Subject:	**Conceptual-Level Seismic Design and Geohazard Evaluation Criteria (Final Draft)**
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1. Introduction

The purpose of this Technical Memorandum (TM) is to provide general guidelines for the conceptual design regarding a seismicity evaluation, as well as the development of seismic design ground motions and potential ground rupture or faulting for various facilities, including:

- Intakes
- Pumping plants
- Levees (excluding existing Delta levees)
- Embankments
- Appurtenant works
- Tunnels
- Shafts
- Buildings
- Bridges
- Other project features planned for the Delta Conveyance Project (Project)

Section 4 discusses the evaluations of liquefaction-induced hazards. The DCA will issue a separate TM that also addresses the seismic design and geohazard evaluation guidelines for the final design.

The material presented in this TM has been prepared in accordance with recognized engineering principles. However, before using the design criteria, the design engineer should exercise competent engineering judgment about its suitability for the complexity, criticality, and importance of the facility being evaluated or designed.

The Delta Conveyance Project (project) consists of new facilities to improve water conveyance from north of the Delta to existing south Delta State Water Project or Central Valley Project facilities (or both), to continue water conveyance to users located south of the Delta. The project facilities would be considered "critical," as long delays in water delivery to south of the Delta could significantly impact human lives and the California economy. Critical facilities designs would consider extended time frames (delays) and relatively high costs for major repair and replacement efforts (such as large pumps or tunnel structures) and interruptions to operations and water delivery. The human-occupied facilities (such as pumping plants) would also be designed for collapse prevention (life safety), as described in the current building codes (such as American Society of Civil Engineers [ASCE] 7 and California Building Code). The new facilities would not be classified as "essential," because they would not directly supply water to people or for fire suppression or be expected to be used in an emergency.

1.1 Organization

- Seismic Ground Motions
- Fault Rupture
- Liquefaction-induced Geohazard Guidelines
- References
- Document History and Quality Assurance

1.2 Facility Descriptions and Jurisdictions

The project includes the following general facilities (Table 1).

Table 1. Delta Conveyance Facilities

Facility	Anticipated Jurisdictional Agency or Code
Intakes	USACE
Embankments	DSOD
Tunnels	Project-specific – DCA and DWR
Temporary Structures	Project-specific – DCA and DWR
Shafts – Permanent	Project-specific – DCA and DWR
Pumping Plant	California Building Code
South Delta Conveyance Facilities – Canals and Gates	Project-specific – DCA and DWR
Levees	State of California Code of Regulations Title 23, USACE
Bridges	Caltrans Seismic Design Criteria

Notes:

Caltrans = California Department of Transportation

DSOD = State of California Division of Safety of Dams

USACE = U.S. Army Corps of Engineers

2. Seismic Ground Motions

To develop this TM, the DCA evaluated the seismicity and associated ground motions for project features via the following tasks:

1) Review existing data and information to identify and characterize seismogenic sources and background seismicity most relevant to any site.
2) Assess site conditions, including subsurface and shear-wave velocity profiles, to identify competent-soil deposits for estimating reference ground motions.
3) Estimate appropriate reference ground motion hazards and develop site-specific design response spectra by conducting a site-specific seismic hazard analysis.
4) Develop site-specific acceleration, velocity, and displacement time histories, as necessary, to evaluate the seismic performance of critical facilities.
5) Evaluate the effects of local soils on ground motions, as necessary.

This section provides guidelines for assessing seismicity and developing site-specific design ground motions for the facilities.

2.1 Seismic Loading and Performance Criteria

DWR's *Seismic Loading Criteria Report for State Water Project* (DWR, 2012a) presents minimum seismic loadings for the State Water Project (SWP) and provides different levels of seismic loading criteria based on the criticality of a facility. The guidelines allow flexibility so the project engineer can judge which criteria to use, because designing all the facilities to the same seismic standard would not be feasible, reasonable, or cost-effective. The *Delta Seismic Design Report* (DWR, 2012b) presents seismic design criteria and recommendations for the development of fault rupture and ground motions for the SWP facilities near the Sacramento-San Joaquin River Delta Region.

The seismic loading and performance criteria for a facility are based on:

- Consequences of failure
- Criticality of the structure for water delivery
- Downtime and cost for the repair of the facility

The loading criteria report (DWR, 2012a) states life-safety protection, post-earthquake emergency access, and difficulty or ease of repair work would be considered. For instance, canals can be repaired within a reasonable timeframe compared to tunnels or the large pumps of a pumping plant.

Table 2 summarizes facility-specific seismic criteria for both structural and geological hazard evaluations. These criteria generally conform with those recommended by DWR for the SWP (DWR, 2012a), with some modifications to incorporate specific jurisdictional requirements (such as DSOD, USACE, and Caltrans) and current practice for similar facilities as recommended by other agencies (such as Bay Area Rapid Transit, East Bay Municipal Utility District, and San Francisco Public Utilities Commission [SFPUC]).

In general, the recommended seismic loadings consider two-level design earthquakes:

1) Operational basis earthquake (OBE) - defined as the probabilistic ground motion with a return period of 475 years (20 percent probability of being exceeded in 100 years)
2) Maximum design earthquake (MDE) – facility-specific and represents the rare events that have low probability of occurrence during the lifespan for which a facility is designed or evaluated

2.2 Facility-specific Seismic Design Criteria

This section presents additional details about the facility-specific design criteria summarized in Table 2.

Intakes: The design of the actual structures at the intake facilities (such as the screens, gates, and reinforced concrete structures) would be subject to the requirements of the USACE. Further, if the sediment basin would not be contained within the jurisdictional levee, the sedimentation basin would be subject to the California DSOD design requirements. For this TM, it is assumed that the sediment basin would not be subject to the California DSOD requirements.

