APPENDIX I

STABILITY ANALYSES

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APPENDIX I

STABILITY ANALYSES

I.1. GENERAL

This appendix presents technical information related to the slope stability analyses that were performed as discussed in the Stability Analyses section of the main report.

I.2. MATERIAL SHEAR STRENGTH PROPERTIES

The following sections provide details on the shear strength models used for rock and bentonite clay materials in the analyses. Properties for all materials included in the analyses are shown in Table 3 of the main report text.

Parameters for all materials included in the models correspond to long-term (i.e., drained), effective stress conditions because the observed landslide movement took place over several months. The terrace soil deposits were modeled using Mohr-Coulomb parameters. The following sections describe our approach for selecting rock and bentonite shear strength parameters.

I.2.1. Generalized Hoek-Brown Strength Criterion

We modeled rock in cross sections K-K' and L-L' using the generalized Hoek-Brown strength criterion, the same strength model used in the stability analysis in our Final Report. However, we modified the geologic strength index, intact rock parameter, and unconfined compression strength factors to represent data collected in borings B-10 and B-11. A description of the Generalized Hoek-Brown model and its parameters are presented in our Final Report. Figure I-1 presents the Hoek-Brown strength criterion model used for the Altamira Shale in the analyses.

I.2.2. User-Defined Nonlinear Strength Model for Bentonite Clay

The same strength model used in the stability analysis in our Final Report was applied for the forward analysis of cross sections K-K' and L-L'. Descriptions of strength behavior and model justification are also presented in our Final Report.

We assumed that drained conditions exist in the eastern flank because of the relatively slow moving nature of the November 2011 landslide, and that the residual shear strength conditions had been reached due to previous displacement to accommodate inter-layer slip during folding. We modeled the clay using a nonlinear envelope defined by a series of torsional ring shear tests, ASTM D7608 (ASTM, 2010), presented in our Final Report.

I.3. HYDROGEOLOGY

The influence of regional hydrogeology, porewater pressure, and hydrostatic forces in landsliding is disused in our Final Report and in the text of this addendum. In the model, we accounted for our groundwater observations at B-10 and B-11 and the effect of elevated porewater pressures acting on the failure plane. As discussed in our report, the VWPs show that multiple confined aquifers are present. However, we did not have evidence to suggest that the material within the failure plane is under flowing artesian conditions. Therefore, we modeled one piezometric surface as an unconfined aquifer as shown in Figures I-2 and I-3. Results of our forward analyses for selected sections are provided in Figures I-2 and I-3.

As done with the analyses in our Final Report, the phreatic correction feature in SLOPE/W was used to account for the curvature of the piezometric surface (i.e., non-vertical equipotential lines) near the slope surface.

I.4. LANDSLIDE GEOMETRY AND MODE

The geomorphology near the landslide is described in our Final Report. For the slope stability analyses, we defined the surface geometry based on the 2011 site specific survey contours for after-sliding conditions. The subsurface geometry was based on interpretation of the subsurface and surface geologic data collected by Shannon &Wilson and City representatives.

I.5. SEISMIC SLOPE STABILITY ANALYSES

We performed seismic analyses according to "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California" as discussed in our Final Report (Blake et al., 2002). Our seismic stability analyses are part of the calculation package included with this appendix. The FS for these pseudo-static, limit equilibrium analyses are presented in Table 4 of the main report and the calculation package attached with this appendix.

Note that if the static FS is approximately 1.0, i.e., the slope is marginally stable, an additional driving force such as earthquake shaking would likely result in unbounded, large-scale movements greater than the predicted coseismic deformations. Because our analyses indicated that the slope is marginally stable under current (static) conditions, we assume that design earthquake shaking will result in ground deformation of a similar magnitude to that observed during the 2011 Landslide.

For the seismic analyses, we included discontinuities in our model and considered various cases where the groundwater level in the discontinuity ranged above or below the unconfined, static piezometric surface measured at the site. For each discontinuity groundwater lever considered, the slope failed the screen analysis, indicating that unstable conditions would occur under design seismic loading.

