FOUNDATION REPORTS
for
BRIDGES

January 2021

DIVISION OF ENGINEERING SERVICES

GEOTECHNICAL SERVICES

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1 INTRODUCTION

The intent of this document is to define the Department's standard of practice for preparation of the Structures Preliminary Geotechnical Report (SPGR), the Preliminary Foundation Report (PFR) and the Foundation Report (FR). Standardized and consistent report presentations for projects statewide benefit the Department's staff, engineering consultants, bidders, and contractors. Geotechnical Services staff as well as any other organization preparing these reports must comply with the requirements presented herein.

1.1 Reporting for Caltrans Project Delivery

Foundation investigation and reporting generally occurs at three stages of the project development process:

- A Structures Preliminary Geotechnical Report (SPGR) to support Advanced Planning Studies, performed during the Work Breakdown Structure activity 150.15 (K Phase).
- A Preliminary Foundation Report (PFR) to support Type Selection, performed during the Work Breakdown Structure activity 160.10 (0 Phase) or 240.70 (1 Phase).
- A Foundation Report (FR) to support the design and construction of the bridge, performed during the Work Breakdown Structure activity 240.80 (1 Phase).

A separate foundation report must be prepared for each bridge structure, with the following additional requirements:

- Left, center, and/or right bridges with the same bridge number should be combined into one report.
- Earth retaining systems located within 150 feet of the bridge, or connected to the bridge, should be addressed in the bridge foundation report.

Prepare reports to succinctly communicate information pertinent to the recommendations in accordance with the report preparation requirements. The following rules must be followed:

- Use proper grammar, spelling and punctuation.
- Present only useful specific information that is relevant to the recommendations.
- Reference or cite existing standards, specifications or policy only when clarifying, modifying, or disallowing the standard, specification or policy.
- Do not include unsubstantiated disclaimers.
- Provide titles and numbers for all figures and tables.
- Tables and figures must be included within the body of the report and located as near as possible to the place where they are first referenced.
- All depth references must have a corresponding elevation in parenthesis.

1.1.1 Reports Prepared by Caltrans Staff

Foundation Reports are written to the Structure Designer, Specification Engineer, and Structure Construction, and are part of the contract.

For reports prepared by Geotechnical Services staff, Foundation Reports must be prepared using the current departmental memorandum format with the subject line of "Foundation Report for *Bridge Name*" or "Preliminary Foundation Report for *Bridge Name*" or "Structures Preliminary Geotechnical Report for *Bridge Name*". Do not include section numbers in the report. First-level and second-level section titles presented in this document (e.g., Geotechnical Conditions, and Geology) must be included in the report. Other section titles are optional.

Do not include the Log of Test Borings (LOTB) and/or As-built LOTB as part of the FR. The Engineering Graphics Unit will send Microstation LOTB files and scanned copies of the As-built LOTB sheets to the Structure Designer for inclusion within the Contract Plans.

Sign and stamp reports in accordance with the *Communications and Reporting* section of the *Offices of Geotechnical Design – Quality Management Plan*.

1.1.2 Reports Prepared by Consultants

Foundation Reports must include the following: cover sheet, table of contents, main contents per this document, and appendices. The cover of the report and any addenda/amendments to the report must include the following information: Caltrans District, County, Route, Post Mile, Bridge Number, Bridge Name, and Expenditure Authorization (EA) number.

The LOTB and/or As-built LOTB must be submitted as part of the FR. Refer to the *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* for direction on the preparation of the LOTB and As-built LOTB.

2 STRUCTURE PRELIMINARY GEOTECHNICAL REPORT (SPGR)

The SPGR is required during the early stages of a project to assist Structure Design in the preparation of an Advanced Planning Study and cost estimate for the District. Often the number, location, and type of bridge(s) are not completely known. As a result, recommendations may be general, and detailed field investigations are usually not warranted. Typical fieldwork consists of a site visit only. The SPGR provides an overview of the existing foundations, site geology, seismicity, and recommendations regarding suitable and unsuitable foundation types. If applicable, the SPGR should also discuss the anticipated field and laboratory work required to support the PFR and FR.

The following topics should be addressed in all Structure Preliminary Geotechnical Reports (SPGR).

2.1 Introduction

Summarize the purpose, scope, and types of work performed to obtain the information supporting the preliminary recommendations. Reference the request memo, preliminary plans by date so the reader knows on what plans the recommendations are based. Do not present an exhaustive list of tasks performed, a few sentences are sufficient.

Example

Per the request dated February 3, 2020, this Structure Preliminary Geotechnical Report has been prepared for the proposed widening of Dry Creek Bridge. The purpose of this report is to summarize the preliminary investigations performed and to provide preliminary foundation recommendations for Dry Creek Bridge. The recommendations presented in this report are based on the Advanced Planning Study dated January 15, 2020, review of As-built plans, previous geotechnical reports, BIRIS records, and a site visit.

2.2 Project Description

Describe the existing and/or proposed structure(s), and pertinent project information relating to the planned improvements. Provide project vertical datum reference.

<u>*Example: New Bridge*</u>

The bridge site is in the city of San Diego on State Route 15 at PM R3.8 which crosses over Interstate 805 (I-805) at PM 15.1. At this site, the proposed bridge replacement is necessary to accommodate the underlying highway improvements, which include the widening of the existing I-805 in order to provide additional High-Occupancy Vehicle (HOV) lanes. Based on the General Plan (dated January 15, 2020), the proposed bridge is a 2-span, cast-in-place, prestressed concrete box girder bridge supported on pile foundations.

All elevations referenced within this report are based on the National Geodetic Vertical Datum of 1929 (NGVD 29), unless otherwise noted.

<u>*Example: Bridge Widening*</u>

The Sweetwater River left and right bridges are located on I-805 in Chula Vista, and are two of several bridges along I-805 which are in the process of being widened to accommodate construction of High Occupancy Vehicle (HOV) lanes in the median. "As-built" information indicates that the existing Sweetwater River left and right bridges consist of five-span, cast-in-place, pre-stressed concrete, box-girder structures, with end-diaphragm abutments that were constructed in 1968. The existing bridges are supported on driven Class 70C concrete piles at all support locations. The proposed work includes median widening between the left and right bridges and removal and replacement/widening of a portion of the deck of the right bridge. The center widening is proposed to consist of a six-span, cast-in-place, pre-stressed concrete, box-girder structure, with end-diaphragm abutments.

The 1968 As-built plans did not include a vertical datum reference. It is assumed that the elevations are based on the National Geodetic Vertical Datum of 1929 (NGVD 29), however it is recommended that structure design verify this assumption.

2.3 Exceptions to Policies and Procedures

List exceptions to Departmental policies and procedures relating to the SPGR. Approved *Request for Exception* forms must be included in the Appendix. Omit this section if there are no exceptions.

2.4 Geotechnical Investigation

Provide an overview of the geotechnical investigation(s) that support the preliminary foundation recommendations.

<u>*Example*</u>

The As-built LOTB show that a subsurface investigation, consisting of three mud rotary borings, was performed in 1969. Additionally, a site visit was performed on February 23, 2020 to review site access and creek conditions. During the site visit the creek was flowing between Piers 2 and 3 with a water depth of approximately 2 feet.

2.5 Geotechnical Conditions

2.5.1 Geology

Identify the pertinent geologic map and the geologic unit(s) at the structure site. Describe relevant geologic features such as faults, bedding, major joint attitudes, and folds if they influence the design and construction of the structure.

<u>*Example*</u>
The Geologic Map of Santa Ana 30' x 60' Quadrangle shows that the site is underlain by Quaternary alluvium.

2.5.2 Surface Conditions

Describe site topography, surface water and drainage conditions, cuts and fills, geologic hazards such as landslides and rockfall, and land use history that may affect the proposed structure. Identify existing structures, facilities, and utilities near the proposed structure that may affect its design and construction.

<u>*Example*</u>
The topography is relatively flat and the site appears free of geologic hazards.

2.5.3 Subsurface Conditions

Provide a generalized description of the known subsurface conditions. The information included within this section may include, but is not be limited to:

- Types of soil/rock, depths to generalized layer breaks, and corresponding elevations
- Pertinent soil/rock conditions such as unsuitable materials (collapsible, expansive foundation materials)

Do not re-create an As-built LOTB in detail in this section. A generalized discussion or table is sufficient.

<u>*Example*</u>
Based on the 1966 As-built Log of Test Borings, the alluvial soil at the site can be separated into three general units. The upper unit consists of very loose to slightly compact silty sand with gravel that extends from the ground surface to a depth of about 15 feet (~ Elev. 950 feet). The middle unit consists of slightly compact to dense sand to a depth of approximately 35 feet (~ Elev. 930 feet). The lowermost unit consists of dense to very dense gravelly sand and sandy gravel with isolated zones of sandy silt and gravel. This unit extends to the maximum depth of the borings, which is approximately 60 feet below the ground surface (~ Elev. 905 feet).

2.6 Groundwater

Report groundwater elevation(s) and dates of measurements. Use of a table is recommended if there are numerous borings and/or measurements.

Table X: Summary of Groundwater Data

Location or Boring ID	Ground Surface Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Date Measured

<u>Example: Groundwater Present</u>

During the 1998 subsurface investigation, groundwater was encountered in both borings. Groundwater levels varied from elevation 945 feet (depth of 20 feet) in February to elevation 938 feet (depth of 27 feet) in August.

