Appendix E: Geological Supporting Information

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Wallace Kuhl
& ASSOCIATES

Geotechnical Engineering Report

GREEN ISLAND ROAD LOGISTICS CENTER

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Geotechnical Engineering Report
GREEN ISLAND ROAD LOGISTICS CENTER
Green Island Road
American Canyon, California
WKA No. 12883.01
November 10, 2020

INTRODUCTION

As authorized, we have completed a geotechnical engineering study for the proposed Green Island Road Logistics Center to be constructed north of Green Island Road in American Canyon, California (see Figure 1). The purposes of this study have been to explore the existing site, soil, and groundwater conditions across the site, and to provide geotechnical engineering conclusions and recommendations for use by the other members of the design team in design and construction of the proposed industrial development. This report presents the results of our study.

Scope of Services

Our scope of services for this project has included the following tasks:

1. Site reconnaissance;
2. Review of previous geotechnical studies that included the project site and for a property within the near vicinity of the site;
3. Review of United States Geological Survey (USGS) topographic maps, historical aerial photographs and available groundwater information;
4. Subsurface explorations, including the drilling and sampling of 52 borings to depths ranging from approximately 10 to 31½ feet below the existing site grades;
5. Collection of representative bulk samples of near-surface soils;
6. Laboratory testing of selected soil samples;
7. Engineering analyses; and,
8. Preparation this report.

Figures and Attachments

This report contains a Vicinity Map as Figure 1; a Site Plan showing the approximate boring exploration locations as Figure 2; and, Logs of Soil Borings as Figures 3 through 54. An explanation of the symbols and classification system used on the logs is included as Figure 55. Appendix A contains general information regarding project concepts, exploratory methods used during our field investigation, and laboratory test results not included on the boring logs. Appendix B contains *Guide Earthwork Specifications* that may be used in preparation of contract documents.

Proposed Development

Based on conversations with Mr. Joe Lavaich of Buzz Oats; review of *Preliminary Concept Site Plan* prepared by RMW Architecture Interiors dated August 8, 2020, we understand the proposed logistics center will include the construction of six warehouse/office buildings identified as Buildings A through F. The warehouse/office structures will encompass building footprints ranging from about 188,300 square feet (sf) up to 715,500 sf. We anticipate that the structures will be constructed of concrete tilt-up panels with interior concrete slabs-on-grade floors. Below grade basements are not anticipated. Structural loads for the buildings are anticipated to be heavy on the order of 150 kips for interior column loads and about 5 kips per lineal foot for exterior walls. Associated improvements will include the construction of underground utilities, landscaping, exterior flatwork, below-grade loading docks, retaining walls, asphalt pavements, and two storm water retention ponds. We anticipate that the storm water retention ponds will be about 5 to 10 feet in depth with unlined side slope gradients of three horizontal to one vertical (3H:1V) or flatter.

Grading plans were not available; however based on the existing site topography, we anticipate maximum excavations and fills on the order of one to five feet for development of the logistics center.

Supplemental Information

Supplemental information used in the preparation of this report included review of the following reports prepared by others and Wallace-Kuhl and Associates (WKA) for studies that included the site and for properties near the project site:

- Cornerstone Earth Group, 2017, *Geologic Fault Investigation New Sanitary Sewer Main* prepared for the Giovanni Property which is located within the site, Project No. 582-4-1;
- Wallace-Kuhl & Associates, Inc., 1994, *Geotechnical Engineering Report* prepared for Lot 5 of the Green Island Industrial Park, WKA No. 2652.02, March 1994;
- Wallace-Kuhl & Associates, Inc., 1994, *Geotechnical Engineering Report* prepared for the Graton Beverage Facility located on Lot 15 of the Green Island Industrial Park, WKA No. 2651.01, April 1994;

- Wallace-Kuhl & Associates, Inc., 1995, *Geotechnical Engineering Report* prepared for Spec Buildings 59 and 188 of the Green Island Industrial Park, WKA No. 2651.04, May 1995;
- Wallace-Kuhl & Associates, Inc., 1996, *Preliminary Geotechnical Engineering Report* prepared for Mezzetta Court Spec Building 197 of the Green Island Industrial Park, WKA No. 3142.01, February 1996;
- Wallace-Kuhl & Associates, Inc., 1996, *Geotechnical Engineering Report* prepared for Mezzetta Court Spec Building 90 of the Green Island Industrial Park, WKA No. 3296.01, September 1996;
- Wallace-Kuhl & Associates, Inc., 1998, *Geotechnical Engineering Report* prepared for Spec Building 102 of the Green Island Industrial Park, WKA No. 3744.01, July 1998;
- Wallace-Kuhl & Associates, Inc., 1998, *Geotechnical Engineering Report* prepared for Spec Building 91 on Lot 3 of the Green Island Industrial Park, WKA No. 3745.01, July 1998;
- Wallace-Kuhl & Associates, Inc. 2000, *Geotechnical Engineering Report* prepared for the Sutter Home Distribution Center Expansion located within the Green Island Industrial Park, WKA No. 4408.01, May 2000;
- Wallace-Kuhl & Associates, Inc, 2005, *Geotechnical Engineering Report* prepared for the Yandell Warehouse located within the Green Island Industrial Park, WKA No. 6445.02, March 2005; and,
- Wallace-Kuhl & Associates, Inc., 2008, *Geotechnical Engineering Report* prepared for the Biagi Brothers Wine Distribution Center located adjacent to the west of the subject property, WKA No. 8005.01, March 2008.

FINDINGS

Site Description

The project site is located north of Green Island Road and about a quarter mile west of Highway 29 in American Canyon, California and encompasses a total area of approximately 208-acres. The site occupies land identified as Napa County Assessor Parcel Numbers 057-130-034, -036, and -038. The site is bounded to the north by commercial and industrial developments, beyond which is the Napa County Airport; to the east by Union Pacific Railroad lines, beyond which is commercial and industrial developments; to the south by commercial and industrial developments, Green Island Road, and Union Pacific Railroad lines; and, to the east by commercial and industrial developments.

The topography of the site is sloping gently from southeast to northwest. The highest portion of the site, the farthest southeast area, has a maximum surface elevation of about +55 feet msl and the lowest portion of the site, the farthest northwest area, has a surface elevation of about +25 feet msl, based on topographic information shown on the *USGS 7.5-Minute Topographic Map of the Cuttings Wharf Quadrangle, California*, dated 2018.

A drainage swale, dry at the time of our explorations, runs from north to south within the western portion of the site, a portion of this drainage swale is diverted through a large diameter pipe that is covered by soil to allow vehicle access between the two sides of the swale. The swale is approximately two feet deep.

At the time of our field explorations, performed between September 21, 2020, and October 2, 2020, the site was vacant pasture land covered with grassy low-lying vegetation.

Historical Aerial Photographic Review

We reviewed historical aerial photographs of the site available from our files and the Google Earth software. Available photographs were taken in the years 1948, 1958, 1968, 1982, 1993 and 2002 through 2020.

Review of the photographs taken from 1948 through 2020 show the site was an agricultural fallow area used for livestock grazing. A seasonal creek located in the northwest portion of the site trends in a northwest-west direction. There are three east-west trending smaller tributaries to the creek along the northern property line and about 200 and 350 feet south of the northern property line, and a north-south trending tributary approximately 1400 feet east of the western property line. Seasonal wetland depressions are observed throughout the site throughout each year. The locations of observed wetlands, are shown on the Site Plan, Figure 2.

General Site Geology

The site is located in the southern portion of Napa Valley which is characterized as a relatively large north-west tending alluvial valley located within the Northern California Coast Range geomorphic province. Various authors have mapped the local geology of the site area. These maps differ in scale and detail but agree that a majority site as being underlain by the late Pleistocene to Holocene age alluvial fan deposits (Qf) [Bezore, et al., 2002], which are bound to the north and west by older geologic units including Late Pleistocene fan deposits (Qpf) and early to middle Pleistocene alluvial fan and terrace deposits (Qoa). The Qf unit is relatively younger than and was deposited over the Qpf and Qoa units. The Qf unit is described as gently sloping, fan-shaped alluvial surfaces. Sediments include sand, gravel, silt and clays that are moderately to poorly sorted and moderately to poorly bedded. The Qpf unit is described as gently sloping, fan-shaped alluvial surfaces where late Pleistocene age is indicated by slight dissection and/or the development of alfisols. The Qoa unit is described as moderately to deeply dissected alluvial deposits capped by alfisols, ultisols, or soils containing a silica or calcic hardpan.

The western portion of the site is mapped as Huichica Formation (QTh), part of the Sonoma Assemblage, of early Pleistocene and Pliocene age. The formations is described as fluvial gravel, sand, silt and clay.

Subsurface Soil Conditions

A total of 52 borings (D1 – D52) were performed at the site between September 21, 2020, and October 2, 2020. The approximate locations of the explorations are shown on the Site Plan, Figure 2. We also collected bulk samples of surficial soils at three locations within proposed pavement areas (B1 – B3).

The borings reveal the surface and near-surface soil conditions at the site consist of moderate to high plastic, stiff to hard, lean clay to depths about five to 10 feet below the existing ground surface (bgs). Underlying the upper clay soil were interbedded silty lean clays, sandy lean clays, clayey sands and silty sands to the maximum depth explored of 51½ feet bgs. The surface and near-surface soils across the site were observed to be relatively loose with frequent desiccation cracks to depths of about 6 to 18 inches.

The soil conditions encountered in the explorations for this study are consistent with soil conditions encountered in previous studies referenced in this report. For specific information regarding the soil conditions at a specific exploration location, please refer to the Logs of Soil Borings, Figures 3 through 54.

Groundwater

Groundwater was encountered in six of the 52 borings performed during our field work, at depths ranging from approximately 12 to 20 feet bsg. The borings were backfilled with neat-cement grout as required by the Napa County upon completion of the subsurface exploration program. Please note that the borings may not have been left open long enough for groundwater to reach static equilibrium.

To supplement the groundwater data obtained from the borings, we reviewed available groundwater information at the California Department of Water Resources (DWR) website. The DWR periodically monitors groundwater levels in wells across the state. Their website shows one well located near the sites border with the commercial and industrial development near the middle of the site. The well is identified as Well No. 04N04W13E001M with ground surface elevations of about +44 feet msl, which is similar to the elevation at the project site. Groundwater data for this well was recorded from March 25, 1930 to at least April 18, 1962. Data shows the highest recorded groundwater depth at the well was about +33.5 feet msl (about 10.5 feet below the ground surface) at the well on March 28, 1952. The lowest recorded groundwater elevation was about +21.4 feet msl at the well (about 22.6 feet below the ground surface) at the well on September 6, 1961.

These groundwater conditions are consistent with the groundwater levels observed during the field explorations performed for this study. Similar groundwater can be expected during the proposed construction.

CONCLUSIONS

Fault Rupture

According to mapping by the California Geologic Survey, the western portion of the project site lies within a mapped Earthquake Fault Zone and Seismic Hazard Zone. In 2017, a geologic fault investigation was performed by the Cornerstone Earth Group to locate the potential surface trace of the West Napa Fault as part of a proposed sewer main alignment project. The investigation included a review of previously completed investigations performed by other consultants to locate and/or "clear" sites of fault surface traces, drilling an array of borings, and excavating three observation trenches along the suspected fault alignment. The West Napa Fault was confirmed as trending through the western portion of the project site. The approximate fault trace is shown on the Site Plan, Figure 2.

Further evaluation for the presence or absence of the West Napa Fault or the potential for fault rupture is beyond our scope of services for this project. The referenced Cornerstone Earth Group report should be submitted to the local building official to satisfy the requirements of the Alquist Priolo Fault Hazard Zone Act, if required.

Soil Expansion Potential

Laboratory testing of three representative near-surface clay soil samples collected from the proposed building footprint revealed these soils possess moderate to high plasticity when tested in accordance with the American Society of Testing and Materials (ASTM) D4318 test method (see Figures A1 through A4). Laboratory test results of the five near-surface clay soil samples also revealed these soils possess Expansion Indices ranging from 33 to 108, equivalent to a "*low to high*" expansion potential when tested in accordance with the ASTM D4829 test method (see Figures A5 through A9). These results are consistent with testing performed on nearby sites.

Based on these test results, the test results from previous studies referenced in this report and our local experience, the near-surface clay soils at the site are considered capable of exerting significant expansion pressures on building foundations, interior floor slabs, exterior flatwork, and pavements. In our opinion, the near surface soils supporting slab on grade concrete will require removal and replacement with imported, non-expansive soils (Expansion Index < 20), or chemical amendment of the clay soils (i.e., lime-treatment) to reduce the expansion potential of the near surface clays.

Liquefaction Potential

A common secondary hazard as a result of strong ground shaking is the potential for soil liquefaction and subsidence. Liquefaction describes a phenomenon in which saturated soil in the upper 40 to 50 feet of subgrade loses shear strength and deforms as a result of increased pore water pressure induced by strong ground shaking. As the excess pore pressures dissipate following an earthquake, volume changes within the liquefied soil layer will occur, which can manifest as ground surface settlement, ground rupture (sand boils or ground cracking), and lateral slope displacement (commonly referred to as lateral spreading). Soils most susceptible to liquefaction are saturated, loose to medium dense sand and silt or clay with plasticity indices (PI) less than 12 and moisture contents greater than 85 percent of the soils liquid limit (LL) (Bray and Sancio, 2006).

A preliminary screening suggests that a majority of the subsurface soils encountered during our investigation were generally too high in plasticity (i.e., clay) and/or too stiff/dense to be susceptible to liquefaction. Strata of silty sand, clayey sand and poorly graded “clean” sand, however, was encountered at borings D21 and D32 at depths of about 30 feet below the ground surface (bgs). Accordingly, a liquefaction analysis was performed in accordance with an approach outlined by the National Center for Earthquake Engineering Research (NCEER) and summarized by Youd, et al (2001). A groundwater depth of 12 feet (corresponding to the estimated high groundwater depth encountered), a peak ground acceleration adjusted for site class effects (PGAM) of 0.98g based on a return period of 2,475 years (2% probability of exceedance in 50 years), and a mode magnitude earthquake of 6.9 was used in the analysis. The mode magnitude earthquake was determined using the using the 2014 USGS National Seismic Hazard Mapping Project (NSHMP) Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation website.

The results of our analysis suggest that strata of silty and poorly graded sands with penetration resistance (N-value) less than 30 blows per foot (bpf) and clayey sands with N-values less than about 20 bpf are potentially susceptible to liquefaction. Using empirical procedures (Ishihara and Yoshimine 1992), the seismically-induced settlement at the top of the liquefiable sand layers is estimated to range from about ¾ inch to 2 inches. This liquefaction is not expected to be widespread since the potentially liquefiable soil layers appear to be discontinuous and confined. Furthermore, ground surface settlement should be significantly less due to bridging effects within the overlying soil. Typical practice for these conditions is to assume a localized differential settlement on the order of one-half of the total estimated settlement to account for soil variability and engineering uncertainties. In our opinion, the anticipated maximum differential settlement due to liquefaction should be on the order of 1 inch over a horizontal distance of 100 feet.

The methods used to estimate liquefaction settlement assume that there is a sufficient cap of non-liquefiable material to prevent ground cracking or sand boils. For ground rupture to occur, the excess pore water pressure within the liquefiable soil layer needs to be great enough to break through the overlying non-liquefiable soil layer, which could cause additional ground deformation and settlement. The work of Youd and Garris (1995) indicates that the potentially liquefiable soils are overlain by a sufficient cap of non-liquefiable soil to prevent ground rupture; therefore the above total settlement estimates appear to be reasonable.

<u>Seismic Site Class</u>

Based on the soil conditions encountered at the exploration locations and our experience with similar soil conditions in the vicinity of the site, it is our opinion that the soils at the project site are not vulnerable to potential failure or collapse under seismic loading and can be designated as Site Class D in determining seismic design forces for this project in accordance with Section 1613A.3.2 of the 2013 CBC, which references Chapter 20 of American Society of Civil Engineers (ASCE) Standard 7-10. While relatively thin discrete layers of granular soils have been determined to be liquefiable, these layers appear to be discontinuous and confined. Furthermore, the estimated cumulative seismically induced settlement is relatively small (about 1 inch or less). In our opinion, the stiff clay soil profile under the site will control the seismic behavior of the soil in a seismic event.

