COMMENTS ON CONCEPTUAL ENGINEERING AND CEQA FOR THE WATERFIX PROJECT

Abstract

These comments analyze the conceptual engineering of the WaterFix tunnels, and whether the analysis is sufficient for CEQA purposes. The conclusion is that the analysis is inadequate and incomplete for CEQA analysis of risks to water supply, related risks to surface property and life, and related CEQA required monitoring, mitigation, and reporting of significant impacts.

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Introduction

The construction of two forty foot diameter tunnels in soft soils consisting of sedimentary layers of sand and peat is a significant engineering challenge. Given the large diameter of the tunnels, the amount of water they will be carrying, and the sedimentary deposits surrounding the tunnels, significant preliminary engineering is required to document that the proposed conceptual design will have sufficient structural integrity to protect the project, itself, the water supply, and structures and people on the surface.

The Final EIR/EIS gives a list of seismic design standards that will apply to the design of the WaterFix facilities in section 9.2.2.6, Regulatory Design Codes and Standards for Project Structures. Section 9.2.2.6 states that risks to people and property will be minimized by conformance with these standards. The strongest standards are the American Society of Civil Engineers 07-10 standards, Minimum Design Loads for Buildings and Other Structures. [1] The Final Draft Conceptual Engineering Report (Final Draft CER) [2] produced by the Department of Water Resources fails to use the ASCE Maximum Considered Earthquake to analyze the proposed design of the tunnels, and states that the final seismic design criteria have not yet been adopted. There is also no discussion of leakage and movement criteria for the tunnels under long term operations. Criteria for acceptable surface settlement during tunnel construction also appear not to have been adopted. Without these clear and specific design criteria, the tunnels are not sufficiently specified for adequate analysis of the impacts of the project construction and operation under CEQA.

According to the Final Draft Design and Construction Enterprise Agreement (DCE Agreement)[3], the option of a single-pass liner design has been chosen in part because of considerations of cost and construction timing (p. 9.) However, DWR's preliminary analysis showed that a single-pass liner could leak in an earthquake, and DWR's engineers recommended that the option of a second steel liner be retained until the feasibility of the single-pass liner was demonstrated. The option appears not to have been retained. The potential problems with a single-pass liner are obfuscated in the analysis in the Final Draft CER, which uses significantly weaker ground-motion assumptions than the preliminary engineering or the ASCE standards. This gap in the analysis is of particular concern where the tunnels pass under important structures, including Delta island levees and channels, the Stockton Deep Water Shipping

[1] American Society of Civil Engineers, Standard 7-10, "Minimum Design Loads for Buildings and Other Structures," 2010. Available at http://www.avant-garde-engineering.com/ASCE_7.pdf Accessed on January 16, 2017. Incorporated by reference.

[2] California Department of Water Resources, Final Draft Conceptual Engineering Report for the Modified Pipeline/Tunnel Option (MPTO), July 1, 2015. Available at http://www.waterboards.ca.gov/waterrights/water_issues/programs/bay_delta/california_waterfix/exhibits/docs/petitioners_exhibit/dwr/dwr_212.pdf. Accessed on January 16, 2017. Incorporated by reference.

[3] California Department of Water Resources, Final Draft Agreement Regarding Construction of Conveyance Project between the Department Of Water Resources and the Conveyance Project Coordination Agency, 2015. Available at http://cms.capitoltechsolutions.com/ClientData/CaliforniaWaterFix/uploads/Draft_Final_DCE_Agreement_Combined.pdf. Accessed on January 16, 2017. Incorporated by reference.

Channel, State Route 4, State Route 12, the Mokelumne aqueduct, and natural gas and other product and services pipelines. Failure under a levee or channel could result in catastrophic flooding, endangering human life. Rapid failure under State Route 4 or 12 could also cause loss of life.

Assessments, monitoring, and mitigation under CEQA cannot be adequately addressed until adequate preliminary analyses of the probability of seismic-induced tunnel lining and ground failures, of settlement during tunnel boring, and of tunnel leakage are completed as summarized below.

1. Seismic design criteria
    - To adequately assess the risk to life and property, the maximum considered ground motions in the seismic analyses of the tunnels and the intake/outfall facilities in the CER must use the Maximum Considered Earthquake as defined in the American Society of Civil Engineers 7-10 standards. The results of these failure analyses must be disclosed in the Final EIR/EIS.
    - The CER needs to discuss the Occupancy Category of the tunnels and intake facilities, as defined by the Ameriacan Society of Civil Engineers 7-10 standards, and acceptable damage in a Maximum Considered Earthquake. The acceptable damage levels need to consider both the possibility of catastrophic failure and loss of life.
    - The CER and the Final EIR/ EIS needs to disclose what the potential repair times for the tunnels would be under acceptable damage criteria, as well as impacts on surface structures. The Final EIR/EIS needs to discuss mitigation for potential damage to surface structures.
2. Soil conditions
    - The Conceptual Engineering Report needs to classify the soils found in the preliminary borings, and do further analysis for class F soils. Inhomogeneity of the soils also needs to be considered.
    - The assumptions in the Final Draft CER that ground motions fall by 50% at 100% depth are inconsistent with data from similar sites in the CSMIP Strong-Motion Geotechnical Array, which shows that peak ground acceleration falls by at most 70%. This assumption must be corrected for an adequate preliminary analysis.
    - Preliminary seismic analysis (2010) showed that substantial liquefaction should be expected in soft ground below, at, and above the tunnels.
    - Assumptions about ground motion in the liquefaction analyses in the Final Draft CER have been weakened to 10% in 50 years. The Final Draft CER states that under those assumptions, liquefaction does not need to be considered in the tunnel design. The CER needs to consider the risk of liquefaction under the ASCE Maximum Considered Earthquake and the Final EIR/EIS needs to disclose the risk.
3. Seismic structural analysis
    - Preliminary seismic structural analysis (2010) showed that de-stressing of the tunnel joints could occur in an earthquake, resulting in leakage. The Final

EIR/EIS and the CER must disclose the results of the analysis and how the issues are to be resolved.

- To show that the tunnels would not have catastrophic failure in a Maximum Considered Earthquake, the CER must have a seismic structural analysis for a Maximum Considered Earthquake, as defined by the American Society of Civil Engineers 7-10 standards.

4. Leakage analysis and monitoring
    a. The Final Draft CER does not include any preliminary estimates of leakage of the tunnel liner design, and leakage analysis and potential mitigation is deferred
    b. Initial analysis showed that the tunnel joints could leak when the tunnels were initially filled.
    c. The Final EIR/EIS does not specify any acceptable leakage criteria to be used in the design or construction, and does not discuss monitoring or remediation during tunnel operations.
5. Construction criteria and monitoring
    - The Final Draft CER does not include any preliminary estimates of settlement during tunnel boring or any detailed discussion of procedures to evaluate and control settlement.
    - Settlement criteria, and monitoring and mitigation procedures need to be specified for the CEQA process to have adequate consideration of construction impacts.
6. No discussion in the Final Draft CER covers inspection and monitoring programs to detect ground movements (up/down), tunnel lining leaks, or potential actions or requirements to repair leaks or movements.

This summary listing clearly demonstrates the inadequacy and incompleteness of the conceptual risk analyses for tunnel construction, and for interruption of tunnel operations and integrity for the proposed design. Without these analyses, CEQA considerations of impacts of the proposed project cannot be adequately assessed. Given that one of the stated goals of the project is to protect water supply in the event of an earthquake, this is a major omission. The risks to local infrastructure, as well as local lives and property from tunnel failure, also needs to be explicitly and adequately considered

Without an adequate preliminary seismic analysis, these risks cannot be adequately assessed, and the CEQA analysis is inadequate.

