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FINAL GEOLOGY and GEOTECHNICAL INVESTIGATION REPORT

Gray Lodge Wildlife Area Water Supply Project
Biggs-West Gridley Water District
Butte County, CA

SAGE Project No. 10-066.00

Prepared for:

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December 22, 2011

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December 22, 2011
SAGE Project No. 10-066.00

Mr. Randy Hopkins
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286 W. Cromwell Avenue
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Re: Final Geology and Geotechnical Investigation Report
Gray Lodge Wildlife Area Water Supply Project
Biggs-West Gridley Water District
Butte County, California

Dear Mr. Hopkins:

Sanders & Associates Geostructural Engineering, Inc. (SAGE) is pleased to submit this final report presenting the results of our geology and geotechnical investigation for the proposed Gray Lodge Wildlife Area Water Supply Project in Butte County, California. Our services have been performed in general accordance with the scope of services provided as Exhibit A (Revision 4, dated June 29, 2011) to our contract with Provost & Pritchard Engineering Group, Inc. (P&P). Of notable exception is the work pertaining to the Cassady Lateral, which has been eliminated from the project altogether.

The proposed project is located on approximately 16.25 miles of the existing Biggs-West Gridley Water District's (BWGWD) irrigation canal in Butte County. The project site begins at the intersection of Highway 99 and the canal, north of the city of Biggs, continues southwest past the cities of Biggs and Gridley, and terminates at the northern border of the Gray Lodge Wildlife Area at. The canal is divided into the following laterals: Belding, Schwind, Traynor, Cassady, and Rising River. Each lateral, with the exception of Cassady, will be improved as part of the project. From north to south, the canal intersects the Union Pacific Rail Road, Afton Road, Farris Road (twice), Colusa Highway (twice), W. Liberty Road, and W. Evans Reimer Road. The canal is surrounded mostly by agricultural land typically used for rice production. Residential and agricultural structures, farm equipment, fencing, overhead utilities, and canal structures infrequently crowd the project site.

The project consists of improving or replacing 31 individual structures along the canal, which consist of bridges, siphons, flumes, checks, and farm crossings. In addition, the canal will be graded to "smooth" the channel to improve the hydraulics and portions will be widened to increase capacity. Furthermore, a new segment of canal will be excavated along the Belding Lateral which will reduce canal curvature between approximate stations 252+00 and 262+00.

Our investigation was conducted to evaluate surface and subsurface conditions along the project site; assess the potential for adverse geologic conditions which may impact the feasibility and/or constructability of the proposed structures; and to obtain information to develop geotechnical design criteria for design of the proposed structures.

The report submitted herewith contains recommendations regarding foundation design, pavement design, and site grading that should be reviewed in their entirety. These recommendations are based on limited and widely-spaced subsurface exploration and laboratory testing at the major structures identified herein. Consequently, variations between expected and actual soil conditions may be found during construction. We recommend that SAGE be retained to observe earthwork and foundation installation to assist in identifying such variations and to evaluate whether our recommendations remain valid for the actual geotechnical conditions encountered during construction.

At the time this report was prepared, the project was still under design. We understand that additional structures are being considered along the Belding, Traynor and Schwind Laterals. Furthermore, work may ultimately be performed on other laterals within the BWGWD system that were not included in our subsurface exploration in order to meet water conveyance objectives for the project. Once completed, we should be provided updated plans showing additional proposed structures and improvements. We will review the plans and provide our recommendations on whether additional subsurface investigation is warranted. The need for supplemental investigation will depend on the type of structure proposed and its distance from existing subsurface borings.

Please call us should you have questions.

Sincerely yours,
Sanders & Associates Geostructural Engineering, Inc.

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BUTTE COUNTY, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geology and geotechnical investigation for the proposed Gray Lodge Wildlife Area Water Supply Project. The purpose of the project is to improve water conveyance to the Gray Lodge Wildlife Area in accordance with Biggs-West Gridley Water District's (BWGWD) Cooperative Agreement with the Bureau of Reclamation. The canal system is operated and maintained by BWGWD and includes the following irrigation laterals: Belding, Schwind, Traynor, Cassady, and Rising River. Each lateral, with the exception of Cassady, will be improved as part of the project. The project is located in Butte County, north of the Gray Lodge Wildlife Area and west of Highway 99 and the cities of Biggs and Gridley (see Figure 1).

Improvement of water conveyance will be achieved by retrofitting or replacing up to 31 individual structures throughout the canal system, as well as modifying the canal cross-section to improve hydraulics. The proposed structures include cast-in-place reinforced concrete county road bridges, flumes, checks, and siphons. The structures were originally identified in the *Design Data Report for Conveyance of Refuge Water Supply to Gray Lodge Wildlife Area (DDR),* dated August 2009 (CH2MHill, 2009). Our field investigation was completed at the major structure locations from the DDR, which were also included in the 30% design submittal drawings (Provost & Pritchard, 2011a). Table 1 below provides a summary of the proposed major structures and overall dimensions. Loading on the structures will vary by location and purpose, but are generally expected to consist of a combination of retained soil loads, hydraulic pressure from the canal and water table, and vehicular surcharges from farm equipment. Minor structures on the project include turnouts and culverts that are proposed to be removed, relocated, and/or replaced; these minor structures have not been included in Table 1.

Subsequent to our field investigation, Provost & Pritchard (P&P) implemented changes to the quantity and type of the structures proposed for the project. Although, based on the structures identified on the 30% plans, Table 1 reflects the revised structure types and/or dimensions shown in the 90% design submittal drawings (Provost & Pritchard, 2011b). However, several new major structures are shown on the 90% design plans that were not included in the DDR or 30% drawings. These new structures are not included in Table 1, nor are they explicitly addressed in this report. Care should be taken on using information contained herein for structures not listed in Table 1 because of the potential for unexpected subsurface conditions at unexplored locations. Our recommendations to perform a supplemental investigation at the additional structure locations are presented in Section 8.0.

Additional improvements will consist of regrading the canal. Grading is expected to consist of "smoothing" the channel invert and sides to improve the hydraulics, widening portions of the canal to increase capacity, and widening the access roads on top of the canal. Locally, this will require cuts and fills, generally on the order of 1 to 2 vertical feet, along the canal bottom and berms. However, portions of the berms along the canal will be reconstructed or widened which will require cuts and fills of up to 10 vertical feet.

TABLE 1

Proposed Major Structure Characteristics

STRUCTURE TYPE	OVERALL DIMENSIONS (FEET)			DESIGN FLOW (CFS)	STRUCTURE NAME/LATERAL
	Length (1)	Width (2)	Height (3)		
Check	14 to 79.5	42 to 66.3	6.5 to 10.6	120 to 750	Garcia/Belding Banion/Belding North/Belding #349/Belding #376/Belding #422/Belding Bonslett/Belding #058/Schwind #102/Traynor #059/Rising River
Headgate	36 to 59.5	42 to 57.2	9.3 to 9.8	100 to 380	Ashley/Belding Traynor/Traynor
Headgate/Crossing	51	41	11.6	270	Division 2/Belding
Flume	79.5 to 193	55 to 116	7.8 to 14	100 to 850	Razorback/Belding Garcia/Belding Fields/Belding Schwind/Schwind Nugent/Traynor
Siphon	118 to 124	32 to 62	16.2 to 20.5	85 to Por. 850	U.P.R.R./Belding Liberty Rd/Schwind
Bridge	29.5 to 35	35.4 to 55	5.4 to 12.3	120 to 750	Afton Rd./Belding N. Farris Rd./Belding S. Farris Rd./Belding Colusa Hwy./Traynor Evans Reimer Rd./Rising River
Farm Crossing/Bridge	28 to 32	39.5 to 65.9	6.9 to 13	220 to 380	#407/Belding #443/Belding Bonslett/Belding #077/Traynor
Farm Crossing/Culvert	52	40.5 to 42	7.6 to 8.2	85	#071/Schwind #100/Schwind

Notes: (1) Length: measured parallel to canal, includes wingwalls
(2) Width: measured perpendicular to canal, includes wingwalls
(3) Height: measured above canal invert

2.0 SCOPE OF SERVICES

We performed this investigation in general accordance with our scope of services presented with the agreement between Provost & Pritchard Engineering Group, Inc. (P&P) and Sanders & Associates Geostructural Engineering, Inc. (SAGE) dated June 29, 2011. Our scope of services consisted of a review of existing geotechnical and geologic data for the site and vicinity, review of aerial photographs, preparation of a subsurface exploration plan and drilling schedule, coordinating our exploration, performing a subsurface exploration program including forty-one (41) soil borings, installing twelve (12) piezometers, and laboratory analysis of select soil samples to evaluate site-specific subsurface conditions for the proposed project. However, due to site access constraints, design progress, and other factors, options for improving, bypassing, and/or eliminating the Cassady Lateral are being evaluated. Therefore, only thirty-seven (37) borings were drilled and only six (6) piezometers were installed. The evolution of our field exploration is discussed further in Appendix A.

Based on the results of our field and laboratory investigations, we performed geotechnical engineering analyses to develop conclusions and recommendations regarding:

- Subsurface conditions and engineering properties of soils;
- Regional seismicity and seismic hazards;
- Bearing capacity, expected settlement, and friction factor for shallow foundations;
- Vertical and lateral capacity of pile foundations for farm crossings;
- Retaining wall design parameters, including active, passive, and at-rest soil pressures;
- Fill quality and compaction;
- Utility installation;
- Temporary shoring;
- Construction dewatering;
- Asphalt and concrete pavements;
- Typical soil types and grain sizes for scour analysis;
- Canal slope stability;
- Bore & jack design parameters, including jacking and receiving pits;
- Lane's creep ratio; and
- Seismic design parameters.

3.0 GEOLOGIC SETTING

The site is located in the Great Valley geomorphic province of California, which is an alluvial plain approximately 50 miles wide and 400 miles long in the central part of California. The Great Valley is a structural depression that has been filled with a thick sequence of Mesozoic and Tertiary marine sediments covered by Quaternary alluvial sediments. Subsequent deformation has folded these older sediments into a northwest-trending asymmetrical syncline with its axis off center toward the Coast Ranges.

The Great Valley province is characterized by meandering fluvial systems, particularly along the Sacramento River, which drains the northern part of the Great Valley. In general, coarse-grained (sand and gravel) alluvial fan deposits are typically found along the perimeter of the valley and near the meandering Sacramento River; fine-grained (silt and clay) alluvial deposits are typically found towards the center of valley and on the floodplains of the river.

The project site is composed largely of undivided basin deposits and modern alluvium. The basin deposits provide the rich farmland soil present within the project area. Locally, deposits of the Upper and Lower members of the Modesto Formation are present. These deposits consist of sediments derived from volcanic rocks from the Cascade Mountains to the northeast of the site. The Modesto Formation can generally be found bordering the existing streams and drainages in the project area (Helley and Harwood, 1985).

4.0 AERIAL PHOTOGRAPH REVIEW

We reviewed available historic stereo-paired aerial photographs covering the project vicinity on file at the Shields Library, UC Davis. The objective of our review was to develop a limited history of canal system, and to identify canal sections that may have been altered or abandoned in the vicinity of the proposed improvements. In all, we reviewed 5 sets of aerial photographs flown between 1958 and 1984, ranging in scale from 1:20,000 to 1:40,000. A table of aerial photographs reviewed is included in the reference section of this report. A digital photograph was taken of each aerial photo reviewed which will remain on file with SAGE.

The canal system appears to be largely unchanged from 1958 to the present day. Locally, canal alignments have been altered or abandoned; however, the canal changes visible in the photographs are outside of the areas of the proposed improvements. The fields surrounding the canals have greatly changed from grading on-contour to grading large flat open areas.

5.0 SITE AND SUBSURFACE CONDITIONS

The site is located to the north of the Gray Lodge Wildlife Area and west of Highway 99 in Butte County. The irregularly-shaped site comprises approximately 16.25 miles of water supply canal examined for the purposes of this report (see Figures 1 and 2).

The site is primarily used for agricultural purposes and is relatively flat but has an overall gradient down to the southwest.[1] The highest point on the canal system is at the northern end of the Belding Lateral and has an approximate elevation of 105.5 feet (NAVD88) on the berm crest. The elevation along the berm crest gradually decreases to 72.5 feet at the southwestern terminus (Schwind Lateral) and 81.5 feet at the southeastern terminus (Rising River Lateral) of the canal system.

The canal was constructed by excavating in natural ground and placing the excavated materials along the edges to construct berms (embankment fills). Therefore, the base of the canal is located below the surrounding ground surface. Generally speaking, the crest of the berms is approximately 2 to 10

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1 Site grades estimated from "Gray Lodge Wildlife Area Water Supply Project, 30% Design Submittal Plans", Sheets PP-01 through PP-42, prepared by Provost & Pritchard Consulting Group, dated 8/19/2011.

feet above the adjacent farmland. Typically, the canal is unlined and has a trapezoidal-shaped cross-section. The canal side slopes have variable inclinations but are generally steeper than 2H:1V and flatter than 1.5H:1V. There are localized areas as steep as 1H:1V and as flat as 2.7H:1V. Portions of the canal side slopes are covered with rip-rap, particularly near structures and at sharp curves in the canal.

The canal is paralleled on both sides by a dirt road on the crest of a raised berm. The dirt roads were typically dry and well compacted at the surface due to operation of farm and BWGWD equipment. At some locations, the roads were recently disced and wetted by farming equipment. Gravels, cobbles, and to a lesser extent, weeds and clamshells sparsely cover the roads. In some instances, concrete debris, shotgun shells, fencing materials, irrigation pipes, and other debris are also present. The surrounding farmland is typically used for rice production, and as a result, the adjacent fields were filled with standing water at the time of our investigation. The marshy conditions made the fields and edge of roadways very soft and muddy. In addition, drainage ditches are located along the toe of the canal berms that are generally full of water. While most of the adjacent lands are used for rice farming, there are areas where orchards abut the canal system.