Table 2. Recommended Seismic Loading and Performance Criteria

Facility	Performance Objectives	Seismic Loads for Structure Evaluations	Seismic Loads for Geologic Hazard Evaluations	Notes
Intakes	• MDE: Some inelastic responses and repairs • OBE: Elastic responses with no damage	• MDE: Envelope of 975-year probabilistic and 84th-percentile deterministic ground motions • OBE: 475-year probabilistic ground motion	• Same OBE and MDE ground motions	• USACE ER 1110-2-1806 • Easier to repair • Part of Intakes would be constructed in the levee
Embankments	• MDE: No uncontrolled release of water • OBE: Some repairs, with reduced operations after EQ	• MDE: Envelope of 975-year probabilistic and 84th-percentile deterministic ground motions • OBE: 475-year probabilistic ground motion	• Same OBE and MDE ground motions	• DSOD (2018) Guidelines • Currently, DSOD uses a single deterministic design EQ • Easier to repair
Tunnels	• MDE: Some inelastic responses; no joint openings • OBE: Elastic responses with no damage	• MDE: Envelope of 2,475-year probabilistic and 84th-percentile deterministic ground motions • OBE: 475-year probabilistic ground motion	• Same OBE and MDE ground motions	• Based on criticality of facility and difficulty and high costs to repair (long down time)
Shafts - Permanent	• MDE: Some inelastic responses; no joint openings • OBE: Elastic responses with no damage	• MDE: Envelope of 2,475-year probabilistic and 84th-percentile deterministic ground motions • OBE: 475-year probabilistic ground motion	• Same OBE and MDE ground motions	• Based on criticality of facility and difficulty and high costs to repair (long down time)

Table 2. Recommended Seismic Loading and Performance Criteria

Facility	Performance Objectives	Seismic Loads for Structure Evaluations	Seismic Loads for Geologic Hazard Evaluations	Notes
Pumping Plant	• Continued operations after EQ • Life safety	• 2019 CBC (⅔MCE)	• Use MCE ground motions, without the ⅔ factor	• California Building Code, Title 24, Part 2 (Volumes 1 & 2) • Occupancy Category IV and importance factor of 1.5 • SFPUC Seismic Design Criteria (2014) • Long-lead Items for repair and/or replacement
South Delta Conveyance Facilities – Canals and Gates	• MDE: Some inelastic responses and repairs • OBE: Elastic responses with no damage	• MDE: 975-year probabilistic ground motion • OBE: 475-year probabilistic ground motion	• Same OBE and MDE ground motion	• State Water Project Seismic Loading Criteria Report (DWR, 2012a) • If tunnels are considered, use tunnel criteria
Bridges	• SEE: • Ordinary bridges: total collapse prevention • Recovery bridges: require repairs, but no replacement/ collapse • FEE: Elastic responses with minimal damage	• SEE: 975-year probabilistic ground motion • FEE: 225-year probabilistic ground motion	• SEE: 975-year probabilistic ground motion • FEE: 225-year probabilistic ground motion	• Caltrans Seismic Design Criteria (SDC, Version 2.0, 2019)

Table 2. Recommended Seismic Loading and Performance Criteria

Facility	Performance Objectives	Seismic Loads for Structure Evaluations	Seismic Loads for Geologic Hazard Evaluations	Notes
Temporary Structures – Retrieving and Launching Shafts, Construction Levees and Cofferdams, and Others	• Shafts: Some inelastic responses. No joint or panel openings • Construction levees/cofferdams: No uncontrolled release of water • Others: To be determined for each facility	• 200-year probabilistic ground motion or higher based on risk assessment	• 200-year probabilistic ground motion or higher based on risk assessment	• Temporary structures shall be evaluated for consequences of failure during construction • 200-year probabilistic ground motion has a 5% probability of exceedance in 10 years

Notes:

% = percent

CBC = California Building Code

EQ =Earthquake

FEE = Functional Evaluation Earthquake

MCE = Maximum Considered Earthquake, as defined in ASCE 7-16 and 2019 CBC.

SEE = Safety Evaluation Earthquake

Embankments: Embankments would be planned at the Southern Forebay. For the MDE, an upper-bound envelope of the 84th-percentile (median plus 1 sigma) deterministic ground motion from the controlling faults and the 975-year average return period ground motion (10 percent probability of being exceeded in 100 years) is recommended for the embankments. The deterministic 84th-percentile ground motion is the upper end of the range (67th- to 84th-percentile) recommended by the DSOD for moderate slip rate (0.1 to 1.0 millimeters per year [mm/year]) faults and extremely high-consequence dams or embankments (DSOD, 2018).

Tunnels and Permanent Shafts: Repair costs associated with tunnels would probably be greater than repair costs for other facilities, such as canals and embankments, due to the tunnels' great depths. Based on an anticipated longer repair time, a higher seismic performance standard is recommended for tunnels and shafts than for other facilities with an upper-bound envelope of 2,475-year and 84th-percentile deterministic ground motions for MDE. Generally, the deterministic 84th-percentile ground motion would control the design envelope. However, the distant major active faults may contribute to the probabilistic ground motion at the 2,475-year average return period and the longer periods of motions and control the envelope in this range.

Pumping Plant: The pumping plant would be designed using ASCE 7 or CBC criteria, with appropriate importance factors and modification factors to the MCE, as described in the codes.

South Delta Conveyance Facilities – Canals and Gates: Ground motions with average return periods of 475 years and 975 years would be recommended for the OBE and MDE, respectively, for the South Delta Conveyance facilities. Note, these ground motions would be higher than the design ground motion recommended by DWR for urban levee system, where a probabilistic 200-year average return period ground motion was used (DWR, 2011; 2012).

Bridges: Bridges would be considered critical for emergency response purposes, because heavy equipment would need to be transported during an emergency following earthquakes. Consistent with offsite highways, seismic design loads and performance requirements for bridges would follow the Caltrans seismic design criteria.

Temporary Structures: Temporary structures would be needed during construction, including launching and retrieving shafts and construction levees or cofferdams. These structures would be expected to provide protection during construction for many years (about 10 years), and they would be designed to resist the more frequent earthquakes with a minimum average return period of 200 years (or a 5 percent probability of exceedance in 10 years). Each temporary structure would be evaluated for its risk of failure, and consequences to the project costs and schedule delays. If the consequences were judged to be high, earthquake ground motion with longer return period would be considered.

Existing Delta Levees: The existing levees in the Delta would be evaluated as providing "temporary" protection to the project infrastructure, since they would only be relied upon during construction. The existing levees that meet PL84-99 are judged to have robust cross sections, based on the levee height and peat thickness, and should provide sufficient risk mitigation due to seismicity during construction. Critical sections of the levees where failure may impact the project would be evaluated based on site-specific data on a case-by-case basis.

2.3 Ground Motion Development

As indicated, design ground motions would be developed by conducting site-specific seismic hazard analyses or by using the code-based seismic design parameters. Seismic hazard analyses would be conducted using both probabilistic and deterministic methods.