1. Specification Seismic Design Criteria for the WaterFix tunnels and intakes

a) ASCE Standards for Seismic Analysis

The Final EIR/EIS, Section 9.2.2.6, Regulatory Design Codes and Standards for Project Structures, lists the American Society of Civil Engineers' Standard 7-10, Minimum Design Loads for Buildings and Other Structures.[4]According to section 21-2.2 (p. 236)

[4] American Society of Civil Engineers' Standard 7-10, Minimum Design Loads for Buildings and Other Structures, 2010. Available at http://www.avant-garde-engineering.com/ASCE_7.pdf. Accessed on January 29, 2017.

the Maximum Considered Earthquake for structures with occupancy category II or III should be consistent with ground motions having a 2% probability of exceedance in 50 years. This generally corresponds to motions having a return period of about 2,475 years. The Delta Risk Management Strategy report[5] does provide probabilistic seismic hazard curves and ground motion values for a 2% in 50 years event, which is consistent with ASCE standard 7-10. (Table 6-5 p. 61.)

b) ASCE Occupancy Category of Conveyance Structures

The occupancy category of the tunnels and the intakes has not been assigned. Standard 7-10 of the American Society of Civil Engineers[6], defines Occupancy Categories for structures. The standard states in section 1.5.1:

a. Each building or other structure shall be assigned to the highest applicable occupancy category or categories. (p. 2.)

According to the Table 1-1 in ASCE 7-10, Occupancy Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads, the Occupancy Category III includes the following:

b. Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to:
   i. Power generating stations
   ii. Water treatment facilities
   iii. Sewage treatment facilities
   iv. Telecommunication centers

   (Table 1-1, p. 32.)

Designation of the State Water Project as a critical structure would tend to put the WaterFix tunnels and intakes in occupancy category III. The tunnels must not be assumed to be redundant in the event of a large earthquake. Seismic conditions which impact one tunnel are likely to impact the other tunnel and related project structures.

The potential risk to important structures in the Delta from tunnel failure, including Delta island levees, State Route 4, State Route 12, the Mokelumne aqueduct would also put the tunnels in category III. Any residential structures that could be affected by a tunnel failure would put the tunnels in occupancy category II.

Incorporated by reference.

[5] Delta Risk Management Strategy, Final Report, Section 6, Seismic Risk Analysis, 2009. Available at http://www.water.ca.gov/floodsafe/fessro/levees/drms/docs/Risk_Report_Section_6_Final.pdf. Accessed on January 16, 2017. Incorporated by reference.

[6] American Society of Civil Engineers, Standard 7-05, "Minimum Design Loads for Buildings And Other Structures," 2006. Available at http://www.dres.ir/sazeh/DocLib9/ASCE%207-05%20Minimum%20Design%20Loads%20for%20buildings%20and%20other%20Struc.pdf Accessed on January 16, 2017. Incorporated by reference.

c.) Ground motions used in the CER and Final EIR/EIS

The ground motions used in the Final Draft CER are significantly weaker than the ASCE Maximum Considered Earthquake. According to the Final CER, section ES.3.2, Seismic Considerations, "[t]he design level of ground motion for the Intake Facilities has a 10 percent chance of being exceeded in 50 years, while the design level for the tunnels has a 5 percent chance of being exceeded in 50 years." (p. 31.) These generally correspond to motions having a return period of about 500 years and 1,000 years.

The tunnel design criteria of 5% in 50 years is the same as the "Design Basis Earthquake" of the California High Speed Train Project. The California High Speed Train Project distinguishes the two requirements in section 2.2 of Technical Memo 2.10.4, Interim Seismic Design Criteria[7]:

> No Collapse Performance Level (NCL):
>
> HST facilities are able to undergo the effects of the Maximum Considered Earthquake (MCE) with no collapse. Significant damage may occur that requires extensive repair or complete replacement, yet passengers and personnel are able to evacuate safely.
>
> Safety Performance Level (SPL):
>
> HST facilities are able to undergo the effects of the Design Basis Earthquake (DBE) with repairable damage and temporary service suspension. However, normal service can resume within a reasonable time frame, and passengers and personnel can safely evacuate. Only short term repairs to structural and track components are expected.
>
> (p. 13)

For the Delta tunnels, the equivalent to the "no collapse" criteria for the High Speed Train Project tunnels is a "no lining failure" criteria. Without an analysis of the performance of the tunnel lining and other critical project facilities in a Maximum Considered Earthquake, and associated risk of loss of life, the seismic analysis in the Conceptual Engineering Report is incomplete, and so is the evaluation of potential seismic effects for the CEQA/NEPA process.

d.) Army Corps of Engineers Standards

The Final EIR/EIS includes the US Army Corps of Engineers' standard EM 1110-2-6050, Response Spectra and Seismic Analysis for Concrete Hydraulic Structures, 1999, in the list of applicable standards. The USACE standards potentially allows analysis with a maximum design earthquake (MDE) less than the ASCE maximum considered earthquake (MCE):

[7] California High Speed Train Project, Technical Memo 2.10.4, Interim Seismic Design Criteria, June 2009. Available at http://www.tillier.net/stuff/hsr/TM-2.10.4-Interim-Seismic-Criteria-R0-090608-.pdf. Accessed on January 16, 2017. Incorporated by reference.

> For a maximum design earthquake (MDE) which is a maximum level of ground motion for which a structure is designed or evaluated, the associated performance requirement is the that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated. The damage during an MDE event could be substantial, but should not be catastrophic in terms of loss of life, economics, and social and environmental impacts.

However, for critical structures, the USASCE maximum considered earthquake must be used:

> For critical structures (structures of high downstream hazard whose failure during or immediately following an earthquake could result in loss of life), the MDE is set equal to the MCE (the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence). For other than critical structures, the MDE is selected as a lesser earthquake than the MCE.

Failure of the tunnel lining could cause significant leakage, which could potentially cause significant ground and surface movement and losses. Sewer, drain, and water leaks have caused large sinkholes within their right-of-way. Sinkholes and blowouts are a major risk to both surface facilities and to people occupying or using surface facilities. Failure of the tunnel lining could also necessitate lengthy and expensive repairs. For these reasons, impacts of seismically induced tunnel lining failure must be explicitly considered, both in the selection of seismic design criteria and in the CEQA process.

e.) Bridge and Highway Standards

Section 9.2.2.6 of the Final EIR/EIS also lists several standards related to bridges and highways, including the following:

- American Association of State Highway and Transportation Officials (AASHTO) Guide 36 Specifications for LRFD [load and resistance factor] Seismic Bridge Design, 1st Edition, 2009
- Caltrans (California Department of Transportation) Seismic Design Criteria (SDC), Version 1.6, 17 Nov 2010.
- Federal Highway Administration Seismic Retrofitting Manual for Highway Structures, Parts 1 28 and 2, 2006.

It should be clear that these criteria are for bridges and for retrofits of highway structures, and have not been evaluated for large hydraulic structures. The Federal Highway Administration (FHWA) does have a civil engineering manual for design and construction of road tunnels, [8] which is cited in the Conceptual Engineering Report. The

[8] U.S. Department of Transportation, Federal Highway Administration, Technical Manual for Design and Construction of Road Tunnels — Civil Elements, December 2009. Previously cited.

manual states that it is typical for transportation tunnels to be designed to the American Society of Civil Engineers criteria, because collapse could be catastrophic:

> The collapse of a modern transportation tunnel...during or after a major seismic event could have catastrophic effects as well as profound social and economical impacts. It is typical therefore for modern and critical transportation tunnels to be designed to withstand seismic ground motions with a return period of 2,500 years, (corresponding to 2% probability of exceedance in 50 years, or 3% probability of exceedance in 75 years). (p. 402)

f.) Department of Water Resources Standards for Seismic Analysis

The Final EIR/EIS lists the Department of Water Resources' 2012 State Water Project Seismic Loading Criteria Report (SLC Report)[9] as one of the applicable design standards. The SLC Report states that the Department of Engineering could find no seismic design documents for any previously constructed pipelines or tunnels. It also does not specify any minimum seismic standards for future construction of pipelines or tunnels. DWR's SLC report states, with respect to water supply pipelines:

> Similar to SWP canals, little documentation exists regarding the seismic loading criteria used in the design of existing pipelines including the recently designed pipelines. DWR does not currently use any analytical model to predict the behavior of buried pipelines during earthquake occurrences. This is partly because earthquake loads may not be a concern for pipelines below the ground surface. Furthermore, AWWA manuals do not explicitly include seismic loading criteria for water pipelines.
> (p. 16)

The assertion in DWR's SLC report that earthquake loads may not be a concern for pipelines below the ground surface is incorrect. Earthquakes in California have resulted in significant damage for buried pipelines.[10] Although DWR's SLC report goes on discuss recommendations for seismic standards by the American Lifelines Association, the report does not even treat them as guidelines. The section on tunnels is even briefer, stating only:

> The seismic loading criteria that were used in the design of existing SWP tunnels also have not been found. Many references, including the "Seismic Design of Tunnels – A Simple State-of-the-Art Design Approach" monograph (Jaw-Nan

[9] Department of Water Resources Division of Engineering, State Water Project Seismic Loading Criteria Report, 2012. Available at http://www.water.ca.gov/pubs/swp/swp_seismic_loading_criteria_report/swp_seismic_loading_criteria_report.pdf
[10] The assertion that earthquake loads may not be a concern for buried pipelines is incorrect. The American Lifelines Alliance (ALA), a partnership between FEMA, USGS, USDOT, BuRec, and the National Institute of Building Science, developed a database in 2001 which includes pipeline damage rates from 18 earthquakes from 1923-1995. Eight were in California. The results were published in ALA Seismic fragility formulations for water systems, ALA (2001). Available at http://www.americanlifelinesalliance.com/pdf/Part_2_Appendices.pdf. Incorporated by reference.