We explored the subsurface soil conditions by drilling 37 test borings across the site between July 19, 2011 and August 10, 2011. The borings were advanced through the dirt roads on the canal berm crests to depths varying between 26.5 feet and 51.5 feet, depending on the structure and improvement type at that location. See Tables 2 through 5 for boring locations based on canal lateral and structure type. Additional information regarding the field investigation is provided in Appendix A.

5.1 Belding Lateral

The Belding Lateral is the northernmost main lateral of the canal within the BWGWD boundary. This lateral has both east-west and north-south trending alignments and has a total length of approximately 9.2 miles. The northernmost segment of the lateral, between Highway 99 and the Traynor Lateral (5.6 miles), has depths ranging from 9 to 10 feet below the top of the berm. The southernmost segment between the Traynor and Schwind Laterals (3.6 miles) has depths ranging from 6 to 8 feet. A total of 21 borings (SB-1 to SB-22, excluding SB-15) were completed on this lateral - 14 on the northern segment and 7 on the southern segment. The locations of the borings are shown on Figures 3 through 5.

We have provided a generalized summary of the subsurface conditions encountered at each structure along the Belding Lateral in Table 2. Typically, the materials encountered on the Belding Lateral are fine-grained and lean. The majority of the subsurface materials were very stiff to hard clay or sandy clays. The sands within the clay were very fine grained, nearly classifiable as silt, and contributed to the lean nature of the material. Many of the borings had a layer of soft to hard fat clay of varying thickness, usually within the upper 10 feet of the boring. Intermediate and variable layers of silts or sands were also common, particularly below 15 feet. The ground water depth ranged between 4.5 and 20.5 feet with an average of 8.7 feet.

Borings SB-13, SB-14, and SB-18 through SB-21 encountered subsurface materials with increased granular soil (sand and gravel) content. SB-13 was typically very stiff sandy silt below 8 feet but contained some loose silty gravel at 13 feet below the berm crest. SB-14 was predominantly medium dense to loose fine grained sand to 10 feet below the ground surface, followed by the typical clayey material. In SB-18 through SB-21 we typically encountered fine grained sands and some gravels with fewer layers of clay. The sands had low fines contents and were not cohesive. This, in combination with the varying density of the sands, caused the sands to flow into the drill hole and seize the drilling equipment, which made sampling difficult.

TABLE 2

Summary of Belding Lateral Borings

STRUCTURE NAME AND TYPE [1]	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS [2] (DEPTHS IN FEET)
Razorback Flume	SB-1	31.5	0-6: Clay, v. stiff (*fill*) 6-8.5: Clay, with sand, very stiff 8.5-13.5: Silt, with sand, hard 13.5-31.5: Alternating layers of Silt with sand, hard and Sand, dense
	SB-2	31.5	0-6: Silt, v. stiff (*fill*) 6-24: Clay, variable sand, m. stiff to hard 24-31.5: Silt and Clay, hard
U.P.R.R. Siphon and Ashley Headgate	SB-3	26.5	0-5: Clay, v. stiff (*fill*) 5-8: Clay, soft to v. stiff 8-13: Sand, clayey, m. dense 13-19: Silt, sandy, v. stiff 19-25: Sand, m. dense 25-26.5: Silty Clay, hard
	SB-4	31.5	0-4.5: Clay, sandy, stiff (*fill*) 4.5-13: Clay, stiff to hard 13-19: Sand, with silt, m. dense 19-31.5: Silt, sandy, v. stiff to hard
Garcia Check	SB-5	26.5	0-4: Clay, sandy, stiff (*fill*) 4-8: Clay, sandy, v. stiff 8-13: Sand, clayey, m. dense 13-18: Silt, sandy, v. stiff 18-26.5: Clay, with sand, stiff to hard
Garcia Flume	SB-6	31.5	0-5.5: Clay, sandy, stiff (*fill*) 5.5-8: Clay, m. stiff 8-15: Sand, clayey and silty, m. to v. dense 15-18: Silt, sandy, hard 18-31.5: Clay, sandy, soft to hard
	SB-7	31.5	0-4: Clay, with sand, stiff (*fill*) 4-13: Clay, variable sand, stiff 13-31.5: Clay, variable sand, hard

STRUCTURE NAME AND TYPE (1)	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS (2) (DEPTHS IN FEET)
Afton Rd. Bridge	SB-8	41.5	0-4: Clay, sandy, v. stiff (*fill*) 4-23: Clay, sandy, stiff to hard 23-28: Silt, sandy, m. stiff 28-33: Clay, hard 33-38: Sand, with silt, loose to m. dense 38-41.5: Clay, sandy, hard
	SB-9	41.5	0-3.5: Clay, sandy, v. stiff (*fill*) 3.5-23: Clay, variable sand, stiff to hard 23-31: Sand, m. dense to dense 31-34: Clay, variable sand, hard 34-36: Sand, with clay, m. dense 36-41.5: Clay, variable sand, v. stiff to hard
Banion Check	SB-10	26.5	0-4: Clay, v. stiff (*fill*) 4-18: Clay, variable sand, v. soft to hard 18-25: Sand, variable silt, loose to m. dense 25-26.5: Clay, sandy, m. stiff
Fields Flume	SB-11 SB-12	31.5	0-4.5: Clay, variable sand, stiff (*fill*) 4.5-31.5: Clay, variable sand, m. stiff to hard
N. Farris Rd. Bridge and North Check	SB-13	26.5	0-3.5: Clay, sandy, stiff (*fill*) 3.5-8: Clay, m. stiff 8-15.5: Sand and Gravel, silty/clayey, loose to m. dense 15.5-25.5: Silt, sandy, v. stiff 25.5-26.5: Clay, sandy, hard
Division 2 Headgate	SB-14	31.5	0-6: Sand, loose to medium dense (*fill*) 6-31.5: Clay, variable sand, m. stiff to hard
Check #349	SB-16	26.5	0-4: Clay, with sand, m. stiff (*fill*) 4-13: Clay, variable sand, stiff to hard 13-16: Sand, v. dense 16-18: Clay, sandy, hard 18-20.5: Sand, clayey, m. dense 20.5-26.5: Silt and Clay, variable sand, stiff to hard
Check #376	SB-17	26.5	0-2: Clay, m. stiff (*fill*) 2-26.5: Clay, variable sand, m. stiff to hard
Farm Crossing #407 Bridge	SB-18	26.5	0-1: Clay, stiff (*fill*) 1-4: Clay, stiff 4-9: Sand, clayey, v. loose 9-23: Sand, dense 23-26.5: Clay, hard
Check #422	SB-19	26.5	0-1: Clay, with sand, stiff (*fill*) 1-8: Clay, with sand, stiff 8-15: Sand, variable clay, m. dense 15-23: Clay and Silt, variable sand, v. stiff 23-26.5: Sand, m. dense

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STRUCTURE NAME AND TYPE (1)	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS (2) (DEPTHS IN FEET)
Farm Crossing #443 Bridge and S. Farris Rd. Bridge	SB-20 SB-21	41.5	0-3: Clay, stiff (*fill*) 3-18: alt. layers of Clay, soft to v. stiff and Sand, m. dense 18-23: Silt, sandy, stiff to v. stiff 23-33: Sand, variable silt, loose to v. dense 33-41.5: Sand and Gravel, m. dense to v. dense
Bonslett Check and Farm Crossing Bridge	SB-22	25.5	0-3: Clay, stiff 3-8: Silt, sandy, hard 8-22: Clay, variable sand, hard 22-25.5: Sand, with gravel, dense

Notes: (1) Structure names follow the naming convention established in the 90% Design Plans.
(2) These are generalized summaries. Refer to the actual boring logs in Appendix A for detailed subsurface profiles at each boring location.

5.2 Schwind Lateral

The Schwind Lateral has a north-south trending alignment that begins at the end of the Belding Lateral, approximately 1 mile north of Colusa Highway, and continues down to W. Liberty Road. The total length of the canal is approximately 2.1 miles. The depth of the canal ranges from 6 to 8 feet below the top of the berm. From north to south, the borings along this lateral are designated SB-23 to SB-29. The locations of the borings are shown on Figure 6.

The subsurface materials were typically medium stiff to hard clay to sandy clay. In all but SB-29, a 3- to 7-foot-thick layer of fat clay was encountered at or just below the ground surface. Borings SB-26 and SB-27 transitioned to silty sand, sandy silt, and clean fine grained sand between 13 and 22 feet. Ground water was encountered between 3.8 and 8 feet, with an average of 5.8 feet.

TABLE 3
Summary of Schwind Lateral Borings

STRUCTURE NAME AND TYPE (1)	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS (2) (DEPTHS IN FEET)
Schwind Flume/Crossing	SB-23 SB-24	26.5	0-26.5: Clay, variable sand, m. stiff to v. stiff (*upper 2' fill in SB-24*)
Check #058 Crossing	SB-25	26.5	0-2.5: Clay, stiff (*fill*) 2.5-9: Clay, variable sand, stiff 9-14: Silt, v. stiff 14-26.5: Clay, variable sand, v. stiff
Farm Crossing #071 Culvert	SB-26	26.5	0-2.5: Clay, m. stiff (*fill*) 2.5-13: Clay, variable sand, m. stiff to stiff 13-18: Sand, silty, m. dense 18-21: Silt, sandy, v. stiff 21-26.5: Clay, sandy, v. stiff

STRUCTURE NAME AND TYPE [1]	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS [2] (DEPTHS IN FEET)
Farm Crossing #100 Culvert	SB-27	26.5	0-3: Clay, m. stiff *(fill)* 3-13: Clay, m. stiff to v. stiff 13-18: Silt, sandy, hard 18-22: Sand, with silt, m. dense 22-26.5: Clay, v. stiff to m. stiff
W. Liberty Rd. Siphon	SB-28 SB-29	31.5	0-2: Clay, variable sand, stiff (*fill*) 2-31.5: Clay, variable sand, stiff to hard (*upper 3' fill in SB-29*)

(1) Structure names follow the naming convention established in the 90% Design Plans.
(2) These are generalized summaries. Refer to the actual boring logs in Appendix A for detailed subsurface profiles at each boring location.

5.3 Traynor Lateral

The Traynor Lateral has a north-south trending alignment that begins at the Belding Lateral near boring SB-14, approximately 1.7 miles north of Colusa Highway, and continues south to W. Liberty Road. The total length of the canal is approximately 3.3 miles. The depth of the canal along this lateral ranges from 9 to 14 feet below the top of the berm. From north to south beginning at the Belding Lateral split, the borings are designated SB-15 and SB-30 to SB-34. SB-33 and SB-34 are on either side of the Colusa Highway on a particularly narrow portion of the canal berms. Two additional borings were completed along this lateral in order to construct piezometers identified as TRA2 and TRA4. The locations of the borings and piezometers are shown on Figures 4, 7, and 8.

The soils observed from the Traynor Lateral borings were the most variable of the project, but were typically composed of fine grained materials. The fine grained material ranged from very soft to hard sandy clays, clays, sandy silts, and silts. SB-15, SB-31, SB-33, and SB-34 also have intermittent layers of medium dense to dense, fine- to coarse-grained sands with varying fines contents, except for layer of very loose sand encountered in the embankment at SB-15. In SB-30, very soft, highly plastic clays were encountered. Ground water was encountered between 6.5 and 13 feet, with an average of 8.5 feet.

TABLE 4
Summary of Traynor Lateral Borings

STRUCTURE NAME AND TYPE [1]	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS [2] (DEPTHS IN FEET)
Traynor Headgate	SB-15	26.5	0-6: Sand, variable clay and silt, v. loose to loose (*fill*) 6-26.5: Clay, variable sand, v. soft to hard
Nugent Flume	SB-30	31.5	0-3: Clay, sandy, stiff (*fill*) 3-9: Clay, very soft 9-14: Silt, soft 14-31.5: Clay, variable sand, stiff to hard

STRUCTURE NAME AND TYPE (1)	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS (2) (DEPTHS IN FEET)
Farm Crossing #077 Bridge	SB-31	26.5	0-3.5: Clay, sandy, v. stiff (*fill*) 3.5-8: Clay, with gravel, v. stiff 8-13.5: Sand and Silt, m. dense/v. stiff 13.5-26.5: Clay, variable sand, hard
Check #102	SB-32	26.5	0-4: Clay, m. stiff (*fill*) 4-8: Clay, m. stiff 8-15.5: Silt, sandy, v. stiff to hard 15.5-26.5: Clay, v. stiff to hard
Colusa Hwy. Bridge	SB-33	51.5	0-4.5: Clay, stiff to m. stiff (*fill*) 4.5-13: Clay, soft to hard 13-20.5: Sand, dense 20.5-28: Clay, v. stiff 28-33: Sand, silty, dense 33-43: Clay, variable sand, hard 43-51.5: alt. layers of Sand, m. dense and Clay, stiff to hard
	SB-34	51.5	0-3.5: Clay, with sand, stiff (*fill*) 3.5-15.5: Clay and Silt, variable sand, stiff to hard 15.5-18: Sand, with silt, v. dense 18-33: Clay, variable sand, v. stiff to hard 33-38: Sand, silty, m. dense 38-51.5: Clay, v. stiff to hard
Piezometer	TRA2	13	0-4: Clay, v. soft (*fill*) 4-5.5: Clay, soft 5.5-13: Silt, stiff to v. stiff
Piezometer	TRA4	13	0-4.5: Clay, m. stiff (*fill*) 4.5-6: Clay, stiff 6-13: Silt, sandy, very stiff

(1) Structure names follow the naming convention established in the 90% Design Plans.
(2) These are generalized summaries. Refer to the actual boring logs in Appendix A for detailed subsurface profiles at each boring location.