The following steps would be considered when performing seismic hazard analyses for a given project site using either probabilistic or deterministic methods:

1) Characterize subsurface conditions based on the available subsurface information.
2) Identify and characterize historical seismicity, potential seismic sources, fault parameters, geometry, and locations.
3) Perform probabilistic and deterministic seismic hazard analyses (PSHA and DSHA, respectively) to generate acceleration response spectra for a reference site condition (such as the surface of a competent subsurface soil layer or bedrock at depth).
4) Incorporate directivity effects from the faults that are near the project site.
5) Perform magnitude and distant de-aggregation analysis to identify controlling earthquake magnitudes and distances.
6) Perform site-specific site response analyses to calculate response spectra at or near the ground surface, as well as whether different site conditions than the reference site, and whether soft soils or liquefiable soils (or both) are present in the subsurface.

2.3.1 Subsurface Characterization

Engineers would use the available information during the conceptual-level design phase to characterize subsurface conditions and exercise engineering judgment, as necessary, to identify soft soil layers and potentially liquefiable soils in the subsurface and foundation earth materials. For ground motion predictions, a reference site condition would be determined, corresponding to the top of a competent subsurface soil layer or bedrock encountered at a given depth below ground surface. The reference site condition would be defined by the “time-average” shear wave velocity (V_s) in the upper 100 feet (V_{s30}), calculated using V_s data at the site.

It is recommended that V_{s30} data be obtained from direct measurements of V_s at a site. In cases where measured site-specific V_s data are not available, the engineers could consider applicable nearby data or use other properties, such as standard penetration test (SPT) blow-count (N-value), cone penetration test (CPT) data, and density, to estimate the V_{s30}. Uncertainty in the V_s profile would be considered, especially when SPT N-values or CPT data (or both) were used to estimate V_s data. To the extent they are available, the relationships developed for Delta soils would be used. If the subsurface conditions vary significantly along the project alignment, the alignment could be divided into several segments and each segment assigned its own reference V_{s30}.

The reference ground motions estimated using the New Generation Attenuation (NGA) West 2 Ground Motion Models (GMMs, discussed in Section 2.3.3.2) for a specified V_{s30} value represent the outcropping soil and rock motions for an assumed V_s profile at depth used in the development of the NGA West 2 GMMs. For the final design, adjustments to the reference ground motions would be applied to account for the differences in site response of the assumed and site-specific V_s profiles.

2.3.2 Identification and Characterization of Potential Seismogenic Sources

All significant "local" seismogenic sources within a 200-kilometer (km) radius of the site and potentially controlling major distant seismogenic sources would be identified and characterized related to geometry and location, and geological conditions that affect the development of seismic source parameters for use in the estimation of the earthquake ground motions. The required parameters for seismic hazard analysis, compiled or estimated from available data for each potential seismogenic source, are:

- Style of faulting (such as strike-slip, reverse, oblique, or normal)
- Fault traces and closest source-to-site distance
- Segment and total fault length and estimated rupture scenarios and length
- Seismogenic depth
- Fault dip
- Slip rate or recurrence intervals
- Maximum or characteristic magnitude (M_{max} or M_{charac})

A seismogenic source can be represented by a planar fault or a distributed (areal) source. A fault is well, to moderately well, defined in its location and activity, and it has been identified as a potential source of seismic events in the past. The fault must be judged to be at least potentially active, and be capable of contributing to the ground motion hazards at the site. An aerial source is used to characterize background earthquakes not associated with known faults.

2.3.3 Seismic Hazard Analysis

Both PSHA and DSHA procedures would be used to develop acceleration response spectra for the reference site condition.

2.3.3.1 Probabilistic and Deterministic Seismic Hazard Analyses

PSHA would be performed using an industrially accepted computer program. The PSHA program employs the analytical procedure to compute seismic hazard originally developed by Cornell (1968). The probabilistic method allows for the explicit inclusion of ranges of possible interpretations in model components, including seismic source geometry and parameters and ground motion models. It estimates levels of ground motions at a location for different likelihoods (probabilities) of occurrence. Uncertainties in source geometry and parameters can be incorporated into the probabilistic analysis through a logic tree that reflects the quality of the available information. When sufficient data are available, both time-independent and time-dependent source models would be considered for the final design.

DSHA evaluates ground motions at a location generated by earthquakes on nearby controlling seismic sources. The ground motions are estimated using ground motion models, regardless of the likelihood of occurrence. The median (50th-percentile), median plus ½ sigma (67th-percentile) and median plus 1 sigma (84th-percentile) ground motions from the occurrences of maximum earthquakes on these controlling seismic sources would be estimated. For background seismic sources, the maximum earthquakes would be placed at a horizontal distance of 15 km from the site.

2.3.3.2 Ground Motion Models

The following five NGA West 2 GMMs for active tectonic regions and crustal earthquakes are recommended for the estimation of ground motions:

1) Chiou, B.S.J. and R.R. Youngs (2014)
2) Boore, D.M., J.P. Stewart, E. Seyhan, and G.M. Atkinson (2014)
3) Abrahamson, N.A., W.J. Silva, and R. Kamai (2014)
4) Campbell, K.W. and Y. Bozorgnia (2014)
5) Idriss, I.M. (2014)

The applicability of these GMMs to the scenarios that control the hazard at a site would be evaluated. Other GMMs would also be considered, as judged appropriate, depending on specific site conditions and fault mechanisms. The Cascadia subduction zone (CSZ) in the Pacific Northwest is not expected to contribute to the project site hazards. However, if it was determined that the CSZ contributes to the predicted ground motions, especially for longer period vibrations or longer return period ground motions (or both), the latest GMMs for subduction zones that are most applicable to the magnitude and distance ranges that contribute significantly to the hazards would be used. For DSHA, ground motions would be estimated using the geometric mean of selected GMMs, while for PSHA, weights would be assigned to the selected models as part of the ground motion characterization logic tree.

These GMMs were developed using global ground motion data that include both the uncertainties from multiple events and active regions (ergodic models). For the final design, considerations would be given to develop a non-ergodic Delta-specific GMM, including Delta-specific basin parameters.

2.3.3.3 Design Horizontal Response Spectra

The results of PSHA are expressed in terms of relationships between amplitudes of horizontal peak ground acceleration (PGA), or response spectral accelerations, and annual frequencies of exceedance. These relationships are commonly known as hazard curves. Horizontal uniform hazard spectra (UHS) (that is, response spectra having the same annual probability of exceedance for all vibratory periods) can then be developed for the reference site condition from the computed hazard curves.