<u>2019 CBC/ASCE 7-16 Seismic Design Criteria</u>

The 2019 CBC references the ASCE Standard 7-16 for seismic design. The following seismic parameters provided in Table 1 were determined based on the site latitude and longitude and the web interface developed by the Structural Engineers Association of California (SEAOC) and the California Office of Statewide Health Planning and Development (OSHPD) (https://seismicmaps.org).

Since S_1 is greater than 0.2 g, a ground motion hazard analysis will be required (Section 11.4.8 of the 2019 CBC) in accordance with Section 21.2 of ASCE 7-16. However, if Exception Note No. 2 in Section 11.4.8 of ASCE 7-16 applies (specifically if $T \leq 1.5 \times T_s$), the 2019 CBC coefficient values F_v, S_{M1}, and S_{D1} presented in Table 1 below are valid for this project. Further evaluation of the seismic design parameters presented in Table 1, including the seismic Site Class will be required during preparation of the design-level geotechnical report.

The seismic design parameters summarized below in Table 1 may be used for seismic design of the planned improvement at the site.

TABLE 1 – 2019 CBC/ASCE 7-16 SEISMIC DESIGN PARAMETERS

Latitude: 38.1993° N Longitude: 121.2670° W	ASCE 7-16 Table/Figure	2019 CBC Figure/Section/Table	Factor/ Coefficie	2019 CBC
0.2-second Period MCE	Figure 22-1	Figure: 1613.2.1(1)	S_S	2.156 g
1.0 second Period MCE_R	Figure 22-2	Figure: 113.2.1(2)	S_1	0.780 g
Soil Class	Table 20.3-1	Sections: 1613.2.2	Site Class	D
Site Coefficient	Table 11.4-1	Tables: 1613.2.3 (1)	F_a	1.000
Site Coefficient	Table 11.4-2	Tables: 1613.2.3(2)	F_v	1.700*
Adjusted MCE Spectral Response Parameters	Equation 11.4-1	Equations: 16-36	S_{MS}	2.156 g
	Equation 11.4-2	Equations: 16-37	S_{M1}	1.326 g*
Design Spectral Acceleration Parameters	Equation 11.4-3	Equations: 16-38	S_{DS}	1.437 g
	Equation 11.4-4	Equations: 16-39	S_{D1}	0.884 g*
Seismic Design Category	Table 11.6-1	Tables: 1613.2.5(1)	Risk Category I to IV	D
	Table 11.6-2	Tables: 1613.2.5(2)	Risk Category I to IV	D

Notes: MCER = Risk-Targeted Maximum Considered Earthquake; g = gravity

* = The value is valid if the requirements in Exception Note No. 2 in Section 11.4.8 of ASCE 7-16 are met. If not, a ground motion hazard analysis will be required (Section 11.4.8 of the 2019 CBC) in accordance with Section 21.2 of ASCE 7-16

Bearing Capacity and Building Support

The subgrade soils encountered generally consist of stiff to hard, low compressible clay that should provide adequate support for the anticipated structural loading provided the recommendations in this report are followed. The surface and near-surface soils across the site were relatively loose and had frequent desiccation cracks ranging from about 6 to 18 inches in depth. Accordingly, these soils should be deep ripped or scarified, moisture conditioned, and compacted in-place prior construction and/or placement of engineered fill.

Groundwater

Groundwater was encountered in six of the 52 borings drilled during our field explorations, at depths ranging from approximately 12 to 20 feet below existing site grades. Review of available groundwater data revealed groundwater depths at the site likely fluctuated from about 10½ to 22 feet bgs.

Groundwater levels at the site should be expected to fluctuate throughout the year based on variations in seasonal precipitation, time of year, tidal fluctuations, local irrigation practices, and the proximity to drainage canals/ditches.

Based on explorations performed at the site and available groundwater data, we anticipate excavations extending below 10 feet below the ground surface may encounter groundwater and require dewatering (depending on the time of year).

Seasonal Water

During the wet season, infiltrating surface runoff water will create a saturated surface condition due to the relatively low permeability of the near-surface soils. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soil, intended for use as engineered fill, will require a prolonged period of dry weather and/or considerable aeration to reach a moisture content that allows achieving the required compaction. This should be considered in the construction schedule for the project.

Pavement Subgrade Quality

Laboratory test results performed on three representative bulk samples of near-surface clay soils from proposed pavement areas of the site revealed these soils will provide poor support characteristics for pavements. Accordingly, relatively thick pavement sections are required to compensate for the lower support characteristics of these soils. Laboratory test results revealed the clay soil possess a Resistance ("R") value of 5 when tested in accordance with California Test 301 (see Figures A10 through A12). Based on our findings and previous experience in the project area, it is our opinion an R-value of 5 is appropriate for design of pavements at the site supported on untreated subgrade soil.

In lieu of supporting pavement directly on expansive clay, pavement sections are also presented assuming the upper 12 inches of subgrade soil is amended by mixing it with lime (lime-treatment). This procedure strengthens and reduces the plasticity/expansion characteristics of the treated clay and, thus, tends to reduce future maintenance and associated costs. Furthermore, the required aggregate base section is thinner for pavement supported on lime treated soil, thus reducing the relative cost of the lime-treatment.

The performance of lime stabilized soils is dependent on uniform mixing of the quicklime into the subgrade soils, and providing a proper curing period following compaction. An experienced soil stabilization contractor, combined with a comprehensive quality control program, is essential to achieve the best results with lime stabilized soils.

Representative near-surface clay soils from proposed pavement areas of the site were mixed with four percent dolomitic quicklime and subjected to an R-value test. Laboratory test results revealed the treated clay soils possess an R-value of 82, 84, and 84 when tested in accordance with California Test 301 (see Figures A10 through A12). Based on the Caltrans Highway Design Manual, a maximum R-value of 50 should be used for design of pavements to be supported on a treated subgrade. Therefore, an R-value of 50 is appropriate for design of pavements at the site supported on treated surface and near-surface sand lean clay soils. Additional recommendations regarding lime-treatment of the pavement subgrade soils are provided in the Pavement Design section of this report.

Excavation Conditions

The surface and near-surface soils at the site should be readily excavated using conventional earthmoving and trenching equipment. Shallow excavations (less than 5-feet deep) in the clay encountered should stand vertically for a period long enough for typical foundation and utility, unless they become wet or are disturbed. The sand encountered, however, is cohesionless and may cave and/or slough soon after it is exposed in the excavation. Where encountered, the contractor should be prepared to brace or shore the excavations, as necessary.

Temporarily sloped excavations and shored excavations less than 20 feet in depth should be constructed in accordance with federal, local and OSHA standards (29 CFR Part 1926) under the guidance of the Contractors qualified “competent person.” For preliminary evaluation, the silts and clays encountered would classify as Cal-OSHA Type B soil, while the sands would classify as Type C soils. In no case should the information provided be interpreted to mean that Wallace-Kuhl & Associates is assuming responsibility for site safety or the Contractor's activities.

Excavated materials should not be stockpiled directly adjacent to an open excavation to prevent surcharge loading of the excavation sidewalls. Excessive truck and equipment traffic should be avoided near excavations. If material is stored or heavy equipment is stationed and/or operated near an excavation, a shoring system must be designed to resist the additional pressure due to the superimposed loads.

Soil Suitability for Use in Fill Construction

The existing on-site soils, including the fill soils, are considered suitable for use as engineered fill, if they do not contain significant quantities of organics, rubble and deleterious debris, and are at a proper moisture content to achieve the desired degree of compaction. Organically laden topsoil should not be reused as engineered fill.

During the wet season, infiltrating surface runoff water will create a saturated surface condition due to the relatively low permeability of the near-surface soils. It is probable that grading operations attempted following the onset of winter rains and prior to prolonged drying periods will be hampered by high soil moisture contents. Such soil, intended for use as engineered fill, will require a prolonged period of dry weather and/or considerable aeration to reach a moisture content that allows achieving the required compaction. This should be considered in the construction schedule for the project.

Permeability Characteristics of Near-Surface Soils

The soils encountered in the area of the proposed detention ponds (borings D10, D11, D12, D18, D37, D43, D44, and D45) generally consisted of moderate to high plastic clay to depths ranging from about five to greater than 10 feet bgs, followed by silty and clayey sand. To assist in determining the permeability characteristics of the clays underlying the site, two laboratory hydraulic conductivity (ASTM D5084) tests were performed on relatively undisturbed soil samples obtained from borings D12 and D44. The results showed that the soil had permeability rates of about 9.8×10^{-9} and 1.7×10^{-6} centimeters per second (cm/sec.). The results are presented in Figures A13 and A14. Based on our previous experience, when compacted the permeability of these clays could be expected to be 1×10^{-7} cm/sec. or slower. Based on various publications (Hazen[1], Aryani[2] and Das[3]) and previous experience, for preliminary design a

[1] Allen Hazen, *Some Physical Properties of Sand and Gravels, with Special Reference to their Use in Filtration*, (Massachusetts: State Board of Health, 1892).

permeability rate of between $1x10^{-4}$ and $1x10^{-5}$ cm/sec can be anticipated for the silty and clayey sands encountered, respectively.

Depending on their final depths, we anticipate the ponds may need to be lined with clay to improve their "water-holding" capacity. The liner, if constructed, should be designed by the project Civil Engineer. As previously discussed, the native clays are moderate to highly plastic and may exhibit significant shrinking and cracking during warm weather conditions when the retention ponds are dry. When the ponds are again filled, the infiltration characteristics of the ponds could be affected for a period of time. If the native clay or a clay liner will be exposed within the ponds, consideration should be given to overlying the clays with a 12 to 18 inch thick layer of low plasticity silt or sand to reduce moisture fluctuations or by reducing the plasticity of the clay by intermixing or blending the soils with a low plasticity soils. If blending is considered, laboratory tests should be performed to evaluate the permeability of the mixed soil. If requested, WKA can provide additional criteria for either of these alternatives.

<u>Soil Corrosion Potential</u>

The samples of representative, near-surface clay soil was submitted to Sunland Analytical Lab of Rancho Cordova, California for laboratory testing to determine minimum resistivity, pH, and chloride and sulfate concentrations to help evaluate the potential for corrosive attack upon reinforced concrete and buried metal. The results of the corrosivity testing are summarized in Table 2; copies of the corrosion test reports are presented in Figures A15 through A26.

Table 2a: Corrosion Test Results

Analyte	Test Method	D6 (0'-5')	D14 (0'-5')	D22 (0'-5')
pH	CA DOT 643 Modified*	6.48	6.89	5.41
Minimum Resistivity	CA DOT 643 Modified*	540 Ω-cm	220 Ω-cm	880 Ω-cm
Chloride	CA DOT 422	323.0 ppm	758.8 ppm	61.6 ppm
Sulfate	CA DOT 417	167.4 ppm	821.2 ppm	185.2 ppm
Sulfate-SO4	ASTM D-516m	179.4 mg/kg	783.1 mg/kg	194.2 mg/kg

[2] Cyrus Aryani, *Applied Soil Mechanics and Foundation Engineering, Volume 1* (California State University, Sacramento, Aryani, 2004)
[3] Braja M. Das, *Fundamentals of Geotechnical Engineering, Second Edition* (Ontario: Thomas Learning, Stenquist, 2005).

Table 2b: Corrosion Test Results (cont.)

Analyte	Test Method	D33 (0'-5')	D40 (0'-5')	D49 (0'-5')
pH	CA DOT 643 Modified*	5.78	6.77	6.72
Minimum Resistivity	CA DOT 643 Modified*	380 Ω-cm	640 Ω-cm	1,070 Ω-cm
Chloride	CA DOT 422	379.7 ppm	374.8 ppm	9.0 ppm
Sulfate	CA DOT 417	443.5 ppm	295.9 ppm	3.0 ppm
Sulfate-SO4	ASTM D-516m	493.8 mg/kg	313.5 ppm	3.1 ppm

Notes:
* = Small cell method CA DOT = California Department of Transportation
Ω-cm = Ohm-centimeters ppm = Parts per million

The California Department of Transportation Corrosion and Structural Concrete Field Investigation Branch, *Corrosion Guidelines*, Version 2.1, dated January 2015, considers a site to be corrosive to foundation elements if one or more of the following conditions exists for the representative soil and/or water samples taken: has a chloride concentration greater than or equal to 500 ppm, sulfate concentration greater than or equal to 2000 ppm, or the pH is 5.5 or less. Based on this criterion, areas of the on-site, near-surface clay soil are considered unusually corrosive to steel reinforcement properly embedded within PCC for the samples tested. The low resistivity may indicate a higher corrosion potential to metal in direct contact with soil. Based on the chloride concentration criterion, the surface and near-surface clay soils tested can lead to corrosion of steel reinforcement in concrete and steel structures by breaking down the protective layer of oxides normally present on the steel surface.

Caltrans indicates soil or water with a pH of 5.5 or less can react with the lime in concrete to form soluble reaction products that can more easily leach out of the concrete. This is of special concern where the sulfate concentrations in the soil are above 2,000 ppm, as the more brittle concrete (caused by the leaching of soluble reaction products) can be more susceptible to cracking which can allow for sulfate attack of the rebar reinforcement. Test results indicate a maximum sulfate concentration of 821.2 ppm at the site, which is well below the 2,000 ppm threshold. Acidic (low pH) conditions also can cause discoloration resulting in a yellowish or rust color distributed over the concrete surface. Considering the low sulfate concentrations, we do not anticipate the low pH of the soils to be a significant factor in development of the property.

Table 19.3.1.1 – Exposure Categories and Classes, of American Concrete Institute (ACI) 318-14, Section 19.3 – Concrete Design and Durability Requirements, as referenced in Section 1904.1 of the 2016 CBC, indicates the severity of sulfate exposure for the samples tested is Exposure Class S0 (water-soluble sulfate concentration in contact with concrete is low and

injurious sulfate attack is not a concern). The project structural engineer should evaluate the requirements of ACI 318-14 and determine their applicability to the site.

Wallace-Kuhl & Associates are not corrosion engineers. Therefore, if it is desired to further define the soil corrosion potential at the site a corrosion engineer should be consulted.

RECOMMENDATIONS

General

The recommendations presented below are appropriate for typical construction in the late spring through fall months. The on-site soils likely will be saturated by rainfall in the winter and early spring months, and will likely not be compactable without drying by aeration or chemical treatment. Should the construction schedule require work to continue during the wet months, additional recommendations can be provided, as conditions dictate.

Relative compaction should be based on the maximum dry density as determined in accordance with the ASTM D1557 Test Method.

Site preparation should be accomplished in accordance with the provisions of this report and the appended specifications. A representative of the Geotechnical Engineer should be present during all earthwork operations to evaluate compliance with the recommendations and the guide specifications included in this report. The Geotechnical Engineer of Record referenced herein should be considered the Geotechnical Engineer that is retained to provide geotechnical engineering observation and testing services during construction.

Site Clearing

Prior to grading, the planned construction areas should be cleared of all surface trash, rubble, and deleterious debris, if any, to expose firm and stable soils, as determined by the Geotechnical Engineer's representative. The area of removal should extend at least five feet beyond the edge of all exterior flatwork and pavements, where practical. Any rubble and debris, if encountered, should be removed from the site.

Any existing underground utilities designated to be removed or relocated from the site should include removal of all trench backfill and bedding materials. The resulting excavations should be restored with engineered fill placed and compacted in accordance with the recommendations included in this report.

Surface vegetation/organics and organically laden soil within construction areas should be stripped from the site. Debris from the stripping operations should not be used in general fill construction areas supporting concrete slabs, pavements or any other surface improvements. Discing of the organics into the surface soils may be a suitable alternate to stripping, depending on the condition and quantity of the organics at the time of grading. The decision to utilize discing in lieu of stripping should be made by the Geotechnical Engineer, or his representative, at the time of earthwork construction. Discing operations, if approved, should be observed by the Geotechnical Engineer's representative, and be continuous until the organics are adequately mixed into the surface soils to provide a compactable mixture of soil containing minor amounts of organic matter. Pockets or concentrations of organics will not be allowed.

Any depressions resulting from site clearing operations, as well as any loose, soft, disturbed, saturated, or organically contaminated soils, as identified by the Geotechnical Engineer's representative, should be cleaned out to firm, undisturbed soils and backfilled with engineered fill in accordance with the recommendations of this report. It is important that the Geotechnical Engineer's representative be present during clearing operations to verify adequate removal of any surface and subsurface items, as well as the proper backfilling of resulting excavations.