5.4 Cassady Lateral

Three borings and one piezometer were originally included in our scope and scheduled to be completed on the Cassady Lateral, which trends from the terminus of the Traynor Lateral and continues west approximately 2.8 miles. Our scope for exploration on the Cassady Lateral has been put on hold as P&P evaluates options to potentially eliminate the Cassady Lateral from the project scope altogether. As such, this report does not explicitly address proposed improvement along the Cassady Lateral.

5.5 Rising River Lateral

The Rising River Lateral extends from the terminus of the Traynor Lateral at W. Liberty Road and extends southwest to the intersection of the canal with W. Evans Reimer Road. The total length of the canal is approximately 1.5 miles. The depth of the canal along this lateral ranges from 6 to 7 feet below the top of the berm. Borings continue southwest and end with SB-39 and SB-40 on either side of W. Evans Reimer Road. One additional boring was completed along this lateral in order to construct a piezometer identified as TRA6. The locations of the borings and piezometer are shown on Figure 8.

The materials encountered in borings SB-38 through SB-40 were similar to those encountered over the entire project. The predominant material was typically stiff to hard clay to sandy clay with silt being less common. Locally, soft clay was encountered near the elevation of the existing canal invert in SB-39. Layers of fat clay and silty to clean fine grained sands were intermittent throughout the Rising River Lateral. At SB-38, SB-39, and SB-40, ground water was encountered at 5, 7, and 12 feet, respectively.

TABLE 5

Summary of Rising River Lateral Borings

STRUCTURE NAME AND TYPE (1)	BORING NAME	TOTAL DEPTH (FT)	SUBSURFACE CONDITIONS (2) (DEPTHS IN FEET)
Check #059	SB-38	26.5	0-1: Clay, soft (*fill*) 1-26.5: Clay, variable silt, soft to hard
W. Evans Reimer Rd. Bridge	SB-39	41.5	0-4: Clay, sandy, stiff (*fill*) 4-8: Clay, sandy, soft 8-21.5: Sand, variable silt and clay, m. dense to dense 21.5-28: Clay, variable sand, v. stiff to stiff 28-41.5: alt. layers of Sand, loose to m. dense and Clay, stiff to v. stiff
	SB-40	41.5	0-8: Clay, stiff to v. stiff 8-11.5: Sand, silty, m. dense 11.5-28: Clay and Silt, stiff to v. stiff 28-35.5: Sand, variable silt, loose to m. dense 35.5-41.5: Clay, sandy, hard and Sand, silty, dense
Piezometer	TRA6	13	0-2.5: Clay, stiff (*fill*) 2.5-6: Clay, stiff 6-13: Silt, sandy, very stiff

(1) Structure names follow the naming convention established in the 90% Design Plans.
(2) These are generalized summaries. Refer to the actual boring logs in Appendix A for detailed subsurface profiles at each boring location.

5.6 Erosion Characteristics

Clay soils, such as those that comprise a majority of the near surface soils at the site, are generally considered to be resistant to erosion by flowing water. However, some naturally deposited clay soils can deflocculate in the presence of water and would therefore be prone to erosion and piping. This type of clay is known as dispersive clay and is the result of the depositional environment. As part of this investigation, five pinhole dispersion tests were completed to provide a qualitative determination of whether the clay soils at the site might be dispersive. The results of the tests are included in Appendix B and indicate the clay soils are nondispersive to slightly dispersive. The turbidity at the end of the tests was reported as clear to barely visible.

6.0 SEISMICITY

6.1 Regional Seismicity

Seismicity is defined as the geographical and historical distribution of earthquakes, or more simply, earthquake activity. The potential for ground shaking at the site is related to earthquake activity that might occur along nearby or distant faults. Based on historical earthquake activity and fault hazard mapping, the general site region is considered to have a relatively low to moderate potential for seismic activity.

Based on our review of available published geologic maps, U.S. Geological Survey Quaternary Fault and Fold Database, and State of California Alquist-Priolo maps, there are no active[2] surface fault traces mapped in the site vicinity (Saucedo and Wagner, 1992; USGS, 2010; Hart and Bryant, 2007). An unnamed, northeast-trending, concealed fault is depicted on the 1:250,000 scale geologic map as crossing the southwestern corner of the project area (Saucedo and Wagner, 1992), but the fault is not zoned as active (Hart and Bryant, 2007). Although there are no Quaternary (movement within the last 1,600,000 years) faults mapped in the immediate site vicinity, there are several Quaternary faults mapped in the project region:

- The Foothills fault system is a group of northwest-trending faults that tectonically separate distinctive belts of Paleozoic and Mesozoic rocks for more than 200 miles along the western foothills of the Sierra Nevada (Clark, 1960). The fault system terminates near Lake Oroville, approximately 15 miles east of the site. During the last five million years, the Sierra Nevada has been uplifted as a tilted block by active faults along the steep eastern escarpment of the mountain range. In response to this uplift, microseismicity and small fault displacements have occurred along the Foothills fault system. On August 1, 1975, a magnitude 5.7 earthquake and associated surface ruptures occurred near Oroville (Sherburne and Hauge, 1975), focusing attention on the Foothills fault system as a potential area of active faulting (Harwood et al., 1981). The Foothills fault system is not currently zoned as active under the State of California Alquist-Priolo Earthquake Fault Zoning Act, except for the Cleveland Hill fault which experienced ground rupture during the 1975 Oroville earthquake (Hart and Bryant, 2007; CDMG, 1977). The Cleveland Hill fault is located approximately 15 miles northeast of the site. The maximum moment magnitude earthquake estimated for the Foothill fault system is M_w 6.5, with a recurrence interval of about 12,500 years (CDMG, 1996).
- The Chico Monocline is located approximately 15 miles north of the site, and is composed of a northwest-trending, southwest dipping flexure along the northeast side of the Sacramento Valley (Harwood et al, 1981). The monocline formed between 1.0 and 2.6 million years ago from uplift of the northern Sierra Nevada due to rupture along a concealed

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[2] Active faults are defined as those exhibiting either surface ruptures, topographic features created by faulting, surface displacements of Holocene (younger than about 11,000 years old) deposits, tectonic creep along fault lines, and/or close proximity to linear concentrations or trends of earthquake epicenters.

fault beneath the monocline(Harwood and Helley, 1987). The Chico Monocline fault is not currently zoned as active under the State of California Alquist-Priolo Earthquake Fault Zoning Act (Hart and Bryant, 2007).

- The Corning Fault/Willows Fault Zone, located approximately 25 miles northwest of the site, trends parallel to the Chico Monocline fault and exhibits uplift on the eastern side of the zone similar to the Chico Monocline.
- The Resort Fault Zone, located approximately 40 miles southwest of the site, is composed of an approximate 1-mile wide normal fault zone with right-lateral movement (McLaughlin and others, 1989).
- Several minor unnamed faults also exist within the Sutter Butte Mountains to the south of the site.

6.2 Seismic Hazards

An earthquake on a segment of one of the regional faults could result in low to moderate ground shaking at the site. We evaluated the anticipated level of shaking to determine if seismic hazards, such as liquefaction or ground fault rupture, could impact the project site. Our evaluation of the potential seismic hazards at the site is presented in the following subsections.

6.2.1 Ground Shaking

We expect the site will experience low to moderate ground shaking. The intensity of ground shaking at the site depends on many factors, including the size of the fault generating an earthquake event, the distance from the fault rupture to the project site, and the duration of strong ground shaking.

Based on review of the United States Geologic Survey (USGS) Probabilistic Hazards Curves (2002) and design parameters for use with the 2010 California Building Code (CBC), the estimated peak ground acceleration (PGA) at the site is about 0.21 g for Site Class D (deep soil deposits), which corresponds to a low to moderate level of shaking. Design parameters for use with the 2010 CBC are presented later in this report.

6.2.2 Soil Liquefaction and Associated Hazards

Soil liquefaction is the sudden and rapid reduction in the shear strength of a soil due to an increase in excess pore pressure caused by cyclic loading under undrained loading conditions, most commonly, strong ground shaking. In the case of complete soil liquefaction, physical properties of the soil become similar to a heavy fluid rather than a soil, and a nearly complete loss of shear strength can occur. Soils most prone to liquefaction are clean, fine-grained, uniformly graded sands. However, sand with varying amounts of silt and clay, non-plastic silts, some fine gravel, and sensitive clays may also liquefy and/or lose strength during strong cyclic loading. Phenomena associated with liquefaction include sand boils, flow failure, lateral spreading, differential settlement, loss of bearing strength, and ground fissures.

Because liquefaction occurs due to the buildup of pore-water pressure within the soil skeleton, potentially liquefiable soils are generally below the groundwater table. Static groundwater was encountered between 4.0 to 20.5 feet of the ground surface with an average depth of 8 feet. The shallow levels are likely due to heavy irrigation in the rice fields and water levels in the canal. The typical subsurface material, consisting of clay and silt with varying amounts of fine grained sand, have a low susceptibility of liquefaction when below the water table due to their consistency and/or plasticity using the methods presented by Boulanger and Idriss (2006).

The submerged sand layers encountered beneath the site were evaluated for their liquefaction potential. A screening-level evaluation was performed first, followed by a detailed analysis with layers found to be potentially liquefiable. The methodologies presented by Seed and Idriss (1982) and Youd et. al. (2001) were used in our screening-level and detailed analyses. In performing the analyses, a PGA of 0.21 g and an M_w of 6.5, which are both consistent with the regional seismicity, were assumed.

The screening-level evaluation conservatively neglected fines content and found that potentially liquefiable layers were those layers that had N_{60}[3] values $\leq$ 23 (i.e., N_{60} > 23 has a $FOS_{liq} \geq 1.3$). The results of the screening-level evaluation indicate that 22 different sand layers in 18 borings are potentially liquefiable. The layers range in thickness from 2 to 10 feet thick, consist of sand with variable amounts of silt and clay, and have N_{60} values ranging from 1 to 21. The depth of the sand layers is highly variable between borings, typical of alluvial deposits.

Utilizing the results of our screening-level evaluation, a detailed liquefaction analysis was then performed on each layer determined to be potentially liquefiable. Using the methods presented above, including the use of the actual or estimate fines content, 9 of the 22 layers are considered liquefiable with factors of safety ranging from 0.5 to 1.2. The potential settlement due to liquefaction of these layers was calculated using the relationship by Tokimatsu and Seed (1987), resulting in settlements of 0.34 to 2.82 inches. A summary of the liquefiable layers and potential settlement due to liquefaction is presented in Table 6.

TABLE 6

Summary of Liquefiable Layers

BORING NAME	STRUCTURE NAME	DEPTH TO TOP OF LAYER (FT)	DEPTH TO BOTTOM OF LAYER (FT)	TOTAL LAYER THICKNESS (FT)	POTENTIAL SEISMIC SETTLEMENT (IN)
SB-8	Afton Rd. Bridge	23	28	5	1.32
SB-9		34	36	2	0.34
SB-18	Crossing #407	4	9	3.5 (1)	1.35

[3] See Appendix A for the definition of N_{60}.

BORING NAME	STRUCTURE NAME	DEPTH TO TOP OF LAYER (FT)	DEPTH TO BOTTOM OF LAYER (FT)	TOTAL LAYER THICKNESS (FT)	POTENTIAL SEISMIC SETTLEMENT (IN)
SB-20	S. Farris Rd. Bridge	23	33	10	2.82
SB-21		6	8	2	0.52
SB-33	Colusa Hwy. Bridge	48	50.2	2.2	0.45
SB-34		33	38	5	0.72
SB-39	W. Reimer Rd. Bridge	28	31	3	0.86
SB-40		28	35.5	7.5	2.25

(1) A portion of this layer is located above the groundwater level.

Based on our analysis, we conclude that the potential for liquefaction at the site is locally high during a seismic event. It is noted, however, that our subsurface exploration was not directed toward a liquefaction investigation: hollow-stem auger methods were used instead of rotary wash. Therefore, the results of our liquefaction analysis are likely conservative.

The primary hazard posed by liquefaction is differential settlement. Because we believe the layers are largely discontinuous over large horizontal distances, the risk of lateral spreading is relatively low. Furthermore, due to the presence and thickness of non-liquefiable cohesive soils above the liquefiable layers, the potential for sand boils is also generally low. The potential for sand boils and abrupt settlement is higher near SB-18 an SB-21, where the potentially liquefiable deposits were encountered at much shallower depths.

We note that our investigation scope was limited to structure locations which are a small part of the overall canal system. A detailed evaluation of the liquefaction potential for the entire canal system was beyond the scope of our investigation, as was the identification of improvement measures. Based on the types of structures proposed for this project, it is our opinion there is low likelihood that the above-calculated settlements will adversely affect the canal performance and operations. Regardless, it is our understanding that improvements to offset the effects of liquefaction are beyond the scope of the proposed construction.

6.2.3 Seismically Induced Densification

Seismically induced densification of non-saturated sand (sand above the groundwater table) due to earthquake vibrations may also cause settlement. However, the soil deposits encountered at the site have either sufficient density and/or cohesion such that the risk of seismically induced densification is considered negligible.

6.2.4 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. No known active or potentially active faults are mapped as crossing or projecting towards the alignment. Therefore, we conclude the risk of fault offset at the site from a known active fault is low.

7.0 DISCUSSION AND RECOMMENDATIONS

From a geotechnical standpoint, the proposed structures and improvements to the canal system are feasible as planned provided the recommendations presented in the remainder of this report are incorporated into the design and construction.