I.6. REFERENCES

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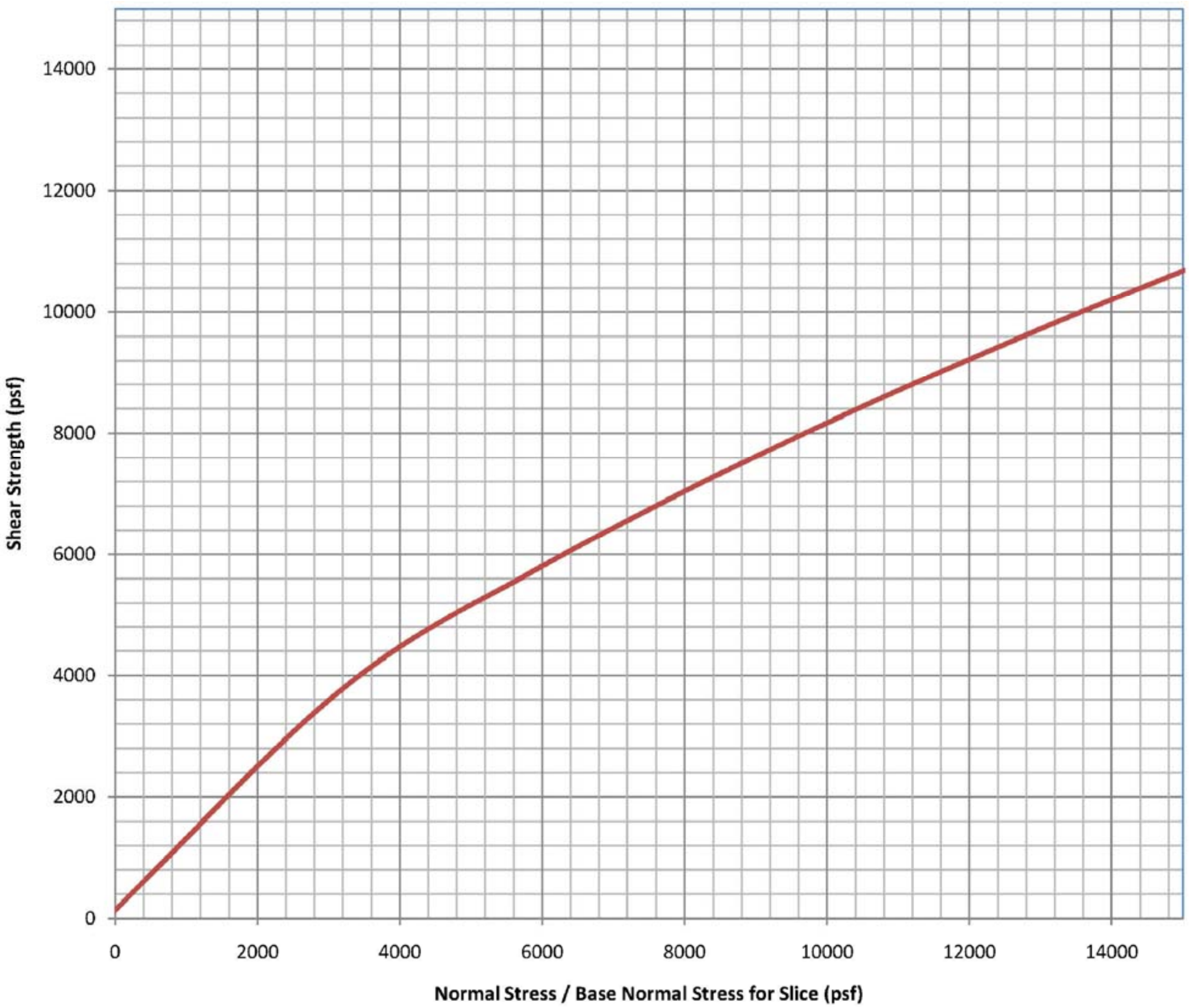

NOTE

Based on Hoek-Brown Strength Criterion as Presented in:

- Hoek, E., and Brown, E.T., 1997, Practical estimates of rock mass strength: International Journal of Rock Mechanics and Mining Sciences & Geomechanics Abstracts, v. 35, no. 8, p. 1165-1186. , and
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White Point Landslide
San Pedro District
Los Angeles, California