<u>Site Preparation</u>

The near-surface soil underlying the site consists of moderate to highly plastic clay that, based on our experience and testing, can exhibit significant expansion characteristics. The following presents recommendations for general subgrade preparation. Subsequent sections should be reviewed for specific or supplemental recommendations to address the expansive soil conditions.

The surface and near-surface soils encountered during our investigation were relatively loose with frequent desiccation cracks. Furthermore, we anticipate that clearing operations will likely cause additional disturbance to the upper soils. Therefore, in all areas that will support concrete slabs, engineered fill or pavement, should be thoroughly scarified to a depth of at least 12 inches, brought to a uniform moisture content at least two percentage point above the optimum moisture content, and compacted to not less than 90 percent of the maximum dry density per ASTM D1557 specifications. In pavement areas, the relative compaction of the upper 6 inches of final soil subgrade should be increased to 95 percent of the maximum dry density.

The performance of pavement is critically dependent upon uniform and adequate compaction of the soil subgrade, as well as all engineered fill and utility trench backfill within the limits of the pavements. Final pavement subgrade preparation (i.e. scarification, moisture conditioning and compaction) should be performed after underground utility construction is completed and just prior to aggregate base placement.

Pavement subgrades should be stable and unyielding under heavy wheel loads of construction equipment. To help identify unstable subgrades within the pavement limits, a proof-roll should be performed with a fully-loaded, water truck on the exposed subgrades prior to placement of aggregate base. The proof-roll should be observed by the Geotechnical Engineer's representative.

If construction begins during the summer or fall, there is a potential that the surface clayey soils may be desiccated deeper than the recommended depth of scarification. Should this condition exist, the site should be continuously watered for a sufficient period of time to close the desiccation cracks.

The prepared subgrade soils should be protected from disturbance until covered by capillary break material or aggregate base. Disturbed subgrade soils may require additional processing and recompaction just prior to construction of these improvements, depending on the level of disturbance.

All subgrade preparation must be performed in the presence of the Geotechnical Engineer's representative who will evaluate the performance of the subgrade under compaction loads and identify any loose or unstable soil conditions that could require remediation. Construction bid documents should contain a unit price (price per cubic foot) for additional excavation due to unsuitable materials and replacement with engineered fill.

Engineered Fill Construction

Engineered fill consisting of on-site or import materials should be placed in lifts not exceeding six inches in compacted thickness, with each lift being thoroughly moisture conditioned to at least two percent above the optimum moisture content for clay soils and to the optimum moisture content for granular/silty soils (import fill materials). Soils should be uniformly compacted to at least 90 percent relative compaction.

On-site soils encountered in our explorations are considered suitable for use as engineered fill, provided they are at a workable moisture content to achieve required compaction, and do not contain rubbish, rubble, deleterious debris, and organics. However, clay soils should not be used in fills within the upper 12 inches of final subgrade for the building pad or exterior flatwork, unless the clay soils are lime-treated as described in the Lime Treatment Alternative section of this report.

Imported fill materials should be compactable, well-graded, granular soils with a Plasticity Index of 15 or less when tested in accordance with ASTM D4318; an Expansion Index of 20 or less when tested in accordance with ASTM D4829, and should not contain particles greater than three inches in maximum dimension. In addition, we recommend that the contractor supply a certification for any imported fill materials that designates the fill materials do not contain known contaminants per Department of Toxic Substances Control's guidelines for clean fill, and have corrosion characteristics within acceptable limits. Imported soils should be approved by the Geotechnical Engineer prior to being transported to the site.

All earthwork operations should be accomplished in accordance with the recommendations contained within this report and the *Guide Earthwork Specifications* provided in Appendix B. We recommend the Geotechnical Engineer's representative be present on a regular basis during all earthwork operations to observe and test the engineered fill and to verify compliance with the recommendations of this report and the project plans and specifications.

Cut and Fill Slopes

We anticipate that cut and fill slopes less than 10 feet in vertical height may be constructed to provide level building areas. In our professional opinion, fill slopes should be inclined no steeper than 2(h):1(v). This slope recommendation is based on our experience with similar conditions since no detailed slope stability analysis was performed to justify steeper slopes. Given this inclination, however, there is a modest risk that displacement, movement, and/or soil sloughing could occur in the event of strong seismic ground shaking. For the native soils and compacted fill conditions anticipated, we expect this movement to be relatively shallow, requiring limited cleanup and dressing to restore the slopes to their original condition. If this risk is unacceptable to the owner, the slopes should be flattened to 3(h):1(v) or flatter.

Since the proposed detention pond embankments will be immersed below water, the embankments should be constructed at an inclination of 3(h):1(v) or flatter. Like discussed above, some shallow seated displacement or movement should be anticipated in the event of strong seismic ground shaking. If this condition is unacceptable, the embankments should be flattened.

Paved interceptor drains should be provided along the tops of slopes where the tributary area flowing toward the slope has a drainage path greater than 40 feet, measured horizontally. The interceptor drains or terraces should be sloped to a suitable drainage device and disposed off-site well below the toe of the slope. Drop inlets and storm sewers should not be installed near the crests of slopes because leakage can result in maintenance problems or possible slope failure. The slopes should be inspected periodically for erosion, and if detected, repaired immediately. To reduce erosion and gulling, all disturbed areas should be planted with erosion-resistant vegetation suited to the area. As an alternative, jute netting or geotextile erosion control mats can be installed per the manufacturer's recommendations. Slopes should be over-built and cutback to design grades and inclinations.

Utility Trench Backfill

Utility trench backfill within structural areas (e.g. building, exterior flatwork, pavements, etc.) should be mechanically compacted as engineered fill in accordance with the following recommendations. Bedding and initial backfill around and over the pipe should conform to the pipe manufacturer's recommendations and applicable sections of the governing agency standards.

Based on explorations performed at the site and available groundwater data, we anticipate excavations extending below 10 feet below site grade may encounter groundwater (depending on the time of year) and may require dewatering. Where groundwater is encountered, the use of sumps, submersible pumps, deep wells or a well point system could be used as methods to lower the groundwater level. The dewatering method used will depend on the soil conditions, depth of the excavation and amount of groundwater present within the excavation. Dewatering, if required, should be the contractor's responsibility. The dewatering system should be designed and constructed by a dewatering contractor with local experience. We recommend the selected dewatering system lower the groundwater level to at least two feet below the bottom of the proposed excavations.

It is likely that materials excavated from trenches will be at elevated moisture contents and will require significant aeration or a period of drying to reach a compactable moisture content. We recommend bid documents contain a unit price for the removal and drying of saturated soils, or replacement with approved import soils.

We recommend that on-site soil be used as trench backfill, especially below the non-expansive or lime-treated material within the footprint of the interior concrete slabs. Utility trench backfill should be placed in maximum eight-inch lifts (compacted thickness), thoroughly moisture conditioned to at least two percent above the optimum moisture content, and mechanically compacted to at least 90 percent relative compaction. Within the upper 12 inches of final subgrade for the interior concrete slabs and exterior flatwork, trench backfill should consist of granular material placed and compacted as described in the Engineered Fill Construction section of this report, unless the lime-treatment alternative is selected. Where the top 12 inches of the interior concrete slabs and exterior flatwork areas consist of lime-treated soils, the upper 12 inches of trench backfill should consist of controlled density fill (CDF) or aggregate base compacted to at least 95 percent relative compaction. Within the upper six inches of untreated pavement subgrade soils and upper 12 inches of lime-treated pavement subgrade soils, compaction should be increased to at least 95 percent relative compaction at no less than two percent above the optimum moisture content.

We recommend that all underground utility trenches aligned nearly parallel with new foundations be at least three feet from the outer edge of foundations, wherever possible. Trenches should not encroach into the zone extending outward at a one horizontal to one vertical (1H:1V) inclination below the bottom of foundations. Additionally, trenches parallel to existing foundations should not remain open longer than 72 hours. The intent of these recommendations is to prevent loss of both lateral and vertical support of foundations, resulting in possible settlement.

Foundations

The proposed structures may be supported upon continuous and/or isolated spread foundations. Due to expansive soil considerations, the foundations should extend at least 18 inches below lowest adjacent soil grade. Lowest adjacent soil grade is defined as the grade upon which the capillary break material is placed or exterior soil grade, whichever is lower. Continuous foundations supporting should maintain minimum widths of 15; while isolated spread foundations should be at least 24 inches in plan dimension. Foundations should be continuous around the perimeter of the building to reduce moisture variations beneath the structures. If shrinkage cracks appear in the footing excavations, the excavations should be thoroughly moistened to close all cracks prior to placement of concrete.

Foundations bearing on undisturbed or compacted native soils, engineered fill, or a combination of those materials may be sized for maximum allowable "net" soil bearing pressure of 3,000 pounds per square foot (psf) for dead plus live load. A one-third increase in the allowable bearing pressure may be applied when considering short-term loading due to wind or seismic forces. The weight of the foundation concrete extending below lowest adjacent soil grade may be disregarded in sizing computations.

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on the foundation criteria discussed above and the assumed foundation loads, foundations are anticipated to experience a maximum total static settlement on the order of about ¾ to 1 inch, and differential settlement on the order of about ½ inch for 50 lineal feet or the shortest distance of the structure, whichever is less.

As discussed in the Liquefaction Potential section of the Conclusions, it is estimated that up to 1 inch of differential settlement over a horizontal distance of 100 feet could occur in the event of strong ground shaking (earthquake with a return period of 2,475 years or 2% probability of exceedance in 50 years).

All foundations be adequately reinforced to provide structural continuity, mitigate cracking and permit spanning of local soil irregularities. The structural engineer should determine final foundation reinforcing requirements.

Resistance to lateral foundation displacement may be computed using an allowable friction factor of 0.30, which may be multiplied by the effective vertical load on each foundation. Additional lateral resistance may be computed using an allowable passive earth pressure equivalent to a fluid pressure of 300 psf per foot of depth, acting against the vertical projection of the foundation. These two modes of resistance should not be added together unless the frictional component is reduced by 50 percent since full mobilization of the passive resistance requires some horizontal movement, effectively reducing the frictional resistance. We recommend that all foundation excavations be observed by the Geotechnical Engineer's representative prior to placement of reinforcement and concrete to verify firm bearing materials are exposed.

Interior Floor Slabs

Conventional floor slabs in combination with shallow spread foundations may be used for support of the proposed structures. As discussed in the Soil Expansion Potential section of the Conclusions, the near-surface soil underlying the site and anticipated clay fill can exhibit significant expansion characteristics.

The most direct method to reduce expansive soil risks and improve the subgrade conditions would be to support the proposed floor slabs on a layer of compacted, non-expansive fill. This procedure consists of placing at least 12 inches of non-expansive fill directly below the proposed floor slab system. The non-expansive fill should be moisture conditioned to at or above its optimum moisture content and compacted as engineered fill. Specific requirements for import fill are presented in the Engineered Fill section. The non-expansive soil pads could be prepared by removing and replacing the native clay, raising the building pads above existing site grade, or a combination of both. A capillary break or other slab support system placed directly below the floor slabs should not replace in whole or part the non-expansive fill layer. The zone of non-expansive soil should extend laterally at least 3 feet outside the perimeter of the structures. Prior to placement of the non-expansive fill, the moisture content of the underlying clay soil should be checked. If the soil moisture content is found to be less than1 percentage point above the optimum moisture content, the soil moisture content should be raised using liberal sprinkling, flooding or another suitable method. A representative of the Geotechnical Engineer should perform a field check of the soil moisture content and relative compaction prior to placement of the non-expansive fill.

A second approach (lime treatment) consists of mixing the upper 12 inches of subgrade soils within the proposed floor slab area with dolomitic or high calcium quick lime and compacting the soil as engineered fill. The subgrade preparation, spreading, mixing, compacting and lime type should meet the requirements outlined in Section 24 of the Caltrans Standard Specifications. The zone of lime-treated soil should extend laterally at least 3 feet outside the perimeter of the proposed structure. Based on our previous experience, 4½ pounds of quick lime per cubic foot of soil may be assumed for planning purposes. The lime treated subgrade soils should be compacted to at least 90 percent relative compaction.

At least 2 to 3 days prior to spreading or mixing the lime, the moisture content of the underlying, untreated clay soil should be checked. If the soil moisture content is found to be dry of optimum, the soil moisture content should be raised using liberal sprinkling, flooding or another suitable method. A representative of the Geotechnical Engineer should be on-site during treatment operations to document spreading, mixing and compaction operations and provide supplemental/revised recommendations, if warranted, based on the soil conditions observed.

Following lime treatment, the treated soil should be properly cured by continual sprinkling with water to keep the surface damp, combined with light rolling to keep the surface knitted together. We suggest that the subgrade soils be covered with Class 2 aggregate base or crushed rock within 2 to 3 days of lime treatment in an effort to reduce drying. Periodic sprinkling is still required to keep the surface damp. As an alternative, the treated soil could be cured as discussed in Section 24 of the Caltrans Standard Specifications.

Lime treatment increases the pH of the soil and may not promote plant growth. Accordingly, the landscape designer should be consulted during pre-construction to verify that future landscaping is suitable for lime treated soils. If the landscaping is not suitable, the lime-treated soils should be completely removed and replaced prior to planting.

The interior concrete slabs should be at least four inches thick, however, the project structural or civil engineer should determine final floor slab thickness, reinforcement and joint spacing. Temporary loads exerted during construction from vehicle traffic, cranes, forklifts, other construction equipment, rack loads, and storage of palletized construction materials, etc. should be considered in the design of the thickness and reinforcement of the interior concrete slabs-on-grade.

Provided the building pad is constructed in accordance with the recommendations included in this report (12 inches of non-expansive, granular soils or lime-treated clay soils, and six inches of aggregate base where applicable), a soil modulus of reaction (k_s) of 200 kips per cubic foot (kcf) or 115 pounds per cubic inch (pci) may be utilized for design of floor slabs subjected to vehicle/fork lift traffic or any other loading conditions described above.

Moisture Penetration Resistance

It is likely that the subgrade soils below floor slabs will become very moist or wet at some time during the life of the structures. This is a certainty if the subgrade soils are constructed during the wet season or poor drainage conditions exist adjacent to structures. For this reason, it should be assumed that interior floor slabs with moisture-sensitive floor coverings or coatings will require protection against moisture or moisture vapor penetration through the slabs.

Interior floor slabs for the planned buildings should, as a minimum, be underlain by a layer of free-draining crushed rock/gravel, serving as a deterrent to migration of capillary moisture. The crushed rock/gravel layer should be between four- and six-inches-thick and graded such that 100 percent passes a one-inch sieve and less than five percent passes a No. 4 sieve. Additional moisture protection may be provided by placing a vapor retarder membrane (at least 10-mils thick) directly over the crushed rock/gravel. The water vapor retarder membrane should meet or exceed the minimum specifications as outlined in ASTM E1745 and be installed in strict conformance with the manufacturer's recommendations. For warehouse portions of the proposed buildings that will not have moisture-sensitive floor coverings or coatings, the vapor retarder membrane should be placed directly over 4 to 6 inches of compacted aggregate base.

Floor slab construction practice over the past 30 years or more has included placement of a thin layer of dry sand or pea gravel over the vapor retarder membrane. The intent of the sand/pea gravel is to aid in the proper curing of the slab concrete. However, during the wet seasons moisture can become trapped in the sand or pea gravel, which can lead to excessive moisture vapor emissions from floor slabs. As a consequence, we consider use of the sand/pea gravel layer as optional. The concrete curing benefits should be weighed against efforts to reduce slab moisture vapor transmission.

It is emphasized that the crushed rock/grave and the vapor retarder membrane suggested above provides only a limited, first line of defense against soil-related moisture issues and will not "moisture proof" the slab. Nor do these measures provide an assurance that slab moisture transmission levels will be within tolerable levels to prevent damage to floor coverings or other building components. If increased protection against moisture vapor penetration is desired, a concrete moisture protection specialist should be consulted. The design team should consider all available measures for slab moisture protection. It is commonly accepted that maintaining the lowest practical water-cement ratio in the slab concrete is one of the most effective ways to reduce future moisture vapor penetration of the completed slabs.