A geotechnical consideration for the site is the stability of the existing canal berms. The berms were constructed decades ago when the canal system was constructed. The materials that comprise the berms are similar to the native soils but locally contain gravels and cobbles near the surface. The berms were likely poorly compacted, if at all, which is typical of historic berm and levee construction. We understand breaches in the canal berms are fairly common and are quickly repaired by BWGWD maintenance crews. As such, the overall stability of the berms is considered marginal with factors of safety close to 1.0. Detailed evaluation of berm stability along the entire canal system is beyond the scope of this investigation, and would only reinforce the notion of poor to marginal berm stability.

Based on discussion with the design team, we understand rebuilding the canal berms to increase the factor of safety consistent with current engineering standards is beyond the scope of the project. We do note, however, that where the canal is widened, the stability of berms constructed with new fill placed in conformance with this report will be increased. In addition, we understand the inclination of the side slopes of the existing and widened berms is proposed to be flattened to 2:1 which should also increase the stability.

Another geotechnical consideration is the condition of the subgrade soils adjacent to the existing berms, where widening of the canal is proposed. As previously discussed, there are drainage ditches adjacent to the berms which are typically full of water and covered with vegetation. Likewise, the agricultural fields are heavily irrigated much of the year or are wet from seasonal rainfall. Similar conditions will be encountered in the Traynor Lateral where a new segment will be constructed through existing farmland to straighten the canal alignment. Accordingly, the areas of canal berm improvements are expected to be wet and soft at the time of construction. Recommendations are provided below for grading under these conditions.

The following subsections present our general recommendations regarding site grading, foundation design and construction, retaining structure design, permanent and temporary slopes, shoring, dewatering, seismic design, and pavement construction.

7.1 Site Grading

7.1.1 Demolition

Site demolition should include the removal of existing foundations, utilities, and other below grade improvements, if any, that will interfere with the proposed construction plans. We anticipate that structural demolition will include concrete footings, abutments, slabs, and wing walls for the weirs, crossings, siphons, and bridges. Demolition of these elements is anticipated to require excavation of

3 to 5 feet below grade. Demolition excavations should be properly backfilled with engineered fill according to recommendations provided later in this section.

As part of our investigation, six (6) piezometers were installed on the crest of the berms as discussed in Appendix A. The piezometers extend to depths of 13 feet below the ground surface. We also understand there were previously existing piezometers beyond the toe of the berms. If the piezometers are proposed to be abandoned as part of the grading operations, they must be abandoned in accordance with Butte County Environmental Health Department requirements.

7.1.2 Fill Materials

We believe most of the materials excavated for construction, once processed, will be suitable for reuse as engineered fill, provided they are free of organics, trash, and other debris; have a plasticity index (PI) of 20 or less; and do not contain oversize particles larger than four inches in least dimension. Highly plastic CH clay materials excavated from the site may be used as general site fill provided they are mixed with low to non-plastic soils such that the effective plasticity index meets the above requirements. However, in areas where surface improvements will be constructed or there is little to no tolerance for movement, highly plastic clay soil should be removed and replaced with on-site or imported material with a low expansion potential. In addition, expansive clayey soil should not be used as backfill beneath or behind any proposed structural improvements.

Soil excavated from within the canal or the adjacent irrigation ditches will likely have a moisture content well above optimum. Therefore, these materials could require significant drying to reduce the moisture content to a level at which they can be compacted.

If imported fill is required, it should be free of organics, trash, and other debris; should not contain oversize particles larger than four inches in minimum dimension; and should have a relatively low expansion potential (defined by a liquid limit less than 40 and a plasticity index less than 15).

Samples and/or index test results of all fill material, including on-site fill, should be submitted to the Geotechnical Engineer for approval at least 72 hours before it is to be used on site. Where imported fill is required, the fill supplier should provide analytical test results or other suitable environmental documentation at least three days before use at the site indicating the proposed fill material is free of hazardous materials, such as heavy metals or petroleum hydrocarbons.

7.1.3 Subgrade Preparation

The ground surface in areas to be graded should be stripped to remove all existing vegetation and other deleterious materials including all rubbish and debris. It is estimated that stripping depths of 1 to 2 inches may be necessary; however, the actual depth of stripping should be determined in the field by the Geotechnical Engineer. Any material that is deemed to be topsoil and requiring stripping may not be used as structural fill. If any trees exist in areas to receive fill, the rootball and associated roots that are greater than ½-inch diameter must be removed. The organic material should be removed from the site.

The exposed ground surface should be kept moist during subgrade preparation. The exposed native soil should be scarified to a minimum depth of eight inches, moisture conditioned, and compacted to at least 85 percent relative compaction[4]. Subgrade preparation should extend at least five feet horizontally beyond the area of planned improvement, where possible. The contractor should expect significant drying will be required where subgrade is prepared below the normal canal water surface elevation.

7.1.4 Subgrade Stabilization

Where the subgrade conditions are wet, soft, and/or unstable, such as along the landside toe of the canal berms, it will be necessary to stabilize the subsurface prior to fill placement. The more economical stabilization measure typically consists of aeration (drying) of the wet soil to reduce its moisture content to a compactable level. However, depending on climatic conditions, several days to several weeks of relatively warm, dry weather may be required to dry the soil to an acceptable level. In addition, it is often necessary to turn the material several times per day to promote uniform drying. The soil will be deemed sufficiently aerated when the required degree of compaction can be achieved and the resulting subgrade surface is firm and unyielding.

Another stabilization alternative consists of overexcavation of the wet/soft/unstable soil. For this alternative, the area should be overexcavated to a depth of two feet, or to competent, non-yielding soils, whichever is less. Where competent materials are exposed, they should be scarified and recompacted in accordance with previous recommendations for subgrade preparation. Where the excavation is still unstable at two feet, additional stabilization measures may be required as determined by the Geotechnical Engineer and may include additional excavation, treatment with lime, placement of a geotextile stabilization fabric and granular material, etc. The type of stabilization used and the amount of overexcavation required should be addressed on a case-by-case basis by a SAGE engineer during construction.

7.1.5 Fill Compaction Requirements

Upon satisfactory preparation of the subgrade as recommended above, engineered fill may be placed. Engineered fill should be placed in 8-inch thick loose lifts, moisture conditioned to an above optimum moisture content, and compacted to a minimum relative compaction of 90%. The upper six inches of subgrade in planned pavement areas should be compacted to at least 95%. The soil should not be allowed to dry out between the placement of lifts. The contractor should be prepared to keep all soil surfaces moist until they have been covered by improvements. If the soil is allowed to dry out, it should be scarified eight inches, moisture conditioned, and recompacted.

Prior to compaction, each layer should be spread evenly and thoroughly blade mixed to obtain uniformity of material in each layer. The fill should be brought to a water content that will permit proper compaction by either (a) aerating the material if it is too wet, or (b) spraying the material with

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4 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM D1557 laboratory compaction procedure.

water if it is too dry. Compaction should be performed by footed rollers or other types of approved compaction equipment and methods. Compaction equipment should be of such design that they will be able to compact the fill to the specified density. Rolling of each layer should be continuous over its entire area and the equipment should make sufficient passes to ensure that the required density has been obtained. Flooding or jetting is not permitted.

The standard test used to define maximum densities and optimum moisture content of all compaction work shall be the Laboratory Test procedure ASTM D1557 and field tests shall be expressed as a relative compaction in terms of the maximum dry density and optimum moisture content obtained in the laboratory by the foregoing standard procedure. Field density and moisture tests shall be made in each compacted layer by the Geotechnical Engineer in accordance with Laboratory Test Procedure ASTM D6938. When footed rollers are used for compaction, the density and moisture tests shall be taken in the compacted material below the surface disturbed by the roller. When these tests indicate that the compaction requirements on any layer of fill, or portion thereof, have not been met, the particular layer, or portion thereof, shall be reworked until the compaction requirements have been met.

Backfill behind retaining structures, such as the siphon, bridge, and weir wing walls, should be compacted using light (hand-operated) compaction equipment, unless larger equipment is approved by the structural designer. If heavy equipment is used within five feet of the wall, the wall may require design for the additional surcharge pressure exerted by the equipment.

7.2 Temporary and Permanent Slopes

7.2.1 Temporary Slopes

Where excavation is performed for structure construction and is less that about ten feet deep, we anticipate temporary slopes will be used. All temporary slopes should be excavated in accordance with the latest edition of the CAL-OSHA excavation and trench safety standards as a minimum (CCR, 2008). Site soils should be preliminarily classified as Type B according to the CAL-OSHA classification system. The maximum allowable slope for Type B soil is 1H:1V. If granular soils or seepage is observed in the cut face, the soil should be classified as Type C and a maximum slope of 1.5H:1V should be used. Where vertical benches are used at the base of excavations in cohesive soils, the maximum height of the bench should be limited to four feet.

The Contractor should be responsible for all temporary slopes and shoring systems used at the site, and should designate one of their on-site employees as a "competent person" who is responsible for trench and excavation safety. The competent person shall be responsible for determination of the actual CAL-OSHA soil type and shall direct the excavation crews to adjust slopes inclinations if appropriate.

7.2.2 Temporary Shoring

We expect temporary shoring will be required at deeper cuts such as at the siphons beneath the U.P.R.R. and W. Liberty Road. Temporary shoring may consist of trench boxes, soldier pile and lagging system, interlocking sheetpiles, or other continuously faced shoring systems.

All temporary ground support shoring systems used in the construction of the project should be designed, planned, constructed and maintained by the Contractor and should conform to all state and federal safety regulations and requirements. Whenever deep trench excavations are made in soil, unexpected caving of trench walls can occur at any time or place. Adequate protection of workers in excavations and trenches must be provided by the Contractor at all times.

We believe it is most likely the selected shoring system will be internally braced. Braced shoring should be designed to resist a lateral earth pressure distribution with a trapezoidal pressure distribution of 50H in pounds per square foot (psf), as shown in Illustration 1 below (H = total shored height in feet).

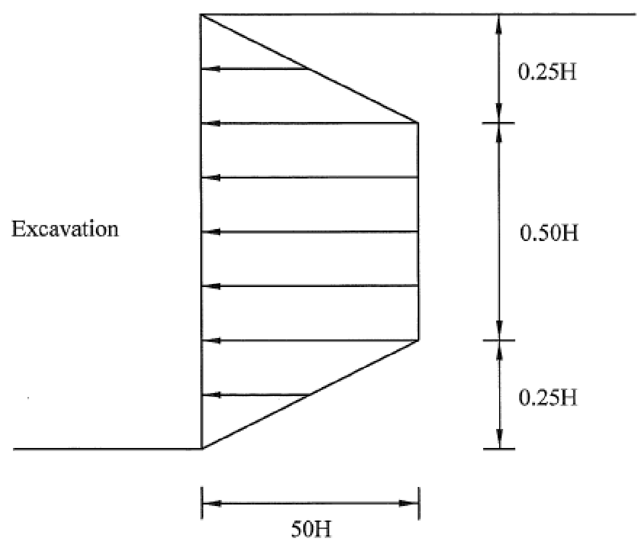

ILLUSTRATION 1

Any surcharge pressures on the ground surface adjacent to the excavation must be added to the pressure distribution shown above, including stockpiles and equipment. In addition, the above pressure distribution assumes that the groundwater level is maintained at least 2 feet below the bottom of the excavation.

If a tied-back shoring system is used, the pressure distribution will depend on the number of levels of tiebacks and stiffness of the shoring system. If a tied-back system is selected, we can provide recommendations for the design soil pressures on a case-by-case basis.

Temporary shoring, where required, will likely be installed in variable layers of soft to very stiff clay and silt and medium dense sand. In addition, intermittent layers of hard clay and silt to very dense sand will likely be encountered, especially on the northern segment of the Belding Lateral. These harder/denser materials may be difficult to excavate or drive sheet piles. The contractor should select excavation and shoring methods appropriate for these types of materials.

If the selected shoring system requires the installation of piers (e.g., a soldier beam and lagging system), pier construction may be difficult where dense sand and hard clays/silts are encountered. The contractor should anticipate the need for augers suitable for these types of materials. Although the materials are dense, these materials may be prone to caving into the drilled shaft due to the groundwater conditions and the low fines content. Therefore, the contractor should have casing on site to support the shaft, if necessary. Alternatively, polymer slurry can be used to support the drilled shaft should caving soils be encountered.

The temporary shoring design is the responsibility of the contractor. The excavation and shoring plans and calculations should be provided to SAGE for review of conformance to the above recommendations.

7.2.3 Permanent Slopes

Where permanent slopes are required, they should be constructed with a maximum inclination of 2H:1V. Fill materials, subgrade preparation, and compaction requirements should be as described above in the "Site Grading" section. To reduce the potential for erosion of the slope face, the compacted slopes should be overbuilt by at least three inches and trimmed back to design grade. Alternatively, the face of the slope can be compacted or track-walked.

If the fill slopes will exceed five feet in height, the toe of the fill should be keyed into the subgrade soil. The keyway should be at least three feet wide and extend at least two feet below existing site grades. The base of the keyway should be scarified and recompacted as described above, resulting in a firm and unyielding subgrade prior to fill placement.

Where the canal berms are proposed to be widened, the new fill should be keyed and benched into the existing soils. The keyed benches should be at least 1 foot wide with a maximum vertical spacing of 2 feet. A typical cross section showing the grading requirements for berms is included in Figure 9.

7.3 Pipe Jacking – Siphons

As part of the project, two siphons will be constructed to transport water beneath the U.P.R.R. and W. Liberty Road. Pipe inverts are expected to be approximately 25 and 19 feet below existing grades for the U.P.R.R. and W. Liberty Road siphons, respectively. The siphon pipelines will be constructed using pipe jacking (trenchless) techniques to minimize disruption to the railroad and to existing utilities beneath W. Liberty Road. Jacking and receiving pits will be required at each end of the pipeline. The pits will require temporary slopes and/or shoring to allow for construction. Recommendations for temporary excavations are provided in Section 7.2 above.