> Wang and Parson Brinckerhoff, 1993) discuss the seismic loading criteria that could be used for tunnels.
> (p. 18)

The SLC Report also states:

> These guidelines are a suggested starting point, but do not take the place of the design engineer's judgment and additional information available for a particular project site. Each design engineer should have the knowledge, experience, and insight into the importance of their facility to select the appropriate seismic design load and subsequently to apply that load in an appropriate manner to the structure. Similarly, this report does not prescribe the procedure or process of analyzing the structure. Again, this is design engineer's responsibility to select the method of analyses that best suit the complexity, criticality, and importance of the facility.
> (p. 3)

Thus the SLC Report does not provide any minimum standards for SWP facility design, simply documentation of that Department of Water Resources considers seismic design criteria to be the responsibility of the design engineer.

The Final Draft CER indicates that the final seismic design criteria for the Modified Preferred Tunnel Option has not yet been adopted (p. 49, "Final design liquefaction analyses should be performed when final seismic design criteria for the MPTO/CCO facilities have been adopted.") It is thus unclear which, if any, of the external standards cited in section 9.2.2.6 of the Final EIR/EIS will apply to the final seismic design of the MPTO/CCO facilities, including the tunnels and the intake facilities. The CEQA analysis is therefore inadequate without final seismic design criteria for the tunnels and the intake facilities. Risks cannot be adequately addressed or mitigated under CEQA process until the final seismic design criteria is specified and disclosed. The final seismic design criteria should also be specified by the Department of Water Resources and the US Bureau of Reclamation before the design of the project progresses to the Design Construction Enterprise.

2.) Soils

a.) Soil classification and spectral response

The standards in ASCE 07-10 state, with respect to soils:

> 20.1 SITE CLASSIFICATION The site soil shall be classified … based on the upper 100 ft (30 m) of the site profile […] Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless 11.4.5 the authority having jurisdiction or geotechnical data determines Site Class E or F soils are present at the site.
> (p.234)

The Delta Risk Management Strategy Seismic Risk Analysis assumed Class D soils.[11] However, many of the project soils are Class F and subject to liquefaction or have greater than 10 feet of peat (equivalent to Soft or Flowing Ground, in tunneling) with significantly different seismic responses. Bartlett, et. al. developed guidance for the Utah Department of Transportation for design of structures constructed on deep, unconsolidated sediments similar to those in the Sacramento Delta. Their report[12] states:

> Research from past earthquakes and ground response modeling studies suggest that at high levels of ground motion, soft soils will yield and behave plastically. This yielding produces a strong nonlinear soil response that affects the amplitude and frequency content of the recorded motion. Response spectra for soft/deep soil sites show a deamplification at higher frequencies and a shift of the predominate response to longer periods, when compared with adjacent rocks sites at similar earthquake distances (Seed et al., 1976; Idriss, 1990; Seed et al., 1992; Chang et al. 1997; Seed et al. 1997).
> (p. 2)

and continues:

> Site-specific response analyses are required for all Site Class F soils, which include liquefiable soils, peat, and high plasticity clays. (MCEER/ATC-49a, b and ASCE 7-05).
> (p. 2.)

Such site-specific response analyses must be developed and used in the Conceptual Engineering for seismic analyses of the tunnels and related structures. A set of initial borings is available, and site-specific response analyses should have been developed for borings close to the revised alignment. Non-uniformity of ground conditions should also have been evaluated. Evaluation of variation in soil conditions is essential to assess differential stresses on the tunnel lining. Iain Tromans' 2004 dissertation, Behaviour of buried water supply pipelines in earthquake zones, has important information on seismic factors associated with damage to buried pipelines.[13] With respect to non-uniformity, Tromans stated:

> Many field observations and theoretical studies have shown, however, that for transient earthquake effects, the level of non-uniformity of ground conditions is also extremely important in the seismic behaviour of buried pipelines (Liang &

[11] Delta Risk Management Strategy, Final Report, Section 6, Seismic Risk Analysis, 2009. Previously cited.
[12] Bartlett, Steven F., Ostadan, Farhang, Abghari, Abbas, Farnsworth, Clifton. Development of Design Spectra for Deep and Soft Soil Sites. Available at http://www.civil.utah.edu/~bartlett/CVEEN6330/spectra.pdf. Accessed on January 16, 2017. Incorporated by reference.
[13] Iain Tromans, Behaviour of buried water supply pipelines in earthquake zones, dissertation, Department of Civil and Environmental Engineering Imperial College of Science, Technology and Medicine, London, 2004. Available at https://workspace.imperial.ac.uk/geotechnics/Public/tromans.pdf. Accessed on January 29, 2017. Incorporated by reference.

> Sun, 2000). Lateral variation of ground conditions has been shown to cause strain concentrations during ground shaking due to significant differences in ground-motion characteristics even over short distances. Strong-motion array measurements have shown variations by a factor of five in velocity over a distance of 200 m and by a factor of two in acceleration over the same distance, all caused by variable site conditions (Zerva, 2000).
> (p. 68)

The Conceptual Engineering Report also uses peak ground acceleration (PGA) to assess the potential damage to the tunnels. Tromans states, "PGA itself is not a particularly good measure of damage to structures, except in certain special cases (i.e. very stiff structures) (p. 46.)" The peak ground velocity (PGV) is a better measure of ground strain on the segmented tunnel lining. As Tromans states,

> The inadequacy of using solely peak ground acceleration (PGA) as a strong-motion parameter for seismic design has been recognised for some time (McGuire, 1978). In the area of lifeline earthquake engineering, as has been shown in Chapter 3, the behaviour of buried pipelines is controlled by the ground strain, which is closely related to peak ground velocity (PGV). Peak ground displacement (PGD) is of particular importance for structures with multiple supports such as above-ground pipelines and bridges, or other large-scale structures with long-period response (Gregor, 1995).
> (p. 125)

Tromans also states:

> Velocity is a parameter less sensitive to high frequency components of the ground motion. As such, the peak ground velocity, PGV is a useful indicator of the effect of ground motion on
> structures such as tall or flexible buildings, which are sensitive to intermediate frequencies. Velocity parameters in general are closely linked to the energy associated with an earthquake
> record, so may be better indicators of structural damage potential (Newmark & Hall, 1982).
> (p. 46)

Correct estimates for peak ground velocity are thus essential to assessing the effects of ground strain on the proposed design. The Delta Risk Management Strategy report seismic risk analysis[14] does not provide estimates of Peak Ground Velocity, but they can be derived from the 1.0 second spectral acceleration, which is provided. Estimates of PGV from the DRMS study should be used for the tunnels engineering where estimates of PGA from the DRMS study are used. Site-specific response curves derived from site borings should include estimates of PGV.

[14] Delta Risk Management Strategy, Final Report, Section 6, Seismic Risk Analysis, 2009. Previously cited.