The 5 percent-damped mean, 84th-percentile, and 95th-percentile UHS for return periods of 144, 475, 975, and 2,475 years would be developed. These return periods correspond to approximately 50, 20, 10, and 4 percent probabilities of exceedance in 100 years, respectively. In addition, UHS for a return period of 200 years would be developed, which represents a 5 percent probability of exceedance in 10 years.

Because UHS consists of spectral values calculated from earthquake occurrences on all seismic sources and because it has the same probability of exceedance over a given exposure time, it is conservative (especially for long linear systems, such as tunnels and levees), since these spectral values are unlikely to occur at the same time from a single earthquake event. Alternatively, for final design, conditional mean spectra (CMS) can be developed in lieu of UHS. The use of CMS could reduce the seismic demands for structures with known dominant periods.

A 5 percent-damped median, 67th-percentile, and 84th-percentile deterministic response spectra would also be developed using the GMMs recommended in Section 2.3.3.2.

The near-field source or directivity effects are most significant for sites located toward the ends of a major fault. These effects tend to increase the median ground motions, as well as their variability. In the Delta, most sites are not located near major faults. Therefore, the effects of fault directivity are expected to be minor and need not be considered. In addition, the NGA West 2 GMMs adequately capture these near-field source or directivity effects. For a few sites where near-field source or directivity effects are expected to be significant, the effects would be evaluated for the final design using either a probabilistic method or a deterministic method.

For the Delta, the site hazard is likely controlled by multiple local faults, each with different fault strike. For the probabilistic method, this would result in inconsistent fault-normal and fault-parallel ground motions (because the orientations and strikes of these faults are not the same). The applicable of fault-normal and fault-parallel ground motions for the Delta region would therefore be evaluated for the final design.

Depending on the applicable codes and guidelines, some structures need to be evaluated against the maximum-oriented ground motions (RotD100), such as those subject to CBC or ASCE-7 criteria, while others are designed using the average ground motions (RotD50). The maximum-oriented ground motions would be developed by applying scaling factors to the average ground motions.

The ground motion response spectra would be provided for a 5 percent damping value for periods up to at least 10 seconds. When applicable, response spectra for longer periods and other damping ratios would be developed for the final design. The needs to develop vertical ground motions would also be evaluated for the final design.

2.3.4 Site Response Analysis

Site response analysis is necessary if one or more of the following apply:

- The foundation subsurface is different than the reference site condition for which the UHS was developed in Section 2.3.3.
- The foundation contains liquefiable soils or soft soils that remain untreated.
- The facility is deemed critical that a facility-specific response analysis is warranted.

The GMMs used in the PSHA and DSHA (described in Section 2.3.2) are not valid for peats, soft soils, or liquefied soils (V_s < 150-180 meters per second [m/s]). In addition, these empirical models cannot fully capture the effects of local soil variation. In this case, design ground motions would be developed at the surface of a competent underlying layer using methods described here, and would be adjusted to account for the effects of local soils.

For the conceptual-level design phase, the following analysis methods are recommended for the development of the design response spectra at the tunnel depths or near the base of foundation or embankment.

1) Perform one-dimensional, free-field, dynamic site response analysis by considering:
   a) Equivalent-linear and total-stress soil models
   b) Three sets of horizontal earthquake time histories and no vertical motions,
   c) The available V_s data or those estimated from SPT N-values and CPT cone data (where direct V_s measurements are not available) (or both)
   d) Variability in soil dynamic properties and V_s profile, especially if V_s values are estimated from SPT N-values or CPT cone data

2) Compare the results of one-dimensional site response analysis with the published site amplification factors developed for Delta soils, such as those developed by Kishida et al. (2009).
3) Coordinate with independent technical reviewers.

Earthquake acceleration time histories are necessary to perform the site response analysis. The procedure described in Section 2.3.5 can be used to develop design time histories. The outputs of a site response analysis would include:

- Profiles of PGA, peak shear stress, and peak shear strain versus depth at both free-field and near the structure locations
- Design response spectra and earthquake time histories at the ground surface and at specified depths for aboveground facilities and at tunnel depth and over shaft depths for belowground facilities, tunnels, and shafts

2.3.5 Development of Acceleration Time Histories

Time history analysis is useful to evaluate the seismic performance of structures, both for the design of new structures and the evaluation of existing structures. This type of analysis requires the development of acceleration time histories, which would consider the following three requirements:

1) Three sets of two horizontal components of motion would be developed and used with both positive and negative polarity to account for the uncertainty and variability associated with ground motions. The most severe response of these time histories would be used as the basis for design and evaluation.
2) To develop these time histories, seed records from past earthquakes or from numerical simulations would be used.
3) Selected seed time histories would be scaled or spectrally matched to the design and target response spectra to obtain appropriate acceleration time histories for use in the design analysis.

2.3.5.1 Selection of Seed Time Histories

As stated, seed time histories would be selected either from past recorded earthquakes or from numerical simulations. It is preferred that recorded time histories from past earthquakes be used rather than synthetic time histories. However, in certain scenarios, recorded time histories are scarce, and the use of synthetic time histories is necessary to generate the desired total number of time histories. When selecting seed time histories, the following three criteria are to be considered:

1) The selected motions would be compatible with facility-specific parameters, such as controlling earthquake magnitude and site-to-source distance (as determined from the hazard de-aggregation), rupture mechanism and site conditions. Particular attention would be given to spectral shape, which is the most important factor for selecting seed time histories.
2) The Arias Intensity and strong duration of the scaled or spectrally matched motions would be consistent with the estimated ground shaking at the site. The empirical equations for predicting Arias Intensity consistent with NGA West 2 GMMs proposed by Abrahamson et al. (2016) would be used to determine the target and design Arias Intensity value.
3) Sets of time history would be selected to contain directivity or near-field pulse characteristics. The number of time histories can be determined using the model of Hayden et al. (2014). The selection criteria for the velocity pulse would be based on the Instantaneous Power (IP) parameter proposed by Zengin and Abrahamson (2020).

Note, recorded ground motions from past earthquakes are presented on the Pacific Earthquake Engineering Research (PEER) Center strong motion database website (http://peer.berkeley.edu/smcat/).

2.3.5.2 Scaling and Matching Seed Time Histories

After selecting the seed time histories, if the motion was not sampled at a time step of 0.005 second, resampling is necessary. Resampling involves calculating the fast Fourier transform (FFT), and then the Inverse FFT with interpolation at the desired sampling rate.