<u>Example: Groundwater Not Present</u>

During the 1998 subsurface investigation, groundwater was not encountered in either boring within the explored depth of 100 feet (~ Elev. 900 feet).

<u>Example: Groundwater Information Not Available</u>

Groundwater information was not available based upon the literature search performed.

<u>Example: Groundwater Information Available Nearby</u>

Groundwater measurements available from a DWR monitoring well, located 800 feet northwest of the proposed structure, had groundwater elevations that varied between 930 feet and 920 feet from 2015 to present.

2.7 As-built Data

Include brief discussion of <u>relevant</u> As-built foundation data, such as:

- Existing foundation types and details (e.g., pile tip elevations)
- As-built geotechnical capacities or resistances.
- Construction reports or records such as pile driving logs, pile load test reports, construction difficulties, etc.

Use the tables in the examples below to present foundation data.

<u>Example: Driven Piles</u>

Construction of the original bridge was completed in 1971 with all three supports supported on driven Alternative (Alt) "X" concrete piles. The 1971 As-built LOTB provided the data in Table 1.

Table 1: Summary of the 1971 As-built Data

Support Location	*Foundation Type*	*Design Load*	*Bottom of Pile Cap Elev. (feet)*	*Min. Penetration Elev. (feet)*	*Avg. Penetration Elev. (feet)*	*Max Penetration Elev. (feet)*
Abutment 1	*12" Driven Alt "X" Pile*	*45 ton*	*958.1*	*929.3*	*927.1*	*926.5*
Bent 2	*12" Driven Alt "X" Pile*	*45 ton*	*935.5*	*920.2*	*918.1*	*916.1*
Abutment 3	*12" Driven Alt "X" Pile*	*45 ton*	*953.6*	*928.1*	*926.5*	*925.3*

<u>Example: Shallow Foundations</u>

The existing Cenda Ditch Bridge consists of a two-span, cast-in-place, slab bridge that was constructed in 1963. Abutments 1 and 3 are end-diaphragm abutments and are supported on spread footings placed in approximately 20 feet of embankment fill material. Pier 2 is supported by seven columns on spread footings founded on rock. The As-built bottom of footing elevations and design loads for the bridge are shown below in Table 2.

Table 2: As-built Information

Location	*As-built Bottom of Footing Elevation (feet)*	*As-built Allowable Footing Pressure (tsf)*	*As-built Design Footing Pressure (tsf)*
Abutment 1	*4499.1*	*2.0*	*2.0*
Pier 2	*4475.0*	*5.0*	*5.0*
Abutment 3	*4490.7*	*2.0*	*2.0*

2.8 Scour Data

Report pertinent scour information obtained from the geotechnical investigations (e.g., BIRIS records, Preliminary Hydraulics Report) including the potential for scour. Use the table in the example to present scour data.

<u>Example: Scour Data Available</u>

The bridge site is underlain by alluvial soil, which is considered potentially scourable. The Structure Hydraulics Branch provided the following scour information in a Preliminary Hydraulics Report dated May 15, 2020 (Table 1).

Table 1: Scour Data

Support Location		*Long Term Scour (Degradation and Contraction) Elevation (feet)*	*Short Term Scour (Local) Depth (feet)*
Left Bridge	*Abut 1*	*2285.6*	*3*
	Abut 2	*2285.1*	*3*
Right Bridge	*Abut 1*	*2291.9*	*3*
	Abut 2	*2291.6*	*3*

Example: No Scour
The bridge does not span a watercourse, therefore there is no scour potential.

Example: Scour Data Unavailable
The bridge spans a watercourse. BIRIS records do not identify any historic scour issues. The Structures Hydraulics Branch has not yet issued a Preliminary Hydraulics Report.

2.9 Corrosion Evaluation

Report and discuss pertinent site corrosion data.

Example: No information available
Historical corrosion data is not available. For preliminary design purposes the site should be considered non-corrosive based on the presence of predominantly cohesionless soil. Corrosion samples will be obtained during the design phase to evaluate the corrosion potential of the site.

Example: Non-Corrosive
Three soil samples and one water sample were collected for corrosion testing during the 2011 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on Caltrans' standards, the site is considered non-corrosive.

Example: Corrosive
During the 2011 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings RC-11-001 and RC-11-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered corrosive based on Caltrans' standards, and corrosion mitigation may be required depending on the type/depth of foundation selected.

Table 1: Soil Corrosion Test Summary

Boring ID	Elevation (feet)	Minimum Resistivity (Ohm-Cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)	Corrosive?
RC-11-001	15.8 to 14.3	1544	7.24	N/A	N/A	No
RC-11-001	-4.2 to -3.2	683	7.94	384	432	No
RC-11-002	-69.1 to -70.6	73	6.86	850	1500	Yes
RC-11-002	-104.1 to -105.6	78	7.71	1000	1600	Yes

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

2.10 Seismic Information

Report all information required in Section 2.10.1 in the SPGR. Referencing a Seismic Report that was delivered separately is not acceptable. Information required in Section 2.10.2 should be summarized while referencing the reader to the applicable report (e.g., Fault Rupture Report).

2.10.1 Ground Motion Hazard

Include the following information:

a. Site coordinates (latitude and longitude in decimal degrees)
b. The estimated time-average shear wave velocity V_{S30} and how it was determined (e.g., CPT or SPT correlations). See *Design Acceleration Response Spectrum (ARS)* module.
c. For 5% probability of exceedance in 50 years (Return Period = 975 years).
    i. The ARS per the *Design Acceleration Response Spectrum* module.
    ii. The horizontal peak ground acceleration (HPGA).
    iii. Deaggregated mean earthquake moment magnitude (M) for the HPGA and the mean site-to source distance (R) for the 1.0 second period spectral acceleration.
d. Ground Motion Data Sheet (see *Forms and Templates*)
e. Soil Profile Classification (Class S1 or Class S2) for the site or support locations per the SDC v2.0, Section 6.1 and Section 6.2.3. Omit if sufficient site data is not available.

<u>Example</u>

The site is susceptible to strong earthquake induced ground motions during the design life of the bridge.

Based on available subsurface information and Standard Penetration Test (SPT) correlations for determining shear wave velocity, the time-average shear wave velocity (V_{S30}) for the upper 100 feet of soil at the site is estimated to be 980 ft/sec.

The Design Spectrum for the Safety Evaluation Earthquake, as specified in Caltrans Seismic Design Criteria with October 2019 interim revisions, Version 2.0 (SDC v2.0), is the probabilistic response spectrum representing the horizontal ground motion at the site with a

5% probability of exceedance in 50 years (return period = 975 years). The USGS's 2014 NSHM is used as the basis to determine the Design Spectrum in the form of the design Acceleration Response Spectrum (ARS).

Caltrans web-based tool ARS Online v3.0 was utilized to determine the design ground motion parameters, including the ARS, for the subject structure site. Based on the ARS Online v3.0 tool, the design PGA = 0.27g, and the deaggregated mean earthquake moment magnitude for PGA is M = 6.7 and mean site-to-source distance for 1.0 second period spectral acceleration is R = 41.2 miles.

The Ground Motion Data Sheet, presenting the design ARS data, plot, and other relevant information, is attached.

The soil at the site is "Class S1" per Section 6.1 and 6.2.3 of the SDC, v2.0.

2.10.2 Other Seismic Hazards

The section must include information on the following seismic hazards, as applicable at the site:

a. Surface fault rupture potential (see *Fault Rupture* module)
b. Liquefaction potential (see *Liquefaction Evaluation* module)
c. Seismically induced total and differential ground settlements
d. Lateral spreading potential (see *Lateral Spreading* module)
e. Seismic slope instability
f. Tsunami risk

Example

The structure is not located within an Alquist-Priolo Earthquake Fault Zone or 1000 feet from any Holocene or younger aged fault. Therefore, per MTD 20-10, the structure is not considered susceptible to surface fault rupture hazards.

Groundwater was not encountered within the As-built borings drilled to depths ranging from 70 to 100 feet (~ Elev. 90 to 60 feet) from the existing ground surface. Dense and/or stiff soils were encountered in these borings below a depth of about 60 feet (~Elev. 100 feet) from the existing ground surface. Based on these groundwater and subsurface soil conditions, the project site is not susceptible to liquefaction or related seismic hazards, including seismic total or differential ground settlement, seismic downdrag and lateral spreading.

The project site and the adjacent areas are relatively flat. The existing abutment and approach embankment slopes consist of dense and stiff compacted fill soil. Based on these soil conditions and the absence of soil liquefaction potential, the existing fill slopes at the site are not considered subject to instability during the design seismic ground motion event.

The site is located more than 0.5 miles from the nearest coastline and is situated above elevation 40 feet, therefore the risk for tsunami does not exist (per MTD 20-13).

2.11 Geotechnical Recommendations

Recommendations must include discussion of the appropriateness of shallow foundations, driven pile foundations, and CIDH concrete pile foundations. Recommendations must be presented in the order of preference with the recommended foundation type(s) presented first; followed by feasible, but not preferred, alternatives; followed by foundation types not recommended. If applicable, include commentary relating to foundation types proposed by the Structure Designer (MTD 3-1, Table 3-2).

Example

The following is a discussion of the foundation system alternatives. This discussion is based upon an understanding of the regional geology and the observations of the subsurface conditions from the 1990 field investigation and construction of the existing bridge in 1992.

