvial fan and fluvial deposits (Qhaf). The alluvial fan deposits are described as brown or tan, medium dense to dense, gravely sand or sandy gravel that generally grades upward, to sandy or silty clay. Near the distal fan edges, the fluvial deposits are typically brown, never reddish, medium dense sand that fines upward to sandy or silty clay.

Source: Helley and Graymer, 1997

Figure 2-1. Geologic Map
Chapter 2
Site Subsurface Conditions

The older Pleistocene and Holocene Era dune sand (Qds) that may occur underlying the alluvial fan and fluvial deposits is described as fine-grained, very well-sorted, well-drained, eolian deposits.

Preliminary Site Subsurface Assessment

Borings A-9, A-10, ESA-3 and R-10 from the Los Vaqueros Pipeline investigation (Attachment 1) were used to assess the approximate foundation conditions. They are the closest borings available from the project site. For reference, Table 2-1 presents the approximate distances of borings used from the substation and the approximate ground surface elevations of these borings.

Table 2-1. Available Borings Nearest the Proposed Substation

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Approx. Distance from Substation¹ (feet)</th>
<th>Approx. Ground Surface¹ (feet)</th>
<th>Soil Type</th>
<th>Depth to Bedrock Surface² (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-9</td>
<td>500</td>
<td>El. 88</td>
<td>Lean, gravelly, sandy, or silty clay</td>
<td>8</td>
</tr>
<tr>
<td>A-10</td>
<td>800</td>
<td>El. 80</td>
<td>Lean, gravelly, sandy, or silty clay</td>
<td>&gt;21.5</td>
</tr>
<tr>
<td>ESA-3</td>
<td>310</td>
<td>El. 127</td>
<td>Lean, gravelly, sandy, or silty clay</td>
<td>3</td>
</tr>
<tr>
<td>R-10</td>
<td>270</td>
<td>El. 125</td>
<td>Lean, gravelly, sandy, or silty clay</td>
<td>3</td>
</tr>
</tbody>
</table>

Notes:
¹ Distances and elevations from Sheet LV30 LP-C-03 (Montgomery Watson, 1998)
² From Sheet LV30 LP-B-02 (Montgomery Watson, 1994); bedrock is described as sandstone, claystone, shale and siltstone.

The clayey soils identified in borings are generally consistent with the sandy and silty clays (Qhaf) shown in the geological map (Helley and Graymer, 1997). The borings appear to indicate that the thickness of clay deposits increases at lower ground surface elevations (Elevation (EL) 80 feet). Blow counts are not available in these borings to assess soil strength.

No groundwater table readings were found in available drawings.
Chapter 3
Approximate Foundation Conditions

Site Class

By default, the site class for this project is assumed to be D (stiff soil), subject to confirmation when blow counts, undrained shear strength and/or shear wave velocity become available.

Liquefaction Potential

Based on the liquefaction susceptibility map of the San Francisco Bay Area (USGS, 2006), the project site is approximately located at the transition between moderate and very low liquefaction potential areas.

The available borings (Table 2-1) nearest the project site encountered clayey soils which are not susceptible to liquefaction. Boring A-10 was relatively shallow (less than 21.5 feet) and possibly ended before encountering the underlying sand and/or gravel deposits. From the geological map (Helley and Graymer, 1997), the sand and gravel deposits below clay deposits are described as medium dense to dense. As such, the liquefaction potential of the sand and gravel deposits is assumed to be relatively low. A confirmatory liquefaction potential assessment using blow counts, undrained shear strength, or shear wave velocity is required.

Other Geotechnical Hazards

The Oakley area is known to present localized problems associated with expansive clays, unstable colluvium during ground shaking and prolonged rainfalls, and unstable cut slopes (City of Oakley, 2016). The lowland soils are generally impervious and potentially corrosive, which could be the case where the ground surface is located at EL 80 feet or below.