These analyses of soil seismic properties are an essential part of planning, design, and construction of large tunnels in deep, wet, soft, non-uniform alluvial deposits and must be supported by appropriate borings, samplings, and surveys. Without adequate analyses, the feasibility of the proposed tunnel lining design and the economic feasibility of the proposed tunnel alignment cannot be assessed. The Department of Water Resources should not proceed to a Design and Construction Enterprise without demonstrating the safety and feasibility of the conceptual level design. Similarly without such consideration, the CEQA process remains incomplete with regard to impacts and mitigations.

b.) Assumptions of Attenuation of Seismic Ground Motions

Seismic ground motions attenuate with depth. However, the Final Draft Conceptual Engineering Report may be assuming too much attenuation of ground motion with depth. The Federal Highway Administration (FHWA) provides a table of attenuation values in their tunnel design manual, and states (p. 407):

> The ratios of ground motion values at tunnel depths to those at the ground surface may be taken as the ratios summarized in Table 13-1 unless lower values are justified based on site-specific assessments.

Table 13-1 Ground Motion Attenuation with Depth

Tunnel Depth (m)	Ratio Of Ground Motion At Tunnel Depth To Motion At Ground Surface
≤ 6	1.0
6 -15	0.9
15 -30	0.8
≥ 30	0.7

The value of 70% of PGA at 30 meters (98 feet), would therefore be acceptable to use for the design of the tunnels. The Final Draft CER cites the FHWA manual value of 70%, but then increases attenuation of at-depth PGA to 50%:

> The proposed depths of the tunnels are between 100 to 200 feet bgs. For the conceptual level design, and in the absence of more rigorous analyses, a value of approximately one-half of the surface PGA was assumed for structural analyses of the buried tunnel linings. (p. 46.)

However, seismic data from the CSMIP Strong-Motion Geotechnical Array shows that the assumption of 50% attenuation may be wrong. Following the Santa Monica freeway collapse in the Northridge Earthquake, CalTrans has done long-term seismic monitoring of boreholes in sites with deep soft alluvial deposits, including La Cienega in Los Angeles, Meloland in El Centrol, Eureka, and Vincent Thomas in Long Beach.[15]

A 2005 study by Chinese engineers, Variation of earthquake ground motion with depth, Hu Jin-jun & Xie Li-li,[16] analyzed the data from these sites. The 2005 study found that peak ground acceleration (PGA) in the deep alluvial soils falls off slowly for large magnitude earthquakes (Section 3.1.2, Soil site, p. 77.) The graph from section 3.1.2 is reproduced on the following page and indicates that a peak ground surface acceleration is reduced by 30% down to 70% of that at surface at a depth of 40-50 meters (120-160 feet) and is approximately correct for large magnitude events in deep alluvial deposits. An attenuation value of 50% is too large, compared to 30% of PGA.

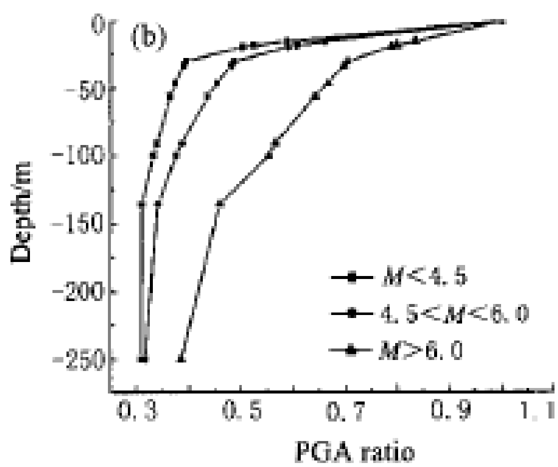

Figure 1 Hu Jin-Jun and Xie Li, Li, PGA variation with depth at soil site

c.) Liquefaction Analysis

The deep, soft soils in the project footprint are subject to liquefaction. Appropriate assessments of the potential for liquefaction at or above the tunnels is essential for assessing the risk to water supply and to surface structures. Liquefaction at the level of the tunnels can give rise to high ground strains. If the tunnels are buoyant, that is, less dense than the surrounding ground, liquefaction above the tunnels could also cause significant vertical movement of the tunnel segments, which could result in major leakage especially in the vicinity of the tunnels with their shafts.

[15] Recent Data Recorded from Downhole Geotechnical Arrays, Vladimir Graizer, Anthony Shakal and Pat Hipley, SMIP2000 Seminar Proceedings. Available at http://www.conservation.ca.gov/cgs/smip/docs/seminar/SMIP00/Documents/paper2_graizer.pdf. Accessed on January 16, 2017. Incorporated by reference.

[16] Variation of earthquake ground motion with depth, Jin-jun, H. & Li-li, X. Acta Seimol. Sin. (2005) 18: 72. doi:10.1007/s11589-005-0008-x. Available at http://link.springer.com/article/10.1007/s11589-005-0008-x Accessed on January 16, 2017. Incorporated by reference.

Liquefaction analyses in the initial 2010 study by the Department of Water Resources found that substantial, continuous liquefaction of the soil column down to 100 feet could be expected:

> The seismic event assumed for the liquefaction analysis had a magnitude of 7.5 and a peak ground acceleration of 0.49 g. The average shear wave velocity for the uppermost 40 feet of soil (Vs, 40) was assumed to be 500 ft/sec.
>
> All of the borings analyzed included soils that are potentially liquefiable, although to different extents. Substantial, continuous liquefaction of the soil column can be expected down to elevation -100 feet, based on the borings analyzed. Below this depth only isolated pockets of liquefaction are observed. (p. 4-13.)

However, the Final Draft CER assumed a lower earthquake magnitude and peak ground acceleration (PGA):

> For purposes of the preliminary liquefaction analyses, a horizontal PGA corresponding to the probabilistic 85th percentile, 1,000-year ground motion was used for the forebay locations, and the probabilistic median 500-year ground motion was used for all other facility locations. (p. 49)

Results for the main tunnels was a 40-60% reduction in the estimated depth of liquefaction:

> For the Main Tunnels, extensive liquefaction of the upper 40 to 60 feet is predicted in areas with soft and loose soils, and liquefaction-induced settlement of the Main Tunnel drive shafts and reception shafts working pad fills can be expected. (p. 49.)

The conclusion in the Final Draft CER was that liquefaction would occur substantially above the depth of the tunnels and did not need to be considered at the tunnel profile depths and in tunnel design. However, this conclusion was based on 10% in 50 years ground motion, substantially weaker than the ASCE Maximum Considered Earthquake. The seismic analysis is simply incomplete without analysis of liquefaction under stronger motions.

3.) Structural Analysis for Conceptual Design

a.) Appropriate levels of structural analysis for preliminary design

The California High Speed Train Project has developed preliminary seismic design benchmarks. These benchmarks are suitable for a preliminary conceptual analysis, and include criteria for seismic design of tunnels. The California High Speed Rail Project's Technical Memorandum on 15% Seismic Design Benchmarks[17] states:

> Generally, seismic response of tunnels is dominated by the surrounding ground response, and not the inertial properties of the tunnel itself. The focus of tunnel seismic design shall be on the free-field deformation of the surrounding ground and its interaction with the tunnel.
>
> For 15% Design, two types of deformations which characterize the seismic response of tunnels shall be evaluated:
>
> 3. Longitudinal axial and curvature deformations (see Figure 6-1)
> 4. Transverse ovaling or racking deformation (see Figure 6-3)
>
> The Federal Highway Administration (FHWA) has created an online technical manual [17, http://www.fhwa.dot.gov/bridge/tunnel/pubs/nhi09010/13a.cfm][18] which summarizes approximate and simplified closed-form procedures for these deformations. These procedures shall be used to evaluate the 15% seismic design of tunnels, with regard to deformation and strain demands.
> (p. 12)

The Department of Water Resources 2010 initial design report[19] shows that a simplified closed form analysis of the tunnel lining was done. In Section 10.1, Design Criteria, the report recommended that "a second pass system using a steel liner installed in the areas of higher internal pressures should be maintained as an option until the development of the design and testing during preliminary engineering prove the feasibility of the desired lining system." (p. 10-1.) There is no further discussion of the seismic analysis of the tunnel lining for preliminary engineering in the 2015 Final Draft CER. However, the Design Construction Enterprise Agreement[20] indicates that only a single-pass liner has been chosen for design development (p. 9.)