- *Driven Displacement Piles: Driven displacement piles such as Standard Plan precast prestressed concrete piles or closed end pipe piles are recommended for support of the new structure.*
- *Driven Non-Displacement Piles: Driven non-displacement open-ended pipe piles or H-piles are feasible for foundation support, however installed pile lengths are expected to be variable and difficult to predict in these subsurface conditions, particularly for the H-pile alternative. Driven displacement piles are preferable to driven H-piles.*
- *Cast-in-Steel-Shell (CISS) Concrete Piles: CISS piles are feasible for foundation support, however installed pile lengths are expected to be variable and difficult to predict.*
- *Large Diameter Drilled Shafts (CIDH Concrete Piles): Large diameter drilled shafts, those with diameters greater than 24 inches, are not recommended for support. Saturated granular foundation soils exist at this location. Caving and flowing soils are expected, and "wet" construction methods would be required.*
- *Small Diameter Drilled Shafts (CIDH Concrete Piles): Small diameter drilled shafts, those with diameters of 24 inches and less, are not recommended for support. Saturated granular foundation soils exist at this location. Caving and flowing soils are expected, and "wet" construction methods would be required.*
- *Spread Footings: The foundation conditions are not suitable for spread footings because of the presence of loose material in the upper 15 feet.*

2.12 Additional Field Work and Laboratory Testing

Describe the anticipated scope and types of fieldwork and testing that may be required to complete the geotechnical investigation. Discuss the potential need for entry permits, task orders, groundwater monitoring, access road construction, lane closures, etc.

Example

The available site information will not provide adequate data to complete the design recommendations for Dry Creek Bridge. Therefore, a field investigation consisting of borings, seismic CPT, and laboratory testing will be performed to characterize the site.

The District Project Manager must initiate the process of obtaining drilling clearances (e.g., environmental permits, right of entry, categorical exemptions, etc.) so that drilling, preliminary design, and the Preliminary Foundation Report can be completed prior to the end of PA&ED. For foundation investigation details, the District Project Manager may contact the Office of Geotechnical Design X.

Any questions regarding the above recommendations should be directed to the attention of NAME and PHONE.

2.13 Report Copy List

The SPGR must be addressed to the Structure Designer and copies provided to those listed under Report Distribution in the *Communications and Reporting* module.

2.14 Appendices

Appendix I: Ground Motion Data Sheet

3 PRELIMINARY FOUNDATION REPORT (PFR) and FOUNDATION REPORT (FR)

The PFR is prepared after completion of the SPGR and Advanced Planning Study, and prior to the Structure Type Selection. The number, location, types of foundations and associated loads will be better defined, and the site investigation may be complete. The amount of information provided in the PFR will be relative to the information provided by Structure Design and the extent of geotechnical investigation completed.

The FR expands on data provided in the PFR and updates the foundation recommendations based upon final loads provided by Structure Design. The FR becomes part of the contract documents via its inclusion in the Information Handout per Standard Special Provision 2-1.06B, “Supplemental Project Information.”

The following topics, if applicable, must be addressed in the Preliminary Foundation Report and Foundation Report.

3.1 Introduction

Summarize the scope and types of work performed to obtain the information supporting the foundation recommendations.

Example: Preliminary Foundation Report with 0-Phase Drilling

Per the request dated May 3, 2020, this Preliminary Foundation Report has been prepared for the proposed widening of Dry Creek Bridge. The purpose of this report is to summarize the investigations performed and to provide preliminary foundation recommendations for Dry Creek Bridge. The recommendations presented in this report are based on the draft general plan dated January 15, 2020, a subsurface investigation consisting of borings at the abutments, and preliminary loads and scour information provided by Structure Design. Borings were not completed at Bent 2 because access permits were not yet available.

Example: Foundation Report

Per the request dated May 3, 2020, this Foundation Report has been prepared for the proposed widening of Dry Creek Bridge. The purpose of this report is to summarize the investigations performed and to provide foundation recommendations for Dry Creek Bridge. The recommendations presented in this report are based on the general and foundation plans dated January 15, 2020, a subsurface investigation, and loads and scour information provided by Structure Design.

3.2 Project Description

Describe the existing and/or proposed structure(s), and pertinent project information relating to the planned foundation improvements. Provide project vertical datum reference.

Example

The bridge site is located in the city of San Diego on State Route 15 at PM R3.8, which crosses over Interstate 805 (I-805) at PM 15.1. At this site, the proposed bridge replacement is necessary to accommodate the underlying highway improvements, which *include the widening of the existing I-805 in order to provide additional High-Occupancy Vehicle (HOV) lanes. All elevations referenced within this report are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise noted. To convert an elevation at this site from National Geodetic Vertical Datum of 1929 (NGVD 29) to NAVD 88, add 2.3 feet to the NGVD 29 elevation.*

Based on the General Plan (dated January 15, 2020), the proposed bridge is a 2-span, cast-in-place, prestressed concrete box girder structure supported on pile foundations.

3.3 Exceptions to Policies and Procedures

Discuss exceptions to Departmental policies and procedures relating to the PFR/FR. Approved *Request for Exception* forms must be included in the Appendix. Omit this section if there are no exceptions.

3.4 Geotechnical Investigation

Provide an overview of the geotechnical investigation(s) performed to support the geotechnical recommendations including the number of boreholes/CPT soundings with maximum depth(s) and elevation(s), and the types of field and/or downhole testing (e.g., in-situ, geophysical).

<u>Example</u>

The Geotechnical Investigation included a review of the as-built borings from the 1966 investigation and drilling three borings in June 2020. The 1966 foundation investigation consisted of one 3-inch mud rotary boring and eight 1-inch driven soil tube borings. In June 2020, three mud rotary borings were drilled to a maximum depth of 80 feet (~ Elev. 230 feet) using a CS2000 drill rig. The Standard Penetration Test (SPT) was performed at regular intervals to evaluate the engineering properties of the earth materials. The type(s) and location(s) of field testing are shown on the LOTB sheets.

3.5 Laboratory Testing Program

Provide an overview of the laboratory testing program, if performed, to support the geotechnical recommendations. Briefly explain what the tests were used for (e.g. soil classification, settlement, strength parameters).

<u>Example</u>

During the June 2020 field investigation, soil samples were collected from borings RC-20-001 and RC-20-002 for soil classification and corrosion evaluation (Particle Size Analysis, Plasticity Index, Corrosion Testing). A summary of the test results is provided in the Appendix, and the test sample locations are shown on the Log of Test Borings.

3.6 Geotechnical Conditions

3.6.1 Geology

Identify the pertinent geologic map and the prominent geologic unit(s) at the structure site. Describe relevant geologic features such as faults, bedding, major joint attitudes, and folds if they may influence the design and construction of the structure.

Example
The Geologic Map of Santa Ana 30' x 60' Quadrangle shows that the site is underlain by Quaternary alluvium.

3.6.2 Surface Conditions

Describe site topography, surface water and drainage conditions, cuts and fills, geologic hazards such as landslides and rockfall, and land use history that may affect the proposed structure. Identify existing structures, facilities, and utilities near the proposed structure that may affect its design and construction.

Example
The topography is relatively flat and no geologic hazards have been identified.

3.6.3 Subsurface Conditions

Provide a generalized description of the subsurface conditions. The information included within this section may include, but is not be limited to:

- Types of soil/rock, depths to generalized layer breaks, and corresponding elevations
- Pertinent soil/rock conditions such as unsuitable materials (collapsible, expansive foundation materials)

Do not re-create the LOTB(s) in detail in this section. A generalized discussion or table is sufficient.

Example
Based on the 2020 site investigation, the alluvial soil at the site can generally be separated into three units. The upper unit consists of very loose to medium dense silty sand with gravel that extends from the ground surface to a depth of about 15 feet (~ Elev. 950 feet). The middle unit consists of dense sand to a depth of approximately 35 feet (~ Elev. 930 feet). The lowermost unit consists of dense to very dense gravelly sand and sandy gravel with isolated zones of sandy silt and gravel. This unit extends to the maximum explored depth of the borings, which is approximately 60 feet below the ground surface (~ Elev. 905 feet).

3.7 Groundwater

Report groundwater elevation(s) and dates of measurements. Use of the following table is recommended if there are numerous borings and/or measurements. Discuss surface water conditions that might influence the design or construction of the foundations. State the groundwater elevation(s) (e.g., liquefaction, pile design) used for analyses and design.

Summary of Groundwater Data

Location or Boring ID	Ground Surface Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Date Measured

Example

As-built LOTB's from the April 1968 subsurface investigation indicate that groundwater was encountered in several borings at that time, and ranged from elevation 19.0 feet to elevation 21.2 feet (NAVD88 datum). During the 2020 subsurface investigation groundwater was measured in Boring RC-20-001 at elevation 15.3 feet, and in Boring RC-20-002 at elevation 13.9 feet. During the 2020 subsurface investigation, groundwater was measured in boring RC-20-003 at elevation 17.1 feet, which corresponded to the level of the water in the riverbed at that time. The groundwater elevation used for design was 21 feet.

3.8 As-built Data

Include brief discussion of relevant As-built data, such as:

- Existing foundation types and details.
- As-built geotechnical capacities or resistances.
- Construction reports or records such as pile driving logs, pile load test reports, construction difficulties, etc.

Use the tables in the examples below to present foundation data.