If these potential hazards are confirmed by future geotechnical investigation, possible mitigation methods are:

Expansive Soils

- Replacement of Expansive Soils – Expansive soils can be removed and replaced with non-expansive soils. Alternatively, the expansive and non-expansive soils can be mixed together to reduce the expansion index to acceptable levels. The expansive soils could be present in the upper clay deposits.

- Reinforcement of Slabs on Grade and Footings – Relatively small slabs on grade can be reinforced using wire mesh in the concrete that provides tensile capacity to resist soil movement. Another technique is to use steel reinforcing bars for post-tensioning of the
slab on grade. Moreover, the sub-grade can be wetted 24 hours prior to pouring the concrete to condition the soil to the recommended moisture content that minimizes expansion.

- **Addition of Gravel under Slabs on Grade** – Four to 8 inches of crushed aggregate can be compacted under the slab on grade to help reduce soil movement. As soil swells, the soil particles infiltrate the voids in gravel reducing the uplift pressure on the slab.

**Unstable Colluvium**
Colluvium, if found on the slopes probably above ground surface EL 80 feet, must be completely removed from the footprints of the proposed structures to prevent slope instability.

**Unstable Cut Slopes**
Shale (rock), if exposed during excavation, should be immediately protected by a shotcrete layer to prevent weathering that could lead to shear strength reduction and consequent slope instability. Possible exposure of shale during excavation is more likely to occur where the ground surface is above EL 80 feet.

**Corrosive Soils**
California Department of Transportation (2015) provides corrosion guidelines including laboratory tests and mitigation methods that may be used. Corrosive soils are more likely to be encountered below ground surface EL 80 feet, such as at end of the 84-inch diameter suction pipeline near Contra Costa Canal.
Chapter 4
Preliminary Geotechnical Recommendations

Foundation Type

Based on a limited geotechnical information available from the Los Vaqueros Pipeline, shallow foundations are deemed to be applicable to the project site which is assumed to be Site Class D. The base of the slab on grade and footings should be located at a minimum depth of 18 inches below the final ground surface level after grading.

It is, however, noteworthy to mention that localized incompetent foundation soils could occur beneath the proposed structures located at or below ground surface EL 80 feet. Since accurate subsurface conditions are presently unknown, it is beneficial that the structures at these ground elevations be buried as deep as possible, preferable close to the bottom of the Contra Costa Canal, to reduce potential differential settlement and lateral displacement.

If future geotechnical investigations show that incompetent foundation soils occur below the proposed structures, possible solutions include removal and replacement of incompetent foundation soils, and ground improvement using stone columns or deep soil mixing columns to limit ground movements to acceptable levels.

Bearing Capacity

A minimum bearing capacity of 1,500 pounds per square foot can be used for permanent vertical loads applied on slabs on grade and shallow footings founded directly on clay deposits.

The bearing capacity can be increased to 2,000 pounds per square foot for the foundations of semi-buried structures founded on medium dense to dense sand deposits. The top of the sand deposits in the project area is presently unknown, but it could be around EL 40 feet, to be confirmed by future geotechnical investigation.

For temporary loads (except for seismic loads from earthquakes because the soils could undergo strength loss during ground shaking), the bearing capacity can be increased by a multiplying factor of 1.3.

If a larger bearing capacity is required, geotechnical investigation will be necessary.

Lateral Earth Pressure

Retaining walls and buried structures are designed to resist both lateral earth pressures and additional lateral loads caused by surcharge loads.
Approximate unit weights of equivalent fluid are shown in Table 4-1 considering the condition of the wall (restrained or unrestrained at the top of the wall), soil backfill type (imported clean crushed rock or native backfill), and groundwater condition behind the wall (drained or undrained). Walls that are restrained at the top are required to be designed to resist “at rest” lateral pressures. Walls that are unrestrained at the top may be designed for “active” lateral earth pressures only if some movement of the wall is tolerable to allow the strength of the soil to be mobilized behind the wall.