If the seed time histories are to be spectrally matched, the time domain spectral matching procedure as coded in RSPMATCH (Abrahamson, 2003 or later version) is recommended for use. The quality of the results would be assessed by measuring the tolerance to which the matched motions converge towards the design and target spectrum in the period range of interest, and how well the matched motions compare to the original motions in the time domain. In particular, the matched displacement and velocity time histories need to be reasonable and reflect some of the predominant characteristics of the original motions. The following two matching and scaling criteria would be satisfied:

1) The standard deviation of the misfit for horizontal spectrum for a given set would be less than 0.2 natural log unit at any period over the period range of interest.
2) The standard deviation of the misfit for horizontal spectrum for the entire sets would be less than 0.1 natural log unit over the period range of interest.

In addition, the spectrally matched time histories would satisfy the Arias Intensity target for the design-level ground motion (within 16th and 84th-percentile target values, as determined in Section 2.3.5.1). The spectrally matched motions would be baseline corrected before their use in analyses, including site response analysis.

3. Fault Rupture

This section provides guidelines for assessing a permanent ground displacement (PGD) hazard due to fault rupture from a major seismic event at the proposed Project facilities. PGDs due to fault rupture (or fault-related folding) could be a significant hazard for critical infrastructure that intersect or are adjacent to active faults. Several key parameters would be considered as part of the engineering evaluation and design for PGDs due to fault rupture (DWR, 2012b and ANSI/ANS-2.30-2015) that include:

- Fault location and uncertainty
- Estimate of expected co-seismic surface displacement and uncertainty
- Style of faulting (such as direction of displacement; horizontal and vertical components of slip)
- Distribution of fault displacement (such as folding, knife-edge dislocation or distributed shear across a zone)

The approach for evaluating fault rupture hazard for project features would consist of the following steps:

1) Review, compile, and analyze existing geological, geotechnical, and seismological data to understand the site's geological and tectonic setting.
2) Assess site conditions at the fault crossing by supplementing the data compilation described here with a literature review, field mapping, and subsurface data collection, as necessary.

3) If a facility crosses or is adjacent to an active fault, estimate the location, style, rate, and amount of potential PGD by conducting a site-specific fault displacement hazard analysis (FDHA) that can be used to evaluate the performance of critical facilities.
4) Estimate the amount of PGD using both deterministic and probabilistic methods (outlined here).
5) Prepare a FDHA report that documents the methods, assumptions, results, and uncertainties considered in the fault displacement hazard characterization.

The following subsections provide guidelines for assessing PGDs related to fault rupture for Project facilities and features.

3.1 Design Fault Displacement and Performance Criteria

Similar to ground motion hazard (such as PSHA), the design fault displacements and performance criteria selected for a facility would be based on a number of different criteria, some of which are outlined in guidelines from several agencies: SFPUC (Cheng and Sadden, 2018), DSOD (2018), Los Angeles Department of Water and Power (Davis, 2019), California High-Speed Train Project (CAHSRA, 2015) and ANSI/ANS-2.30-2015), as summarized:

- Consequences of failure
- Fault activity and slip rate
- Criticality of the structure for water delivery
- Downtime and cost to repair the facility

Unlike the approach described for strong ground shaking, the recommended seismic loadings for fault rupture consider only a single design-level earthquake, defined as:

- **MDE** – Facility-specific and represents a rare event that has a low probability of occurrence during the life-time of the facility. Depending on the criticality of structure and the impact of fault rupture, the MDE could be based on either the probabilistic or deterministic fault displacement estimates at various return periods or exceedance probabilities (such as 2,475-year return period).

Table 3 lists the design fault displacements and performance criteria recommended for the facilities located near the Clifton Court Forebay area and along the tunnel alignment.

Table 3. Recommended Fault Displacements and Performance Criteria for Structures near Clifton Court Forebay and Along Tunnel Alignments

Facility	Performance Objectives	Fault Displacement	Notes
Embankments	• MDE: No uncontrolled release of water	• MDE: Greater of 2,475-year probabilistic and 84th-percentile deterministic displacements	• Consequence Hazard Matrix of DSOD (2018) • SFPUC Performance Class Designation (2014)
South Delta Conveyance Facilities – Canals and Gates	• MDE: Some inelastic responses and repairs	• MDE: Greater of 2,475-year probabilistic and 50th-percentile deterministic displacements	• Consequence Hazard Matrix of DSOD (2018) • SFPUC Performance Class Designation (2014)

Table 3. Recommended Fault Displacements and Performance Criteria for Structures near Clifton Court Forebay and Along Tunnel Alignments

Facility	Performance Objectives	Fault Displacement	Notes
Tunnel and Permanent Shafts	• MDE: Some inelastic responses and repairs but tunnel should not be compromised due to fault rupture	• MDE: 2,475-year probabilistic fault displacement	• LA Metro and HSR guidelines

Note:

MDE is the maximum level of ground displacement (or folding) for which a structure is designed or evaluated. Depending on the structure and the impact of fault rupture on the proposed structure, the MDE could be based on either the probabilistic or deterministic fault displacement estimates at various return periods or exceedance probabilities.

The recommended return periods and deterministic percentiles for fault displacement were determined based on the criteria listed earlier, as well as other agency guidelines for similar structures or facilities, and they can be modified following the DCA's classification and quantification of these criteria. Depending on the fault characteristics (style of deformation, direction of slip, width of deformation, and the like) and the amount of the permanent ground deformation, coupled with an understanding of the cost-benefit ratio of a particular mitigation design compared to the criticality of the structure, the DCA may consider an enhanced performance level design for the proposed structure. Higher fractile values (such as 95 percentile value) may also be considered for the probabilistic estimates due to larger uncertainties associated with fault rupture predictive models.

A hazard consequence matrix, similar to the "Fault Displacement Consequence Hazard Matrix" developed by the DSOD (Fraser and Howard, 2002 and DSOD, 2018), could be considered to determine the appropriate return periods and deterministic percentiles used for the MDE fault displacement hazard and level of design. Another document, prepared by the American Nuclear Society, and entitled ANSI/ANS-2.30-2015, could be consulted with respect to FDHA methodology and risk-based approach.

3.2 Estimating Surface or Near-surface Fault Displacement

The following guidelines would be considered when performing fault displacement hazard analyses for a given site using both the deterministic and probabilistic methods.