Example: Driven Piles

Construction of the original bridge was completed in 1971 with all three supports supported on driven Alternative (Alt) "X" concrete piles with design loads of 45 tons. The 1971 As-built LOTB provided pile driving information, which included the minimum, average, and maximum penetration elevations for the piles. The bottom of pile cap elevations listed were obtained from the As-built foundation plan. Table 1 presents a summary of the 1971 As-built Data.

Table 1: Summary of the 1971 As-built Data

Support Location	*Foundation Type*	*Design Load*	*Bottom of Pile Cap Elev. (feet)*	*Min. Penetration Elev. (feet)*	*Avg. Penetration Elev. (feet)*	*Max Penetration Elev. (feet)*
Abutment 1	*12" Driven Alt "X" Pile*	*45 ton*	*958.1*	*929.3*	*927.1*	*926.5*
Bent 2	*12" Driven Alt "X" Pile*	*45 ton*	*935.5*	*920.2*	*918.1*	*916.1*
Abutment 3	*12" Driven Alt "X" Pile*	*45 ton*	*953.6*	*928.1*	*926.5*	*925.3*

<u>*Example: Shallow Foundations*</u>

The existing Cenda Ditch Bridge consists of a two-span, cast-in-place, slab bridge that was constructed in 1963. Abutments 1 and 3 are end-diaphragm abutments and are supported on spread footings placed in approximately 20 feet of embankment fill material. Pier 2 is supported on seven columns, each with a spread footing founded on rock. The As-built bottom of footing elevations and design loads for the bridge are shown below in Table 2.

Table 2: As-built Information

Location	*As-built Bottom of Footing Elevation (feet)*	*As-built Allowable Footing Pressure (tsf)*	*As-built Design Footing Pressure (tsf)*
Abutment 1	*4499.1*	*2.0*	*2.0*
Pier 2	*4475.0*	*5.0*	*5.0*
Abutment 3	*4490.7*	*2.0*	*2.0*

3.9 Scour Data

Report pertinent scour information obtained from the geotechnical investigations (e.g., BIRIS records, Hydraulics Report) including the potential for scour. Use the table in the example to present scour data from the Hydraulics Report.

If the field investigation reveals geologic information that contradicts the hydraulics report, then the Geoprofessional must discuss the findings in the PFR/FR and provide that information to the author of the hydraulics report so that the scour recommendations can be re-evaluated.

<u>*Example: Scour Data Available*</u>

The bridge site is underlain by alluvial soil, which are considered potentially scourable. The Structure Hydraulics Branch provided the following scour information in a report dated January 15, 2020 (Table 1).

Table 1: Scour Data

Support Location		*Long Term Scour (Degradation and Contraction) Elevation (feet)*	*Short Term Scour (Local) Depth (feet)*
Left Bridge	*Abut 1*	*2285.6*	*3*
	Abut 2	*2285.1*	*3*
Ruight Bridge	*Abut 1*	*2291.9*	*3*
	Abut 2	*2291.6*	*3*

Example: No Scour

The bridge does not span a watercourse, therefore there is no scour potential.

Example: Scour Data Unavailable

The bridge spans a watercourse. BIRIS records do not identify historic scour issues. The Structures Hydraulics Branch has not yet provided a Hydraulics Report to this Office.

3.10 Corrosion Evaluation

Include and update the corrosion data from the SPGR based on new findings and field investigations. If corrosion testing was not completed during the foundation investigation, provide justification for the corrosion recommendations.

Example: Non-Corrosive

Three soil samples and one water sample were collected for corrosion testing during the 2020 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on Caltrans' standards, the site is considered non-corrosive.

Example: Corrosive

During the 2020 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings RC-20-001 and RC-20-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered corrosive based on Caltrans' standards, and corrosion mitigation is required.

Table 1: Soil Corrosion Test Summary

Boring ID	*Elevation (feet)*	*Minimum Resistivity (Ohm-Cm)*	*pH*	*Chloride Content (ppm)*	*Sulfate Content (ppm)*	*Corrosive?*
RC-11-001	*15.8 to 14.3*	*1544*	*7.24*	*N/A*	*N/A*	*No*
RC-11-001	*-4.2 to -3.2*	*683*	*7.94*	*384*	*432*	*No*
RC-11-002	*-69.1 to -70.6*	*73*	*6.86*	*850*	*1500*	*Yes*
RC-11-002	*-104.1 to -105.6*	*78*	*7.71*	*1000*	*1600*	*Yes*

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

3.11 Seismic Information

Update the seismic information required for the SPGR based on new findings and/or investigations. Summarize analyses and evaluations performed, and recommendations relating to seismic design.

3.11.1 Ground Motion Hazard

Include the following information:

a. Site coordinates (latitude and longitude in decimal degrees)
b. The estimated time-average shear wave velocity V_{S30} and how it was determined (e.g., geophysics, seismic CPT or SPT correlations). For sites with a low V_{S30}, a site-specific ground response analysis may be required to determine the final design ARS. See the *Design Acceleration Response Spectrum* module.
c. For 5% probability of exceedance in 50 years (Return Period = 975 years).
   i. The ARS per the *Design Acceleration Response Spectrum* module.
   ii. The horizontal peak ground acceleration (HPGA).
   iii. Deaggregated mean earthquake moment magnitude (M) for the HPGA and the mean site-to source distance (R) for the 1.0 second period spectral acceleration.
d. Ground Motion Data Sheet (see *Forms and Templates*)
e. Soil Profile Classification (Class S1 or Class S2) for the site or support locations per the SDC, Section 6.1 and Section 6.2.3. For the PFR, omit if sufficient site data is not available.
f. <u>If requested:</u> provide site data and support for the development of ground motion time-histories for bridge project sites (typically for sites with low V_{S30}).
g. <u>If requested:</u> include the design ARS for the Functional Evaluation Earthquake with a Return Period = 225 years (see *Design Acceleration Response Spectrum* module).

<u>*Example*</u>

The site is susceptible to strong earthquake induced ground motions during the design life of the bridge.

Based on information obtained during the recent site exploration and seismic Cone Penetration Test for determining shear wave velocity, the time-average shear wave velocity (VS30) for the upper 100 feet of soil at the site is estimated to be 850 ft/sec.

The Design Spectrum for the Safety Evaluation Earthquake, as specified in Caltrans Seismic Design Criteria with October 2019 interim revisions, version 2.0 (SDC v2.0) is the probabilistic response spectrum representing the horizontal ground motion at the site with a 5% probability of exceedance in 50 years (return period = 975 years). The USGS's 2014 NSHM is used as the basis to determine the Design Spectrum in the form of the design Acceleration Response Spectrum (ARS).

Caltrans web-based tool ARS Online v3.0 was utilized to determine the design ground motion parameters, including the ARS, for the subject structure site. Based on the ARS Online v3.0 tool, the design PGA = 0.27g, and the deaggregated mean earthquake moment magnitude for PGA, M = 6.7 and the mean site-to-source distance for 1.0 second period spectral acceleration, R = 41.2 miles.

The Design Ground Motion Data Sheet, presenting the design ARS data, plot, and other relevant information, is attached.

The soil at the site is "Class S1" per the Sections 6.1 and 6.2.3 of the SDC, v2.0.

3.11.2 Other Seismic Hazards

The section must include information on the following seismic hazards:

a. Surface fault rupture potential (see *Fault Rupture* module)
b. Liquefaction potential (see *Liquefaction Evaluation* module)
c. Effects of Liquefaction, including
    i. Seismically-induced ground surface settlements at each support location
    ii. Downdrag at each support location with pile foundations (see *Downdrag* module)
    iii. Lateral spreading potential (see *Lateral Spreading* module)
d. Seismic slope stability
e. Tsunami risk (if applicable)

Discuss the findings and results of other seismic-design analyses, all applicable and necessary geotechnical seismic design recommendations (e.g., residual shear strengths for liquefied soil layers, seismic downdrag, lateral spreading loads/displacements, nominal bearing resistances of foundations with and without considering liquefaction for seismic retrofit projects, seismic lateral earth pressures, liquefaction mitigation measures, etc.).

<u>*Example: No Hazards*</u>

The site has been determined not to have potential for surface fault rupture, liquefaction, seismic-induced slope failure, or tsunami.

<u>*Example: No Surface Fault Rupture*</u>

The structure is not located within an Alquist-Priolo Earthquake Fault Zone or 1000 feet from any Holocene or younger aged fault. Therefore, per MTD 20-10, the structure is not considered susceptible to surface fault rupture hazards.

<u>Example: Surface Fault Rupture</u>

The structure is located within an Alquist-Priolo Earthquake Fault Zone. Therefore, per MTD 20-10, the structure is susceptible to surface fault rupture hazards. Per the attached Fault Rupture Report dated March 15, 2020, the horizontal displacement is estimated to be one foot and the vertical displacement is estimated to be 4 inches.

<u>Example: Liquefaction</u>

Due to the presence of loose to medium dense alluvial material and shallow ground water beneath the site, the potential for soil liquefaction is present at the site. Liquefiable zone elevations at the abutment and pier locations are summarized in Table 1.

<u>Example: Effects of Liquefaction</u>

Liquefaction-induced settlement of the ground surface and pile downdrag are anticipated and summarized in Table 1. Implications of liquefaction on the pile tip elevations will be addressed in the Geotechnical Recommendations section.