Table 4-1. Approximate Unit Weights of Equivalent Fluid for Below-Grade Retaining Structures

<table>
<thead>
<tr>
<th>Earth Pressure Condition</th>
<th>Approximate Unit Weight of Equivalent Fluid (pound per cubic foot)</th>
<th>Imported Clean Crushed Stone Backfill</th>
<th>Native Silty Clay Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>At-rest, drained¹</td>
<td>58</td>
<td>61</td>
<td></td>
</tr>
<tr>
<td>At-rest, undrained²</td>
<td>95</td>
<td>92</td>
<td></td>
</tr>
<tr>
<td>Active, drained¹</td>
<td>37</td>
<td>42</td>
<td></td>
</tr>
<tr>
<td>Active, undrained²</td>
<td>83</td>
<td>83</td>
<td></td>
</tr>
</tbody>
</table>

Notes:

1 Does not include unit weight of water
2 Includes unit weight of water.

The “drained” lateral pressures assume that the walls are fully back-drained to prevent the buildup of hydrostatic pressures and also assume a level backfill. If walls are not fully back-drained, “undrained” pressures should be used.

For walls with inclined backfill, an additional unit weight of equivalent fluid of 1 pound per cubic foot (pcf) per every 2 degrees of slope inclination should be used.

For imported clean crushed stone backfill in direct contact with the wall, drained condition prevails above the groundwater level and undrained condition prevails below the groundwater level. For the sake of cost-effectiveness, backfilling using crushed stone may be limited to a width of 3 feet from the wall such that the rest of backfill can be completed using native soil. Use 4-inch diameter perforated polyvinyl chloride pipe at the base of crushed stone backfill. Placement of a concrete pavement or a 2-feet-thick compacted clay liner will be required at the top of crushed stone backfill to reduce rainwater infiltration.

For native soil backfill in direct contact with the wall, use undrained condition regardless of the groundwater level. This is to account for potential perched groundwater level and shrinkage cracks that could develop at the interface of the wall and the native soil backfill above the perched groundwater level.

Walls subjected to surcharge loads are required to be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively.
During an earthquake event, additional loads will be applied to the retaining walls and buried structures. As such, the retaining structures are required to be designed to resist a lateral earth pressure that includes an additional unit weight of equivalent fluid to the unit weights presented in Table 4-2. Transient loads produced by vehicular traffic or heavy construction equipment do not need to be considered in the retaining wall design, unless they produce lateral pressures that exceed the pressures produced under earthquake loading conditions.

### Table 4-2. Additional Unit Weights for Earthquake Loads

<table>
<thead>
<tr>
<th>Earth Pressure Condition</th>
<th>Additional Unit Weight of Equivalent Fluid</th>
<th>Pressure Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>At-rest</td>
<td>20</td>
<td>20 \cdot H</td>
</tr>
<tr>
<td>Active</td>
<td>20</td>
<td>20 \cdot (H-z)</td>
</tr>
</tbody>
</table>

Note: A peak ground acceleration of 0.5 g was used (Attachment 2).

Key:
- \( H \) = height of retaining structure from final ground surface level to the top of wall footing
- \( z \) = depth below final ground surface level.

The following parameters presented in Table 4-3 are assumed for the backfill materials. The cohesion intercept is assumed to be zero.

### Table 4-3. Approximate Backfill Parameters

<table>
<thead>
<tr>
<th>Backfill Type</th>
<th>Angle of Internal Friction (degrees)</th>
<th>Moist Unit Weight (pound per cubic foot)</th>
<th>Saturated Unit Weight (pound per cubic foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>¾-in. Crushed stone</td>
<td>34</td>
<td>130</td>
<td>135</td>
</tr>
<tr>
<td>Native sandy/silty clay</td>
<td>27</td>
<td>110</td>
<td>115</td>
</tr>
</tbody>
</table>

### Lateral Load Resistance

Lateral load resistance for the proposed structures and retaining walls can be developed by friction between the foundation bottom and the supporting subgrade. An ultimate friction coefficient of 0.34 is considered applicable between concrete and ground.