No apparent consideration is noted regarding seismic responses of tunnels and related shaft/caissons both during construction/boring and operations. Similarly, analyses have not considered or provided for analyses with orientation of seismic waves in relationship to the project orientation (e.g., cross-wise and parallel [90°, 45°, 0°] with tunnel right of

[17] California High Speed Train Project, Technical Memorandum 2.10.5, 15% Seismic Design Benchmarks. Available at http://www.hsr.ca.gov/docs/programs/eir_memos/Proj_Guidelines_TM2_10_5R00.pdf Accessed on January 16, 2017. Incorporated by reference.
[18] U.S. Department of Transportation, Federal Highway Administration, Technical Manual for Design and Construction of Road Tunnels — Civil Elements, December 2009. Previously cited.
[19] California Department of Water Resources," Draft Report Of the Initial Analysis & Optimization of the Pipeline/Tunnel Option," December 17, 2010. Previously cited.
[20] California Department of Water Resources, Final Draft Agreement Regarding Construction of Conveyance Project between the Department Of Water Resources and the Conveyance Project Coordination Agency, 2015. Previously cited.

way). Without adequate consideration of the failure potential for the project during seismic events and its impacts, the CEQA process remains inadequate if not incomplete.

b.) Separation of Liner Segments

The California High Speed Train Project addresses the design of liners for earth tunnels in section 3.3.4.4.1 of Technical Memo 2.10.4, Interim Seismic Design Criteria. With respect to segmented tunnel liners, the Technical Memo states:

> 3.3.4.4.8 Stability When segmental linings are used for a bored tunnel, the stability of the segments has to be shown by detailed finite element model using nonlinear soil continuum and proper contact surfaces at the interfaces of each segment. Racking/ovaling analysis shall be performed to examine the separation of the segments and stability of the entire system.
> (p. 33.)

The 2010 initial analysis by the Department of Water Resources showed that the tunnel lining joints could “de-stress”:

> The compressive stresses induced by the earthquake are less than the assumed compressive strength of the concrete; however, temporary de-stressing of segment joints could occur, resulting in temporary increase in exfiltration.
> (p. 4-13.)

The failure to disclose the results of the 2010 initial analysis of the tunnel lining joints in the Conceptual Engineering Report or to discuss mitigation strategies in the CEQA assessments raises significant concerns. The Final Draft EIR/EIS should commit to providing a publicly distributed racking/ovaling analysis for the final lining design,that uses appropriate seismic criteria.

c.) Safety Factors for Tunnel Design

The Department of Water Resources’ Final Draft CER has a detailed discussion of criteria for seismic safety factors for bridges (p. 9-32), intakes and outlets (p. 9-40), pile foundations (p. 9-43), and levees (p. 9-78), but does not discuss criteria for safety factors in the design of the WaterFix tunnels. General requirements specify that tunnels should be designed with a minimum safety factor of two (2.0). Safety factors are an essential part of standard engineering practice and prospective assessment of impacts and mitigation during construction and operations.

The California High Speed Train Project provides the following design criteria for "earth tunnel liners" in section 3.3.4.4.1 of Technical Memo 2.10.4, Interim Seismic Design Criteria.[21]

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[21] California High Speed Train Project, Technical Memo 2.10.4, Interim Seismic Design Criteria, June 2009. Available at http://www.tillier.net/stuff/hsr/TM-2.10.4-Interim-Seismic-Criteria-R0-090608-.pdf Accessed on January 16, 2017. Incorporated by reference.

3.3.4.4.1 Earth Tunnel Liners - General

Earth tunnel liners shall be designed to sustain all the loads to which they will be subjected with minimum factor of safety of two. Such loads shall include:

[…]

3. Erection loads including external grouting loads.

4. Earth pressure shall be calculated using 2D finite element analysis methods based on the best available geotechnical data. In lieu of this computer analysis, no less than full overburden shall be used.

5. Hydrostatic pressure.

6. Self-weight of the tunnel structure.

7. Loads due to imperfect liner erection, but not less than 0.5 percent diametrical distortion.

8. Additional loads due to the driving of adjacent tunnels.

9. Effects of tunnel breakouts at cross-passages, portals, and shafts.

[…]

12. Seismic loads as indicated in this document.

Provisions shall be made in the liner segments for corrosion prevention and the elimination of stray currents from the surrounding ground area. Provisions for ground structure interaction and lateral support of surrounding ground shall be included.

(p. 31.)

Similar criteria must be apply to the WaterFix tunnels and expressly incorporated into the project description for the CEQA/NEPA considerations.

2.) Interface with Access Shafts

Given the ground plasticity and potential liquefaction of the soft ground surrounding the tunnels, the issue of differential movement of the tunnels, intakes/outlets, and access shafts is substantial. These must be carefully analyzed and their impacts adequately addressed and mitigated.

The California High Speed Train Project provides the following design criteria for interfaces between the main tunnel and access shafts in section 3.3.4.4.9 of Technical Memo 2.10.4[22]:

> 3.3.4.4.9 Interface Joints...Interfaces between the bore tunnel structures and the more massive structures, such as the cut and-cover structures, mined station sections, and ventilation/access structures,...shall be designed as flexible joints to accommodate the differential movements. The design...differential movements shall be determined by the designer in consultation with the Geotechnical Engineer.
> (p. 33.)

Differential movements between the WaterFix tunnels, intakes/outlets, and access shafts also need a differential analysis and appropriate assessment of impacts and required mitigation. This is especially important because the access shafts will be fixed vertically in very large concrete slabs to protect the shafts from flooding, while the tunnels will be bedded in deep alluvial deposits.

3.) Leakage

a.) Leakage analysis

Leakage through the tunnel liner due to ongoing operations could cause failure of the lining. The Conceptual Engineering Report states:

> In addition to strength requirements, leakage control through the liner is essential to ensure liner performance. Excessive leakage through the liner could lead to potential soil erosion, hydraulic fracturing and loss of liner support. Water leakage from the tunnel to the surrounding area also translates to economic loss. (p. 142.)

In comments on the 2015 WaterFix Revised Draft EIR/Supplemental EIS, East Bay Municipal Utilities District (East Bay MUD) expressed concerns about the impacts of long term tunnel leakage on the overlying Mokelumne Aqueduct.[23] Similar concerns apply to other critical structures that are in the tunnel path, including State Route 4 and State Route 12, and Delta levees and channels.

Impact: Tunnel lining failure of BDCP Conveyance Tunnels

> Long-term degradation of segmental concrete lining may result in failure of the lining. In the event that the tunnel lining fails and results in a tunnel collapse or blowout, a collapse during operations would result in major ground movement extending to the ground surface and potentially sinkholes or blowout. With such events, resulting ground movement would likely result in failure of the existing Mokelumne Aqueducts.

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[22] California High Speed Train Project, Technical Memo 2.10.4, Interim Seismic Design Criteria, June 2009. Previously cited.
[23] East Bay Municipal Utilities District, comments on the 2015 WaterFix Revised Draft EIR / Supplemental EIS, Attachment 3. Available at http://www.restorethedelta.org/wp-content/uploads/2015/11/EastBayMUD.pdf

(Attachment 3, p.15.)

East Bay MUD's recommended design mitigations included the following:

1) Design of the segmental lining for long term performance,
[…]
4) Use of a higher level of design and longer design life for the segments which may include the need for a more robust lining system,
5) Additional reinforcement, stronger or more durable concrete, multiple gaskets, and stronger joints,
6) Use of a carrier pipe surrounded with backfill grout inside the segmental concrete lining.
(Attachment 3, p.15.)

The Department of Water Resources (2010) did an initial analysis of the tunnel design, and the analysis showed that the tunnels could have significant leakage (exfiltration) when they are first operated:

> The analyses showed that a tunnel at a depth to springline of 80 ft would have a considerable amount of exfiltration (Table 4-2), and the impact on groundwater could reach the surface, while a tunnel at depth of 160 ft would experience a moderate amount of exfiltration and would have a negligible impact on the groundwater.