HOEK-BROWN STRENGTH CRITERION FOR ALTAMIRA SHALE

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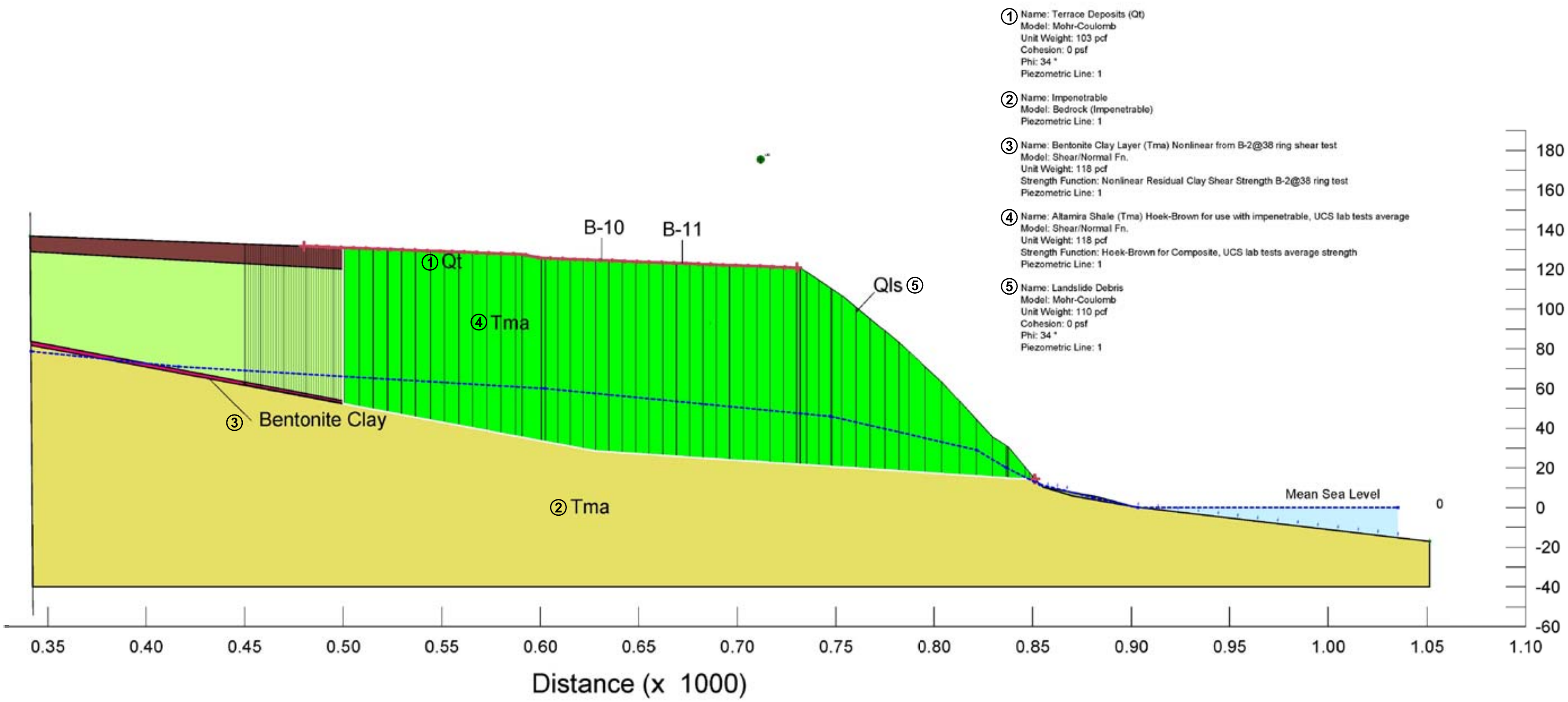
FIG. I-1



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SLOPE/W MODEL FOR FORWARD ANALYSES CROSS SECTION L-L'

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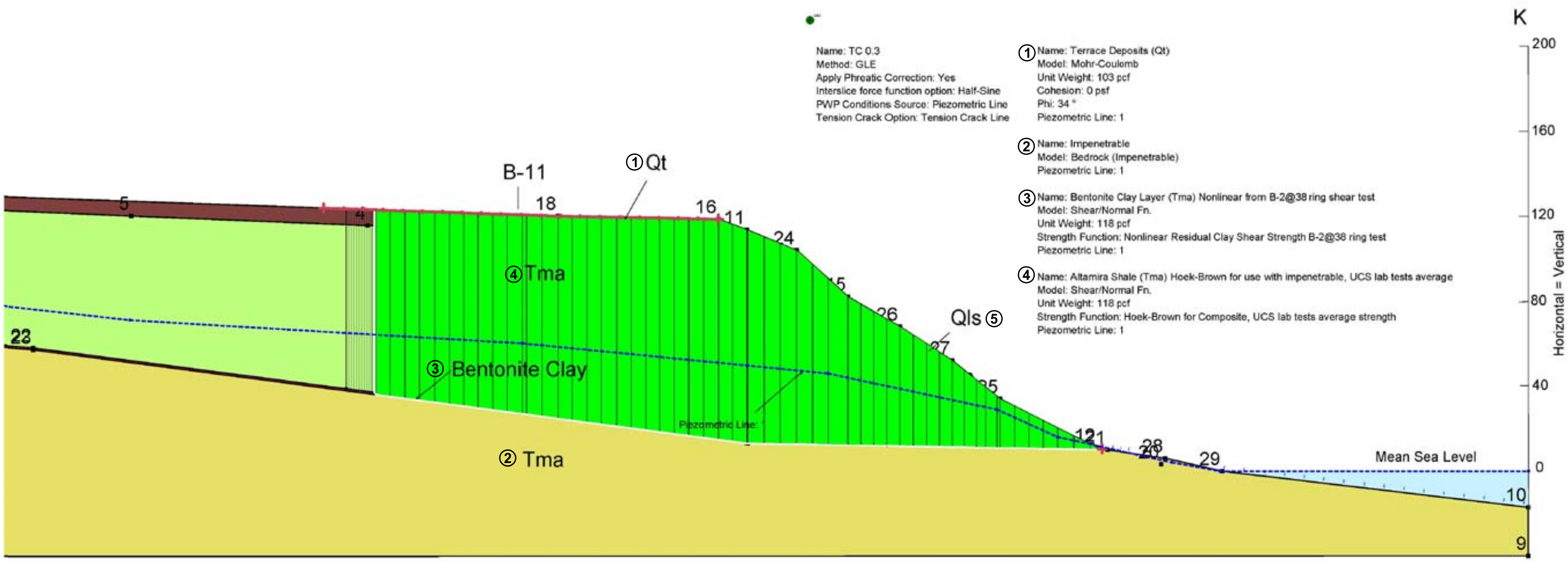
FIG. I-2



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SLOPE/W MODEL FOR FORWARD ANALYSES CROSS SECTION K-K'

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FIG. I-3