Table 1: Liquefaction Potential at Old River Bridge

Support	*Liquefaction Elevation (feet)*	*Estimated Seismic-induced Settlement (inches)*	*Downdrag Zone Bottom Elevation (feet)*	*Estimated Downdrag Load (kips/pile)*
Abutment 1	*Elev. 20 to 15* *Elev. 0 to -10*	*3*	*-5*	*150*
Pier 2	*Elev. 10 to -5*	*4*	*-3*	*50*
Abutment 3	*Elev. 20 to 10*	*3*	*12*	*100*

Note: Downdrag loads calculated for 24-inch CIDH concrete piles at the Abutments and 60-inch CISS at Pier 2.

<u>Example: Lateral Spreading Potential</u>

Due to the presence of liquefiable soils at shallow depths and relatively high design horizontal peak ground acceleration, an initial lateral spreading hazard assessment was performed at each abutment by ignoring all lateral resistance contributions from the foundation piles. The analysis was performed in accordance with Steps 1 through 4 of the Lateral Spreading Analysis Example of the Geotechnical Manual. Results of the analysis indicate a lateral spreading hazard potential at both abutments.

Additional lateral spreading analyses were performed for each abutment in accordance with MTD 20-15 and the Lateral Spreading Analysis Example module of the Geotechnical Manual. The pile restraining force versus displacement plots (MTD 20-15 Figure 5, Curve 3) developed for the two abutments are shown in Figures X and Y.

Due to the relatively flat ground surface conditions, lateral spreading potential does not exist at Pier 2.

<u>Example: Seismic Slope Stability</u>

Seismic slope stability analyses were performed to evaluate the overall stability at the proposed abutment slopes. The pseudo static analysis was performed with a horizontal seismic coefficient (k_h) equal to 0.15g.

Two-dimensional slope stability analyses were performed and the results are included in the appendix. The analyses found the minimum value of factor of safety at the proposed abutment slopes to be approximately 1.25 (resistance factor = 0.8), which meets the accepted minimums for stable abutment slopes (per AASHTO LRFD).

<u>Example: No Tsunami Risk</u>

The site is located about 0.25 miles from the nearest coastline. However, the ground surface elevation at the bridge location ranges from 100 to 120 feet above mean sea level. The site is not located within the tsunami inundation zone shown in California Official Tsunami Inundation Map for the <u>X</u> County (Interactive Map accessed on mm/dd/year).

Based on the above information and per MTD 20-13, a tsunami hazard does not exist at the site.

<u>Example: Tsunami Risk</u>

The site is located about 0.25 miles from the nearest coastline and the ground surface elevation at the bridge location ranges from 10 to 50 feet above mean sea level. The site is located within the tsunami inundation zone shown in California Official Tsunami Inundation Map for the <u>X</u> County (Interactive Map accessed on mm/dd/year).

Based on the above information and per MTD 20-13, a tsunami hazard exists at the site.

3.12 Geotechnical Recommendations

Provide complete and concise recommendations for bridge foundations by addressing the topics in the applicable portions (i.e., Shallow Foundations, Driven Pile Foundations, and/or CIDH Concrete Pile Foundations) of this section. Include recommendations for earth retaining structures connected to the bridge, or located within 150 feet of the bridge. Refer to *Foundation Reports for Earth Retaining Systems* for reporting requirements.

Present and/or discuss the following:

1. Identify all structures addressed in this section
2. Date of plans used for analysis (e.g., General Plan, Foundation Plan, Retaining Wall Layout Sheets)

<u>Example</u>

The following recommendations are for the proposed Dry Creek Bridge (Br. No. 54-1200) and associated wing walls as shown on the General Plan dated May 14, 2020, and a Foundation Plan received via electronic mail on June 28, 2020.

3.12.1 Shallow Foundations

Provide complete and concise recommendations by addressing the topics in the applicable portions of this section. Discuss any considerations which influence foundation type selection, analysis, and design (e.g., scour, groundwater, ground improvement).

Present the following in the Preliminary Foundation Report where the geotechnical investigation is complete:

1. Foundation Data table (after MTD 4-1, Attachment 4, Table 1), from the Structure Designer
2. A description of the material on which the footing is to be placed
3. A description of the ground line conditions (e.g., flat, sloped)
4. Preliminary Foundation Data Tables
   a. End Supports (Abutments) table (after MTD 4-1 Attachment 2, Table 1)
   b. Intermediate Supports (Bents and Piers) table (after MTD 4-1 Attachment 2, Table 2)

Table X: Foundation Data

Support Location	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Footing Dimensions (feet)		Permissible Settlement under Service Load (inches)
			B	L	
Abutment 1					
Bent 2					
Abutment 3					

Table X: Preliminary Foundation Data for Abutments

End Supports (Abutments)

Support Location:
Foundation Material (Soil or Rock)[1]:
Friction Angle or Undrained Shear Strength for Sliding:
Permissible Settlement (in):
Resistance Factor (Strength) – φ_b:
Resistance Factor (Seismic) – φ_b:

	Total Number of B' =			
No	Effective Footing Width	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B' (feet)	q_n (ksf)	q_{pn} (ksf)	q_R (ksf)
1				
2				
3				
4				
5				

1. Select “Soil” or “Rock” depending on design methodology used.
2. Based on L’ =____ ft.

Table X: Preliminary Foundation Data for Bents and Piers

Intermediate Supports (Bents and Piers)

Support Location:
Foundation Material (Soil or Rock)[1]:
Friction Angle or Undrained Shear Strength for Sliding:
Permissible Settlement (in):
Resistance Factor (Strength) – φ_b:
Resistance Factor (Seismic) –φ_b:

Total Number of unique *L′/B′* Ratios[2]	
Total Number of *B′*s per *L′/B′* Ratios[3]	

No	Effective Footing Width	Effective Footing Size Ratio	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B′ (ft)	*L′* / *B′*	*qn* (ksf)	*qpn* (ksf)	*qR* (ksf)
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					

1. Select “soil” or “rock” depending on design methodology used.
2. Indicates total number of curves used to show variations of q_n, q_{pn}, and q_R vs. *B’*.
3. Indicates total number of points on each curve to show variations of q_n, q_{pn}, and q_R vs. *B’*.

Example: Shallow Foundations (PFR)

At Abutments 1 and 2 support locations, spread footings are recommended. The subsurface information gathered for the site indicate that the abutment footings will be founded on dense sand. The foundation recommendations were designed in accordance with the AASHTO LRFD Bridge Design Specification with CA Amendments. The spread footings are located in close proximity to a descending slope and were designed as "footing on a slope." The following Foundation Geotechnical Data Tables provide preliminary recommendations for all support locations.

Present the following in the Foundation Report:

1. Information from the Structure Designer
   a. Foundation Data table (after MTD 4-1, Attachment 4, Table 1)
   b. Summary of Controlling Loads table (after MTD 4-1, Attachment 5, Table 1)
2. A description of the material on which the footing is to be placed.
3. A description of the ground line conditions (e.g., flat, sloped)
4. Foundation Design Recommendations for Spread Footing table (after MTD 4-1, Attachment 5, Table 2).
5. Spread Footing Data Table (after MTD 4-1, Attachment 5, Table 3).
6. Calculated resistance factor for overall global stability and local slope stability of the foundation (Service Limit State and Extreme Event Limit State).

If applicable, present the following additional items in the Foundation Report:

7. If spread footings are to be constructed below groundwater level, identify the type of excavation (Type A or Type D) required at all applicable support locations (See Bridge Design Aids, Section 11 - Estimating).
8. If unsuitable native soil underlies the proposed footing, specify sub-excavation and replacement with structure backfill.
9. Discussion of the influence of the new footing on the adjacent structures and/or utilities.

Table X: Foundation Data

Support Location	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Footing Dimensions (feet)		Permissible Settlement under Service Load (inches)
			B	L	
Abutment 1					
Bent 2					
Abutment 3					

Table X: Summary of Controlling Loads

Support Location	L (feet)	B (feet)	Controlling Loads						
			M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)	P_{total} (kips)	P_{perm} (kips)	*Load Combination*
Abutment 1				N/A	N/A				
Bent 2									
Abutment 3				N/A	N/A				

Table X: Foundation Design Recommendations for Spread Footing

Support Location	Footing Size (feet)		Bottom of Footing Elevation (feet)	Minimum Footing Embedment Depth (feet)	Total Permissible Support Settlement (inches)	Service Limit State	Strength Limit State *(φ_b=)*	Extreme Event Limit State *(φ_b=1.0)*
	B	L				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Abutment 1						(B' =)	(B' =)	N/A
Bent 2						(B' =)	(B' =)	(B' =)
Abutment 3						(B' =)	(B' =)	N/A

For each contact stress and bearing resistance in the table, include the associated effective footing width (B') in parentheses. See *Shallow Foundations* module for example.

Table X: Spread Footing Data Table

Support Location	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength/Construction Factored Gross Nominal Bearing Resistance (φ_b=) (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance (φ_b=1.0) (ksf)
Abutment 1			N/A
Bent 2			
Abutment 3			N/A

<u>Example: Shallow Foundations (FR)</u>

At Abutments 1 and 4 support locations, spread footings are recommended. The foundation recommendations are based on the information provided by Structure Design in the following tables and were designed in accordance with the AASHTO LRFD Bridge Design Specification with CA Amendments. The spread footings are in close proximity to a 1.5:1 descending slope and were designed as "footing on a slope." The following Foundation Design Recommendations table and Spread Footing Data Table provide final recommendations for Abutments 1 and 4.

Groundwater will be encountered during construction of the footings at the proposed abutments, therefore show Structure Excavation Type D on the plans.