As an alternative for active earth pressure condition, a passive resistance equal to an equivalent fluid pressure weighing 190 pcf acting against the vertical face of the foundation could be used for drained conditions. If foundations are poured neat against the soil, the friction and passive resistance can be used in combination. Considering displacement compatibility on both sides of the retaining structure, the passive resistance above assumes that only about 50 percent of the maximum passive resistance would be mobilized at maximum active lateral earth pressure. The passive resistance for undrained soils can be obtained using a unit weight of equivalent fluid of 70 pcf. The factor of safety is evaluated by analyzing the equilibrium of forces, i.e., comparison between the resisting force as obtained above and the driving force. This alternative of lateral load resistance is not applicable to the at-rest lateral earth pressure condition because the passive resistant would not be mobilized without lateral displacement of the retaining structure.
Groundwater Control

Possible presence of groundwater should be assumed for excavations deeper than 10 feet at the foundation site of the pump station, and pipeline segments located below ground surface EL 90 feet.

It is deemed that groundwater infiltration in relatively shallow excavations in clay deposits can be controlled by sump.

Relatively deep excavations may undergo base instability of the excavation triggered by uplift pressure, a condition that would require groundwater control using a dewatering system prior to the start of excavation.

Soil Excavation

It is anticipated that the excavations for the semi-buried structures and pipelines will be in soil and rock. Rock excavation is addressed below in the section entitled “Rock Excavation.”

Soil excavation is expected to be in lean clay, gravelly clay sandy clay and silty clay based on the material descriptions summarized in Table 2-1. The soil excavation for the proposed structures can be made with conventional earthmoving equipment.

It is recommended that permanent cut slopes in soil be designed at an inclination no steeper than 2H:1V.

Temporary excavation slopes of 1.5H:1V may be used. For shallow excavations less than 5 feet in depth, temporary slopes of 1H:1V may be used. Alternatively, temporary excavation support may be used such as sheet piles, soil mixed walls, soldier piles and lagging, strut supported excavation, or other applicable methods. Sheet pile driving will be extremely difficult for very dense foundation materials, principally with measured SPT N-values larger than 50 blows per foot, a condition which would require tie-backs.

For utility excavations, trench shields may be used for excavation support, provided that some lateral movement and/or settlement of the ground surface adjacent to the trench can be tolerated. Otherwise, if a line drawn at a 45-degree angle from the bottom of edge of the excavation to the ground surface intersects a utility, foundation, or other permanent structures, which cannot tolerate movements, use of continuous (braced) shoring is recommended. This solution limits ground movement adjacent to the excavation or trench.

It is recommended that surface drainage be directed away from the top edge of all excavations.

Stantec recommends that the contractor be familiar with applicable local, state, and federal regulations for both temporary construction slopes and shoring, including the current Occupational Safety and Health Administration Excavation and Trench Safety Standards. Stantec further recommends that the contractor be contractually identified as solely responsible for the design and construction of temporary construction slopes and shoring.
Rock Excavation

Rock surface is most likely to be encountered in excavations to EL 46 feet at the pump station (Sheet M2) necessary for the installation of the 72-inch diameter pump cans and along the 84-inch diameter discharge pipeline. The top of bedrock could also be locally exposed in excavations along the proposed road adjacent to the substation. Other excavations could also expose the rock surface.

Boring logs A-9 and EAS-3 suggest that most probable rock types are sandstone, siltstone, claystone and shale (Attachment 1). The top of bedrock could be relatively shallow within 10 feet below ground surface. Blasting is not expected for any of the proposed facilities.

Rock excavation can be performed using multi-ripper (Attachment 3), or similar products. The ripper which has a width of 13 to 64 inches can be attached to an excavator or backhoe with a weight of 6 to 300 kilo pounds, respectively. The multi-ripper can be used to excavate a wide range of materials including frozen ground, coral, sandstone, limestone, shale, decomposed granite and caliche.

Trench excavation in rock can be accomplished using multi-ripper bucket (Attachment 3), or similar products. The bucket width is 18 to 54 inches which is attached to an excavator or backhoe with a weight of 6 to 300 kilo pounds, respectively. The trench width for the pipeline is expected to be approximately 11-foot-wide, such that multiple passes will be necessary to open the trench. The multi-ripper bucket is able to open vertical cuts in rock.