The siphon pipeline is expected to consist of rubber gasketed, reinforced concrete pipe (RGRCP). The RGRCP is proposed to be 96-inch diameter and 54-inch diameter for the U.P.R.R. and W. Liberty Road siphons, respectively. The concrete pipe should be designed by qualified and experienced engineers familiar with jacked concrete pipe installations. Table 7 provides criteria that may be used by the engineer for design of the pipelines. The information in the table should be supplemented by a thorough review of the logs of test borings in Appendix A and the applicable laboratory test results.

TABLE 7
Pipe Jacking Design Criteria

STRUCTURE NAME	APPLICABLE BORINGS	EXPECTED SOIL TYPE AT PIPE LEVEL (1)	RELATIVE DENSITY/ CONSISTENCY	MOIST SOIL UNIT WEIGHT (PCF)	WATER LEVEL OVER PIPE (FT) (2)	K'_{μ}
U.P.R.R.	SB-3 and SB-4	SC, ML, SM, SP-SM	Medium dense/ very stiff to hard	121	4.4 to 8.3	0.165
W. Liberty Road	SB-28 and SB-29	CL, ML	Stiff to hard	125	2.3 to 5.6	0.110

(1) USCS soil type, see logs of borings in Appendix A
(2) Based on level at the time of drilling and pipe elevation from 60% submittal plans for U.P.R.R. and 90% submittal plans for W. Liberty Road; the U.P.R.R. is not included in the 90% submittal plans.

7.4 Utility Installation and Backfill

As a minimum, any pipe bedding should extend a distance of at least D/4 (with D equal to the outside pipe diameter) below the bottom of the pipe. However, the bedding should not be less than four inches thick. Either clean sand or pea gravel bedding material of at least the required minimum thickness is adequate for trenches above the groundwater level. For pipes below the groundwater table, clean, open-graded 3/4-inch drain rock should be used for pipe bedding. Clean rock should be separated from submerged sandy gravel and soils using a non-woven geotextile filter fabric (Mirafi 160N or equivalent). After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of at least six inches with sand or fine gravel, which should then be mechanically tamped to at least 90 percent relative compaction.

Trench backfill should meet the requirements presented above for general site fill. The materials excavated from the trenches can generally be reused to backfill those trenches, provided the material meets the fill criteria previously presented and can be compacted to the required degree of compaction. If expansive silt and clay is encountered, it should only be used if the potential for ground movement (settlement or heave) is tolerable.

If fill with less than 10 percent fines (e.g., clean sand) is used, the entire depth of the fill should be compacted to at least 95 percent relative compaction. Pea gravel, rod mill, and open-graded gravel should be mechanically tamped in 12-inch loose lifts. Jetting of trench backfill is not allowed. Special care should be taken when backfilling utility trenches in any pavement areas. Poor compaction may cause excessive settlements, resulting in damage to structures or pavements which are constructed over the trenches.

In order to reduce the potential for utility/pipeline trenches to act as a conduit for groundwater and canal supply water, a sand-cement slurry or concrete plug should be constructed where utilities or pipes pass through structures or the canal berms. Alternatively, the trench can be backfilled with Class 2 permeable material that meets Caltrans Standard Specification Section 68. The permeable material should extend at least 1 foot around all sides of the pipe but should be no closer than 2 feet from the face of the berm on the water side. The final 2 feet should be backfilled with compacted native clay soil.

7.5 Dewatering

We expect the grading operations for the canal berm improvements will be performed during planned outages when the water level will be drawn down. However, based on the subsurface information obtained during this investigation, proposed excavations may extend below the groundwater table and in some instances will be below the existing invert of the canal. Where this occurs, we anticipate that a significant amount of dewatering will be required for installation of proposed footings and structures below a depth of 4 feet below the canal invert.

The predominant materials expected within the zone of structure construction consist of relatively low permeability clays. However, there will likely be localized areas where higher permeability sands and gravels will be encountered. Consequently a combination of upstream flow diversions/cutoffs and local pumping will be necessary to effectively dewater excavations required for new foundation construction.

7.6 Foundation Support

7.6.1 General

Foundation construction for most of the structures will likely consist of slab and/or strip foundations depending on the application and location. Slab foundations will likely be incorporated in construction of long-crested weirs, flumes, siphons, and trapezoidal bridge crossings, while strip foundations may be employed for construction of check structures, farm crossings, headgates, and wing walls. Driven precast concrete piles will be used in lieu of footings for four of the farm crossings. Specific recommendations for these foundation types are presented in the following sections.

7.6.2 Foundation Preparation

Generally, foundation preparation for the anticipated improvements is expected to consist of demolishing existing structures (in part or whole), overexcavating any organic materials, soft silt and clay materials from the subgrade, scarifying the exposed soils and recompacting the subgrade. Recommendations for subgrade preparation are provided in section 7.1.3.

7.6.3 Foundation Design-Footings/Slabs

7.6.3.1 Footings

As discussed above, structures are anticipated to consist of weirs, siphons, flumes, farm crossings, and bridges. These structures are anticipated to be supported within clay and silt soils. At a minimum, foundations should consist of continuous spread footings bearing in firm native or recompacted soil. Footings should be at least 18 inches wide and have an embedment depth of at least 18 inches below the canal invert. Where a concrete slab lining or rip rap is constructed to protect the foundation from scour, the minimum footing depth may be reduced to 12 inches below the canal invert. We anticipate that a wide range of footing widths may be used in constructing the possible improvements. Recommended allowable bearing capacities (in pounds per square foot) for dead plus live loads and total loads (including wind and seismic), based on the minimum footing width and depths presented above, are summarized in Table 8 below:

TABLE 8

ALLOWABLE BEARING CAPACITY VS. FOOTING WIDTH RECOMMENDATIONS

FOOTING WIDTH (FEET)	FOOTING DEPTH (FEET)	BEARING CAPACITY (PSF) DEAD PLUS LIVE LOAD	BEARING CAPACITY (PSF) TOTAL LOAD
1.5	1	1,300	1,740
	1.5	1,700	2,270
3	1	1,800	2,400
	1.5	2,200	2,930
4	1	2,150	2,870
	1.5	2,550	3,400
5	1	2,500	3,340
	1.5	2,900	3,870
6	1	2,800	3,740
	1.5	3,200	4,270
7	1	3,100	4,140
	1.5	3,500	4,670
8	1	3,500	4,670
	1.5	3,900	5,200

To control settlement, we recommend maximum allowable bearing capacities of 3,900 psf for dead load plus live loads and 5,200 psf for total loads. We estimate total settlement using these values will be less than 3/4-inch.

Lateral loads can be resisted by a combination of passive pressure acting on the vertical face of the footings and friction on the base of the footings. Passive pressure on the face of the footing should be computed using an equivalent fluid weight (triangular distribution) of 260 pounds per cubic foot (pcf) for dry conditions, and 126 pcf when submerged. These values assume level soil is present in front of the footing. The passive resistance from the upper foot of soil, or the maximum depth of scour, whichever is greater, should be neglected unless the ground surface is confined by a slab or pavement. Resistance along the base of the footing/slab should be computed using a frictional coefficient of 0.30. The values presented for passive and frictional resistance can be used in combination and include factors of safety of at least 1.5 to reduce the potential for lateral movement. We have summarized the above recommendations in a Lateral Loading Diagram included as Figure 10.

7.6.3.2 Slabs

Slab foundations will likely be incorporated in construction of long-crested weirs, flumes, siphons, and trapezoidal bridge crossings. For the soils anticipated along the bottom of the canal, a modulus of subgrade reaction of 75 psi/in is applicable for design of the slabs. A global allowable bearing capacity of 750 psf is also applicable but may be increased for localized loads to a maximum of 1,000 psf. It is recommended that slabs be a minimum of 10 inches thick.

7.6.3.3 Foundation Construction

The foundation excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The Geotechnical Engineer should check foundation excavations after cleaning but prior to placement of reinforcing steel to confirm the excavations are bottomed in suitable bearing material and have been cleaned properly. To limit the potential for disturbance during placement of reinforcing steel, the contractor should consider overexcavating 2 to 3 inches below the design bottom-of-foundation elevation and placing a concrete rat slab using a 2-sack sand/cement slurry.

If loose or soft soil is encountered at the bottom of a foundation excavation, it should be removed and replaced with additional concrete. Alternatively, the overexcavated area can be filled with a 2-sack sand/cement slurry as discussed above. The bottoms and sides of footings should be maintained in a moist condition until concrete is placed.

7.6.4 Foundation Design-Piles

7.6.4.1 Vertical Capacity

We understand that farm crossings #407, #443, Bonslett, and #077 will be supported on driven, 12-inch-square or 14-inch-square, prestressed, precast concrete piles. The piles will gain support through friction between the soil and the sides of the pile and, in some cases, end bearing at the pile tip. The vertical capacity of the pile cap should be neglected. We have calculated vertical capacities for both pile sizes and presented equations for calculating them in the following table.

TABLE 9

VERTICAL PILE CAPACITY

PILE TYPE	AXIAL CAPACITY (KIPS)	UPLIFT CAPACITY (KIPS)
12-inch concrete, square	(1.9 x L) + 4.0	(1.4 x L) + W_P
14-inch concrete, square	(2.2 x L) + 5.4	(1.65 x L) + W_P

Notes: (1) L = Length of pile in feet
(2) W_P = Weight of pile in kips
(3) Capacities are allowable and include a safety factor of 2, net of pile weight
(4) Capacities are valid for minimum pile spacing of three pile widths, center to center, or greater

7.6.4.2 Lateral Capacity

The LPILE program (Version 5, Ensoft) was utilized to evaluate the response of laterally-loaded 12- and 14-inch square driven prestressed concrete piles, under both free- and fixed-pile head conditions. Curves of lateral load versus deflection and maximum bending moment versus depth are provided in Figures 11 through 13 for free- and fixed- pile head conditions.

The analyses were based on single, 12- and 14-inch square piles, spaced at least 6 pile diameters center to center. If pile spacing is closer than 6 pile diameters, the load associated with any given pile deflection will be reduced because of pile interaction (group) effects. The lateral load reduction factors will vary depending on the number of piles, the direction of loading, and the location of the pile within the pile group. Typical reduction factors for the type of pile cap expected for the farm crossings range between 0.6 and 0.9. Specific reduction factors can be provided once the number of piles and the cap geometry have been determined. In addition to the lateral load capacity of the piles, the pile cap may be designed to resist lateral loads using the passive pressure recommendations previously presented in Section 7.6.3.1.

7.6.4.3 Installation Considerations

Determination of pile-driving equipment for this project should take into account the "matching" of the pile hammer with the pile size and length. Special consideration should be given to selecting a hammer that can deliver enough energy to the tip of the piles to drive them efficiently without damaging them. The hammer selected should be appropriate to supply sufficient energy to the pile tip to penetrate very stiff to hard clay and dense sand encountered below the site. The contractor should consider whether predrilling is necessary to reduce the potential for pile damage. We estimate that hammers having manufacturer rated energies from 40 foot-kips to 70 foot-kips will be suitable for the recommended piles.

7.7 Seismic Design

For design in accordance with the 2010 California Building Code, the following design parameters should be used:

TABLE 10
SEISMIC PARAMETERS

PARAMETER	VALUE
S_S	0.564
S_1	0.236
Site Class	D
F_a	1.349
F_v	1.928
S_{MS}	0.760
S_{M1}	0.455
S_{DS}	0.507
S_{D1}	0.304

7.8 Retaining Structure Design

7.8.1 Lateral Earth Pressures

Retaining structures that may be constructed at the site include check structures, headgates, siphons, and/or wingwalls for various structures. The retaining structures will be subject to lateral loads from soil, water, and surcharge loads behind the wall, such as a vehicular surcharge. We have summarized our recommendations below in a Lateral Pressure Diagram included as Figure 10. For static conditions and level backfill, walls designed to rotate at the top can be designed for "active" earth pressure conditions using an equivalent fluid weight of 40 pcf. If the walls are fixed against rotation, they should be designed for "at-rest" conditions using an equivalent fluid weight of 60 pcf. These values assume the walls retain firm native soils or recompacted fill and are drained to prevent hydrostatic water pressures from acting on the wall.

Where the walls will be submerged, and/or water is not expected to drain from behind the walls (i.e., hydrostatic pressures will act on the wall), they should be designed for a combination of buoyant active/at-rest pressures plus the hydrostatic water pressure. For these conditions, we recommend using buoyant active and at-rest pressures of 18 and 28 pcf, respectively, plus the hydrostatic water pressure of 62.4 pcf.

In our experience, the design of walls shorter than 10 feet are not controlled by seismic forces. It is up to the design engineer to determine whether seismic forces should be considered in the design. For seismic conditions, we recommend all walls be designed for an active earth pressure plus a seismic pressure increment. The seismic pressure increment can be computed using a uniform pressure (rectangular distribution) of 5H in psf, which is distributed over the entire retained height (H) of the wall. This distribution results in an equivalent resultant force that acts at a height of 0.6H from the base of the wall. However, the active earth pressure plus seismic pressure increment need not exceed the at-rest pressure on the wall.

If traffic, such as maintenance vehicles, will act behind the walls, a vehicle surcharge should be included in the design. The vehicle surcharge for active and at-rest conditions should consist of a uniform pressure of 80 or 120 psf, respectively, applied over the entire height of the wall where the load begins immediately behind the wall. Refer to Figure 10 where the load is set back from the back of the wall, such as where a backslope exists. Alternatively, the vehicle surcharge can be modeled as a 250 psf uniform vertical surcharge placed behind the wall. Larger surcharge pressures may be required for large construction equipment and should be evaluated on a case-by-case basis.