The subsurface information gathered for the site indicate that the Abutment 1 footing will be founded in sedimentary rock formation. At Abutment 4, unsuitable native soils underlie the proposed footings. It is recommended that the native materials be removed to a depth of 5 feet (Elev. 15 feet) below the bottom of footing, and be replaced with structure backfill or concrete to the bottom of footing elevation. The bottom of sub-excavation elevations for the abutments are listed in Table 1.   The limits of the sub-excavation and replacement shown on the plans must conform to the limits specified in Standard Specification 19-5.03B for compaction of embankments under retaining wall footings without pile foundations.

Table 1: Bottom of Sub-Excavation Elevation

Support Location	*Bottom of Sub-Excavation Elevation (feet)*
Abutment 4	*15*

Global stability was analyzed at Abutment 4. The calculated resistance factors for global stability were found to satisfy current requirements for both the Service-I Limit State (static), as well as the Extreme Event Limit State (pseudo-static). Table 2 summarizes the global stability analysis results. Abutment 1 was not analyzed because it is founded on sedimentary rock formation.

Table 2: Global Stability Analysis Summary (Abutment 4)

Service Limit State	*Calculated Factor of Safety*	*Calculated Resistance Factor*	*2017 AASHTO LRFD Resistance Factor Requirement*
Service-I Limit State (Static)	*1.5*	*0.65*	*≤0.65*
Extreme Event Limit State (Pseudo-Static)	*1.11*	*0.89*	*≤0.90*

3.12.2 Deep Foundations

3.12.2.1 Driven Pile Foundations

Provide complete and concise recommendations by addressing the topics in the applicable portions of this section. Discuss any considerations which influence type selection, analysis, and design (e.g., scour, liquefaction, lateral spreading, ground water).

Present the following in the Preliminary Foundation Report where the geotechnical investigation is complete:

1. Information provided by Structure Designer
    a. Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1, Table 3-2)
2. Preliminary Foundation Design Recommendations table, that includes the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} and ϕ_{qp} = 0.7 for side resistance and tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 and 10.5.5.2.3-1.
    b. Round the Required Nominal Resistance up to the nearest 10 kips.

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Table X: Preliminary Foundation Design Recommendations

Support Location	Pile Type	Cutoff Elevation (feet)	Required Nominal Resistance (Strength Limit State) (kips)		Preliminary Tip Elevation (feet)
			Compression (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	
Abutment 1					
Pier 2					
Abutment 3					

Add note if applicable:

- Cutoff elevations not provided by Structure Design and are estimated by Geotechnical Services

Present the following in the Foundation Report:

1. Information provided by Structure Designer
    a. Foundation Design Data Sheet (MTD 3-1, Attachment 1, Table 3-4)
    b. Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5)
2. Foundation Design Recommendations table (after MTD 3-1, Attachment 1, Table 3-6), that includes the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} and ϕ_{qp} = 0.7 for side resistance and tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 and 10.5.5.2.3-1.
    b. Round the Required Nominal Resistance (Strength and Extreme Limit State) and Factored Design Loads (Service Limit State) to the nearest 10 kips.
3. Pile Data Table (MTD 3-1, Attachment 1, Table 3-7)
4. The Required Nominal Driving Resistance is the side resistance of all penetrated soil, including scourable, downdrag, and liquefiable layers, plus the tip resistance. Show this value in the Foundation Design Recommendations table and the Pile Data Table.

If applicable, present the following:

5. Present the wall thickness for driven steel shells or cast-in-steel-shell piles in the Foundation Design Recommendation and Pile Data tables.
6. If the Required Nominal Resistance does not equal the Required Nominal Driving Resistance (e.g., scour susceptible layer, liquefiable layer), explain why.
7. For projects where a pile drivability study has been performed during the design phase (for pipe piles and steel shells), provide the minimum pile wall thickness determined by the pile drivability study.
8. If a Standard Plan pile or steel H-pile requires modification (e.g., increased wall thickness, adding driving tips or lugs), provide recommendations so that the modifications will be shown on the project plans.

    Example

    Install lugs on all steel "H" piles prior to driving. It is recommended that the pile detail sheets or abutment detail sheets show the lugs as illustrated in the Bridge Construction Records and Procedures Manual, Bridge Construction Memo 130-5.0, except that the lugs be located 10 feet from the pile tip.

    Example

    Show Modified Class 200, Alternative "W" steel pipe pile details on the project plans. The modified pipe pile must be shown with a flat circular steel plate or conical steel tip with a minimum thickness of ¾ inch welded to the pile tip, similar to the Alternative "V" pile tip detail shown in the Standard Plans.

9. If the design calculations account for liquefaction, discuss how the effect of liquefaction was incorporated in the pile foundation recommendations.
10. When the foundation soil is designated as Class S2 soil, the Structure Designer will request soil parameters for the lateral analyses for both non-liquefied and liquefied conditions. In some cases, complete lateral analyses may also be requested. Present the data in the Appendix.

11. If the design calculations account for seismic downdrag then add the following:
"The design loads and design tip elevations were adjusted to account for seismic downdrag. The additional seismic downdrag loads calculated by Geotechnical Services were provided to Structure Design, and appropriate load factors were applied by Structure Design and incorporated into the Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5) provided by Structure Design to this Office."

12. If the design calculations account for static downdrag, discuss how the effect of static downdrag was incorporated in the pile foundation recommendations.
13. When a pile cap excavation is anticipated to extend below the groundwater surface elevation, the Geoprofessional must discuss with the Structure Designer and identify the "type" of structure excavation (Type A or Type D) required at all applicable support locations (See Bridge Design Aids, Section 11 - Estimating).

Example
Show Type D excavation on the plans at Piers 2 and 3.

14. For CISS piles, state the top of soil plug elevation and the seal course thickness (if applicable) required for the tip resistance design of CISS piles.

Example
At Abutment 4, a soil plug is utilized to develop internal side resistance in the lower portion of the CISS pile for tip resistance design. The top of the soil plug elevation must be at elevation 252 feet. A seal course thickness of 5 feet is required to counteract the hydrostatic forces of the groundwater and to allow for the pile reinforcement and concrete to be poured in the dry.

Table X: Foundation Design Data Sheet

Support Location	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abut 1							
Pier 2							
Abut 3							

Table X: Foundation Factored Design Loads

Support Location	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
			Compression		Tension		Compression		Tension	
	Total Load per Support	Permanent Load per Support	Per Support	Max. per Pile	Per Support	Max. per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1							N/A	N/A	N/A	N/A
Pier 2										
Abut 3							N/A	N/A	N/A	N/A

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut-Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Perm.		Comp. (ϕqs=0.7) (ϕqs=0.7)	Tension (ϕqs=0.7)	Comp. (ϕqs=1) (ϕqs=1)	Tension (ϕqs=1)			
Abut 1	Class 140 Alt "V"							N/A	N/A	(a-I) (c)		
Pier 2	CISS 24 x 0.5									(a-I) (b-I) (a-II) (b-II) (c)		
Abut 3	Class 140 Alt "V"							N/A	N/A	(a-I) (c)		

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement

If the design tip elevation for settlement is not calculated because the pile tip is in rock, add the following note:

- Design Tip Elevations for Settlement were not calculated because the pile are tipped in rock.

If applicable:

- The specified tip elevations shall not be raised above the design tip elevations for Tension, Settlement and Lateral Load.
- The lateral design tip elevations provided by Structure Design are the lowest design tip elevation, and are therefore the Specified Tip Elevations. The Required Nominal Driving Resistances are based on the lateral design tip elevations.

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Abutment 1	Class 140 Alt. “V”			(a) (c)		
Pier 2	CISS 24 x 0.5			(a) (b) (c)		
Abutment 3	Class 140 Alt. “V”			(a) (c)		

Present the following Notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a) Compression, (b) Tension, (c) Settlement

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable, add the following note:

- The specified tip elevations shall not be raised above the design tip elevations for Tension, Settlement and Lateral Load.
- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations. The Required Nominal Driving Resistances are based on the lateral design tip elevations.

3.12.2.2 CIDH Concrete Pile Foundations

Provide complete and concise recommendations by addressing the topics in the applicable portions of this section. Discuss any considerations which influence type selection, analysis, and design (e.g., scour, liquefaction, lateral spreading, groundwater, usage of casings or shells). Discuss how the pile was designed (e.g., side and tip resistance, locations where geotechnical resistance was ignored, downdrag zones).

Present the following in the Preliminary Foundation Report where the geotechnical investigation is complete:

1. Information provided by Structure Designer
    a. Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1, Table 3-2)
2. Preliminary Foundation Design Recommendations table, that includes the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} = 0.7 and ϕ_{qp} = 0.5 for side resistance and tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 and 10.5.5.2.4-1.
    b. Round the Required Nominal Resistance up to the nearest 10 kips.