Embankment Fill

On-site stripped soil having an organic content less than 3 percent by volume may be used as fill material. All fill placed on the sites, including on-site soils, will not be allowed to contain rocks or lumps larger than 6 inches in the greatest dimension, with not more than 15 percent larger than 2.5 inches. Use of soils classified as Pt, OH, OL, MH, and CH under the Unified Soil Classification System as fill material is not recommended. Any imported fill will be required to be predominantly granular and, in addition, any fill material used as structural fill or wall backfill will be required to have a plasticity index of 15 percent or less.

Construction of permanent fill slopes at an inclination no steeper than 2H:1V is recommended.

Subgrade Preparation

After completion of clearing and stripping, it is recommended that the soil exposed in areas to receive structural fill or slabs-on-grade be scarified to a depth of 6 inches, and compacted to the requirements for structural fill. Proof-rolling of subgrades will be required by running heavy tracked or rubber tired construction equipment or a smooth steel drum roller (equipment operated or walk-behind) so that the entire subgrade surface is covered by at least two passes of the tire, track or drum. A geotechnical engineer will be required to observe the proof rolling and review the condition of prepared subgrade surfaces. In order to achieve satisfactory compaction of the
subgrade and fill materials, adjustment of the water content may be necessary at the time of construction. This may require water be added to soil that is too dry, or the scarification and aeration of soils that are too wet.

If required, Stantec recommends that areas of unstable soils be over-excavated a minimum of 18 inches below finished subgrade elevation. The bottom of the excavation will be required to be completely covered with a ground stabilization geotextile fabric such as Mirafi 500X, or equivalent, and backfilled with Class 2 aggregate base, or other granular material approved by the geotechnical engineer.

Where rock, or other unyielding material is encountered, the yielding material shall be removed for a minimum depth of two feet below grade and replaced with structural backfill.
Chapter 5
Recommendations for Next Design Phase

This preliminary geotechnical assessment requires confirmatory geotechnical investigation data to be collected within the footprints of the main structures, including the pump station, substation and MCC building. The geotechnical data can be obtained from:

- Borings,
- Cone Penetration Tests,
- Geophysical Investigations, and
- Laboratory Tests.
Chapter 5
Recommendations for Next Design Phase

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Chapter 6
Limitations

The findings, interpretations of data, recommendations, specifications or professional opinions are presented within the limits prescribed by available information at the time the technical memorandum was prepared, in accordance with generally accepted professional engineering and geologic practice and within the requirements of the client. There is no other warranty, either expressed or implied.

The findings of this technical memorandum are based on the readily available data and information obtained from public and private sources. Additional studies (at greater cost) may or may not disclose information, which may significantly modify the findings of this technical memorandum. In the event that there are any changes in the nature, design or location of the project, or if additional subsurface data are obtained or any future additions are planned, the conclusions and recommendations contained in this technical memorandum will need to be reevaluated by Stantec in light of the proposed changes or additional information obtained.
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Chapter 7
References


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Appendix B
Technical Memorandum: Neroly Pump Station Preliminary Geotechnical Assessment

Attachment 1 – Los Vaqueros Project Drawings
Los Vaqueros Reservoir Expansion Investigation, California

April 2018
PLAN

PROFILE

These record drawings have been prepared to incorporate such information and shall not be responsible for any errors or omissions which may be incorporated herein as a result.