7.8.2 Wall Drainage

Walls designed for drained conditions should be properly backdrained over the entire width of the wall. Backdrains and outlets should not extend below the permanent groundwater table. Backdrains can consist of a prefabricated drainage panel (Miradrain 6000 or equivalent) placed against the backside of the wall or an at least 12-inch-wide zone of granular drainage material. The drainage material can consist of 3/4-inch clean crushed rock[5] wrapped in filter fabric (Mirafi 160N or equivalent) or Class 2 permeable material conforming to Section 68-2.02F of the Caltrans Standard Specifications[6]. Where Class 2 permeable material is used, the filter fabric is not required.

The drainage system should extend down to a perforated PVC collector pipe (perforations facing down). The collector pipe should be surrounded on all sides by at least four inches of granular drainage material. Where drainage panels are used, prefabricated collection strips (i.e., AdvanEDGE pipe or equivalent) may also be used in lieu of the PVC pipe surrounded by crushed rock. We should review the manufacturer's specifications for all proposed drainage materials to verify they are appropriate for the intended use. The pipe or collector strip should be sloped to drain to 3-inch-diameter weep holes spaced no greater than 10 feet on center. If the weep holes are placed below the average water level in the canal, the designer should carefully consider the impact of a balanced hydrostatic water pressure on both sides of the wall and whether a rapid drawdown condition exists.

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5 Clean crushed rock should have 100% of the particles passing a 1" sieve and no more than 10% and 5% passing the 3/8-inch and No. 4 sieves, respectively.

6 Where referenced throughout this report, Caltrans Standard Specifications shall refer to the 2010 edition, unless noted otherwise.

7.9 Seepage Control at Structures

Piping is defined as the removal, or erosion, of soil material due to flowing water. Piping most commonly occurs along soil/structure interfaces as water moves from a higher head to a lower head. The potential for piping is a function of the vertical and horizontal paths along the soil/structure interface and the head differential between the upstream and downstream sides of the structure. This is defined as Lane's Weighted Creep Ratio, C_w. The minimum recommended C_w to prevent piping varies depending on the soil type present at the soil/structure interface.

Based on information provided to us in the 30% Design Submittal, the structures are expected to be supported in soils anywhere from 6.5 to 19 feet below the existing crest of the berm where our borings were completed. We have reviewed the structure characteristics along with the subsurface boring logs to estimate an appropriate minimum C_w for each structure. The recommended design values are presented in Table 8.

TABLE 11

Lane's Minimum Recommended Weighted Creep Ratio, C_w

STRUCTURE NAME	EXPECTED SOIL TYPE	RECOMMENDED MINIMUM C_w
Razorback Flume	Fine sand and silt	8.5:1
U.P.R.R. Siphon	Fine sand and silt	8.5:1
Garcia Check	Clayey sand	5:1
Garcia Flume	Clay with sand to sandy clay	3:1
Afton Bridge	Sandy clay	3:1
Banion Check	Sandy clay	3:1
Fields Flume	Sandy clay	3:1
N. Farris Rd. Bridge North Check	Silty fine sand	7:1
Division 2 Headgate	Sandy clay	3:1
Check #349	Sandy Clay	3:1
Check #376	Clay	2:1
Crossing #407	Fine sand with clay	7:1
Check#422/Crossing#443	Sandy clay	3:1
S. Farris Bridge	Fine sand with clay	7:1
Bonslett Check/Farm Crossing	Sandy silt to sandy clay	5:1
Schwind Flume	Sandy clay	3:1
Crossing/Check #58	Clay	2:1
Crossing #071	Sandy clay	3:1
Crossing #100	Clay	2:1
Liberty Rd. Siphon	Clay with sand	2:1
Traynor Headgate	Sandy clay	3:1
Nugent Flume	Silt	8.5:1
Crossing #077	Sand	7:1
Check #102	Sandy silt	5:1

STRUCTURE NAME	EXPECTED SOIL TYPE	RECOMMENDED MINIMUM C_w
Colusa Hwy. Bridge	Clay to silt with sand	5:1
Check #059	Silt	8.5:1
Evans Reimer Bridge	Sandy clay	3:1

7.10 Asphalt Concrete Pavement

7.10.1 Methodology

We expect the canal improvements and structures may require reconstruction or widening of existing asphalt concrete pavements along the roadways. Recommendations for pavement surfaces are presented below. We emphasize that the performance of the pavement is critically dependent upon adequate and uniform compaction of the subgrade soils, as well as engineered fill and utility trench backfill within the limits of pavements.

The structural design of asphalt concrete (AC) pavement was performed in accordance with Caltrans guidance in the 2008 Highway Design Manual (HDM). This method utilizes a measure of the stiffness and deflection potential of the soil under saturated conditions (R-value) and the expected traffic loading for the site (Traffic Index, TI) to develop the minimum pavement section required.

7.10.2 Design

The soil subgrade beneath pavement areas is expected to consist predominantly of clays with variable amounts of sand and gravel. Four samples were submitted for R-value tests. Two of the tests were performed on highly plastic/expansive clay soil (CH) materials with results of <5. The other two tests were performed on low to medium plasticity clay soil (CL) materials with results of 17 and 19. For reference, Caltrans specifies a minimum R-value of 50 and 78 for aggregate subbase (ASB) and AB, respectively.

Based on our review of the testing data, we have concluded a subgrade R-value of 5 is appropriate for design of any proposed pavements at the site. Based on this R-Value and a range of traffic indices typical for county roads, the recommended pavement sections for asphalt concrete surfaces are summarized in the table below. The appropriate traffic index (TI) should be determined by the project design engineer or Butte County Standards.

TABLE 12

Pavement Sections

Design Traffic Index	Asphalt Concrete (1) (inches)	Aggregate Base (AB)(2) (inches)	Aggregate Subbase (ASB)(3) (inches)
6.0	3.5	12.5	--
		5.5	8.0
7.0	4.0	15.5	--
		6.5	10.0
8.0	4.5	18.5	--
		8.0	11.5
9.0	5.5	20.5	--
		8.5	13.0
10.0	6.0	23.5	--
		10.0	15.0

Notes: (1) Dense-graded hot-mix asphalt concrete; includes safety factor
(2) ¾-inch max Class 2, minimum R-Value = 78
(3) Class 2, minimum R-Value = 50
(4) All layers in compacted thickness to conform to Caltrans Standard Specifications.

The ASB and AB should have minimum R-values of 50 and 78, respectively, and otherwise conform to Sections 25 and 26 of the Caltrans Standard Specifications. The upper six inches of soil subgrade and the entire thickness of ASB and AB should be compacted to at least 95 percent relative compaction. Asphalt pavement used at the site should utilize Performance Graded (PG) binder 64-10 (Typical) or 70-10 (Special) and otherwise conform to Sections 39 and 92 of the Caltrans Standard Specifications and/or Butte County Standards. This PG binder is appropriate for use on "inland valley" roads per Table 632.1 of the 2008 HDM.

7.11 Corrosivity

The corrosion potential of on-site soils to concrete was evaluated in the laboratory using representative samples obtained from the upper 10 feet of the exploratory borings. Laboratory testing was performed to assess the effects of sulfate and chloride content on concrete. Laboratory test result sheets are presented in Appendix B. In summary, the sulfate contents range from 0.9 to 68.1 ppm and the chloride contents range from 11.3 to 182.9 ppm.

Based on a review of the International Building Code (ICC, 2009) and ACI 318-08 Table 4.2.1, the tested soils are considered to have an Exposure Class of S0. In accordance with ACI 318 Table 4.3.1, there is no restriction as to the type of cement used.

8.0 SUPPLEMENTAL SERVICES

Based on our review of the 90% submittal plans (Provost & Pritchard, 2011b) and discussions with P&P, we understand that approximately 39 new water control structures and canal crossings have been added to the project since we performed our original geotechnical exploration. As such, we do not have geotechnical information specific to the locations of these structures.

The majority of the new structures appear to be relatively lightly loaded structures with shallow foundations that can be appropriately designed based on the generalized geotechnical design parameters we have already developed for the project site. We have identified nine structures with more substantial foundation requirements at which we recommend supplemental subsurface exploration be conducted to confirm our existing geotechnical design parameters are appropriate, or to otherwise develop supplemental design parameters. These new structures are presented in Table 13.

TABLE 13

New Structures Recommended for Investigation

STRUCTURE NAME	LOCATION (APPROXIMATE)
Farm Crossing #270/New Canal Segment	Belding Lateral, STA 265+00
Farm Crossing #364	Belding Lateral, STA 363+50
Riley Road Bridge	Belding Lateral, STA 395+45
Green Lateral Headgate	Belding Lateral, STA 499+30
Check #088	Schwind Lateral, STA 88+50
RD 833 Crossing #64 Replacement	Traynor Lateral, STA 64+50
W. Liberty Rd Bridge, RD 833 Crossing #157, and Check #158	Traynor Lateral, STA 158+00
Farm Crossing #172	Traynor Lateral, STA 172+10
Check #186	Traynor Lateral, STA 186+50

Through discussions with P&P, we understand that the first phase of the project includes only the northern portion of the Belding lateral between Sta. 10+00 and Sta. 306+00 (Division 2 and Traynor headgates). All but one of the above structures will be included in future phases of the project. Supplemental investigations at these structure locations, if performed, will be completed under a separate scope of work. The findings and supplemental recommendations will be presented in an addendum report.

9.0 LIMITATIONS

This report has been prepared for the sole use of Provost & Pritchard Consulting Group and the Biggs-West Gridley Water District, and their agents specifically for the design of the proposed canal and structure improvements. The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our site reconnaissance and exploration, our engineering studies, experience, and engineering judgment, and have been formulated in accordance with generally accepted geotechnical engineering practices that exist at the time this report was prepared. No other warranty, expressed or implied, is made or should be inferred. In addition, the recommendations presented in this report are based on the subsurface conditions encountered in widely spaced test borings. Actual conditions may vary. If subsurface conditions encountered in the field differ from those described in this report, we should be consulted to determine if changes to our conclusions or supplemental recommendations are required.

The opinions presented in this report are valid as of the date of this report for the property being evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man. If site conditions vary from those described herein, we should be consulted to evaluate the impact of the changes, if any. In addition, changes in applicable standard of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of SAGE's control. In any case, this report should not be relied upon after a period of three years without prior review and approval by SAGE.

10.0 REFERENCES

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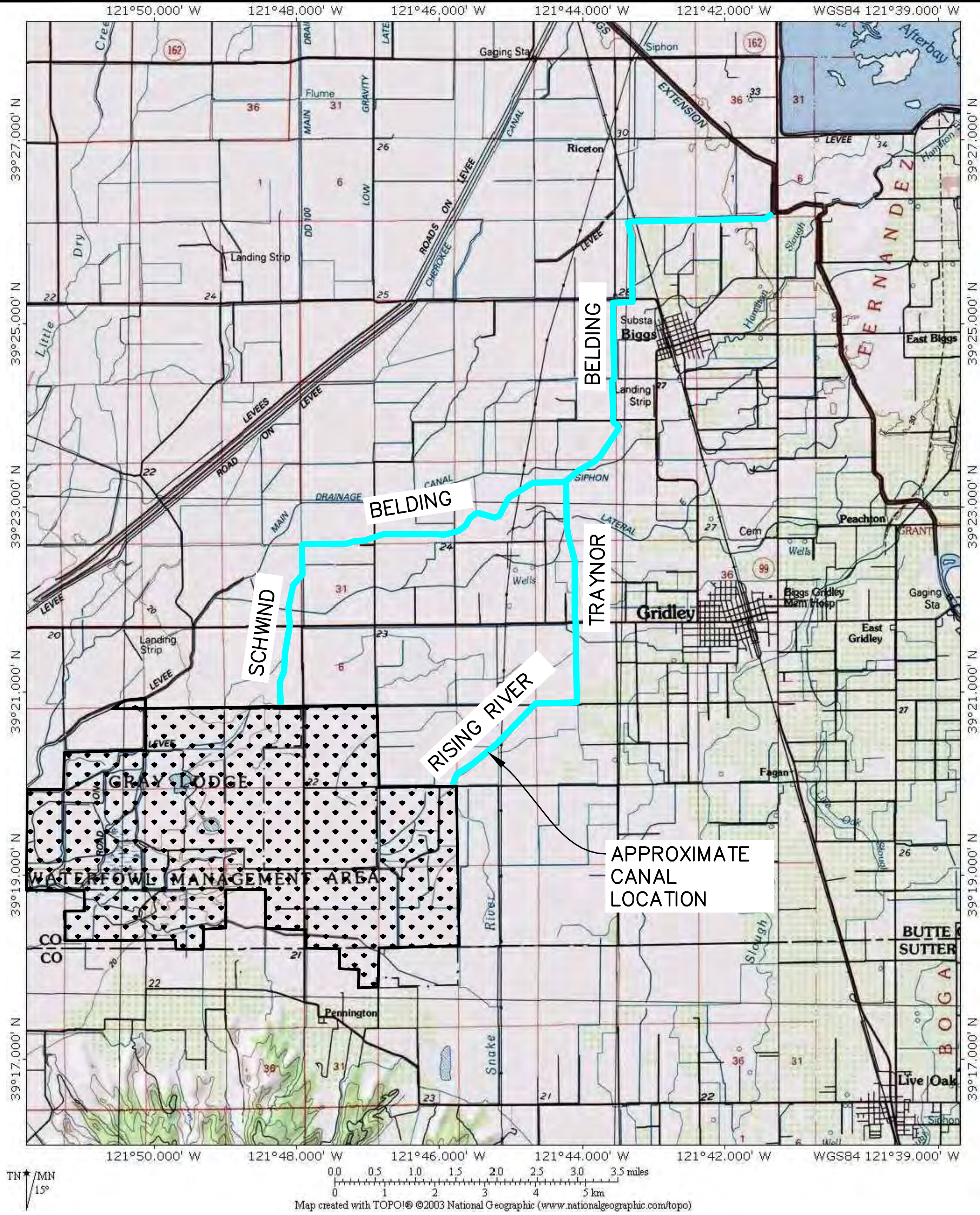
AERIAL PHOTOGRAPHS REVIEWED