3.2.1 Deterministic Fault Displacement Hazard Analysis

In traditional deterministic fault displacement hazard analysis (DFDHA), a single deterministic earthquake is defined, and a level of hazard is calculated, based on that scenario event (such as a 50th, 84th or 90th percentile displacement). The DFDHA may also:

- Consider a range of deterministic earthquake magnitudes based on peer-reviewed literature, historical seismicity, and empirical relationship (such as Wells and Coppersmith, 1994; Petersen et al., 2011; Leonard, 2010; and Hecker et al., 2013).
- Estimate principal fault displacement amplitudes using empirical displacement prediction equations (such as, Wells and Coppersmith, 1994; Petersen et al., 2011; Leonard, 2010; and Hecker et al., 2013).

Secondary (or off-fault) displacements are typically assumed to be some percentage of the principal displacement (such as, less than 25 percent [DSOD, 2018]).

The traditional or simplified method of choosing a single deterministic earthquake is transparent and easy to implement (DSOD, 2002). However, its usefulness is limited to seismotectonic environments where a single deterministic earthquake is easily defined. In more complex environments (such as for an environment such as the San Andreas system in Northern California with connected faults), the traditional approach limits the uncertainty that can be included in the characterization and relies heavily on a geologist's judgement to characterize the "correct" deterministic earthquake. This is also true for poorly characterized faults, such as the West Tracy Fault, where there is little information on the fault's linkage at depth with adjacent faults, as well as regional faults located directly north and south of the fault, such as the South Midland fault.

More recently, several state and local agencies that operate critical infrastructure have begun to use a more sophisticated approach to deterministic fault displacement hazard analyses that explicitly consider epistemic uncertainty in the deterministic earthquake and epistemic uncertainty and aleatory variability in the available empirical fault displacement prediction equations. A logic-tree-based approach developed by Thompson et al. (2018) to capture the uncertainty in the deterministic earthquake would be considered. This approach recognizes that geologists and seismologists are considering other methods and not solely the concept of strict fault rupture segmentation (that is, characteristic behavior and a corresponding single deterministic earthquake). A logic-tree-based DFDHA approach can capture epistemic (or model) uncertainty in the deterministic earthquake and displacement estimation at a given site. The use of a logic-tree framework, in which alternative possibly correct parameters are proposed and assigned weights, can improve the documentation of the geological hazard assessment, and the results of analyses can incorporate a broader range of uncertainty that more accurately reflect the current state of knowledge about fault displacement hazard.

The implementation of a DFDHA would consider the full range of uncertainty in estimating deterministic earthquakes, consider a range of fault displacement prediction equations (such as Wells and Coppersmith, 1994; Petersen et al., 2011; Leonard, 2010; and Hecker et al., 2013), and consider other uncertainties, such as tapered displacement models (Biasi and Weldon, 2009). A DFDHA would describe and provide justification for the DFDHA approach, deterministic earthquake characterization (such as DRMS, 2008), empirical relationships used, and methods of incorporating uncertainty in the analysis.

Note, the current fault displacement empirical prediction equations are ergodic with simple parameterization of fault behavior. As Hecker et al. (2013) showed, the slip at a particular site has much less aleatory variability (non-ergodic variability) when slip data from past earthquakes are used. For final design, the aleatory variability of these models would be re-evaluated if and when the site-specific slip data became available.

3.2.2 Probabilistic Fault Displacement Hazard Analysis

The procedure for probabilistic fault displacement hazard analysis (PFDHA) is similar to that for the ground-shaking PSHA discussed in Section 2, with the following major differences:

- Fault displacement analysis focuses on only nearby faults (less than 2-km radius, including off-fault and secondary displacement) to where displacements are to be estimated.

- Instead of estimating ground-shaking intensity at a site, PFDHA quantifies the magnitude of displacements for a given fault source or sources (such as where multiple seismogenic faults lie within approximately 2 km of the site).

The implementation of the PFDHA would follow the methodologies discussed in Youngs et al. (2003), Petersen et al. (2011), Moss and Ross (2011), or other publications that are based on the more common PSHA of Cornell (1968). There are two fundamental approaches to performing a PFDHA to estimate fault displacement hazard and its uncertainty: (1) the earthquake (or magnitude) approach, and (2) the displacement (or direct) approach of Youngs et al. (2003). The magnitude approach is more commonly used in PFDHA because this type of fault-specific information required for the displacement approach is often rarely available. For many earthquake approach PFDHAs, the seismic source characterization is similar or identical to the fault source characterization used in the PSHAs described here. The displacement approach is used when more detailed information on paleoseismic fault behavior (recurrence and displacement-per-event) is available and estimates of fault displacement hazard can be based on fault-specific information.

The primary result from a PFHDA is a suite of hazard curves (mean and fractile curves) that depict the annual exceedance frequency (or return period) for various displacement amplitudes, similar to the way PSHA provides hazard curves of annual exceedance frequency for ground motion amplitudes. For instance, instead of estimating the annual rate of exceedance of a specified earthquake ground motion at a site, PFDHA estimates the annual rate of earthquake-induced displacement exceeding a specified level, at a site of given dimension. The time-independent rate of exceedance is computed. The initial fault parameters are exactly comparable to fault source characterization that is required for a PSHA, whereby the location and rate of earthquakes are defined for each seismic source. For the Project, a fault displacement characterization would consider the latest DRMS seismic source model(s), updates in the regional seismic hazard model, and new research on fault activity and rate in the Delta region.

Additional terms that are specific to a PFDHA include:

- A conditional probability of non-zero surface rupture (Wells and Coppersmith, 1993; Moss and Ross, 2011; Moss et al., 2019)
- A conditional probability of primary or secondary deformation at the site based on the distance from the principal fault to the facility being evaluated
- A probability of displacement exceedance (such as Petersen et al., 2011)

The PFDHA would consider distributed secondary or discrete off-fault deformation for sites that are not intersected by faults but are within an area of potential secondary or distributed deformation hazard (Petersen et al, 2011). This may be the case for multiple structures built close to or within regions of poorly known seismic sources near or within the Delta.

Hazard is calculated using a single coordinate that represents the approximate center point of a cell (or cells) with given dimensions related to the footprint of the structure. A cell footprint that represents a larger area than the site footprint is justifiable as the historical earthquake rupture datum used to derive the distributed displacement relationships likely underestimate of the true extent of distributed deformation (Youngs et al., 2003).

For low slip rate faults, such as the West Tracy Fault, the range of calculated displacements could be large. The use of mean displacement for design, may, therefore, not be appropriate. Considerations would be given for adopting a higher hazard fractile, such as 90th percentile displacement, for design.