CONTRA COSTA WATER DISTRICT

PIPELINE PLAN AND PROFILE
STA 0+31.60 TO STA 14+00
Appendix B
Technical Memorandum: Neroly Pump Station Preliminary Geotechnical Assessment

Attachment 2 – United States Geological Survey Design Maps

Los Vaqueros Reservoir Expansion Investigation, California

April 2018
User-Specified Input

Report Title: Neroly Pump Station Site 'B'
Tue February 27, 2018 02:24:47 UTC

Building Code Reference Document: ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates: 37.9825°N, 121.7486°W
Site Soil Classification: Site Class D – “Stiff Soil”
Risk Category: IV (e.g. essential facilities)

USGS-Provided Output

\[
\begin{align*}
S_s &= 1.500 \text{ g} & S_{sa} &= 1.500 \text{ g} & S_{sa} &= 1.000 \text{ g} \\
S_1 &= 0.524 \text{ g} & S_{ms} &= 0.786 \text{ g} & S_{ms} &= 0.524 \text{ g}
\end{align*}
\]

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.

For PGA, T_c, C_{es}, and C_{os} values, please [view the detailed report](https://earthquake.usgs.gov/cn1/designmaps/us/summary.php?template=minimal&latitude... 2/26/2018).
Design Maps Detailed Report

ASCE 7-10 Standard (37.9825°N, 121.7486°W)

Site Class D – “Stiff Soil”, Risk Category IV (e.g. essential facilities)

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain $S_s$) and 1.3 (to obtain $S_i$). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1

$$S_s = 1.500 \text{ g}$$

From Figure 22-2

$$S_i = 0.524 \text{ g}$$

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$\bar{v}_s$</th>
<th>$\bar{N}$ or $\bar{N}_{ch}$</th>
<th>$\bar{S}_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Hard Rock</td>
<td>&gt;5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>B. Rock</td>
<td>2,500 to 5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>C. Very dense soil and soft rock</td>
<td>1,200 to 2,500 ft/s</td>
<td>&gt;50</td>
<td>&gt;2,000 psf</td>
</tr>
<tr>
<td>D. Stiff Soil</td>
<td>600 to 1,200 ft/s</td>
<td>15 to 50</td>
<td>1,000 to 2,000 psf</td>
</tr>
<tr>
<td>E. Soft clay soil</td>
<td>&lt;600 ft/s</td>
<td>&lt;15</td>
<td>&lt;1,000 psf</td>
</tr>
</tbody>
</table>

Any profile with more than 10 ft of soil having the characteristics:
- Plasticity index $PI > 20$,
- Moisture content $w \geq 40\%$,
- Undrained shear strength $\bar{S}_u < 500$ psf

F. Soils requiring site response analysis in accordance with Section 21.1

See Section 20.3.1

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²
Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE<sub>r</sub>) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient Fa

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE&lt;sub&gt;r&lt;/sub&gt; Spectral Response Acceleration Parameter at Short Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( S_s \leq 0.25 )</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7 of ASCE 7</td>
</tr>
</tbody>
</table>

Note: Use straight–line interpolation for intermediate values of \( S_s \)

For Site Class = D and \( S_s = 1.500 \) g, \( F_a = 1.000 \)

Table 11.4–2: Site Coefficient Fv

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE&lt;sub&gt;r&lt;/sub&gt; Spectral Response Acceleration Parameter at 1–s Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( S_1 \leq 0.10 )</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
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<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7 of ASCE 7</td>
</tr>
</tbody>
</table>

Note: Use straight–line interpolation for intermediate values of \( S_1 \)

For Site Class = D and \( S_1 = 0.524 \) g, \( F_v = 1.500 \)
Equation (11.4–1):
\[ S_{NS} = F_a S_s = 1.000 \times 1.500 = 1.500 \text{ g} \]

Equation (11.4–2):
\[ S_{M1} = F_v S_1 = 1.500 \times 0.524 = 0.786 \text{ g} \]

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):
\[ S_{DS} = \frac{2}{3} S_{NS} = \frac{2}{3} \times 1.500 = 1.000 \text{ g} \]

Equation (11.4–4):
\[ S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 0.786 = 0.524 \text{ g} \]

Section 11.4.5 — Design Response Spectrum

From Figure 22-12 \(^3\)

\[ T_L = 8 \text{ seconds} \]