<u>Retaining Walls</u>

All retaining walls or below grade walls for the buildings should be designed to resist the lateral soil pressures of the retained soils. Retaining walls that are fixed/restrained at the top should be capable of resisting an “at-rest” lateral soil pressure equal to an equivalent fluid pressure of 60 psf per foot of the wall height (fully drained conditions). Retaining walls above the estimated high groundwater level that will be allowed to slightly rotate about their base (unrestrained at the top or sides) should be capable of resisting an "active" lateral soil pressure equal to an equivalent fluid pressure of 40 psf per foot of wall height (fully drained conditions),

If structural elements, i.e., foundations, roadways, etc., encroach the one horizontal to one vertical (1H:1V) projection from the bottom of retaining walls, the retaining walls should account for surcharge loads resulting from those structural elements. Additionally, any below-grade retaining walls should also account for surcharge loads resulting from construction equipment, vehicles, palletized materials, etc. that encroach the one horizontal to one vertical (1H:1V) projection from the bottom of the below-grade retaining walls. Surcharge loading under the circumstances described above should be evaluated by the retaining wall designer on a case-by-case basis and be included in their design of the walls. The retaining wall designer should evaluate the surcharge load distribution, magnitude of the surcharge resultant force to be applied on the walls, and the location of where the resultant force should be applied on the walls. Surcharge loading on the retaining walls will depend on the specific surcharge load type (e.g. point load, distributed load, etc.) and distance away from the retaining walls.

Retaining wall or below grade walls should be fully drained to prevent the build-up of hydrostatic pressures behind the wall. Retaining walls should be provided with a drainage blanket of Class 2 permeable material, Caltrans Standard Specification, Section 68-2.02F(3), at least one foot wide extending from the base of wall to within one foot of the top of the wall. The top foot above the drainage layer should consist of compacted on-site or imported engineered fill materials, unless covered by a concrete slab or pavement. Weep holes or perforated rigid pipe, as appropriate, should be provided at the base of the wall to collect accumulated water. Drainpipes, if used, should slope to discharge at no less than a one percent fall to suitable drainage facilities. Open-graded ½ to ¾ inch crushed rock may be used in lieu of the Class 2 permeable material provided the rock and drain pipe are completely enveloped in an approved non-woven, geotextile filter fabric. Alternatively, approved geotextile drainage composites, such as MiraDRAIN®, may be used in lieu of the drain rock layer. If used, geocomposite drain panels should be installed in accordance with the manufacturer’s recommendations.

If efflorescence (discoloration of the wall face) or moisture/water penetration of the retaining walls is not acceptable, moisture/water-proofing measures should be applied to the back face of the walls. A moisture/water-proofing specialist should be consulted to determine specific protection measures against moisture/water penetration through the walls.

Structural backfill materials for retaining walls within a one horizontal to one vertical (1H:1V) projection from the bottom of the walls (other than the drainage layer) should consist of imported, granular material or native sand and silt that does not contain significant quantities of rubbish, rubble, organics and rock over four inches in size. Clay, pea gravel and/or crushed rock should not be used for structural wall backfill. Structural wall backfill should be placed, moisture conditioned and compacted in accordance with recommendations provided in the Engineered Fill section of this report.

Foundations for support of retaining or below grade walls should be designed using the appropriate foundation design parameters provided in the Foundations section included in this report.

Exterior Flatwork

The final subgrade for exterior concrete flatwork (i.e., sidewalks, patios, etc.) should be prepared and constructed in accordance with recommendation provided in the Interior Floor Slab section above. The zone of non-expansive fill or lime-treated soils can be reduced to at least 1 foot laterally outside the perimeter of the flatwork.

As an alternative, the subgrade soils could be presoaked by wetting and pre-swelling the subgrade soils prior to placement of concrete, thus reducing the potential for post-construction movement. This approach tends to be the less costly, however, the risk for isolated heaving and cracking is greater since it can be difficult to uniformly moisture condition and completely pre-swell the subgrade soil prior to placement of concrete. Furthermore, pre-soaking also softens and weakens the clay, making this this approach not appropriate where flatwork will support vehicular traffic, heavy concentrated loads, heavy equipment or machinery due to settlement and bearing concerns.

Following subgrade preparation, presoaking consists of wetting or soaking the upper 18 inches of final soil subgrade in order to uniformly raise the soils' moisture content to a uniform, near-saturated moisture condition. The zone of wetting should extend laterally at least one foot outside the perimeter of the flatwork. Presoaking is usually performed using liberal sprinkling, flooding, or other suitable method. The time required for pre-soaking could vary from a few days to over a week depending on the condition of the subgrade soils. If the exposed soils are kept moist or wet following subgrade preparation, the amount and time required for presoaking is often reduced. Likewise, restricting vehicle or equipment traffic following earthwork will decrease the potential for over-compacting the soils and reducing the ability for water to penetrate. A representative of the Geotechnical Engineer should perform a field check of the soil's moisture content and consistency within three days of concrete placement. In hot and/or windy weather, the field moisture check should be performed within 24 hours of concrete placement.

The exterior flatwork concrete should be at least four inches thick and underlain by at least four inches of aggregate base compacted to at least 95 percent relative compaction to provide stability during slab construction and to protect the soils from disturbance during construction. Consideration should be given to thickening the edges of the slabs at least twice the slab thickness where wheel traffic is expected over the slabs. Expansion joints should be provided to allow for minor vertical movement of the flatwork. Exterior flatwork should be constructed independent of other structural elements by the placement of a layer of felt material between the flatwork and the structural element. The slab designer should determine the final thickness, strength and joint spacing of exterior slab-on-grade concrete. The slab designer should also determine if slab reinforcement for crack control is required and determine final slab reinforcing requirements.

Because of seasonal wetting and drying or irrigation of the soil, isolated differential movement and cracking sometimes forms along the outside edges of exterior flatwork. To reduce this risk, consideration should be given placing lateral cutoffs along the outside edges of the flatwork, doweling joints to reduce tripping hazards, and/or stiffening the flatwork by increasing the concrete thickness and including reinforcing steel.

Areas adjacent to new exterior flatwork should be landscaped to maintain more uniform soil moisture conditions adjacent to and beneath flatwork. Final landscaping plans not allow fallow ground adjacent to exterior concrete flatwork.

Practices recommended by the Portland Cement Association (PCA) for proper placement, curing, joint depth and spacing, construction, and placement of concrete should be followed during exterior concrete flatwork construction.

Site Drainage

Final site grading should be accomplished to provide positive drainage of surface water away from the buildings and prevent ponding of water adjacent to foundations, slabs or pavements. The subgrade adjacent to the buildings should be sloped away from foundations at a minimum two percent gradient for at least 10 feet, where possible. We recommend connecting all roof drains to solid drainage pipes which are connected to available drainage features that convey water away from the buildings, or discharging the drains onto paved or hard surfaces that slope away from the foundations. Discharging or ponding of surface water should not be allowed adjacent to the building, exterior flatwork or pavements. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward the buildings.

Pavement Design

The subgrade soils in pavement areas should be prepared and constructed in accordance with recommendations provided in the Subgrade Preparation and Engineered Fill sections.
All aggregate base should be compacted to at least 95 relative compaction.

The moisture content of the prepared subgrade soils should be maintained until placement of the aggregate base by periodic sprinkling with water or other suitable method. If there is a delay between placing the aggregate base and asphalt-concrete, the aggregate base should also be periodically sprinkled or wetted to prevent drying of the underlying soil subgrade. A field check of the subgrade soils moisture condition should be performed by the Geotechnical Engineer's representative prior to placement of the aggregate base.

As an alternative, the upper 12 inches of subgrade soil can be stabilized by mixing it with dolomitic or high calcium quick lime (lime-treatment). The subgrade preparation, spreading, mixing, compacting, lime type and curing should meet the criteria discussed in the Interior Floor Slab section. The zone of lime-treated soil should extend laterally at least two feet outside the perimeter of the proposed pavement subgrade. The lime-treated subgrade soil should be compacted to at least 95 percent relative compaction. A representative of the Geotechnical Engineer should be on-site during treatment operations to document spreading, mixing and compaction operations and provide supplemental/revised recommendations, if warranted, based on the soil conditions observed.

Based on laboratory testing results and previous nearby experience, an R-value of 5 was used for design of pavements supported on compacted native clay and an R-value of 50 was used for lime treated clays. The pavement sections presented in Table 3 have been calculated using traffic indices assumed to be appropriate for the project using pavement design criteria outlined in Chapters 600 to 670 of the *California Highway Design Manual* (Caltrans, 2019), and Sacramento County, November 1, 2009 Street Design Standards. The project civil engineer should determine the appropriate traffic index and pavement section based on anticipated traffic conditions. If needed, we can provide alternative pavement sections for different traffic indices.

Table 3: On-site Pavement Design Alternatives

Traffic Index (TI)	Pavement Use	Untreated Subgrades R-value = 5			Lime-Treated Subgrades Soils (**) R-value = 50		
		Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)	Type A Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Portland Cement Concrete (inches)
4.5	Light Automobile Parking Only	2½*	10	--	2½*	4	--
		3*	8	--	3*	4	--
		--	4	6	--	4	4
6.5	Automobile, Light 2- to 3-axle Truck Traffic, Fire Lanes and Trash	3	16	--	3*	4	--
		4*	14	--	4*	4	--
		--	6	6	--	5	4
8.0	Light-Moderate 4-axle-Truck Traffic	3½	20	--	3½	9	--
		5*	18	--	5*	6	--
		--	7	6	--	6	4
9	Moderate-Heavy 5-axle-Truck Traffic (AADTT ≈ 75)	4½	22	--	4½	11	--
		5½*	21	--	5½*	9	--
		--	8	8	--	6½	4
10.0	Heavy 5-axle-Truck Traffic (AADTT ≈ 170)	5	25	--	5	13	--
		6½*	23	--	6½*	11	--
		--	8	10	--	7½	6

* = Asphalt concrete thickness contains the Caltrans safety factor.

**= Lime-treated subgrade should be at least 12 inches thick and possess a minimum R-value of 40 when testing in accordance with California Test 301.

AADTT = Annual Average Daily Truck Traffic, assuming 5-axles trucks and 20-year ESAL Constant.

In the summer heat, high axle loads coupled with shear stresses induced by sharply turning tire movements can lead to failure in asphalt concrete pavements. Therefore, we recommend that consideration be given to using Portland concrete cement (PCC) pavements in areas subjected to concentrated heavy wheel loading, such as entry driveways and in front of trash enclosures. Alternate PCC pavement sections have been provided above in Table 3. All aggregate base should be compacted to at least 95 percent relative compaction.

We suggest the concrete slabs be constructed with thickened edges in accordance with American Concrete Institute (ACI) design standards, latest edition. Reinforcing for crack control, should be provided in accordance with ACI guidelines. Reinforcement must be located at mid-slab depth to be effective. Joint spacing and details should conform to the current PCA or ACI guidelines. Per the California Highway Design manual, PCC should achieve a minimum modulus of rupture/flexural strength of 625 pounds per square inch (psi) at 28 days. Per PCA guidelines, a minimum compressive strength of 4,000 psi at 28 days is required to achieve the specified modulus of rupture.

All pavement materials and construction methods of structural pavement sections should conform to the applicable provisions of the Caltrans Standard Specifications and Napa County Standards, latest editions.

Efficient drainage of all surface water to avoid infiltration and saturation of the supporting aggregate base and subgrade soils is important to pavement performance. Weep holes could be provided at drainage inlets, located at the subgrade-aggregate base interface, to allow accumulated water to drain from beneath the pavements.

Consideration should be given to using full-depth curbs between landscaped areas and pavements to serve as a cut-off for water that could migrate into the pavement base materials or subgrade soils.

Ancillary Foundations

Foundations for lightly-loaded, ancillary structures not structurally connected to the proposed buildings, such as sound walls, landscape walls, monuments, trash enclosures, or similar structures, may be supported upon conventional spread foundations or drilled, cast-in-place reinforced concrete piers (drilled piers). Drilled piers may be used for support of light poles or similar structures.

Conventional Spread Foundations

Conventional spread foundations should bear on firm, undisturbed ground, engineered fill, or a combination of these materials, as confirmed by the Geotechnical Engineer or his representative. The spread foundations should be at least 12 inches wide and extend at least 18 inches below the lowest adjacent soil grade. The foundations may be sized using a maximum allowable soil bearing pressure of 2,000 psf, with a one-third increase for wind or seismic forces. Lateral foundation resistance may be determined using the factors presented in the Foundations section. The upper 12 inches of subgrade soil should be disregarded when estimating lateral resistance.

Drilled, Cast-in-Place Concrete Piers

Drilled piers should be at least 18 inches in diameter, extend at least five feet below lowest adjacent soil grade, and sized using a maximum allowable end-bearing capacity of 4,000 psf or an allowable skin friction of 250 psf for dead plus live loads, which may be applied over the surface area of the pier. These values may be increased by one-third to include short-term wind or seismic forces. The weight of foundation concrete below grade may be disregarded in sizing computations.

Uplift resistance of drilled pier foundations may be computed using the following resisting forces, where applicable: 1) weight of the pier concrete and, 2) the allowable skin friction of 250 psf applied over the shaft area of the drilled pier. Increased uplift resistance can be achieved by increasing the diameter of the drilled pier or increasing the depth.

The upper 12 inches of skin friction should be neglected for axial capacity or uplift resistance unless the drilled pier is surrounded by slab concrete or pavements for a distance of at least three feet from the edge of the foundation.

Sizing of drilled piers to resist lateral loads can be evaluated using Section 1807.1 of the 2016 CBC. An allowable lateral soil bearing pressure of 200 psf per foot of depth may be used for the CBC parameters S_1 (equation 18-1) and S_3 (equations 18-2 and 18-3). If a deflection of ½ inch at the ground surface is acceptable, this value may be doubled. The upper 12 inches of the subgrade should be neglected when determining lateral resistance.

Reinforcement and concrete should be placed in the pier excavations as soon as possible after excavation is completed to reduce the potential for caving. In no case should the elapsed time between completion of the pier excavation and the start of concrete placement exceed 48 hours. If the piers are designed using the allowable vertical bearing pressure, the bottom of the pier excavations should be free of loose or disturbed soils prior to placement of the concrete. Cleaning of the bearing surface should be verified by the Geotechnical Engineer prior to concrete placement.

If drilled piers are designed using end-bearing capacity and seepage or groundwater is encountered, the water should be pumped from the pier excavation to allow inspection and concrete placement. Otherwise, the concrete should be placed using tremie methods from the bottom of the hole, while keeping the tremie pipe below the surface of the concrete at all times.

Site Drainage

Because of expansive soil concerns, the performance of foundations and concrete slabs relies on how well storm runoff and irrigation water drains from the site. Final site grading should be accomplished to provide positive drainage of surface water away from the buildings and prevent ponding of water adjacent to foundations, slabs or pavements. The subgrade adjacent to the buildings should be sloped away from the building at a minimum two percent gradient for at least five feet, where possible. All roof drains should be connected to non-perforated rigid pipes, which in-turn are connected to available drainage features that convey water away from the buildings or discharging the drainage onto paved or hard surfaces that slope away from the buildings. Landscape berms, if planned, should not be constructed in such a manner as to promote drainage toward the buildings.

Drought Considerations

The State of California can experience extended periods of severe drought conditions. Desiccated clay can shrink and crack and the ability for landowners to use irrigation as a means for maintaining landscape vegetation and soil moisture can be inhibited for unpredictable periods of time. For this reason, landscape and hardscape systems for this development should be carefully planned to prevent the desiccation of soils under and near foundations and slabs. Trees with invasive shallow root systems should be avoided. No trees or large shrubs that could remove soil moisture during dry periods should be planted within five feet of any foundation or slab. Fallow ground adjacent to foundations must be avoided.

To reduce potential for soil creep adversely affecting foundations or exterior flatwork, we recommend a minimum horizontal distance of five feet be provided and maintained between the outside edge of the foundation or flatwork to the nearest adjacent slope (e.g. building pad hinge point), for slopes greater than two feet in height.

Geotechnical Engineering Observation and Testing During Earthwork Construction

Site preparation should be accomplished in accordance with the recommendations of this report and the *Guide Earthwork Specifications* provided in Appendix D. Geotechnical testing and observation during construction is considered a continuation of our geotechnical engineering investigation. Wallace-Kuhl & Associates should be retained to provide testing and observation services during site clearing, preparation, earthwork, and foundation construction at the project to verify compliance with this geotechnical report and the project plans and specifications, and to provide consultation as required during construction. These services are beyond the scope of work authorized for this investigation; however, we would be pleased to submit a proposal to provide these services upon request.