In summary, the fault displacement calculations generated by a PFDHA should provide the DCA and reviewers with the assurance that the fault displacement hazard has been appropriately considered and incorporated into the overall hazard, risk, and reliability assessment of the project.

3.3 Fault Displacement Hazard Analysis

For the conceptual-level design phase, the FDHA should include a broad range of uncertainty so that:

- The full potential impact of fault rupture is understood by the engineering design team.
- Further evaluation and data collection would reduce uncertainty.

The FDHA at the conceptual-level design phase would focus on identifying potential significant engineering challenges posed by PGD due to fault rupture related to the criticality of the structure. The following approach is recommended:

1) Perform a FDHA through the following tasks:
   a) Compile, review, and analyze all available geological and fault data to constrain slip rate, activity, fault location, style and width of faulting, angle of fault intersection and depth, as well as primary and secondary faulting, with a focus on developing broad limits on FDHA parameter uncertainty.
   b) Interview experts to help characterize the range of potential FDHA parameters. Those to be interviewed include experts at universities, the government, and private institutions who are familiar with the geological and tectonic setting of the area or FDHA, or both.
   c) Conduct both PFDHA and DFDHA with a broad range of uncertainty, and use multiple empirical regressions and approaches for estimating fault displacement (as described here).
   d) Prepare FDHA report that presents a detailed discussion of the input, analyses, and results along with a sensitivity analysis that describes the hazard significant parameters and sources of uncertainty.
2) Develop recommendations for further information needs and data collection that are required to reduce uncertainty for hazard significant parameters (such as location, displacement direction and intersection, and style and width of faulting) (the 60% design phase recommendations list possible data collection techniques).
3) Coordinate with engineering design team, as well as independent technical reviewers, to identify whether additional studies are necessary due to the potential impact of the fault-related PGDs, and in consideration of the criticality of the structure.

Additional data collection could be shifted to future design phases if a reduction in uncertainty was required but would not significantly change facility design.

4. Liquefaction-induced Geohazard Guidelines

This section discusses the guidelines for evaluating the liquefaction triggering potential and its related consequences (geological and geotechnical hazards). Structures would be designed to resist the inertia forces and deformations (kinematic forces) resulting from the occurrence of liquefaction and to satisfy the performance criteria presented in Table 2 of this TM.

4.1 Liquefaction Triggering and Cyclic Softening

Liquefaction-triggering potential would be evaluated for sites meeting the one or both of the following two screening criteria:

1) The site has been observed to liquefy in past earthquakes.
2) Soils are saturated and classified as sandy soils, gravelly soils, or silty and clayey soils meeting the screening criteria proposed by Bray and Sancio (2006) or Idriss and Boulanger (2008), or both. The Modified Chinese Criteria for clayey soils proposed by Youd et al. (2001) should not be used.

For sites meeting these screening criteria, liquefaction triggering evaluations need not be performed when the measured SPT resistance, corrected for overburden pressure, fines contents and hammer energy, $(N_1)_{60\text{-}cs}$, is more than 33 blows per foot, or when a CPT cone tip resistance, normalized for overburden pressure and fines contents, $q_{c1N\text{-}cs}$, is more than 185 tons per square feet (tsf).

Potential strength gains due to age of soil deposits would be considered for relatively deeper soil strata. The relationship for Strength Gain Factor in Cyclic Resistance Ratio (CRR) versus age proposed by Hayati and Andrus (2009) would be used in conjunction with the liquefaction triggering curves recommended by Youd et al. (2001) for estimating the increase in $(N_1)_{60\text{-}cs}$ and $q_{c1N\text{-}cs}$ values due to the age of the deposits. Care should be taken because some of the strength gains may already be reflected in the field recorded SPT blow-counts or CPT cone resistance.

4.1.1 Simplified Method

For soils meeting the two criteria, the following simplified methods for liquefaction triggering potential evaluations can be used:

- Youd et al. (2001)
- Seed et al. (2003)
- Boulanger and Idriss (2014)

The method using the CPT data would be the primary method of evaluation. Where sufficient information on the SPT resistance (N-values), such as hammer type, energy measurement, sampler type, are available and the quality of these information and the measured blow-counts can be verified, the SPT method of evaluations can also be used. The method using shear-wave velocity should not be used.

Liquefaction triggering evaluations would be performed to a depth of 75 feet below the final grade, and the factors of safety (FOS) against liquefaction under the design earthquakes would be calculated as a function of depth. If the FOS values calculated using the three simplified methods vary by no more than 20 percent, the average of the 3 results could be used. Otherwise, the lowest FOS values would be reported.

For gravelly soils, special field investigation techniques, such as the Becker Hammer Penetration Test (BPT), Large Sampler Penetration Test (LPT), and small-interval SPT, should be considered to obtain more representative sampler penetration blow-counts. The penetration blow-counts obtained using these techniques would need to be converted to the equivalent SPT N-values for use in the simplified methods. Refer to Harder (1997) and Sy (1997) for BPT conversion and Daniel et at. (2003) for LPT conversion to SPT N-values.

4.1.2 Numerical Modeling Method

The empirical data used to develop the simplified methods for liquefaction triggering potential evaluations are limited to a depth of about 75 to 100 feet. If liquefaction potential evaluation at greater depths was required, such as for tunnels and tunnel shafts, a dynamic site response analysis (as described in Section 2.3.4) could be conducted to obtain the CRR versus depth. For the conceptual-level design, the CRR developed for potentially liquefiable soils at shallower depths (less than 75 to 100 feet) could be used for similar soils at greater depths (less than 75 to 100 feet). The strength gains in CRR due to age of soil deposits would be considered, as discussed in Section 4.1.

4.1.3 Softening of Clay and Plastic Silts

Although clays and plastic silts would not "liquefy" under earthquake cyclic loadings, the excess porewater pressure generated during the loadings could progressively go up to a limiting value, so further loadings beyond this value would cause excessive ground deformations. This phenomenon is referred to as the "cyclic softening" of clayey soils (Idriss and Boulanger, 2008). The amount of the undrained shear strength softening or reduction due to cyclic loadings would be assessed and determined in accordance with Idriss and Boulanger (2008).