Figure 11.4–1: Design Response Spectrum

\[
\begin{align*}
S_a &= S_{DS} (0.4 + 0.6 T / T_o) \\
T_o \leq T \leq T_s : S_a &= S_{DS} \\
T_s < T \leq T_L : S_a &= S_{D1} / T \\
T > T_L : S_a &= S_{D1} T / T^2
\end{align*}
\]
Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCEₐ) Response Spectrum

The MCEₐ Response Spectrum is determined by multiplying the design response spectrum above by 1.5.
Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 \(^{(4)}\)  
\[ PGA = 0.500 \]

**Equation (11.8–1):**  
\[ PGAM = F_{PGA}PGA = 1.000 \times 0.500 = 0.5 \, g \]

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE Geometric Mean Peak Ground Acceleration, PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA ≤ 0.10</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td></td>
</tr>
</tbody>
</table>

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.500 g, \( F_{PGA} = 1.000 \)

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 \(^{(5)}\)  
\[ C_{AS} = 1.057 \]

From Figure 22-18 \(^{(6)}\)  
\[ C_{AS} = 1.083 \]
Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>VALUE OF $S_{0s}$</th>
<th>RISK CATEGORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I or II</td>
</tr>
<tr>
<td>$S_{0s} &lt; 0.167g$</td>
<td>A</td>
</tr>
<tr>
<td>$0.167g \leq S_{0s} &lt; 0.33g$</td>
<td>B</td>
</tr>
<tr>
<td>$0.33g \leq S_{0s} &lt; 0.50g$</td>
<td>C</td>
</tr>
<tr>
<td>$0.50g \leq S_{0s}$</td>
<td>D</td>
</tr>
</tbody>
</table>

For Risk Category = IV and $S_{0s} = 1.000$ g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>VALUE OF $S_{0s}$</th>
<th>RISK CATEGORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I or II</td>
</tr>
<tr>
<td>$S_{0s} &lt; 0.067g$</td>
<td>A</td>
</tr>
<tr>
<td>$0.067g \leq S_{0s} &lt; 0.133g$</td>
<td>B</td>
</tr>
<tr>
<td>$0.133g \leq S_{0s} &lt; 0.20g$</td>
<td>C</td>
</tr>
<tr>
<td>$0.20g \leq S_{0s}$</td>
<td>D</td>
</tr>
</tbody>
</table>

For Risk Category = IV and $S_{0s} = 0.524$ g, Seismic Design Category = D

Note: When $S_1$ is greater than or equal to 0.75g, the Seismic Design Category is E for buildings in Risk Categories I, II, and III, and F for those in Risk Category IV, irrespective of the above.

Seismic Design Category $\equiv$ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. Figure 22-1: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. Figure 22-17: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. Figure 22-18: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf
Appendix B
Technical Memorandum: Neroly Pump Station Preliminary Geotechnical Assessment

Attachment 3 – Multi-Ripper Bucket
Los Vaqueros Reservoir Expansion Investigation, California

April 2018
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Owners Claim: "this is the best attachment that I ever bought"

Highest quality manufacturing in USA to OEM specifications to fit any brand excavator or backhoe above 6,000 pounds. With Swedish Hardox 400 shanks and replaceable teeth!