Section 1803.5.8 “Compacted Fill Material” of the 2016 CBC requires that the geotechnical engineering report provide a number and frequency of field compaction tests to determine compliance with the recommended minimum compaction. Many factors can affect the number of tests that should be performed during construction, such as soil type, soil moisture, season of the year and contractor operations/performance. Therefore, it is crucial that the actual number and frequency of testing be determined by the Geotechnical Engineer during construction based on their observations, site conditions, and difficulties encountered. As a preliminary guideline, we recommend the following minimum tests:

- mass grading: one test per 500 cubic yards of compacted fill or one per day of work, whichever is greater
- final subgrade preparation: one test per 5,000 square feet
- aggregate base compaction: one test per 5,000 square feet
- utility backfill: one test per foot of backfill for every 150 linear feet of trench
- wall backfill: one test per foot of backfill for every 100 linear feet of wall

In the event that Wallace-Kuhl & Associates is not retained to provide geotechnical engineering observation and testing services during construction, the Geotechnical Engineer retained to provide these services should indicate in writing that they agree with the recommendations of this report, or prepare supplemental recommendations as necessary. A final report by the “Geotechnical Engineer” should be prepared upon completion of the project.

<u>Additional Services</u>

We recommend that our firm be retained to review the final plans and specifications to determine if the intent of our recommendations has been implemented in those documents. We would be pleased to submit a proposal to provide these services upon request.

LIMITATIONS

Our recommendations are based upon the information provided regarding the proposed project, combined with our analysis of site conditions revealed by the field exploration and laboratory testing programs. We have used prudent engineering judgment based upon the information provided and the data generated from our study. This report has been prepared in substantial compliance with generally accepted geotechnical engineering practices that exist in the area of the project at the time the report was prepared. No warranty, either express or implied, is provided.

If the proposed construction is modified or relocated or, if it is found during construction that subsurface conditions differ from those we encountered at our exploration locations, we should be afforded the opportunity to review the new information or changed conditions to determine if our conclusions and recommendations must be modified.

We emphasize that this report is applicable only to the proposed construction and the investigated site. This report should not be utilized for construction on any other site. This report is considered valid for the proposed construction for a period of two years following the date of this report. If construction has not started within two years, we must re-evaluate the recommendations of this report and update the report, if necessary.

Wallace - Kuhl & Associates

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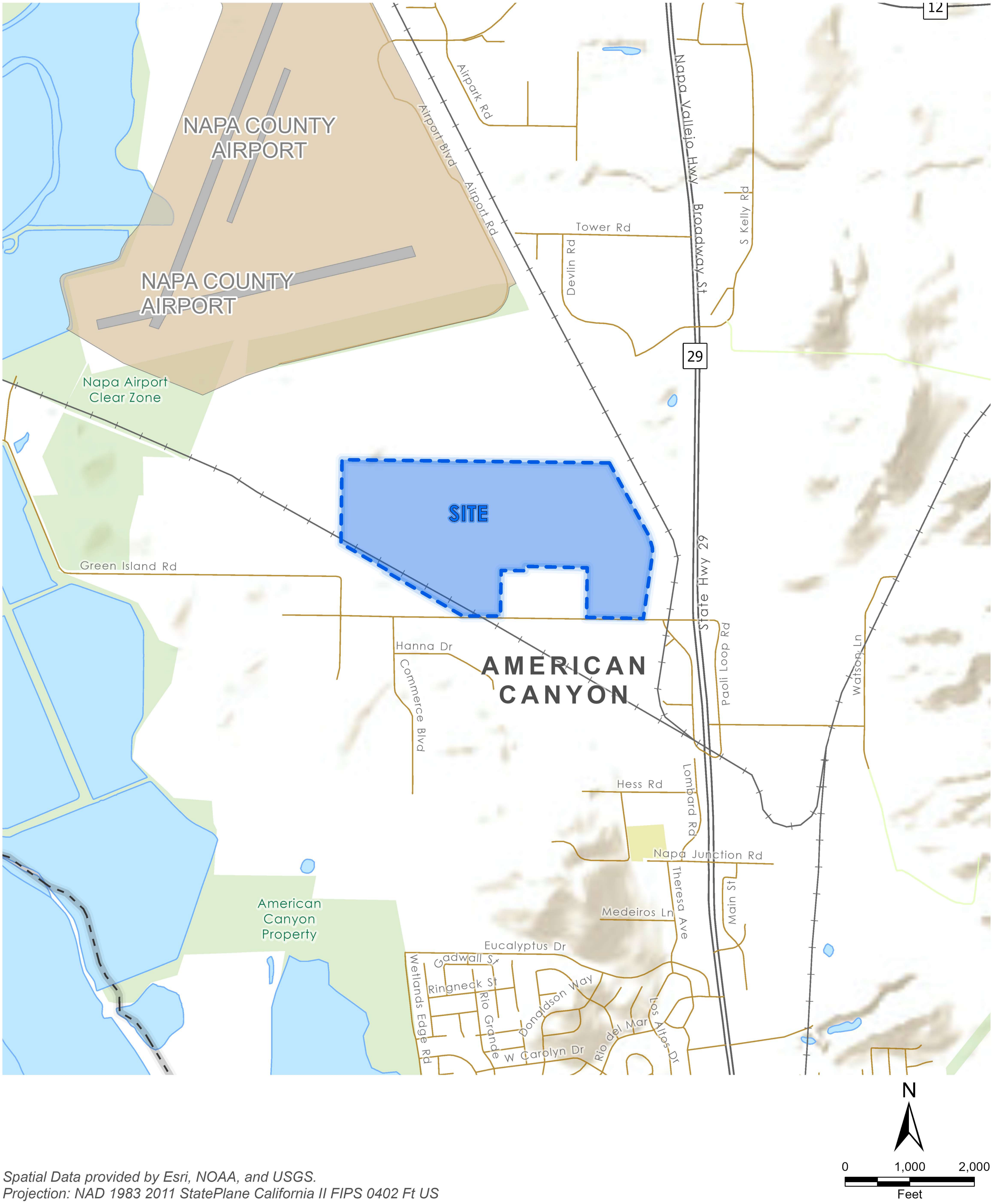

Spatial Data provided by Esri, NOAA, and USGS.
Projection: NAD 1983 2011 StatePlane California II FIPS 0402 Ft US

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VICINITY MAP

GREEN ISLAND LOGISTICS CENTER

American Canyon, California

FIGURE	1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Approximate Soil Boring Location

Approximate Bulk Soil Sample Location

Approximate Earthquake Fault Location (Cornerstone, 2017)

Approximate Wetlands Location

Approximate Site Boundary

N

0 350 700

Feet

Aerial imagery provided by Esri.
Site Plan adapted from maps provided by Buzz Oates, dated 08/06/2020 and CBG Civil Engineers, dated 08/12/2020.
Projection: NAD 1983 2011 StatePlane California II FIPS 0402 Ft US

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SITE PLAN

GREEN ISLAND LOGISTICS CENTER

American Canyon, California

FIGURE	2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D1

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark gray, moist, stiff, sandy LEAN CLAY (CL)		D1-1I	16	16.2	91	PP=3.0
					D1-2I	26	17.3	85	
	5		Yellowish brown, moist, medium dense, clayey SAND with gravel (SC)						
			Reddish brown, moist, very stiff, sandy LEAN CLAY (CL)		D1-3I	19	21.9	105	PP=2.25
	10		Boring was terminated at approximately 10.0 feet below existing ground surface.						

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FIGURE 3

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D2** Sheet 1 of 1

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
				SAMPLE DATA			**TEST DATA**		
	0		Dark gray, moist, hard, silty LEAN CLAY (CL)						
					D2-1I	28	17.4	99	PP>4.5
			brown, with sand, very stiff, iron oxide staining						
	5				D2-2I	34	18.3	102	PP=3.5
	10				D2-3I	22	19.8	107	PP=3.75
			Reddish brown, moist, medium dense, clayey fine to medium SAND (SC)						
	15		Reddish brown, moist, hard, LEAN CLAY (CL)		D2-4I	27			PP=4.5
			Boring was terminated at approximately 16.5 feet below existing ground surface. Groundwater was not encountered.						

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D3

Date(s) Drilled	9/21/20	Logged By	KRL	Checked By	JRY
Drilling Method	Hollow Stem Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	51.5 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	7	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	20.0	Sampling Method(s)	1.4" Standard Penetration Test (SPT)	Drill Hole Backfill	Neat Cement
Remarks	Bulk @ 0-4', EI, PI			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Dark brown, moist, hard, silty LEAN CLAY (CL)						
					D3-1	7	22.9		PP>4.5
	5		Dark brown, moist, hard, sandy LEAN CLAY (CL); iron oxide staining		D3-2	14			PP>4.5 PI
	10		yellowish brown		D3-3	20	16.5		PP>4.5
	15		Yellowish brown, moist, very stiff, sandy LEAN CLAY (CL)		D3-4	21	18.8		PP=3.25
	20		Reddish brown Olive to reddish brown, hard		D3-5	19			PI PP>4.5
	25		Olive to yellowish brown, very stiff		D3-6	23	17.6		PP=2.25

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FIGURE 5

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D3

ELEVATION, feet	DEPTH, feet	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
		hard	D3-7	21	17.4		PP=4.0
	35		D3-8	18			PP=4.25 PI
	40	Olive brown, wet, medium dense, clayey fine to medium SAND (SC)					
		Olive brown, wet, very stiff, LEAN CLAY with sand (CL)	D3-9	15			PP=3.5
	45	Brown, wet, very stiff, sandy LEAN CLAY (CL)					
		At 46.0 feet, a 3" gravel lense	D3-10	33			PP=3.0
	50		D3-11	23			PP>4.5

Boring was terminated at approximately 51.5 feet below existing ground surface.
Groundwater was encountered at approximately 20.0 feet below existing ground surface.

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D4

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
				SAMPLE DATA			TEST DATA		
	0		Dark brown, moist, hard, silty LEAN CLAY (CL)						
					D4-1I	20	16.1	105	PP>4.5
	5		Dark brown, moist, hard, sand LEAN CLAY (CL)		D4-2I	23	15.4	103	PP>4.5
	10		reddish brown, very moist, medium stiff, iron oxide staining		D4-3I	16	21.7	105	PP=1.5
	15		Olive brown, very moist, medium dense, clayey fine to medium SAND with gravel (SC); with iron oxide staining		D4-4I	28			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 6

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D5

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, sandy LEAN CLAY (CL)						
					D5-1I	11	17.4		
			yellowish brown, hard						
	5				D5-2I	31	14.2	108	UCC= 4.2 tsf
			Reddish brown, moist, medium dense, clayey fine to medium SAND (SC)						
	10				D5-3I	27	19.5	111	

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D6

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
				SAMPLE DATA			TEST DATA		
	0		Dark gray, moist, hard, silty LEAN CLAY (CL)		D6 @ 0-5' D6-1I	21	19.4	107	PP=4.5
	5		Reddish brown, moist, medium dense, gravelly SAND with clay (SP)		D6-2I	20	18.1	103	
	10		Brown, very moist, loose, clayey fine to medium SAND (SC)		D6-3I	10	21.5	101	
	15				D6-4I	36			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was encountered at approximately 15.0 feet below existing ground surface.

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FIGURE 8

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D7** Sheet 1 of 1

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark grey, moist, sandy LEAN CLAY (CL)						
					D7-1I	12	17.4	99	
			reddish brown		D7-2I	26	17.0	110	PP=3.25
	5		Reddish brown, moist, medium dense, clayey fine to coarse SAND with gravel (SC)						
			olive to yellowish brown, very moist, increased fines content		D7-3I	22	18.7	109	
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 9

Project: Green Island Logistics Center Project Location: American Canyon, California WKA Number: 12883.01	**LOG OF SOIL BORING D8** Sheet 1 of 1

Date(s) Drilled	9/21/20	Logged By	KRL	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	10.0 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks				Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, hard, LEAN CLAY with sand (CL)						
					D8-1I	13	19.9	98	PP>4.5
	5				D8-2I	19	15.9	112	PP>4.5
					D8-3I	50	13.4	113	PP=4.5
	10		Yellowish brown, moist, very dense, clayey fine SAND (SC)						
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 10

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D9** Sheet 1 of 1

Date(s) Drilled	**9/21/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0	CL	Brown, moist, hard, silty LEAN CLAY (CL); with iron oxide staining						
				X	D9-1I	22	13.5	111	PP>4.5
	5		very stiff	X	D9-2I	31	15.3	112	PP=3.25
		SC	Brown, moist, medium dense, clayey fine to medium SAND (SC)						
	10	CL	Yellowish brown, moist, hard, sandy LEAN CLAY (CL)	X	D9-3I	39	17.2	113	PP>4.5
	15	SC	Brown, moist, medium dense, clayey fine to medium SAND with gravel (SC)	X	D9-4I	27			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encounterd.

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Wallace Kuhl & ASSOCIATES

FIGURE 11

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	# LOG OF SOIL BORING D10 Sheet 1 of 1

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, hard, silty LEAN CLAY (CL)						
					D10-1I	8	20.6	89	PP>4.5
					D10 @ 0-5'				
			very stiff, with fine sand		D10-2I	19	13.8	111	PP=2.25
	5		Yellowish brown, moist, medium dense, clayey fine to medium SAND (SM)						
			increased clay content, slight calcification		D10-3I	42	15.6	112	
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D11

Date(s) Drilled	9/28/20	Logged By	KRL	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	16.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks				Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Brown, moist, very stiff, LEAN CLAY with sand (CL)						
					D11-1I	18	17.2	105	PP=2.25
	5		Yellowish brown, medium dense, clayey fine to medium SAND (SC)		D11-2I	30	21.3	100	
	10		Yellowish brown, moist, very stiff, LEAN CLAY with sand (CL)		D11-3I	20	20.0	105	PP=3.25
	15		medium stiff						
			Yellowish brown, moist, medium dense, clayey fine SAND (SC)		D11-4I	16			PP=1.0

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 13

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D12

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Brown, moist, very stiff, LEAN CLAY (CL); trace sand						
					D12-1I	11	16.9	114	PP=2.5
			dark reddish brown, increased sand content		D12-2I	22			PP=3.5 HC
	5								
			dark yellowish brown		D12-3I	24	18.2	109	PP=4.0
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 14

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D13

Date(s) Drilled	**9/22/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Yellowish brown, moist, hard, silty LEAN CLAY (CL)						
					D13-1I	10	19.1	98	UCC= 5.7 tsf
	5		trace sand		D13-2I	16	16.5	106	UCC= 3.7 tsf
	10				D13-3I	24	18.5	108	PP>4.5
	15		very stiff		D13-4I	20			PP=2.5
			Boring was terminated at approximately 16.5 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 15

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D14

Sheet 1 of 1

Date(s) Drilled	**9/22/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, hard, silty LEAN CLAY (CL)						
					D14-1I	13	17.8	98	PP>4.5
			yellowish brown, very stiff		D14-2I	19	23.6	97	PP=2.75
	5								
			Dark yellowish brown, very moist, medium dense, clayey fine to coarse SAND (SC)		D14-3I	30	16.3	114	
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 16

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D15

Date(s) Drilled	9/28/20	Logged By	KRL	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	16.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks	Bulk @ 0-4'			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark reddish brown, moist, very stiff, sandy LEAN CLAY (CL)						
					D15 @ 0-5' D15-1I	17	15.9	96	PP=2.0
	5		hard		D15-2I	40	14.5	111	PP=4.5
			Dark reddish brown, moist, medium dense, clayey fine to medium SAND (SC)						
	10		Yellowish brown, moist, hard, sandy LEAN CLAY (CL)		D15-3I	25	16.9	110	PP=4.0
	15		slight calcification		D15-4I	40			PP>4.5

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 17

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D16

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Light yellowish brown, moist, sandy LEAN CLAY (CL)						
					D16-1I	14	7.2	94	
			Brown, moist, hard, LEAN CLAY (CL)		D16-2I	26	17.0	108	PP>4.5
	5								
			Yellowish brown, moist, hard, sandy LEAN CLAY (CL)		D16-3I	25	16.8	110	PP=4.25
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 18

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D17

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Dark brown, moist, hard, silty LEAN CLAY (CL)		D17 @ 0-5' D17-1I	22	17.3	112	PP>4.5
	5		Reddish brown, moist, very stiff, sandy LEAN CLAY (CL)		D17-2I	26	17.7	110	PP=3.75
	10		Brown, damp, medium dense, clayey fine SAND (SC)		D17-3I	17	20.4	99	
	15		Reddish brown to olive, wet, very stiff, LEAN CLAY (CL)		D17-4I	24			PP=3.5

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was encountered at approximately 12.0 feet below existing ground surface.