4.2 Liquefaction-induced Hazard Assessments

When the calculated FOS against liquefaction is less than 1.10, the engineering consequences due to liquefaction would be investigated. These consequences may include partial or total loss of soil's shear strength, causing foundation instability, embankment failure, lateral spreading, and excessive ground deformations. Liquefaction could also cause buried structures to float due to buoyancy. Many factors could influence the severity of these consequences, including site topography, subsurface soil heterogeneity, horizontal and vertical extents of potentially liquefiable soils, and effects of foundations and embedded structures. For critical structures, evaluations of these consequences could be warranted even when the FOS against liquefaction was greater than 1.10 (but less than 2.0).

This section outlines the guidelines for assessing these liquefaction-induced geological or geotechnical hazards for conceptual-level design.

4.2.1 Residual Strength

When soil liquefies, the soil loses most of its strength, which could lead to instability. The shear strength of soil at liquefied state is called residual strength. The residual strength would be estimated using the methods proposed by Idriss and Boulanger (2008) based on pre-earthquake SPT N-values and CPT cone data, and considering void redistribution effects, when applicable. The void redistribution could cause localized very low shear strength zone within the liquefied soil layer.

The estimated residual strength would be verified using the following procedures:

- Seed and Harder (1990)
- Olson and Stark (2002)
- Kramer and Wang (2015)

The design liquefied residual shear strengths would be selected by weighting the values predicted by all or some of these methods.

4.2.2 Dynamic Settlement or Compaction

Both free-field dry, unsaturated and saturated seismic-induced settlements would be estimated. The settlement of dry sands can be determined using the procedures proposed by Tokimatsu and Seed (1987). The settlement due to reconsolidation of saturated liquefied sandy soils would be estimated using the weighted average of the following procedures:

- Zhang et al. (2002)
- Ishihara and Yoshimine (1992)
- Idriss and Boulanger (2008)
- Cetin et al. (2009)

The potential of having damage or manifestation at ground surface would be investigated using the method outlined by Ishihara (1985) and Ishihara et al. (2016). This method considers the thickness of liquefied soil layer and that of the overlying non-liquefied layer. As expected, a thicker overlying or protecting non-liquefied soil layer would minimize damage at the ground surface.

Note, the settlement estimated using these procedures is only one-dimensional vertical settlement. It does not include vertical settlements due to lateral spreading (discussed here). The impacts of foundations and buried structures to the estimated settlements would be evaluated, when applicable.

4.2.3 Lateral Spreading

Lateral spreading refers to a sudden movement of soil mass on gently sloping ground during and after the earthquake. Large soil movement can occur when the underlying soil liquefies or there is a nearby open face (free face), or both.

Two analytical approaches would be used for estimating the lateral spreading displacement: (1) the Lateral Displacement Index (LDI), and (2) empirical methods. For the LDI, the methods suggested by Zhang et al. (2004) and Idriss and Boulanger (2008) could be used. For the empirical methods, relationships developed by Youd et al. (2002), Bardet et al. (2002), Faris et al. (2006), and Rauch and Martin (2000) would be considered. Large variation (hence, large uncertainties) in predicted displacement could be expected when using these empirical relationships. Therefore, the design value would be selected by considering the limitations and applicability of each model to the condition being evaluated.

The spatial distribution of predicted displacements and zones of liquefied soil would also be plotted and evaluated, as well as the effects of buried foundations and structures. Displacement patterns can vary significantly over the area of interest, and the impacts to foundations, structures, and embankments would need to be assessed. Lateral spreading displacement is expected to reduce to 50 percent at a length/height (L/H) ratio of 5 to 20 and to less than 20 percent at a L/H ratio of more than 20, where L is the distance from open face and H is the bottom depth of liquefied layer (Idriss and Boulanger, 2008).

4.2.4 Slope Deformations and Flow Failure

Slope stability and deformations under the design earthquakes would be evaluated for embankments and canals. The stability FOS would be determined using the limit equilibrium pseudo-static analysis method by evaluating both the circular and noncircular sliding surfaces. If the calculated FOS was less than 1.10, large deformations or movements (flow failure) could be expected, and the available empirical models to predict slope deformations could not be used. For slopes with FOS greater than 1.10, slope deformations would be determined using the procedures of Makdisi and Seed (1978) and Bray and

Travasarou (2007). Uncertainty associated with these predictions would be considered in determining the design value.

The onset or timing of liquefaction during shaking is critical to evaluate slope stability and deformations. If liquefaction occurred early during shaking, both seismic inertia force and liquefaction would be considered in the stability evaluations. In this case, the residual strengths for liquefied soils and seismic horizontal (and vertical, when applicable) coefficient, k_h, would be applied, leading to a lower FOS and larger deformations. If liquefaction could be shown to occur near the end of shaking (after the strong shaking), the stability and deformations could be determined in two stages: (1) during-earthquake and (2) post-earthquake. For the during-earthquake stage, the soil's undrained shear strengths could be used (such as no liquefaction) in combination with seismic loading. For the post-earthquake stage, the soil's residual strengths could be used without seismic loading.

4.2.5 Buoyancy and Increased Soil Lateral Pressure

Liquefaction could increase lateral earth pressures on walls and buried structures. As soils liquefied, the earth's lateral pressure would approach that of a fluid-like material. Liquefaction could also cause buried pipes and tunnels and structures to become buoyant. The potential for increased earth lateral pressure and buoyancy due to liquefaction would be determined using site-specific data at the locations of walls and buried structures.

4.3 Seismic Hazard Zones Requiring Investigation

The southern portion of the Central Corridor would be located within the Bouldin Island and Woodward Island Seismic Hazard Zones by California Geological Survey (CGS, 2018). These zones are referred to as Earthquake Zones of Required Investigation. The general approach and recommended methods of the required investigations are presented in the CGS *Special Publication 117A – Guidelines for Evaluating and Mitigating Seismic Hazards in California* (2008), including requirements for site investigation study and landslide and liquefaction hazard evaluation procedures.

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6. Document History and Quality Assurance

Reviewers listed have completed an internal quality review check and approval process for deliverable documents that is consistent with procedures and directives identified by the Engineering Design Manager (EDM) and the DCA.

Approval Names and Roles			
Prepared by	**Internal Quality Control review by**	**Consistency review by**	**Approved for submission by**
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This interim document is considered preliminary and was prepared under the responsible charge of Dario Rosidi, California Professional Engineering License GE2392.

Note to Reader

This is an early foundational technical document. Contents therefore reflect the timeframe associated with submission of the initial and final drafts. Only minor editorial and document date revisions have been made to the current Conformed Final Draft for Administrative Draft Engineering Project Report version.