Table 4-2 Summary of Tunnel Exfiltration Estimates

Depth, d (ft)	Exfiltration from tunnel at maximum internal pressure, q (ft^2/sec/ft)	Total exfiltration (ft3/s)	Exfiltration (acre-ft/day)
80	1.4×10^{-2}	1100	2200
120	2.9×10^{-3}	180	360
160	4.5×10^{-4}	24	48

(p. 4-9.)

The tunnel design was modified to move the pumping plant to near Clifton Court Forebay to reduce internal pressure and presumably, potential exfiltration. The Final Draft Conceptual Engineering Report does not disclose the preliminary leakage analysis nor any subsequent leakage analyses, only stating:

> Once detailed geotechnical data is available during preliminary design, the segment liner will be designed to limit water leakage by considering surrounding ground-liner interaction and ground permeability. At the same time, design factors such as effective ground overburden, high strength bolts, shear dowels, post-tensioning system, ferrous push-fit connectors, and proprietary joint connectors will be more fully analyzed as part of the final segment design.
> (p. 142.)

This discussion is inadequate and incomplete to evaluate potential impacts of tunnel liner leakage due to initial or continued operation. Referral to later considerations and prospective measures do not meet CEQA requirements for assessment and mitigation.

b.) Leakage Criteria

In the initial engineering analysis, under section 4.3.5 of Preliminary Evaluation of Gasket Capabilities, [24], the engineers stated:

> Exfiltration (leakage) criteria will need to be developed and adopted during Preliminary Engineering. Search of technical literature identified criteria used for sewer tunnel exfiltration (0.008 gpm x 100 ft x ft resulting in 515 gpm or 2.3 acre-ft/day) and infiltration, or the AWWA criteria for leakage from pressure pipes (3,700 gal/hr or 0.3 acre-ft/day). Depending on the exfiltration criterion adopted for this project, a tunnel at a depth of 160 feet could be acceptable without additional structural restraint. (p. 4-9.)

However, the Final Draft CER does not indicate that any leakage criteria have been adopted. The initial recommended depth of 160 feet was not kept for the North tunnels. The Final Design and Construction Enterprise Agreement[25] states:

> The preliminary tunnel inverts range from 122 to 135 feet below mean sea level (msl) for the North Tunnels and from 147 to 163 feet below msl for the Main Tunnels. (p. 20.)

The CEQA analysis is inadequate without the specification of leakage criteria and evaluation of adequate monitoring or mitigation for leakage under CEQA. The discussion of potential impacts of the operations of the project on the levees under Impact GEO-9: Loss of Property, Personal Injury, or Death from Landslides and Other Slope Instability during Operation of Water Conveyance Features is also incomplete without discussion of potential damage to the levees from leakage.

c.) Leakage Monitoring and Mitigation

i.) Inspection.

While the Conceptual Engineering Report mentions access for inspection, the CER does not indicate an inspection, monitoring, and remediation program and does not discuss contingencies, controls, and recovery following indication and evidence of leakage of the

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[24] California Department of Water Resources," Draft Report Of the Initial Analysis & Optimization of the Pipeline/Tunnel Option," December 17, 2010. Submitted with this report.

[25] California Department of Water Resources, Final Draft Agreement Regarding Construction of Conveyance Project between the Department Of Water Resources and the Conveyance Project Coordination Agency, 2015. Previously cited.

tunnel lining. In comments on the WaterFix RDEIR/SDEIS, East Bay MUD[26] suggested a program of inspection of the tunnel lining, "during and upon completion of construction, and routine inspections during the operational life of the tunnels." A program of inspection, monitoring, and remediation is standard practice in maintenance of water supply pipelines and tunnels.

The Bureau of Reclamation discussed pipeline vulnerabilities and inspections standards in an Environmental Assessment for maintenance of Santa Clara conduit, an 8 foot concrete pipeline between San Luis Dam and the Santa Clara Valley Water District, published in 2007.[27] The Santa Clara Conduit EA states:

> Leaks usually occur at pipeline joints, or connections such as at valves. A leak is easily detected within a vault but can also be detected through pressure drops and localized ground saturation or ponding. Once a month all pipelines would be inspected via helicopter to verify integrity. The helicopter crew scans for saturated ground or ponding near the facilities. Small, buried leaks would be harder to detect in winter months than in the summer when the ground is dry.

The Santa Clara Conduit EA also states:

> ***2.2.2.4 Internal Inspection***
>
> Internal inspections are necessary to determine the integrity of joints and all sections of pipeline, pipeline materials, and equipment, especially in seismically active areas. Internal inspections are planned every 5-10 years, depending on the pipeline. Internal inspection activities usually occur on all pipelines as preventative maintenance although the inspection interval varies by pipeline.

The Santa Clara Conduit pipeline failed in August 2015 and spilled 60 acre-feet of water into nearby Pacheco Creek. [28,29] The pipeline was completed in 1985 and was 30 years old. The last inspection had been in 2008.[30]

ii.) Remediation.

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[26] East Bay Municipal Utilities District, comments on the 2015 WaterFix Revised Draft EIR / Supplemental EIS, Attachment 3, Prevously cited.

[27] Bureau of Reclamation, Environmental Assessment for Maintenance of the Santa Clara Conduit. Available at https://www.usbr.gov/mp/nepa/documentShow.cfm?Doc_ID=2868. Accessed on January 29, 2016. Incorporated by reference.

[28] The pipeline failure was covered by the Mercury News. See http://www.mercurynews.com/2016/03/01/failure-of-key-water-pipeline-into-silicon-valley-may-have-exposed-wider-problems/. Accessed on January 17, 2016. Incorporated by reference.

[29] The District water loss estimate is available at http://valleywater.org/WorkArea/DownloadAsset.aspx?id=12939. Accessed on January 17, 2016. Incorporated by reference.

[30] The District failure analysis is available at http://www.valleywater.org/WorkArea/DownloadAsset.aspx?id=13522. Accessed on January 17, 2016. Incorporated by reference.

In comments on the WaterFix RDEIR/SDEIS, East Bay MUD also suggested procedures for remediation of structural deficiencies in the WaterFix tunnel lining:

> In the event that structural deficiencies of the segmental concrete lining are detected, the situation can be addressed with one or more of the following actions:
>
> > 1) The lining can be improved with localized structural patches,
> > 2) Permeation (cement or chemical) grouting can be used immediately outside the lining,
> > 3) New secondary lining can be placed for full 360 degrees inside the segmental concrete lining,
> > 4) Additionally, compensation grouting can be used to restore lost ground and/or densify the ground to prevent the upward migration of settlement.

No program of mitigation, monitoring, and reporting has been proposed to adequately and completely develop for project start-up, and operations and some form of maintenance. Such program(s) must be developed as part of CEQA prior to finalization of the EIR/EIS. In addition, since the WaterFix tunnels are proposed to be buried at a depth of 120 to 160 feet in saturated soil, remediation could result in significant disruption on the surface, requiring both excavation and dewatering. Procedures for access to and excavation of private lands above the tunnels for remediation need to be developed and discussed as part of CEQA, as well as mitigation for economic losses from remediation.

4.) Tunnel Construction

a) Design criteria

The Final EIR/EIS states that the following guidelines for settlement criteria and calculation of settlement during tunneling.

> In particular, conformance with the following federal design manuals and professional society and geotechnical literature would be used to predict the maximum amount of settlement that could occur for site-specific conditions, to identify the maximum allowable settlement for individual critical assets, and to develop recommendations for tunneling to avoid excessive settlement, all to minimize the likelihood of loss of property or personal injury from ground settlement above the tunneling operation during construction.
>
> > - *Technical Design Manual for Design and Construction of Road Tunnels* (U.S. Department of 33 Transportation, Federal Highway Administration 2009). 34
> > - *A Method of Estimating Surface Settlement above Tunnels Constructed in Soft Ground* (National 35 Research Council of Canada 1983). 36
> > - *Predicting the Dynamics of Ground Settlement and its Derivatives Caused by Tunnelling in Soil* 37 (Attewell and Woodman 1982). 38

- *Predicting the Settlements above Twin Tunnels Constructed in Soft Ground* (Chapman et al. 2004). 39
- *Report on Settlements Induced by Tunneling in Soft Ground* (International Tunneling Association 40 2007). 41
- *Closed-Face Tunnelling Machines and Ground Stability: A Guideline for Best Practice* (British 42 Tunnelling Society 2005).