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D18

Sheet 1 of 1

Date(s) Drilled	**9/22/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, dry, sandy LEAN CLAY (CL)						
					D18-1I	7	8.7	100	
			yellowish brown, moist, very stiff						
	5				D18-2I	19	22.4	99	PP=3.0
			hard						
	10				D18-3I	23	15.9	120	PP>4.5
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 20

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D19** Sheet 1 of 1

Date(s) Drilled	**9/22/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark yellowish brown, moist, very dense, LEAN CLAY with sand (CL)						
					D19 @ 0-5' D19-1I	7	22.8	107	PP=3.0
	5		Dark yellowish brown, moist, very dense, sandy LEAN CLAY (CL)		D19-2I	21	18.0	110	PP=2.75
	10		At 10.0 feet, a 1" fine to medium sand lense		D19-3I	18	18.3	108	PP>4.5
	15		Light yellowish brown to olive brown, moist, clayey fine to coarse SAND (SC)		D19-4I	37			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 21

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D20

Date(s) Drilled	9/22/20	Logged By	KRL	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	10.0 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks				Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Brown, moist, hard, silty LEAN CLAY (CL)		D20-1I	8	22.3	104	PP>4.5
					D20-2I	17	15.5	111	PP=3.75
	5		Yellowish brown, moist, very stiff, sandy LEAN CLAY (CL)						
			Yellowish brown, damp, medium dense, clayey fine to medium SAND (SC)		D20-3I	23	17.4	112	
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 22

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D21

Date(s) Drilled	**9/22/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**51.5 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**7**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**20.0**	Sampling Method(s)	**1.4" Standard Penetration Test (SPT)**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4', EI, PI**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
					SAMPLE DATA		**TEST DATA**		
	0		Brown, moist, stiff, sandy LEAN CLAY (CL)						
					D21-1I	6			PI
	5				D21-2I	17	17.0		
	10				D21-3	9			
					D21-4	8			
			increased moisture content, very stiff, black staining		D21-5	18	20.0		PP=2.0
	15		Brown, moist, medium dense, clayey fine to medium SAND (SC)						
			Brown, moist, very stiff, sandy LEAN CLAY (CL)		D21-6I	13	19.5		PP=3.5
	20				D21-7I	11			PP=3.5 PI
	25		medium stiff At 26 feet, a 2" fine to medium sand lense		D21-8I	10			PP=1.0

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FIGURE 23

LOG OF SOIL BORING D21

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Light reddish brown		D21-9I	11			
	35		increased moistue content		D21-10I	17			
	40		Light reddish brown, wet, medium dense, silty fine to medium SAND (SM)		D21-11I	17			
	45		Brown, wet, medium dense, gravelly fine to coarse SAND (SP)		D21-12I	36			
	50		Yellowish brown, very moist, dense, clayey fine SAND (SC)		D21-13I	41			
			Brown, wet, dense, silty fine to medium SAND (SM)						

Boring was terminated at approximately 51.5 feet below existing ground surface.
Groundwater was encountered at approximately 15.0 feet below existing ground surface.

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D22

Date(s) Drilled	9/22/20	Logged By	KRL	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	16.5 feet
Drill Rig Type	CME 55 HT	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks				Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		Dark reddish brown, moist, hard, silty LEAN CLAY (CL)		D22 @ 0-5' D22-1I	13	19.6	101	PP=4.5
	5		Dark yellowish brown, moist, hard, sandy LEAN CLAY (CL)		D22-2I	25	17.7	107	PP>4.5
	10		Dark yellowish brown, moist, medium dense, clayey fine to medium SAND (SC)		D22-3I	22	15.3	114	
	15		Reddish brown, moist, very stiff, LEAN CLAY (CL); iron oxide staining		D22-4I	16			PP=3.5

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 24

Project: Green Island Logistics Center	LOG OF SOIL BORING D23
Project Location: American Canyon, California	
WKA Number: 12883.01	Sheet 1 of 1

Date(s) Drilled	**9/22/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 55 HT**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, hard, silty LEAN CLAY (CL)						
					D23-1I	9	24.6	89	UCC= 3.9 tsf
	5				D23-2I	16	14.7	112	UCC= 3.1 tsf
			yellowish brown		D23-3I	14	16.4	117	PP=1.25
	10		Yellowish brown, moist, medium dense, clayey fine to medium SAND (SC)						
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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Project: Green Island Logistics Center Project Location: American Canyon, California WKA Number: 12883.01	**LOG OF SOIL BORING D24** Sheet 1 of 1

Date(s) Drilled	**9/29/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark reddish brown, moist, hard, silty LEAN CLAY (CL)						
					D24-1I	8	19.2	87	PP>4.5
			very stiff		D24-2I	23	18.0	108	PP=3.75
	5								
					D24-3I	18	20.3	104	PP=2.0
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 26

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D25

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, stiff, silty LEAN CLAY (CL)						
					D25-1I	10	8.9	90	
			light brown, very stiff		D25-2I	19	24.0	98	
	5								
			with black mottling, sand		D25-3I	21	18.8	106	
	10		Boring was terminated at approximately 10.0 feet below exisiting ground surface. Groundwater was not encountered.						

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FIGURE 27

Project: Green Island Logistics Center Project Location: American Canyon, California WKA Number: 12883.01	**LOG OF SOIL BORING D26** Sheet 1 of 1

Date(s) Drilled	**9/29/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4', EI**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark reddish brown, hard, silty LEAN CLAY (CL)						
					D26 @ 0-4'				PI
					D26-1I	17	17.3	107	PP>4.5
	5		brown		D26-2I	35	19.1	106	PP>4.5
	10		yellowish brown, very stiff		D26-3I	16	19.9	105	PP=3.0
	15		Dark yellowish brown, wet, medium dense, clayey fine SAND (SC)		D26-4I	28			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was encountered at approximately 15.0 feet below existing ground surface.

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FIGURE 28

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D27

Date(s) Drilled	**9/29/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, hard, sandy LEAN CLAY (CL)						
					D27-1I	12	8.6	93	PP>4.5
			Brown, moist, hard, silty LEAN CLAY (CL)		D27-2I	20	19.4	102	UCC= 2.7 tsf
	5								
			with sand, iron oxide staining		D27-3I	29	15.3	114	PP=4.0
	10								
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 29

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D28** Sheet 1 of 1

Date(s) Drilled	**9/29/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Reddish brown, moist, hard, silty LEAN CLAY (CL)						
					D28-1I	12	23.1	98	PP>4.5
	5		brown		D28-2I	26	15.9	107	PP>4.5
			wet, with sand						
	10		Dark yellowish brown, wet, medium dense, clayey fine to medium SAND (SC)						
			Dark yellowish brown, very moist, very stiff, LEAN CLAY (CL)		D28-3I	19	19.4	105	PP=2.0
	15				D28-4I	18			PP=2.5

Boring was terminated at approximately 16.5 feet below exsiting ground surface.
Groundwater was encountered at approximately 13.0 feet below existing ground surface.

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FIGURE 30

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D29** Sheet 1 of 1

Date(s) Drilled	**9/29/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Brown, moist, hard, silty LEAN CLAY (CL)						
					D29-1I	9	22.6	93	PP=4.5
					D29 @ 0-5'				
			very stiff		D29-2I	12	20.9	98	PP=3.25
	5								
					D29-3I	17			
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D30

Date(s) Drilled	**9/29/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		Brown, moist, very stiff, silty LEAN CLAY (CL)						
					D30-1I	12	25.7	98	PP=3.0
	5				D30-2I	23	25.1	94	PP=3.5
	10		Reddish brown, moist, very stiff, sandy LEAN CLAY (CL)		D30-3I	20	24.1	96	PP=3.25
	15				D30-4I	7			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 32

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D31

Date(s) Drilled	9/29/20	Logged By	KRL	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	10.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks				Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Grayish brown, moist, silty LEAN CLAY (CL)						
					D31-1I	13	5.3	84	
			brown		D31-2I	15	17.5	103	PP=2.25
	5								
			yellowish brown, medium stiff		D31-3I	21	15.1	114	PP=1.25
	10		Yellowish brown, moist, medium dense, clayey fine to medium SAND (SC)						
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 33

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D32

Date(s) Drilled	9/29/20	Logged By	KRL	Checked By	JRY
Drilling Method	Hollow Stem Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	51.5 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	8	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	15.0	Sampling Method(s)	2.0" Modified California with 6-inch sleeve & 1.4" Standard Penetration Test (SPT)	Drill Hole Backfill	Neat Cement
Remarks	Bulk @ 0-4'			Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, hard, silty LEAN CLAY (CL)						
					D32-1	12	21.8	103	PP>4.5
	5		Brown, moist, very stiff, sandy LEAN CLAY (CL)		D32-2	17			PP=2.75 PI
	10		yellowish brown, increased sand content		D32-3	13	23.5		PP=2.25
	15		olive brown, medium stiff, iron oxide staining		D32-4	8	24.2		PP=1.0
	20		wet		D32-5	10			PI
	25		very moist		D32-6	11	28.7		PP=1.5

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FIGURE 34

LOG OF SOIL BORING D32

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark yellowish brown, very moist, medium dense, clayey fine SAND with gravel (SC)		D32-7	14	28.5		
	35		Dark reddish brown, wet, very stiff, sandy LEAN CLAY (CL)		D32-8	20			PP=2.0 PI
	40		Reddish brown, wet, medium dense, clayey fine SAND (SC); slight calcification		D32-9	29			
	45		Gray, wet, medium dense, fine to coarse SAND with fine gravel (SP)		D32-10	38			
	50				D32-11	36			

Boring was terminated at approximately 51.5 feet below existing ground surface.
Groundwater was encountered at approxmately 15.0 feet.

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Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D33

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
				SAMPLE DATA			TEST DATA		
	0		Brown, moist, medium stiff, sandy LEAN CLAY (CL)						
					D33-1I	17	18.5	106	PP=1.75
	5		Reddish brown, very moist, silty fine to medium SAND (SM)		D33-2I	22	15.7	107	PP>4.5
			Brown, moist, hard, sandy LEAN CLAY (CL)						
	10		yellowish brown, very stiff		D33-3I	20	23.4	96	PP=3.0
	15		soft		D33-4II	16			PP=0.5
			very stiff		D33-4I				PP=3.5

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

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Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D34

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
				SAMPLE DATA			TEST DATA		
			Brown moist, very stiff, LEAN CLAY with sand (CL)						
					D34-1I	10	20.2	102	UCC= 1.8 tsf
			Light gray, very moist, medium dense, silty fine SAND (SM)		D34-2I	20	17.8	107	UCC= 1.2 tsf
	5								
			Reddish brown, moist, very stiff, sandy LEAN CLAY (CL)		D34-3I	25	17.7	106	PP=3.75
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 36

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D35** Sheet 1 of 1

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.2 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		Olive brown, moist, medium stiff, LEAN CLAY with sand (CL); slight calcification						
					D35-1I	16	20.5	103	PP=1.5
	5				D35-2I	13	27.3	91	PP=1.75
	10		Reddish brown, moist, loose, silty fine SAND (SM)		D35-3I	11	21.7	99	PP=1.25
			Reddish brown, moist, medium stiff, sandy LEAN CLAY (CL)						
	15		wet		D35-4I	15			PP=1.25

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was encountered at approximately 15.0 feet below existing ground surface.

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D36** Sheet 1 of 1

Date(s) Drilled	**9/28/20**	Logged By	**KRL**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, hard, silty LEAN CLAY (CL)						
					D36-1I	10	19.5	101	PP=4.5
			tanish brown, very stiff		D36-2I	33	13.2	109	PP=3.75
	5								
			slightly calcified		D36-3I	18	19.4	104	PP=3.0
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D37** Sheet 1 of 1

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown to dark brown, moist, stiff, silty LEAN CLAY (CL)						
					D37-1I D37 @ 0-4'	13	24.9	93	
			very stiff		D37-2I	28	18.9	107	
	5								
			light brown		D37-3I	29	19.7	112	
	10								
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 39

Project: Green Island Logistics Center **Project Location:** American Canyon, California **WKA Number:** 12883.01	**LOG OF SOIL BORING D38** Sheet 1 of 1

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**11.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Light brown with black mottling, moist, stiff, silty LEAN CLAY (CL)						
					D38 @ 0-4'				
					D38-1I	16	23.6	97	UCC= 2.6 tsf
	5		very stiff		D38-2I	17	19.9	102	UCC= 1.3 tsf
	10		with sand		D38-3I	20			
			Boring was terminated at approximately 11.5 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 40

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D39** Sheet 1 of 1

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, stiff, silty LEAN CLAY (CL)						
					D39-1I	13	16.2	95	
			light brown to brown with black mottling, very stiff		D39-2I	19	16.2	111	
	5								
					D39-3I	23	22.0	105	
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D40** Sheet 1 of 1

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Light brown with gray mottling, moist, very stiff, silty LEAN CLAY (CL)		D40 @ 0-4'				PI
					D40-1I	23	18.8	102	
	5		with black mottling		D40-2I	19	23.7	101	
	10		stiff		D40-3I	11	34.6	87	
	15				D40-4I	15			
			Boring was terminated at approximately 16.5 feet below existing ground surface. Groundwater was not encountered.						

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Project: Green Island Logistics Center Project Location: American Canyon, California WKA Number: 12883.01	**LOG OF SOIL BORING D41** Sheet 1 of 1

Date(s) Drilled	10/1/20	Logged By	DB	Checked By	JRY
Drilling Method	Solid Flight Auger	Drilling Contractor	V&W Drilling	Total Depth of Drill Hole	10.0 feet
Drill Rig Type	CME 75	Diameter(s) of Hole, inches	6	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	Not Encountered	Sampling Method(s)	2.0" Modified California with 6-inch sleeve	Drill Hole Backfill	Neat Cement
Remarks				Driving Method and Drop	140lb auto. hammer with 30" drop

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, medium stiff, LEAN CLAY (CL)		D41-1I	9	18.8	102	
			Grayish brown, moist, stiff, sandy LEAN CLAY (CL)		D41-2I	14	17.7	110	
	5								
					D41-3I	12	31.3	86	
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 43

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D42

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Light brown, moist, stiff, sandy LEAN CLAY (CL)						
					D42-1I	15	18.9	104	
			Light brown, moist, stiff, LEAN CLAY (CL)		D42-2I	16	23.8	99	
	5								
					D42-3I	17	18.5	107	
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

BORING LOG 12883.01 - GREEN ISLAND LOGISTICS CENTER.GPJ WKA.GDT 11/6/20 3:33 PM

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Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D43

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, stiff, silty LEAN CLAY (CL)						
					D43-1I	11	14.5	97	
			Yellowish brown to brown with white and black mottling, very stiff		D43-2I	20	22.5	99	
	5								
			Brown, moist, medium dense, silty SAND (SM)		D43-3I	36	23.1	99	
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

BORING LOG 12883.01 - GREEN ISLAND LOGISTICS CENTER.GPJ WKA.GDT 11/6/20 3:33 PM

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FIGURE 45

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D44** Sheet 1 of 1

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown with white and black mottling, moist, stiff, silty LEAN CLAY (CL)						
					D44-1I	17	18.0	103	
	5		yellowish brown to brown with black mottling		D44-2I	17			HC
	10		no mottling, very stiff		D44-3I	27	24.6	100	
	15				D44-4I	35			
			Boring was terminated at approximately 16.5 feet below existing ground surface. Groundwater was not encountered.						

BORING LOG 12883.01 - GREEN ISLAND LOGISTICS CENTER.GPJ WKA.GDT 11/6/20 3:33 PM

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Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D45** Sheet 1 of 1

Date(s) Drilled	**10/2/20**	Logged By	**JRY**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, stiff, silty LEAN CLAY (CL)						
					D45-1I	12	16.0	103	
			white and black mottling, very stiff, trace sand		D45-2I	30	10.9	113	
	5								
	10				D45-3I	28	22.8	102	
			Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

Project: Green Island Logistics Center Project Location: American Canyon, California WKA Number: 12883.01	**LOG OF SOIL BORING D46** Sheet 1 of 2

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Hollow Stem Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**51.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**8**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, stiff, LEAN CLAY (CL)						
					D46-1I	14	18.6		
	5		Yellowish brown, moist, very stiff, sandy silty LEAN CLAY (CL)		D46-2I	23			PI
	10		slight calcification		D46-3I	24	20.3		
	15		increased plasticity		D46-4I	17	20.3		
	20		increased sand content		D46-5I	20			PI
	25		decreased sand content		D46-6I	26	17.8		

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D46

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Brown, moist, medium dense, clayey fine to coarse SAND (SC)		D46-7I	36	13.3		
	35		Brown, moist, hard, sandy LEAN CLAY (CL)		D46-8I	37			PI
	40		very stiff		D46-9I	28			
	45		Brown, moist, medium dense, clayey fine to coarse SAND (SC)		D46-10I	28			
	50		with gravel, very dense		D46-11I	57			

Boring was terminated at approximately 51.5 feet below existing ground surface.
Groundwater was not encountered.