DATE	PHOTO NUMBER	SCALE	FLOWN FOR	FLOWN BY
7/14/58 8/22/58	AAX-3V-37,38,77 to 79 AAX-10V-128 to 131,137 to 141	1:20,000	USDA	Cartwright Aerial Surveys
6/28/64	AAX-2EE-19 to 23, 80 to 85, 98	1:20,000	USDA	Cartwright Aerial Surveys
7/6/70	AAX-3LL-80 to 83,88,89	1:40,000	USDA	Western Aerial Corporation
7/28/79	179-56 to 58, 93 to 96	1:40,000	USDA	Western Aerial Corporation
3/18/84 3/18/84 3/19/84	WAC-84C 2-266 to 268 WAC-84C 3-89 to 93 WAC-84C 5-62 to 65	1:31,680	-	Western Aerial Corporation

USDA = U.S. Department of Agriculture

FIGURES



LEGEND

APPROXIMATE CANAL LOCATION

GRAY LODGE WILDLIFE AREA

SANDERS & ASSOCIATES GEOSTRUCTURAL ENGINEERING
SAGE
INTEGRATING EARTH & STRUCTURE

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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
SITE LOCATION MAP

BUTTE COUNTY CALIFORNIA

FIGURE 1

1 of 13

DATE: 12/22/11	FILE: Geotech Figs	SCALE: As Shown	JOB NO. 10-066.00

PLOT DATE: Wednesday, December 21, 2011 TIME: 4:21:58 PM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Geotech Figs.dwg

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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
GEOTECHNICAL EXPLORATION MAP - INDEX

BUTTE COUNTY | CALIFORNIA

FIGURE 2
2 of 13

DATE: 12/22/11 | FILE: Geotech Figs | SCALE: 1"=4000' | JOB NO. 10-066.00

PLOT DATE: Thursday, December 22, 2011 TIME: 2:46:01 PM BY: JERRY PASCOE
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGR\DRAWINGS\10-066 Geotech Figs.dwg

PLOT DATE: Wednesday, December 21, 2011 TIME: 5:32:49 PM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGR\DRAWINGS\10-066 Geotech Figs.dwg

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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
GEOTECHNICAL EXPLORATION MAP

BUTTE COUNTY CALIFORNIA

FIGURE 3

3 of 13

DATE: 12/22/11 | FILE: Geotech Figs | SCALE: 1"=1000' | JOB NO. 10-066.00

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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
GEOTECHNICAL EXPLORATION MAP

BUTTE COUNTY CALIFORNIA

FIGURE 4

4 of 13

DATE: 12/22/11	FILE: Geotech Figs	SCALE: 1"=1000'	JOB NO. 10-066.00

PLOT DATE: Wednesday, December 21, 2011 TIME: 5:35:18 PM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGR\DRAWINGS\10-066 Geotech Figs.dwg

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GEOTECHNICAL EXPLORATION MAP

BUTTE COUNTY CALIFORNIA

FIGURE
5
5 of 13

DATE: 12/22/11 | FILE: Geotech Figs | SCALE: 1"=1000' | JOB NO. 10-066.00

PLOT DATE: Thursday, December 22, 2011 TIME: 2:48:22 PM BY: JERRY PASCOE
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGR\DRAWINGS\10-066 Geotech Figs.dwg

THIS BAR IS ONE INCH
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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
GEOTECHNICAL EXPLORATION MAP

BUTTE COUNTY		CALIFORNIA	FIGURE 6 (6 of 13)
DATE: 12/22/11	FILE: Geotech Figs	SCALE: 1"=1000'	JOB NO. 10-066.00

PLOT DATE: Thursday, December 22, 2011 TIME: 3:01:41 PM BY: JERRY PASCOE
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGR\DRAWINGS\10-066 Geotech Figs.dwg

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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
GEOTECHNICAL EXPLORATION MAP

BUTTE COUNTY — CALIFORNIA

FIGURE 7

7 of 13

DATE: 12/22/11 | FILE: Geotech Figs | SCALE: 1"=1000' | JOB NO. 10-066.00

PLOT DATE: Wednesday, December 21, 2011 TIME: 5:44:18 PM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGR\DRAWINGS\10-066 Geotech Figs.dwg

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IF NOT, SCALE ACCORDINGLY

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GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
GEOTECHNICAL EXPLORATION MAP

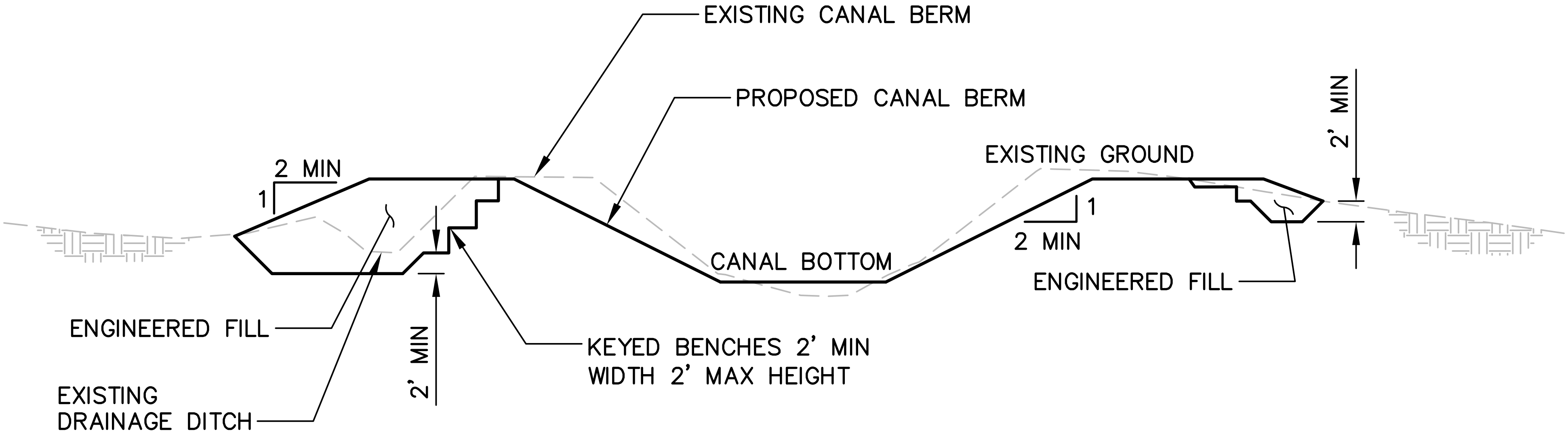
BUTTE COUNTY CALIFORNIA

FIGURE 8

8 of 13

DATE: 12/22/11 | FILE: Geotech Figs | SCALE: 1"=1000' | JOB NO. 10-066.00

PLOT DATE: Wednesday, December 21, 2011 TIME: 5:46:59 PM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Geotech Figs.dwg



GRADING NOTES:

1. VEGETATION IN AREAS TO BE GRADED SHOULD BE STRIPPED TO A DEPTH OF 1 TO 2 INCHES.
2. EXISTING SUBGRADE SOILS SHOULD BE OVEREXCAVATED TO A MINIMUM DEPTH OF 2 FEET BELOW EXISTING GRADES TO EXPOSE FIRM NATIVE SOIL.
3. THE BOTTOM OF THE EXCAVATION SHOULD BE COMPACTED TO A MINIMUM OF 85% RELATIVE COMPACTION (ASTM D1557).
4. ENGINEERED FILL SHOULD BE PLACED IN MAXIMUM 8-INCH THICK LOOSE LIFTS, MOISTURE CONDITIONED TO ABOVE OPTIMUM MOISTURE CONTENT, AND COMPACTED TO AT LEAST 90% RELATIVE COMPACTION.
5. FILL SLOPES SHOULD BE OVERBUILT AND COMPACTED THEN TRIMMED TO THE FINAL FACE.

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BERM CROSS SECTION
GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
BUTTE COUNTY — CALIFORNIA

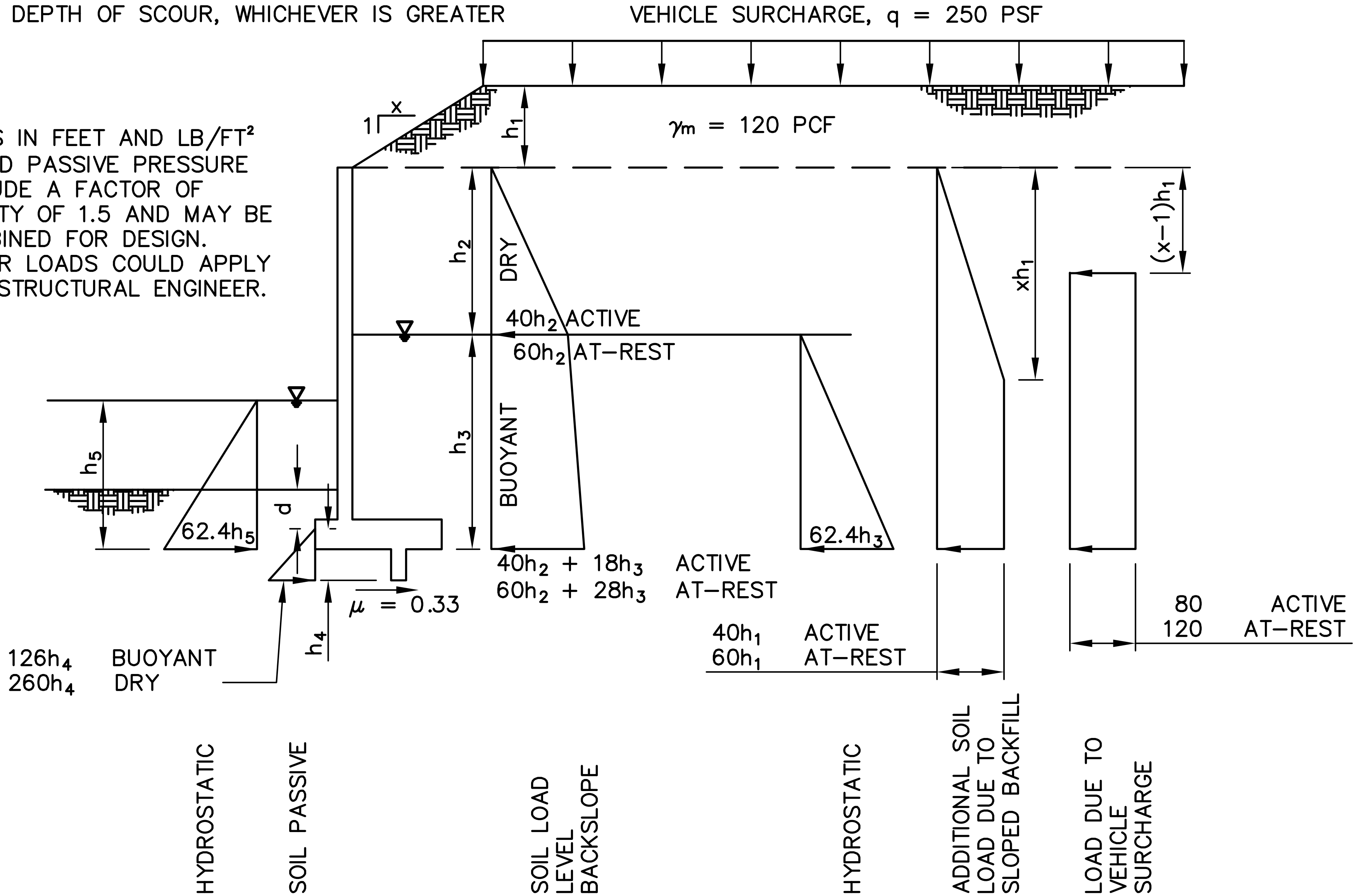
FIGURE 9

DATE: 12/22/11 | FILE: 10-066 Cross | SCALE: NONE | JOB NO. 10-066.00

PLOT DATE: Wednesday, December 21, 2011 TIME: 4:21:33 PM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Cross-Sections.dwg

γ_m = MOIST UNIT WEIGHT
x = HORIZONTAL COMPONENT OF SLOPE INCLINATION (I.E. 2 FOR A 2:1 SLOPE)
μ = COEFFICIENT OF FRICTION
h_1 = HEIGHT OF BACKSLOPE
d = DEPTH TO NEGLECT PASSIVE; 1' MIN OR DEPTH OF SCOUR, WHICHEVER IS GREATER

NOTES:
1. UNITS IN FEET AND LB/FT²
2. μ AND PASSIVE PRESSURE INCLUDE A FACTOR OF SAFETY OF 1.5 AND MAY BE COMBINED FOR DESIGN.
3. OTHER LOADS COULD APPLY PER STRUCTURAL ENGINEER.