(p. 9-197.)

A more complete and up to date discussion of empirical methods is available in "Analysis of ground settlement caused by tunnel construction" by Maraš-Dragojević.[31] This document should have been included in the list, and empirical methods should have been used for preliminary estimates of potential settlement during tunneling. Without such preliminary estimates, and discussion of criteria for acceptable settlement, there cannot be adequate assessment of potential impacts under CEQA. Future "recommendations for tunneling" fail to provide adequate mitigation under CEQA.

b.) Ground loss and settlement due to tunneling

Tunneling boring machines excavate a larger amount of soil than is replaced by the volume of the tunnel lining, which typically causes a wide, shallow settlement trough on the surface. The over-excavation is measured by the volume of ground loss, which is defined as the percent difference between the volume of excavated soil and the volume of the tunnel lining. The volume of the settlement trough on the surface can be as large as the volume of ground loss. If groundwater is drained for tunnel construction, soil layers above the tunnel could settle even further.

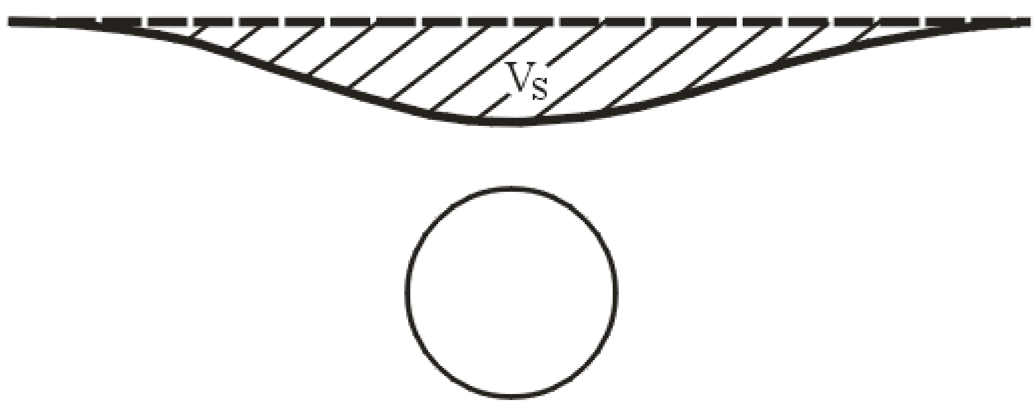

Figure 2 Settlement trough

[31] Maraš-Dragojević, S. 2012. Analysis of ground settlement caused by tunnel construction. Available at http://hrcak.srce.hr/index.php?show=clanak&id_clanak_jezik=126712&lang=en Accessed on January 20, 2016. Incorporated by reference.

East Bay MUD is proposing to construct a 21 foot diameter tunnel in the Delta to replace the Mokelumne Aqueduct. The Conceptual Design report[32] included a section on Ground Loss and Settlement, which states:

> Ground loss and ground settlement commonly occurs as a result of tunnel excavation. A preliminary evaluation of ground loss at the tunnel crown was estimated for this study based on 1 percent face loss, 1 inch overcut of the cutter beyond the shield, and 1 inch ground loss at the end of the shield as the ground moves onto the segments. Annulus grouting is performed at the tail shield between the ground and the liner to replace some ground and reduce settlements, however grouting is not completely effective in preventing ground loss. Based on these assumptions and preliminary calculations, the total ground loss is estimated to be 4 percent of the face which equates to 8 inches across the tunnel at the crown. With careful tunnel excavation, primarily the diligent use of pressurized slurry outside the TBM and immediate complete annulus grouting outside of the concrete segments, these ground losses can be substantially reduced or eliminated, resulting in a combined ground loss of approximately 1 percent. However, these practices are dependent on the contractor's means and methods, and cannot be relied upon for each contract and at every location along all tunnel reaches. Therefore a conservative estimate of 4 percent is used as the basis for comparison in this study.
>
> Note that this lost ground is for routine tunneling and does not include complicating factors such as tunnel curves and work stoppages, nor does it include excessive ground loss such as from over excavation or loss of face pressure. These additional factors can result in substantial ground loss several times the magnitude of ground loss in controlled conditions. Ground settlement will need to be evaluated in detail in future design phases based on actual TBM type, tunneling methods, and details of TBM operation (face pressure), which affect the actual settlement experienced.
>
> Some of these factors can be controlled with a tight design and careful construction, but other factors are unavoidable. Therefore, actual ground loss and settlement could be substantially different from these preliminary estimates.

[32] East Bay MUD, Technical Memorandum Number 2, Delta Tunnel Study Conceptual Design. Available at http://www.waterboards.ca.gov/waterrights/water_issues/programs/bay_delta/california_waterfix/exhibits/docs/EBMUD/ebmud_178.pdf. Accessed on January 28, 2017. Incorporated by reference.

Similar calculations ground loss and settlement should have been included in the Final Draft Conceptual Engineering Report for the WaterFix tunnels, but was not. Without such analysis in the preliminary engineering, there can be no assessment of needed monitoring and mitigation, and the discussion in the Final EIR/EIS is incomplete.

c.) Experience with ground loss during the London Channel Tunnel Rail Link

The Final EIR/EIS refers to Settlements induced by tunneling in Soft Ground, by the International Tunneling and Underground Space Association, 2007[33] The following graphs, from that document, show the ground volume loss in the tunneling for three London segments of the London Channel Tunnel Rail Link. The mean ground loss was around .5% for many segments, but the maximum was over 2.5% in the initial trials. In the Stratford to St Pancras link, once the tunnel ground volume loss exceeded 1, the boring was stopped and the tunnel boring machine was reconfigured for clay soils.

[33] International Tunneling and Underground Space Association, 2007. Settlements induced by tunneling in Soft Ground, report of the Working Group of the International Tunneling Association, Tunnelling and Underground Space Technology 22 (2007) 119–149. Available at https://www.ita-aites.org/en/news/download/76_fc32374761a17cde8d72f432244168b0. Incorporated by reference.