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D47

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, stiff, LEAN CLAY (CL)						
					D47-1I	12	25.3	91	UCC= 2.5 tsf
			Yellowish brown, moist, very stiff, sandy LEAN CLAY (CL)		D47-2I	21			
	5								
			Brown, moist, medium dense, clayey fine SAND (SC)						
					D47-3I	28	14.6	87	
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 49

Project: Green Island Logistics Center Project Location: American Canyon, California WKA Number: 12883.01	**LOG OF SOIL BORING D48** Sheet 1 of 1

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
	0		Dark brown, moist, stiff, LEAN CLAY (CL)						
					D48-1I	16	18.8	96	
	5		Grayish brown, moist, very stiff, sandy LEAN CLAY (CL)		D48-2I	26	27.7	100	
	10		Grayish brown, moist, medium dense, clayey SAND (SC)		D48-3I	36	25.6	94	
	15		Grayish brown, moist, hard, sandy LEAN CLAY (CL)		D48-4I	36			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D49

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, stiff, LEAN CLAY (CL)		D49-1I	14	15.9	93	
					D49 @ 0-5'				
			Dark brown, moist, very stiff, sandy LEAN CLAY (CL); variably cemented		D49-2I	31	13.0	110	
	5								
			Dark brown, moist, dense, clayey fine to coarse SAND with gravel (SC)		D49-3I	46	10.8	114	
	10								

Boring was terminated at approximately 10.0 feet below existing ground surface.
Groundwater was not encountered.

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FIGURE 51

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D50

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE	SAMPLE NUMBER	NUMBER OF BLOWS	MOISTURE CONTENT, %	DRY UNIT WEIGHT, pcf	ADDITIONAL TESTS
			Dark brown, moist, very stiff, silty LEAN CLAY (CL)						
					D50-1I	22	17.0	105	
	5		Brown, moist, very stiff, sandy LEAN CLAY (CL)		D50-2I	24	20.9	91	
	10				D50-3I	33	19.3	106	
	15		hard		D50-4I	41			
			Boring was terminated at approximately 16.5 feet below existing ground surface. Groundwater was not encountered.						

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FIGURE 52

Project: Green Island Logistics Center **Project Location: American Canyon, California** **WKA Number: 12883.01**	**LOG OF SOIL BORING D51** Sheet 1 of 1

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**10.0 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks				Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
			Dark brown, moist, stiff, sandy LEAN CLAY (CL)						
					D51-1I	10	17.9	87	
			brown, hard		D51-2I	39	14.2	88	
	5								
					D51-3I	43	17.7	110	
	10		Boring was terminated at approximately 10.0 feet below existing ground surface. Groundwater was not encountered.						

Project: Green Island Logistics Center
Project Location: American Canyon, California
WKA Number: 12883.01

LOG OF SOIL BORING D52

Date(s) Drilled	**10/1/20**	Logged By	**DB**	Checked By	**JRY**
Drilling Method	**Solid Flight Auger**	Drilling Contractor	**V&W Drilling**	Total Depth of Drill Hole	**16.5 feet**
Drill Rig Type	**CME 75**	Diameter(s) of Hole, inches	**6**	Approx. Surface Elevation, ft MSL	
Groundwater Depth [Elevation], feet	**Not Encountered**	Sampling Method(s)	**2.0" Modified California with 6-inch sleeve**	Drill Hole Backfill	**Neat Cement**
Remarks	**Bulk @ 0-4'**			Driving Method and Drop	**140lb auto. hammer with 30" drop**

ELEVATION, feet	DEPTH, feet	GRAPHIC LOG	ENGINEERING CLASSIFICATION AND DESCRIPTION	SAMPLE DATA: SAMPLE	SAMPLE DATA: SAMPLE NUMBER	SAMPLE DATA: NUMBER OF BLOWS	TEST DATA: MOISTURE CONTENT, %	TEST DATA: DRY UNIT WEIGHT, pcf	TEST DATA: ADDITIONAL TESTS
	0		Brown, moist, very stiff, sandy LEAN CLAY (CL)	Bulk					PI
					D52-1I	30	14.6		
	5		hard		D52-2I	42	16.4	106	UCC= 4.1 tsf
	10				D52-3I	44	16.8	110	
	15				D52-4I	37			

Boring was terminated at approximately 16.5 feet below existing ground surface.
Groundwater was not encountered.

BORING LOG 12883.01 - GREEN ISLAND LOGISTICS CENTER.GPJ WKA.GDT 11/6/20 3:33 PM

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISIONS		USCS[4]	CODE	CHARACTERISTICS
COARSE GRAINED SOILS (More than 50% of soil > no. 200 sieve size)	GRAVELS[1] (More than 50% of coarse fraction > no. 4 sieve size)	GW		Well-graded gravels or gravel - sand mixtures, trace or no fines
		GP		Poorly graded gravels or gravel - sand mixtures, trace or no fines
		GM		Silty gravels, gravel - sand - silt mixtures, containing little to some fines[2]
		GC		Clayey gravels, gravel - sand - clay mixtures, containing little to some fines[2]
	SANDS[1] (50% or more of coarse fraction < no. 4 sieve size)	SW		Well-graded sands or sand - gravel mixtures, trace or no fines
		SP		Poorly graded sands or sand - gravel mixtures, trace or no fines
		SM		Silty sands, sand - gravel - silt mixtures, containing little to some fines[2]
		SC		Clayey sands, sand - gravel - clay mixtures, containing little to some fines[2]
FINE GRAINED SOILS (50% or more of soil < no. 200 sieve size)	SILTS & CLAYS LL < 50	ML		Inorganic silts, gravely silts, and sandy silts that are non-plastic or with low plasticity
		CL		Inorganic lean clays, gravelly lean clays, sandy lean clays of low to medium plasticity[3]
		OL		Organic silts, organic lean clays, and organic silty clays
	SILTS & CLAYS LL ≥ 50	MH		Inorganic elastic silts, gravelly elastic silts, and sandy elastic silts
		CH		Inorganic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
		OH		Organic fat clays, gravelly fat clays, sandy fat clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT		Peat
ROCK		RX		Rocks, weathered to fresh
FILL		FILL		Artificially placed fill material

OTHER SYMBOLS

= Drive Sample: 2-1/2" O.D. Modified California sampler

= Drive Sampler: no recovery

= SPT Sampler

= Initial Water Level

= Final Water Level

= Estimated or gradational material change line

= Observed material change line

Laboratory Tests

CR = Corrosion
PI = Plasticity Index
EI = Expansion Index
UCC = Unconfined Compression Test (TSF)
TR = Triaxial Compression Test
GR = Gradational Analysis (Sieve/Hydro)
FC = Wash (Fines Content)
PP = Pocket Penetrometer Test (TSF)
PID = Photo Ionization Detector Test (PPM)
RV = Resistance ("R") Value

REF = Refusal (>50 blows in 6 inches)

GRAIN SIZE CLASSIFICATION

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS (b)	Above 12"	Above 300
COBBLES (c)	12" to 3"	300 to 75
GRAVEL (g) coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	75 to 4.75 75 to 19 19 to 4.75
SAND coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.75 to 0.075 4.75 to 2.00 2.00 to 0.425 0.425 to 0.075
SILT & CLAY	Below No. 200	Below 0.075

Trace - Less than 5 percent
Few - 5 to 10 percent
Little - 15 to 25 percent
Some - 35 to 45 percent
Mostly - 50 to 100 percent

* Percents as given in ASTM D2488

NOTES:

1. Coarse grained soils containing 5% to 12% fines, use dual classification symbol (ex. SP-SM).
2. If fines classify as CL-ML (4<PI<7), use dual symbol (ex. SC-SM).
3. Silty Clays, use dual symbol (CL-ML).
4. Borderline soils with uncertain classification list both classifications (ex. CL/ML).

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UNIFIED SOIL CLASSIFICATION SYSTEM

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	55
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

APPENDICES

APPENDIX A
General Project Information, Field and Laboratory Test Results

Preliminary Concept Site Plan

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

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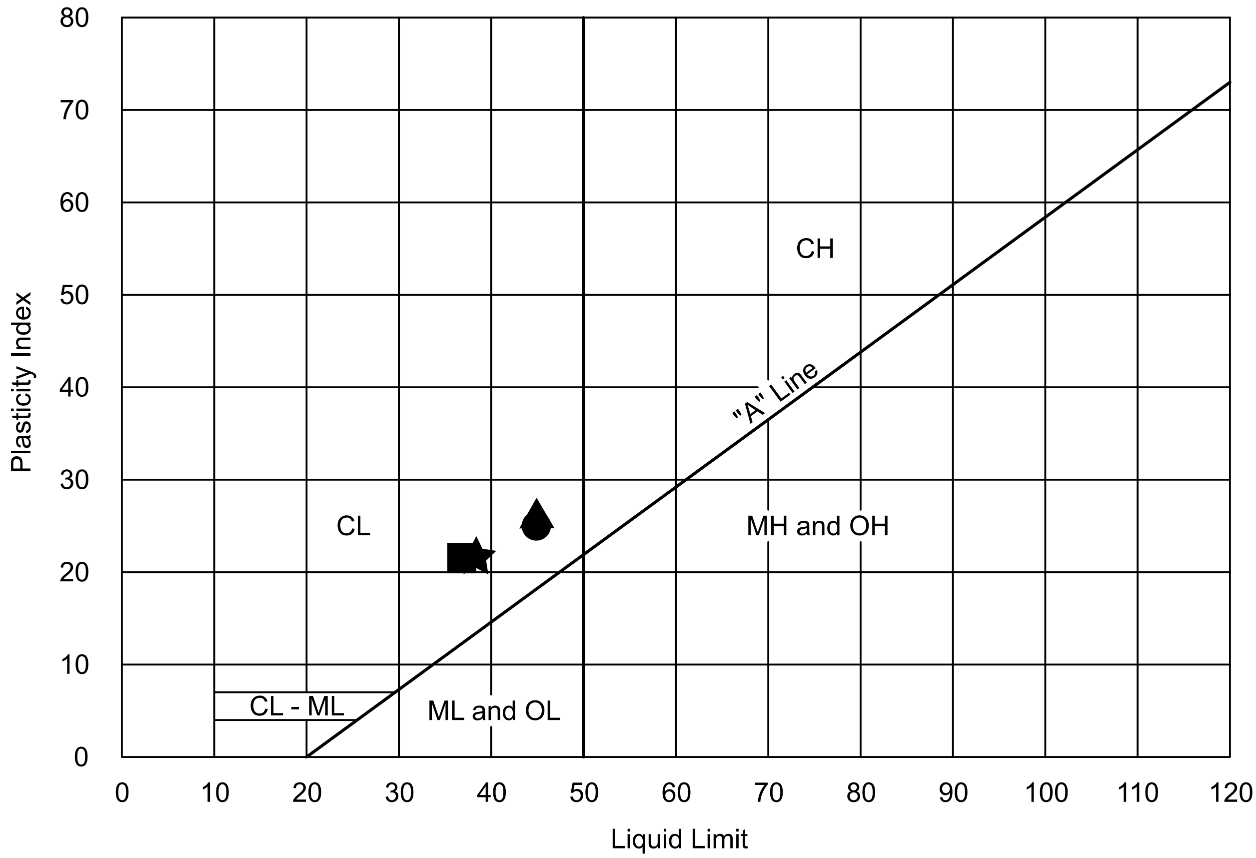
ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D3	0'-4.0'	---	45	25	---	CL
▲	D3-2	6'-6.5'	---	45	27	---	CL
■	D3-5	21'-21.5'	---	37	21	---	CL
★	D3-8	36'-36.5'	---	38	21	---	CL

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ATTERBERG LIMITS

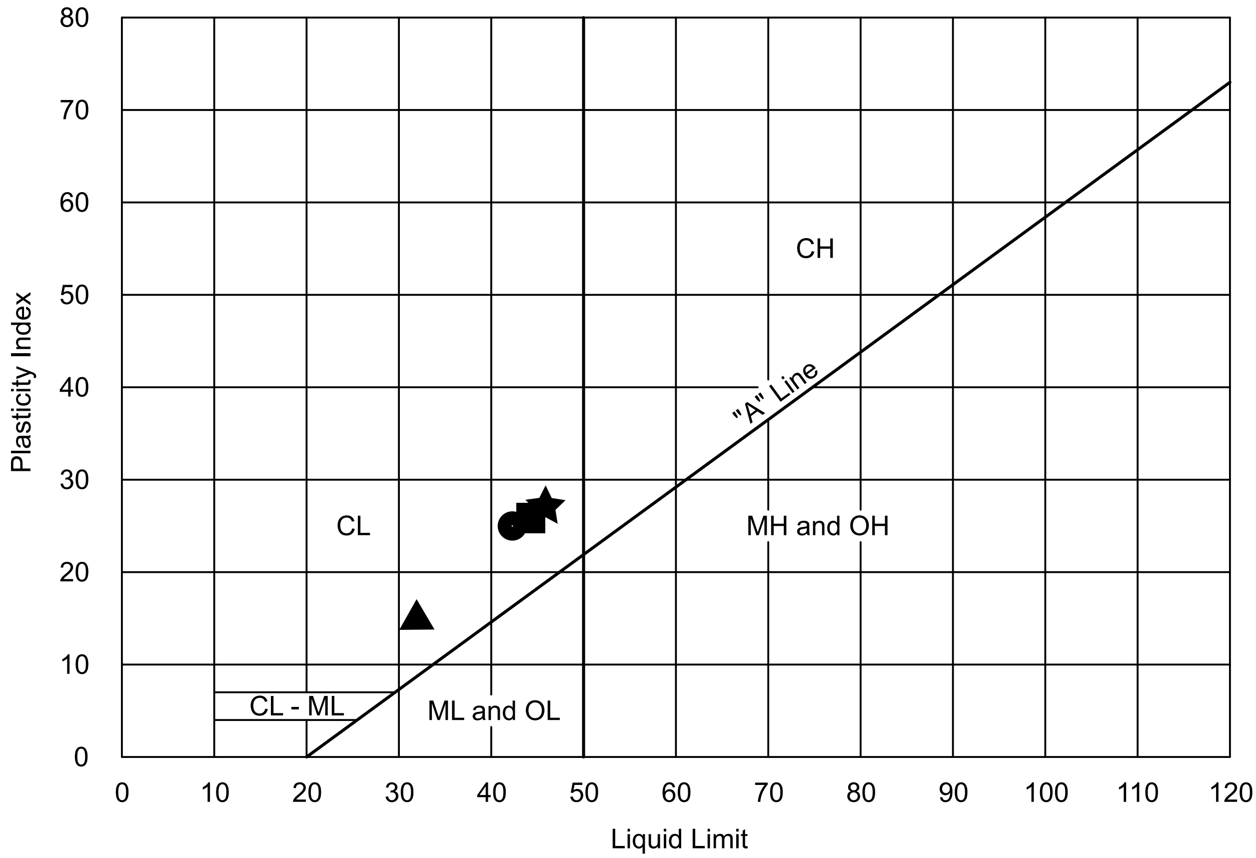
GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D21	0'-4.0'	---	42	25	---	CL
▲	D21-1	3'-3.5'	---	32	15	---	CL
■	D21-7	21'-21.5'	---	44	26	---	CL
★	D26	0'-4.0'	---	46	28	---	CL