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Table X: Preliminary Foundation Design Recommendations

Support Location	Pile Type	Cutoff Elevation (feet)	Required Nominal Resistance (Strength Limit State) (kips)		Preliminary Tip Elevation (feet)
			Compression (ϕqs=0.7) (ϕqp=0.5)	Tension (ϕqs=0.7)	
Abutment 1					
Pier 2					
Abutment 3					

Add note if applicable:

- Cutoff elevations not provided by Structure Design and are estimated by Geotechnical Services

Present the following in the Foundation Report:

1. Information provided by Structure Designer
    a. Foundation Design Information Sheet (MTD 3-1, Attachment 1, Table 3-4)
    b. Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5)
2. Foundation Design Recommendations table (MTD 3-1, Attachment 1, Tables 3-6 or Tables 3-8), with the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} = XX for side resistance, ϕ_{qp} = YY for tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 & 10.5.5.2.4-1.
    b. Round the Required Nominal Resistance (Strength and Extreme Limit State) and Factored Design Loads (Service Limit State) to the nearest 10 kips.
    c. If a CIDH concrete pile is supporting a single column, identify whether the pile is a Type I or Type II shaft in the "Pile Type" column.
    d. For situations where a Permanent Steel Casing is used:
        i. add column "Permanent Steel Casing Specified Tip Elevation (feet)".
    e. For situations where a Driven Steel Shell is used as a Permanent Casing:
        i. add column "Driven Shell Specified Tip Elevation (feet)".
        ii. add column "Required Nominal Driving Resistance (kips)".
    f. For situations where a Rock Socket is used:
        i. Add column "Top of Rock Socket Elevation (feet)" to the Foundation Design Recommendations table and the Pile Data Table.
        ii. Add footnote: "The Bottom of Rock Socket Elevation is equal to the Specified Tip Elevation." to the Foundation Design Recommendations table and the Pile Data Table.
3. Pile Data Table (MTD 3-1, Attachment 1, Tables 3-7 or 3-9), with the following modifications:
    a. Round the Nominal Resistance up to the nearest 10 kips.
    b. If a CIDH concrete pile is supporting a single column, identify whether the pile is a Type I or Type II shaft in the "Pile Type" column.
    c. For situations where a Permanent Steel Casing is used:
        i. add column "Permanent Steel Casing Specified Tip Elevation (feet)".
    d. For situations where a Driven Steel Shell is used as a Permanent Casing:
        i. add column "Driven Shell Specified Tip Elevation (feet)".
        ii. add column "Required Nominal Driving Resistance (kips)".
    e. For situations where a Rock Socket is used:
        i. Add column "Top of Rock Socket Elevation (feet)" to the Foundation Design Recommendations table and the Pile Data Table.
        ii. Add footnote: "The Bottom of Rock Socket Elevation is equal to the Specified Tip Elevation." to the Foundation Design Recommendations table and the Pile Data Table.

If applicable, present and/or discuss the following additional items:

4. If the design calculations account for liquefaction, discuss the how the effects of liquefaction was incorporated in the pile foundation recommendations
5. When the foundation soil is designated as Class S2 soil, the Structure Designer will request the soil parameters for the lateral analysis for both non-liquefied and

liquefied conditions. In rare cases, complete the lateral analysis may also be requested. Present the data in the Appendix.

6. If the design calculations account for seismic downdrag then add the following:

   "The design loads and design tip elevations were adjusted to account for seismic downdrag. The additional seismic downdrag loads calculated by Geotechnical Services were provided to Structure Design, and appropriate load factors were applied by Structure Design and incorporated into the Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5) provided by Structure Design to this Office."

7. If the design calculations account for static downdrag, discuss the how the effect of static downdrag was incorporated in the pile foundation recommendations.
8. If Permanent Smooth-wall Steel Casing, Driven Steel Shell, or CMP is recommended, discuss the reasoning of their selection in the report. State if the structural capacity (from Structure Design) and/or geotechnical side resistance of the casing, shell, or CMP is used in pile design. If a CMP is to be utilized, state that the permanent steel casing must be a CMP.

   <u>Example</u>

   The structural capacity and geotechnical side resistance of the driven steel shell were used in the design of the pile.

   <u>Example</u>

   The structural capacity of the permanent smooth-wall steel casing was used in the design of the pile. The geotechnical side resistance of the permanent smooth-wall steel casing was not used in the design of the pile.

   <u>Example</u>

   The permanent casing may be specified as either smooth-walled steel or a CMP. Neither the structural capacity nor the geotechnical side resistance of the permanent casing was used in the design of the pile.

   <u>Example</u>

   The permanent casing must be specified as a CMP. The upper 20 feet of the CMP was used for the geotechnical side resistance of the pile design. The structural capacity of the CMP was <u>not</u> used in the design of the pile.

9. When a pile cap excavation is anticipated to extend below the groundwater surface elevation, the Geoprofessional must discuss with the Structure Designer and identify the "type" of structure excavation (Type A or Type D) required at all applicable support locations (See Bridge Design Aids, Section 11 - Estimating).

   <u>Example</u>

   Show Type D excavation on the plans at Piers 2 and 3.

Use the applicable Foundation Design Recommendations table and Pile Data Table listed below, and presented on the following pages.

- CIDH Concrete Pile in Soil without Casing
- CIDH Concrete Pile in Soil with Permanent Casing
- CIDH Concrete Pile in Rock Socket without Permanent Casing
- CIDH Concrete Pile in Rock Socket with Permanent Casing

CIDH Concrete Pile in Soil (no Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation. (feet)
						Strength/Construction		Extreme Event			
			Total	Permanent		Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression	Tension		
Bent 2	60” CIDH Concrete Piles									(a-I) (a-II) (c)	
Bent 3	60” CIDH Concrete Piles									(a-I) (a-II) (c)	

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Soil (no Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension		
Bent 2	60” CIDH Concrete Piles			(a) (c)	
Bent 3	60” CIDH Concrete Piles			(a) (c)	

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (c) Settlement.
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevation provided by Structure Design is lowest design tip elevation, and is therefore the Specified Tip Elevation.

CIDH Concrete Pile in Soil (Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Permanent Casing Specified Tip Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
			Total	Permanent		Strength/Construction		Extreme Event				
						Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression (φqs=1.0) (φqp=1.0)	Tension (φqs=1.0)			
Bent 2	60” CIDH Concrete Piles with Casing (Type II Shaft)										(a-I) (a-II) (c)	
Bent 3	60” CIDH Concrete Piles with Casing (Type II Shaft)										(a-I) (a-II) (c)	

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Soil (Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Permanent Casing Specified Tip Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension			
Bent 2	60" CIDH Concrete Piles with Casing (Type II Shaft)				(a) (c)	
Bent 3	60" CIDH Concrete Piles with Casing (Type II Shaft)				(a) (c)	

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (c) Settlement.
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (no Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation. (feet)
						Strength/Construction		Extreme Event				
			Total	Permanent		Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression (φqs=1.0)	Tension (φqs=1.0)			
Bent 2	60” CIDH Concrete Piles										(a-I) (a-II) (b-I) (b-II) (c)	
Bent 3	60” CIDH Concrete Piles										(a-I) (a-II) (b-I) (b-II) (c)	

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevation for settlement is not calculated because the pile tip is in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (no Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension			
Bent 2	60" CIDH Concrete Piles				(a) (b) (c)	
Bent 3	60" CIDH Concrete Piles				(a) (b) (c)	

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement.
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Steel Casing Specified Tip Elevation (feet)
			Total	Permanent		Strength/Construction		Extreme Event					
						Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression (φqs=1.0) (φqp=1.0)	Tension (φqs=1.0)				
Bent 2	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)										(a-I) (a-II) (b-I) (b-II)		
Bent 3	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)										(a-I) (a-II) (b-I) (b-II)		

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- Below the steel casing tip elevations, the CIDH concrete pile diameter is 48”
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Steel Casing Specified Tip Elevation (feet)
		Compression	Tension				
Bent 2	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)				(a) (b) (c)		
Bent 3	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)				(a) (b) (c)		

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement.
- Below the steel casing tip elevations, the CIDH concrete pile diameter is 48”
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

3.12.3 Dynamic Monitoring and Pile Load Testing

Identify support locations for dynamic monitoring. The control zones are identified in the Notes for Specifications.

Example: Dynamic Monitoring

At Piers 2, 4 and 6, dynamic monitoring is to be performed on the first CISS pile installed and will determine the pile acceptance criteria (SS 49-1.01D(4)). The control zones are identified in the Notes for Specifications section of this report.

In consultation with FTI staff, present and/or discuss the following for Pile Load Tests:

1) Control zones and associated support locations for the pile load tests.
2) Location, type and specified tip elevation of the load test pile and anchor piles in the Pile Load Test data table.
3) Type of load test
    a. Compressive (ASTM D 1143)
    b. Tensile (ASTM D 3689)
4) Purpose of test (select one)
    a. Proof test at Nominal Resistance
    b. Load to failure (provide estimate of maximum test load)
5) Identify piles to be dynamically monitored (Per the Standard Specifications, the load test pile and at least one anchor pile).
6) Restrike schedule if pile-setup is anticipated.

Example: Driven CISS Piles – Pile Load Test

Pile load tests in compression must be conducted on a non-production 48-inch CISS pile between Bent 4 of the left and right bridges, installed as required in the Pile Load Test Data table (Table 1). The control zone for the pile load test at Bent 4 will be Bents 2, 3, and 4 of the left and right bridges. During pile installation, the load test pile and one anchor pile will be dynamically monitored.

The compression test must be performed in two stages. For Stage 1 Load Test, load the test pile to the nominal axial geotechnical resistance after driving the test pile to the tip elevation and before removal of the soil plug. For Stage 2 Load Test, load the test pile to the "failure load" after removal of the soil plug, placement a 5-foot thick seal course, and placement of concrete in the test pile. The estimated maximum test load is 2700 kips.