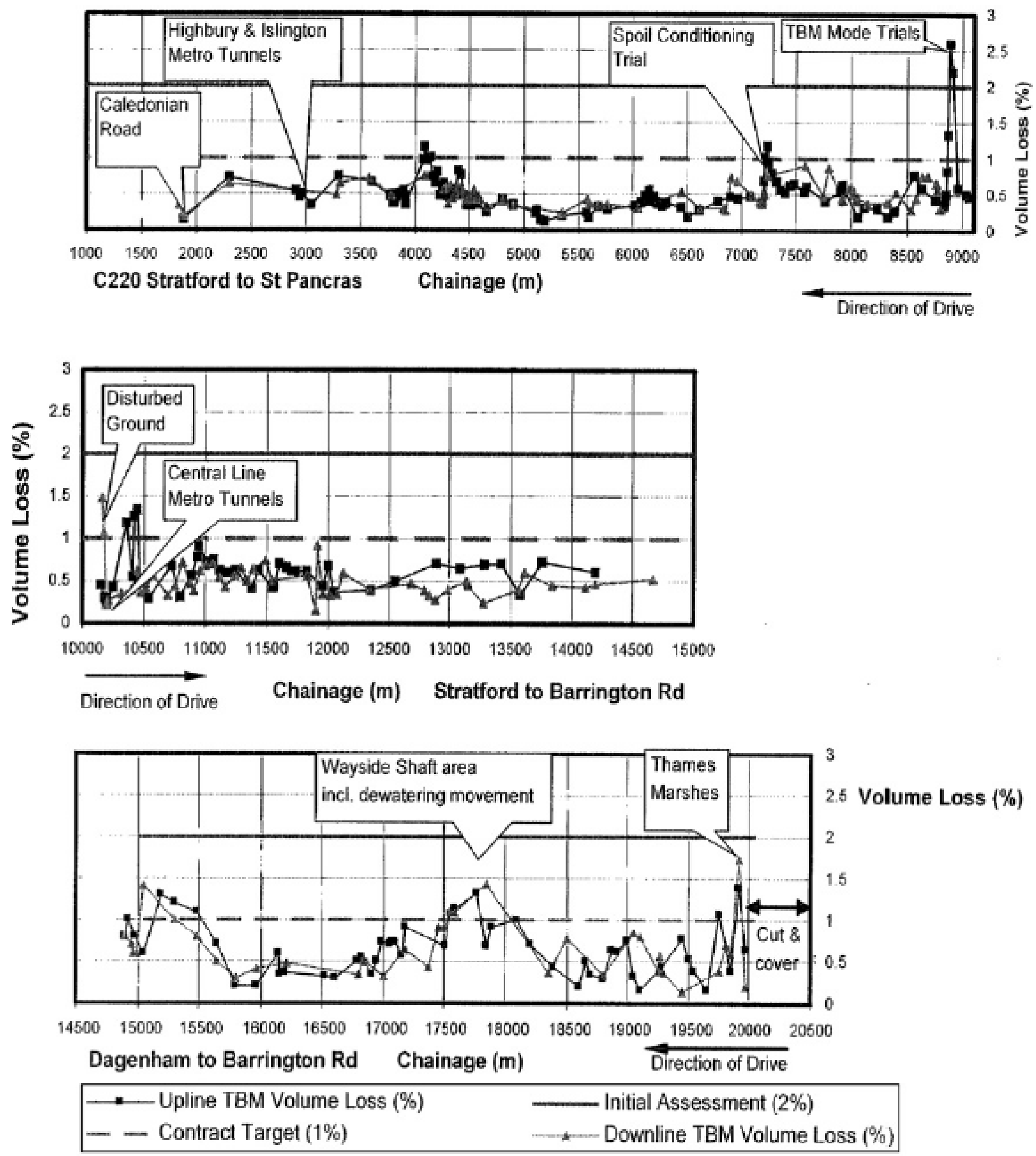

Figure 3 Volume Losses Observed on the CTRL, ITA 2007.

According to Maraš-Dragojević, typical values of ground loss for open-cut tunnel construction in soft soil generally range from 1 to 3 percent (Mair, 2008), while lower values are obtained in closed-cut tunnelling using modern boring machines, such as the EPB shield.

The LCRT construction was tightly monitored and had provisions to stop tunneling when ground loss exceeded 1%. The 1% ground volume loss would be an appropriate criteria for maximum allowed ground loss for tunnel boring. The Final EIR/EIS needs to clarify if that ground loss criteria is being specified, and also discuss whether it is appropriately protective of the surface structures that are in the tunnel alignment. Without specification of ground loss criteria, the CEQA analysis is simply inadequate.

d.) Pre-construction surveys, Monitoring and Restoration

The BART standard specifications on Excavation Support and Protection[34] state the following under 1.0.9 Site Conditionns

A. Pre-construction surveys: The Contractor shall submit to the Engineer, for review and approval, pre-construction surveys for existing structures and facilities located above or adjacent to the new construction and which may be affected by the work. These surveys shall include photographs, maps, plans, written descriptions, and surveyed foundation levels as necessary to fully document pre-construction conditions.

The BART standard specifications on Excavation Support and Protection state the following under part 3, Execution

Section 3.0.1, Detection of Movement

A. For each existing structure or facility within a zone extending upward from the bottom of the excavation on a slope of 2 horizontal to 1 vertical, install settlement detection devices on each footing, foundation, wall, or other feature to be monitored. Settlement detection devices shall be capable of being read to an accuracy of 0.005 foot.

B. Take and record readings not less than once per week during performance of the work.

C. Stop work; notify the Engineer, and take immediate remedial action if movement of the existing structure occurs during performance of the work. All construction activities shall be immediately halted when the settlement of any structure or facility reaches 0.3 inch, and shall not be resumed until after implementation of approved remedial measures.

D. Upon completion of the work, take weekly readings of the measurement points for a period of 4 weeks, or longer if movement persists, and report the results to the Engineer.

E. The detection of movement shall be performed by a qualified licensed civil engineer or land surveyor.

Section 3.12 Restoration

A. Restore existing structures to conditions existing prior to the start of work, including repair of settlement-induced damage

[34] Bay Area Rapid Transit, BART Facilities Standards, Standard Specifications, Revision 3.03, Section 31.50.00, Excavation Support and Protection, January 2014. Available at https://webapps.bart.gov/BFS/BFS_3_0_3_Spec/STDSPEC/31%2050%2000.pdf. Accessed on January 26, 2017. Incorporated by reference.

The CEQA analysis is incomplete without the specification of guidelines for a pre-construction survey, and monitoring for ground movement during the WaterFix tunnel boring. Criteria for work stoppage and mitigation for damage such as BART's restoration commitment needs to be discusses.

e.) Effects on levees

The economic costs of a levee failure due to tunneling damage are potentially very high. The 2004 failure of the Upper Jones Tract, an island of 6,259 acres, cost approximately $120 million to restore. This did not include damage to buildings and crops.

While the effect of the maximum settlement on the freeboard of levees in the Delta is negligible, the horizontal and vertical stresses on the levees from the tunneling movements could cause cracks, especially in levee areas that are prone to slope instability. Cracks in a levee could result in seepage and failure if they happened during times of high flows in the Delta, or if they happened during times of low flow and were not identified and repaired.

Neither the Final EIR/EIS nor the Conceptual Engineering Report discusses seasonal limitations on tunneling under levees, but considering the consequences, this should not be done when storms could cause high flows. The Delta Risk Management Study assessed the conditons of the levees, and assigned them to fragility classes, which were based in part on assessments of the slope of the levee and the soils in the levee. A map of levee fragility classes from the Delta Risk Management Study has been included in the EIR/EIS (Section 3E.) However, no effort has been made to use the levee fragility classes in an assessment of potential effects of tunneling on the levees.

5.) Gas Fields and Wells

Under California law, tunnels are required to be classified by the Division of Occupational Safety and Health, Mining and Tunneling Unit prior to bidding.[35] The Final Draft CER states,

> There are active natural gas fields beneath the anticipated alignment for the tunnels. As these gas fields are present near the proposed tunnel alignment, it is anticipated that the State of California Division of Occupational Safety and Health (Cal/OSHA) might classify the tunnels as "potentially gassy."
> (p. 147.)

But DWR's 2010 initial tunnel analysis included a table of eighteen gas wells located within, or directly adjacent to the proposed Right of Way (Table 7-1, Gas Wells in Pipeline/Tunnel Right-of-Way, p. 7-3.) Some of the wells are currently active. It is unclear whether the Final Draft CER criteria of "beneath the anticipated alignment" includes gas

[35] Division of Occupational Safety and Health, Mining and Tunneling Unit, Underground Classification Requirements. Available at https://www.dir.ca.gov/dosh/documents/mining-and-tunneling/underground-classification-requirements.pdf. Accessed on January 16, 2017. Incorporated by Reference.

wells outside of the tunnel footprint but within or adjacent to the Right of Way or its zone of influence in the CER. All known gas wells within or adjacent to the Right of Way and its Zone of Influence should be included in the CER. Without full consideration, analysis, and disclosure of all gas wells within or adjacent to the Project Right of Way and Zone of Influence, the FEIR cannot adequately assess impacts and appropriate mitigation measures for construction and effects during operations and seismic events. The 2010 initial analysis also stated,

> DWR has neither designed nor constructed a project that passes through a gas field or near existing gas wells, either active or abandoned. Accordingly, and as recommended by the Outside Reviewers, engage the services of a petroleum engineering consultant with experience in the installation and abandonment of gas wells (ideally one familiar with the Delta and its gas wells and fields) to advise the DWR and the DHCCP.
> (p. 7-1)

The Final Draft CER and the Final EIR/EIS need to disclose this recommendation and other recommendations by outside reviewers, and whether the recommended steps will be taken.

6.) Tunnels and Related Project Structures Ground Studies (Development and Mitigation)

Good engineering practices and adequate CEQA project descriptions and settings require objective information, quantitative analyses, rigorous assessments/considerations of impacts (including risks), and meaningful mitigation, monitoring of mitigations, change to mitigation where required, and eventual achievement of levels of mitigation and compensation to convert "significant impact" to those of "less than significant" levels. Current practices and considerations in the Final Draft CER do not appear adequate and sufficient to progress from conceptual to preliminary engineering and assessments along with full implementation of the overall Project.