<table>
<thead>
<tr>
<th>Machine Weight in Pounds</th>
<th>Cat Model Ref</th>
<th>Komatsu Model Ref</th>
<th>John Deere Model Ref</th>
<th>Hitachi Model Ref</th>
<th>Teeth</th>
<th>Width x Tip Radius</th>
<th>Wgt. lbs.</th>
<th>HARDOX Shank Thick</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-11,000</td>
<td>304CR</td>
<td>PC35MR</td>
<td>35D</td>
<td>ZX35U</td>
<td>23SRS</td>
<td>13 x 19</td>
<td>120</td>
<td>1</td>
</tr>
<tr>
<td>11-16,000</td>
<td>308</td>
<td>PC78</td>
<td>410</td>
<td>ZX60</td>
<td>MG10W</td>
<td>18 x 31</td>
<td>390</td>
<td>1.5</td>
</tr>
<tr>
<td>16-25,000</td>
<td>450</td>
<td>PC88</td>
<td>85D</td>
<td>ZX85</td>
<td>MG15W</td>
<td>20 x 32</td>
<td>575</td>
<td>1.5</td>
</tr>
<tr>
<td>25-30,000</td>
<td>312</td>
<td>PC130</td>
<td>120D</td>
<td>ZX120</td>
<td>MG20W</td>
<td>24 x 34</td>
<td>890</td>
<td>2</td>
</tr>
<tr>
<td>30-40,000</td>
<td>315</td>
<td>PC160</td>
<td>160D</td>
<td>ZX160</td>
<td>MG20W</td>
<td>24 x 36</td>
<td>930</td>
<td>2</td>
</tr>
<tr>
<td>40-50,000</td>
<td>320</td>
<td>PC200</td>
<td>200D</td>
<td>ZX200</td>
<td>MG30WR</td>
<td>30 x 38</td>
<td>1,200</td>
<td>2.5</td>
</tr>
<tr>
<td>50-65,000</td>
<td>325</td>
<td>PC220</td>
<td>227R</td>
<td>ZX270</td>
<td>MG30WR</td>
<td>30 x 38</td>
<td>1,260</td>
<td>2.5</td>
</tr>
<tr>
<td>65-85,000</td>
<td>330</td>
<td>PC300</td>
<td>350D</td>
<td>ZX350</td>
<td>MG55WR</td>
<td>36 x 44</td>
<td>2,050</td>
<td>3</td>
</tr>
<tr>
<td>85-110,000</td>
<td>345</td>
<td>PC400</td>
<td>450D</td>
<td>ZX450</td>
<td>MG55WR</td>
<td>36 x 45</td>
<td>2,300</td>
<td>3</td>
</tr>
<tr>
<td>110-160,000</td>
<td>365</td>
<td>PC600</td>
<td>650D</td>
<td>ZX650</td>
<td>MG80W</td>
<td>42 x 52</td>
<td>3,940</td>
<td>4</td>
</tr>
<tr>
<td>160-220,000</td>
<td>385</td>
<td>PC800</td>
<td>800D</td>
<td>ZX850</td>
<td>MG125W</td>
<td>46 x 56</td>
<td>4,930</td>
<td>4</td>
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<tr>
<td>220-300,000</td>
<td>5110</td>
<td>PC1250</td>
<td>1200D</td>
<td>EX1200</td>
<td>MG200W</td>
<td>64 x 70</td>
<td>7,100</td>
<td>4</td>
</tr>
</tbody>
</table>

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Toll Free: 866-928-5800 Email: Sales@Leattach.com Website: www.leadingedgeattachments.com

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*Breaking rock or frost with the maximum breakout force applied individually to each tooth
*Rip the sides of the trench with no depth limitation
*4 times faster than a hammer, at a fraction of the cost, using our patented (US 7,322,133 CA2,521,725)

Owners Claim: "this is the best attachment that I ever bought"

Highest quality manufacturing in USA to OEM specifications to fit any brand excavator, "Pin-Grabber" or Wain-Roy(R) style couplers for excavators or backhoes above 6,000 pounds. With AR400 shanks and replaceable teeth!

<table>
<thead>
<tr>
<th>Machine Weight in Pounds</th>
<th>Cat Model Ref</th>
<th>Komatsu Model Ref</th>
<th>John Deere Model Ref</th>
<th>Hitachi Model Ref</th>
<th>Teeth</th>
<th>Width x Tip Radius</th>
<th>Wgt. Lbs.</th>
<th>HARDOX Shank Thick</th>
<th>Heap Cap. CuYd</th>
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</thead>
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<tr>
<td>6-11,000</td>
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<td>PC88</td>
<td>85D</td>
<td></td>
<td>ZX85</td>
<td>18 x 30</td>
<td>560</td>
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<td>25-30,000</td>
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<td>ZX120</td>
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<td>30-40,000</td>
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<td>160D</td>
<td></td>
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<td>24 x 42</td>
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<td>200D</td>
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<td>ZX200</td>
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