PLOT DATE: Thursday, December 22, 2011 TIME: 11:26:13 AM BY: JERRY PASCOE
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Lateral Loads (fig.10).dwg

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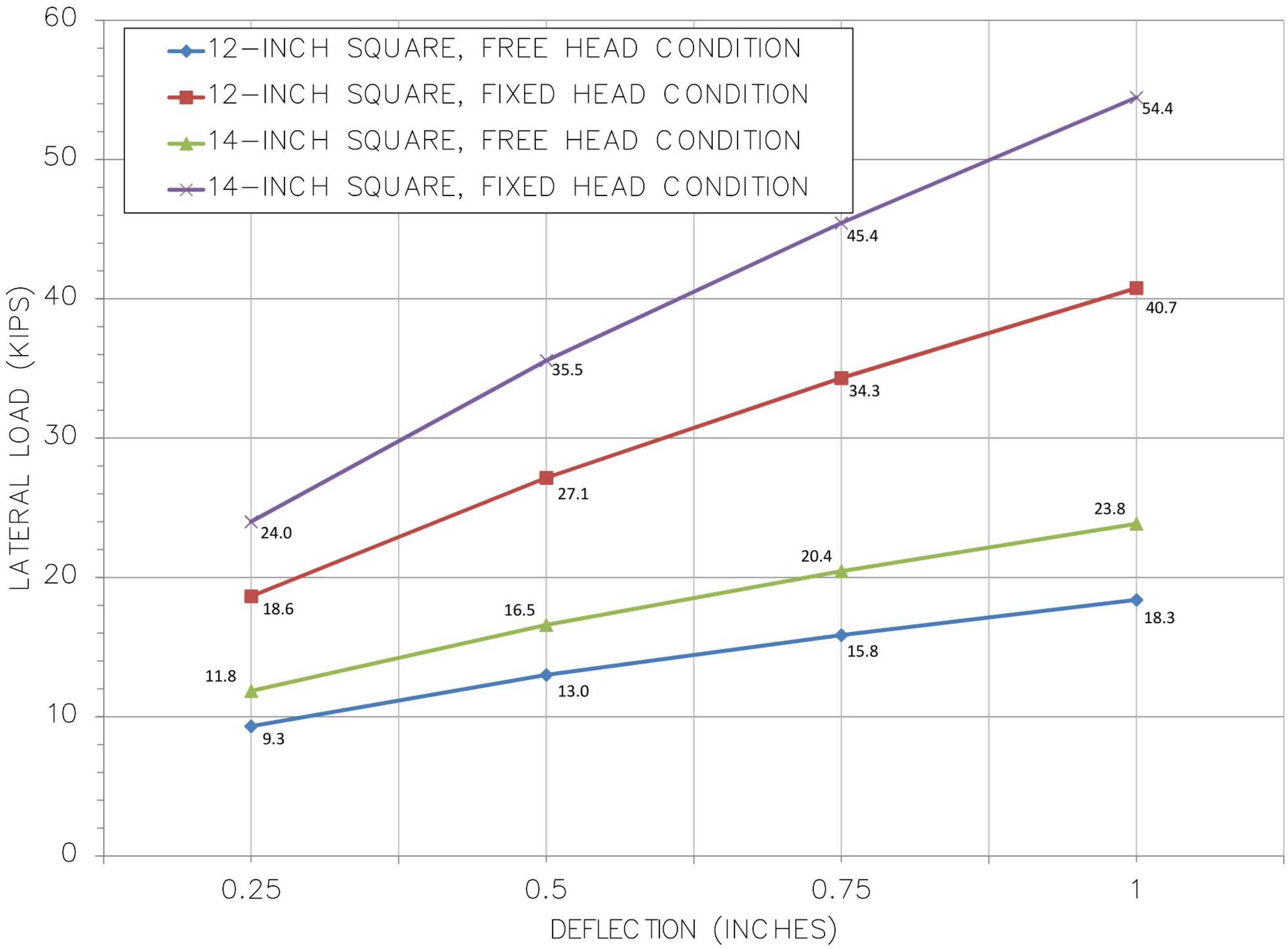
LATERAL PRESSURE DIAGRAM
GRAY LODGE WILDLIFE AREA WATER SUPPLY PROJECT
BUTTE COUNTY CALIFORNIA

FIGURE 10

DATE: 12/22/11 | FILE: 10-066 Lateral | SCALE: NONE | JOB NO. 10-066.00



NOTES:

1. THE DEFLECTION VALUES PRESENTED ARE FOR UNFACTORED SERVICE LOADS.
2. THE DEFLECTION VALUES PRESENTED ARE FOR A SINGLE PILE. GROUP INTERACTION EFFECTS WERE NOT CONSIDERED.

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LATERAL LOAD VS. DEFLECTION

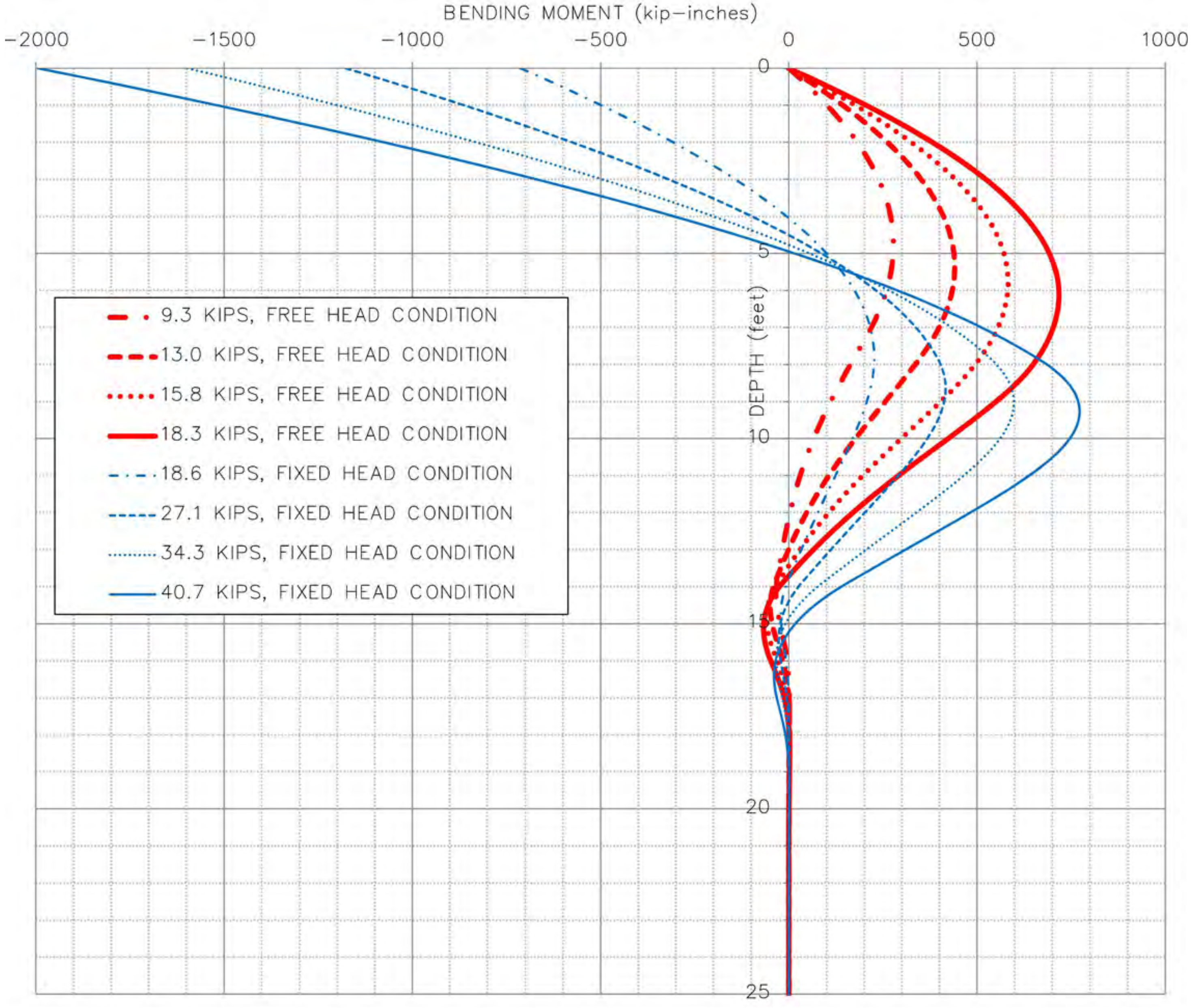
12- AND 14-INCH SQUARE CONCRETE PILES

BUTTE COUNTY — CALIFORNIA

FIGURE 11

DATE: 12/22/11	FILE: 10-066 Load	SCALE: NONE	JOB NO. 10-066.00

PLOT DATE: Thursday, December 22, 2011 TIME: 3:45:18 PM BY: JERRY PASCOE
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Load v Deflection Graphs (fig.11)- portrait.dwg



NOTES:
1. THE BENDING MOMENT VALUES PRESENTED ARE FOR UNFACTORED SERVICE LOADS.
2. THE BENDING MOMENT VALUES PRESENTED ARE FOR A SINGLE PILE. GROUP INTERACTION EFFECTS WERE NOT CONSIDERED.

PLOT DATE: Thursday, December 22, 2011 TIME: 10:23:54 AM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Moment v Depth Graph (fig.12).dwg

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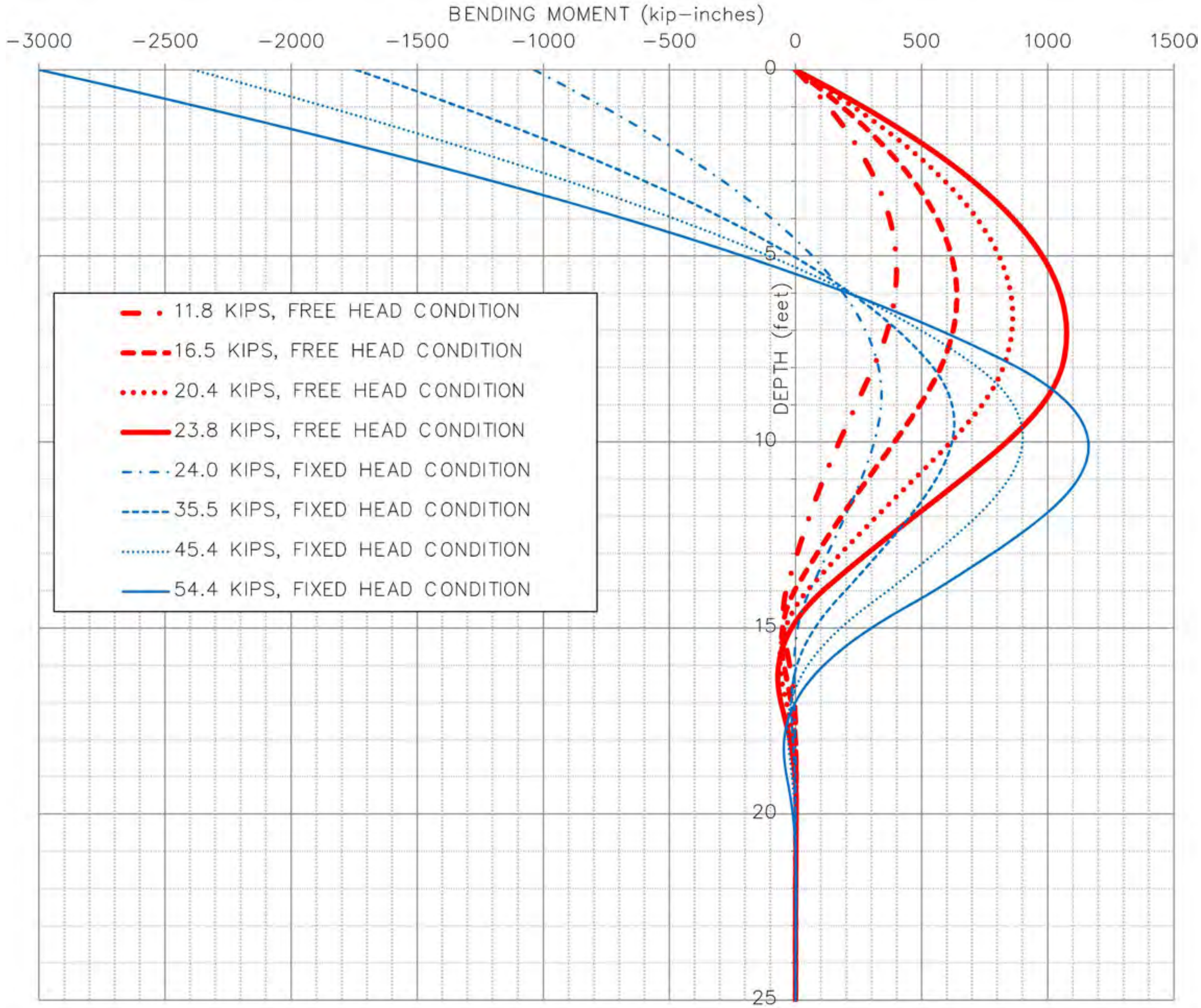
BENDING MOMENT VS. DEPTH
12-INCH SQUARE FREE- AND FIXED-HEAD CONCRETE PILES
BUTTE COUNTY CALIFORNIA

FIGURE 12

DATE: 12/22/11 | FILE: 10-066 Load | SCALE: NONE | JOB NO. 10-066.00



NOTES:

1. THE BENDING MOMENT VALUES PRESENTED ARE FOR UNFACTORED SERVICE LOADS.
2. THE BENDING MOMENT VALUES PRESENTED ARE FOR A SINGLE PILE. GROUP INTERACTION EFFECTS WERE NOT CONSIDERED.

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BENDING MOMENT VS. DEPTH
14-INCH SQUARE FREE- AND FIXED-HEAD CONCRETE PILES
BUTTE COUNTY CALIFORNIA

FIGURE 13

DATE: 12/22/11 | FILE: 10-066 Load | SCALE: NONE | JOB NO. 10-066.00

PLOT DATE: Thursday, December 22, 2011 TIME: 10:23:18 AM BY: RYAN ABERNATHY
FILE: F:\1-PROJECTS\2010\10-066.00 Biggs West Gridley Canal Rebuild\02 GGIR\DRAWINGS\10-066 Moment v Depth Graph (fig.12).dwg