Table 1: Pile Load Test Data

Support Location	*Pile Type*	*Ground Elevation (feet)*	*Cut-Off Elevation (feet)*	*Specified Top of Soil Plug Elevation (feet)*	*Specified Tip Elevation (feet)*	*Required Nominal Driving Resistance (kips)*
Bent 4	*CISS 48 x 1.0 (Load Test Pile)*	*98*	*108*	*40*	*30*	*2350*
	CISS 24 x 0.5 (Anchor Piles)	*98*	*101*	*N/A*	*40*	*800*

3.12.4 Approach Fills

This section may be omitted if the approach fill (fill within 150 feet of the bridge abutment) does not require special considerations for soft or otherwise unsuitable soil.

Present approach fill recommendations in this section. Reporting requirements for embankment construction on soft soil are presented in the *Embankment* module and the *Ground Improvement* module of the Geotechnical Manual.

In cases where settlement-related recommendations are presented, the Geoprofessional must collaborate with the author of the Geotechnical Design Report to assure that the recommendations are compatible. Issues to discuss may include:

- Anticipated settlement magnitude
- Rate of construction
- Settlement periods
- Surcharge locations and heights
- Prefabricated vertical drain locations, spacings, and lengths (see *Ground Improvement* module)
- Use of geosynthetics for separation and/or reinforcement (see *Geosynthetics* module)

3.13 Notes for Specifications

Omit this section for the Preliminary Foundation Report.

This section provides recommendations to the Specifications Engineer for inclusion and editing of Standard Special Provisions and developing NSSPs. Refer to the *Geotechnical Notes for Specifications* module for guidance on how to prepare this report section.

<u>Note:</u> This is a new section in this reporting standard. The information placed in this section was previously in the “Construction Considerations” section. This updated version of FR for Bridges separates communication to the Specifications Engineer (Notes for Specifications) and to Construction (Notes for Construction).

3.14 Notes for Construction

Omit this section for the Preliminary Foundation Report.

Notes for Construction are written to State construction personnel and contractors. Specific geologic conditions that are relevant to construction inspection should be cited in this section to ensure that both the intent of the geotechnical design is met and construction is successful. Include the following Notes if applicable.

3.14.1 Notes for Construction (Shallow Foundations)

1) Include the following instructions (edited for the project site conditions) to address potential disturbance of native material below the specified bottom of footing elevation(s).

Example: Footing on Soil

At all support locations, the spread footings are to be founded on the native alluvium. The structural concrete is to be placed neat against the undisturbed native alluvium at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed to a depth of 1 foot below the disturbance, and replaced at 95% relative compaction.

Example: Footing on Rock

At all support locations, the spread footings are to be founded on the weathered rock. The structural concrete is to be placed neat against the trimmed walls and undisturbed rock at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed and replaced to the bottom of footing elevation with concrete.

2) Include the following instructions to request footing inspections by the Geoprofessional.

Example

All support footing excavations are to be inspected and approved by the Office of Geotechnical Design <u>X</u>, Branch <u>Y</u>. The inspections are to be made after the excavation has been completed to the bottom of footing elevations and prior to placing concrete or rebar in the excavations. The contractor is to allow seven working days for the inspection of each footing excavation to be completed. The Structures Representative is to provide the Office of Geotechnical Design <u>X</u>, Branch <u>Y</u> a one-week notification prior to beginning the seven-day contractor waiting period.

(Note: If sub-excavation and replacement are required, modify the above example to require the inspection to be performed when the contractor completes the sub-excavation and prior to replacement.)

3.14.2 Notes for Construction (Driven Piles)

3) State how the nominal resistance was developed for CISS piles.

Example

The geotechnical resistance of the CISS piles was developed using external side resistance and tip resistance based upon a combination of the end area of the steel shell and the internal side resistance of the portion of the soil plug that will be left in place.

4) Provide cut-off criteria for pile acceptance (Standard Plan and H piles).

Example

Pile acceptance is to be based on Standard Specifications 49-2.01A(4)(c) "Department Acceptance." At Abutments 1 and 4 support locations, any pile that achieves 1½ times the required nominal driving resistance in compression, as shown on the contract plans, within 5 feet of the specified pile tip elevation, may be accepted and cut off with written approval from the Engineer. (i.e. 1½ times the required nominal driving resistance in compression is 580 kips at Abutment 1 and 600 kips at Abutment 4).

3.14.3 Notes for Construction (CIDH Concrete Piles)

5) State if the pile construction is expected to encounter groundwater, and if the slurry displacement method is anticipated for the CIDH concrete piling construction.

   <u>Example</u>

   It is anticipated that concrete placement for the CIDH concrete piles will occur below the groundwater surface, which may necessitate the slurry displacement method.

6) Report how the geotechnical resistance is derived, whether from side resistance and/or tip resistance. Present the highest "Top" elevation and lowest "Bottom" elevation for soil that contribute to pile side resistance in the "CIDH Concrete Pile Side Resistance Zone Elevations" table.

   <u>Example: CIDH Geotechnical Resistance in Soil</u>

   The calculated "Nominal Resistance" of the CIDH concrete piles was based on side resistance only. Tip resistance was not used. The zones used to calculate the side resistance of the CIDH concrete piles are shown in Table 1.

Table 1: CIDH Concrete Pile Side Resistance Zone Elevations

Support Location	*Top of Side Resistance Zone Elevation (feet)*	*Bottom of Side Resistance Zone Elevation (feet)*	*Specified Tip Elevation (feet)*
Pier 2	*165.4*	*151.4*	*145.0*
Pier 3	*165.4*	*153.4*	*147.0*
Pier 4	*168.5*	*156.0*	*150.0*

Example: CIDH Geotechnical Resistance in Rock Socket

The calculated "Nominal Resistance" of the CIDH concrete piles was based on side resistance only. Tip resistance was not used. The zones used to calculate the side resistance of the CIDH concrete piles are shown in Table 2.

If the actual top of rock elevation varies by more than X feet from the elevation presented in Table 2, the Office of Geotechnical Design Y must be contacted for further instruction.

Table 2: CIDH Concrete Pile Side Resistance Zones

Support Location	*Top of Rock Elevation (feet)*	*Top of Side Resistance Zone Elevation (feet)*	*Bottom of Side Resistance Zone Elevation (feet)*	*Specified Tip Elevation (feet)*
Pier 2	*67.5*	*65.5*	*50.5*	*48.5*
Pier 3	*67.5*	*65.5*	*52.5*	*50.5*
Pier 4	*70.5*	*68.5*	*55.0*	*53.0*

3.15 Report Copy List

Reports must be addressed to the Structure Designer and copies provided to those listed under Report Distribution in the *Communications and Reporting* section of the Offices of Geotechnical Design – Quality Management Plan.

3.16 Appendices

The Foundation Report appendices provide detailed information supporting foundation type selection, analyses, and recommendations. Reports prepared by Geotechnical Services staff must include the following (in the order presented, numerated as Appendix I, Appendix II, …):

All Foundation Reports:

- Appendix I: Ground Motion Data Sheet

If produced during the investigation:

- Approved "Request for Exception" forms
- Field-generated Geologic Map and Cross-Sections: Do not include copies of published maps.
- Geophysical Test Reports
- Laboratory Test Data (including Corrosion Test Report) – Organized by test type. In addition to the raw laboratory test results, organize and provide summary tables and graphs developed for the interpretation of laboratory test results.
- Pile Drivability Study
- Soil parameters for lateral analysis and/or P-Y Curves

Optional:

- Photos of Rock Cores
- Photos relevant to the investigation findings, design recommendations, and construction. Photos that illustrate content presented in the text should be embedded in the report if feasible.

Reports prepared by consultants must include the following (in the order presented, numerated as Appendix I, Appendix II, …):

All Foundation Reports:

- Appendix I: Site Map showing project location
- Appendix II: Ground Motion Data Sheet
- Appendix III: Log of Test Borings (including as-built LOTB)
- Appendix IV: Field Exploration and Testing: Data acquired from field exploration and testing such as surface geologic mapping and surface geophysical surveys, logs from the Cone Penetration Test, Pressuremeter, Dilatometer, and in-situ Vane Shear Tests, Borehole Geophysical logging, indicator pile tests, Piezometer Readings, etc.
- Appendix V: Calculation Package
  - The objectives of each calculation, such as time rate of settlement or bearing resistance.
  - List calculation assumptions
  - The geotechnical model used for each calculation
  - The equations used and meaning of the terms used in the equations
  - Copies of the curves or tables used in the calculations and their source.
  - The load and resistance factors, or factors of safety, used for the design
  - If the calculations are performed using computer spreadsheets – step-by-step calculations for one example to demonstrate the basis of the spreadsheet. A computer spreadsheet is not a substitute for the step-by-step calculation.
  - Summary of the calculation results that form the basis of geotechnical recommendations, including a sketch of the design, if appropriate.
- Previous Caltrans review comments and responses

If produced during the investigation:

- Approved "Request for Exception" forms
- Field-generated Geologic Map and Cross-Sections. Do not include copies of published maps.
- Photos of Rock Cores
- Photos that are relevant to the investigation findings, design recommendations, and construction. Photos that illustrate content presented in the text should be embedded in the report if feasible.
- Geophysical Test Reports
- Laboratory Test Data (including corrosion) – Organized by test type. In addition to the laboratory test results, summarize and provide summary tables and graphs developed for the interpretation of laboratory test results.
- Pile Drivability Study
- Soil parameters for lateral analysis and/or P-Y Curves