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ATTERBERG LIMITS

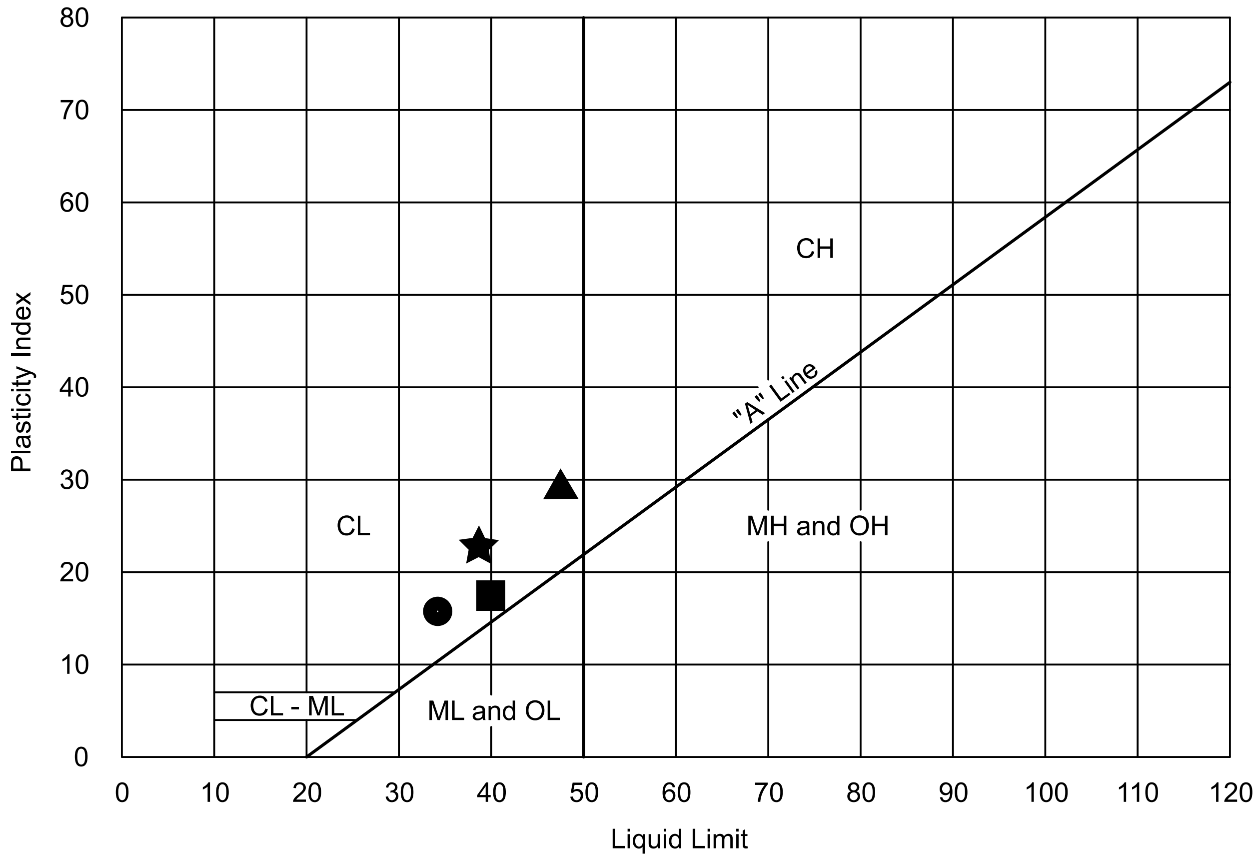
GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D32-2	6'-6.5'	---	34	16	---	CL
▲	D32-5	21'-21.5'	---	48	29	---	CL
■	D32-8	36'-36.5'	---	40	18	---	CL
★	D40	0'-4.0'	---	39	23	---	CL

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ATTERBERG LIMITS

GREEN ISLAND ROAD LOGISTICS CENTER

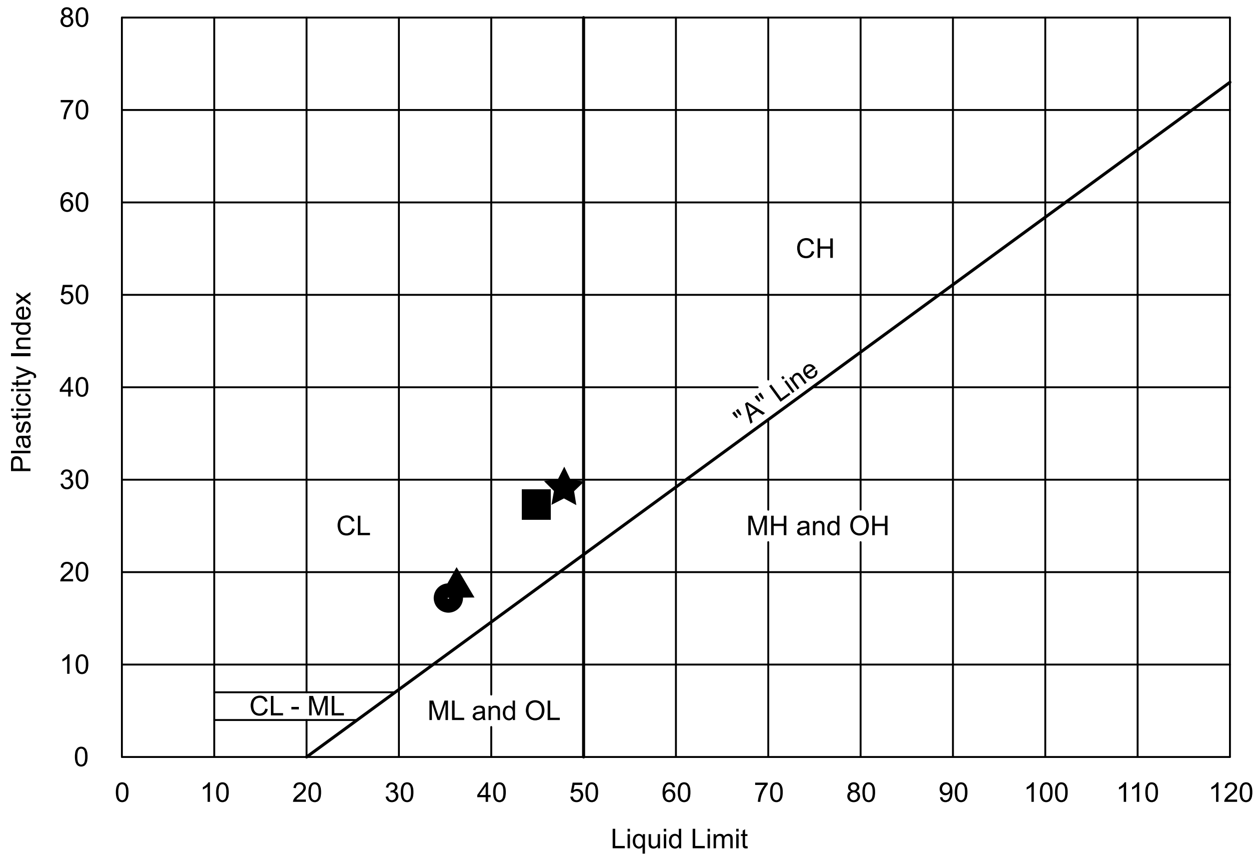
American Canyon, California

FIGURE	A3
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020

WKA NO. 12883.01

ATTERBERG LIMITS

ASTM D4318



KEY SYMBOL	LOCATION	SAMPLE DEPTH	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		PASSING No. 200 SIEVE (%)	UNIFIED SOIL CLASSIFI-CATION SYMBOL
				LIQUID LIMIT (%)	PLASTICITY INDEX (%)		
●	D46-2	6'-6.5'	---	35	18	---	CL
▲	D46-5	21'-21.5'	---	36	19	---	CL
■	D46-8	36'-36.5'	---	45	27	---	CL
★	D52	0'-4.0'	---	48	29	---	CL

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ATTERBERG LIMITS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A4
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark brown, silty lean clay

LOCATION: D3

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 4'	14.5	29.7	93	**39**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	**Low**
51 - 90	Medium
91 - 130	High
Above 130	Very High

* From ASTM D4829, Table 1

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EXPANSION INDEX

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A5
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020

WKA NO. 12883.01

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Brown, sandy lean clay

LOCATION: D21

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 4'	11.5	21.9	105	**33**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	**Low**
51 - 90	Medium
91 - 130	High
Above 130	Very High

* From ASTM D4829, Table 1

Wallace Kuhl & ASSOCIATES

EXPANSION INDEX

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A6
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Dark reddish brown, silty lean clay

LOCATION: D26

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 4'	12.0	25.6	104	**95**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	**High**
Above 130	Very High

* From ASTM D4829, Table 1

Wallace Kuhl & ASSOCIATES

EXPANSION INDEX

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A7
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Light brown, silty lean clay

LOCATION: D40

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 4'	11.0	25.7	105	**108**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	**High**
Above 130	Very High

* From ASTM D4829, Table 1

Wallace Kuhl & ASSOCIATES

EXPANSION INDEX

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A8
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

EXPANSION INDEX TEST RESULTS

ASTM D4829

MATERIAL DESCRIPTION: Brown, sand lean clay

LOCATION: D52

Sample Depth	Pre-Test Moisture (%)	Post-Test Moisture (%)	Dry Density (pcf)	Expansion Index
0' - 4'	11.0	25.7	105	**101**

CLASSIFICATION OF EXPANSIVE SOIL *

EXPANSION INDEX	POTENTIAL EXPANSION
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	**High**
Above 130	Very High

* From ASTM D4829, Table 1

Wallace Kuhl & ASSOCIATES

EXPANSION INDEX

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A9
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Dark brown, silty lean clay

LOCATION: B1 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	109	18.2	686	49	212	*

* Sample extruded, therefore R-Value = 5

MATERIAL DESCRIPTION: Dark brown, silty lean clay with 4% lime added

LOCATION: B1 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	104	21.7	355	8	35	83
2	105	21.4	477	10	43	84
3	103	22.4	255	6	26	81

R-Value at 300 psi exudation pressure = 82

WallaceKuhl & ASSOCIATES

RESISTANCE VALUE TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A10
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Brown, silty lean clay

LOCATION: B2 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	115	15.2	604	31	134	*

* Sample extruded, therefore R-Value = 5

MATERIAL DESCRIPTION: Brown, silty lean clay with 4% lime added

LOCATION: B2 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	109	17.5	360	20	87	85
2	110	17.7	313	21	91	84
3	108	18.1	251	22	95	83

R-Value at 300 psi exudation pressure = 84

WallaceKuhl & ASSOCIATES

RESISTANCE VALUE TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A11
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

RESISTANCE VALUE TEST RESULTS

(California Test 301)

MATERIAL DESCRIPTION: Dark brown, lean clay

LOCATION: B3 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	112	17.4	638	34	147	*

* Sample extruded, therefore R-Value = 5

MATERIAL DESCRIPTION: Dark brown, lean clay with 4% lime added

LOCATION: B3 (0' - 4')

Specimen No.	Dry Unit Weight (pcf)	Moisture @ Compaction (%)	Exudation Pressure (psi)	Expansion (dial, inches x 1000)	Expansion (psf)	R Value
1	122	4.9	350	4	17	87
2	122	5.4	294	1	4	83
3	120	5.8	255	0	0	77

R-Value at 300 psi exudation pressure = 84

WallaceKuhl & ASSOCIATES

RESISTANCE VALUE TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A12
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

HYDRAULIC CONDUCTIVITY TEST REPORT

SAMPLE DATA

Sample Identification: D12-2I
Visual Description:
Date Cast 9/28/20
Remarks:

Sample Depth, ft.: N/A
Sample Type: Tube
Age, days: 8

Lab No.: 56490

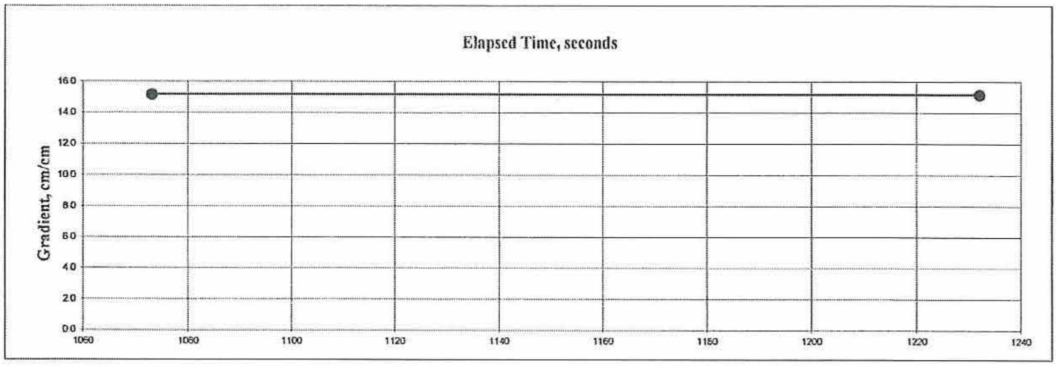
TEST RESULTS

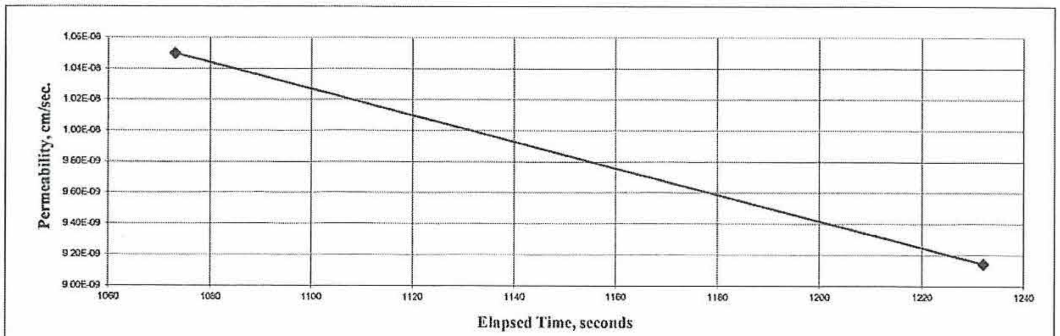
Permeability, cm/sec.: 9.82E-09
Chamber Pressure, psi: 70.0
Back Pressure, psi: 60.0
Consolidation Pressure: 10.0
"B" Coefficient: 79%

Average Hydraulic Gradient: 15.17
Initial Hydraulic Gradient: 15.2
Final Hydraulic Gradient: 15.2
Burrett Area (cm^2): 0.03
Permeant Liquid Used: Tap Water

TEST SAMPLE DATA

Before Test		After Test	
Specimen Height, cm:	5.87	*Specimen Height, cm:*	5.84
Specimen Diameter, cm:	4.78	*Specimen Diameter, cm:*	4.80
Dry Unit Weight, pcf:	101.2	*Dry Unit Weight, pcf:*	100.0
Moisture Content, %:	22.5	*Moisture Content, %:*	25.3
Specific Gravity, Assumed:	2.70	*Specific Gravity (assumed):*	2.70
Percent Saturation:	92.7	*Final Saturation, %:*	99.4





Test Method: ASTM D5084 Method F

PROJECT NUMBER: 20-239 | October 6, 2020

GS GULF SHORE EXPLORATION AND TESTING
3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
Fax: (916) 635-4215

12883.01

HYDRAULIC CONDUCTIVITY TEST REPORT

SAMPLE DATA

Sample Identification: D44-2I
Visual Description:
Date Cast 10/5/20
Remarks:

Sample Depth, ft.: N/A
Sample Type: Tube
Age, days: 1

Lab No.: 56517

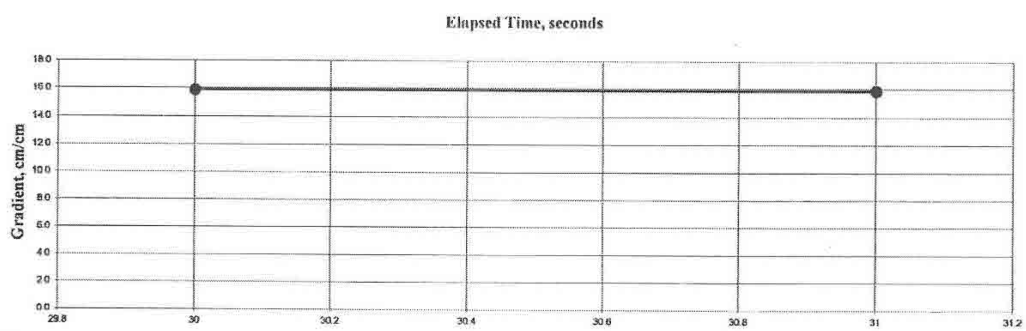
TEST RESULTS

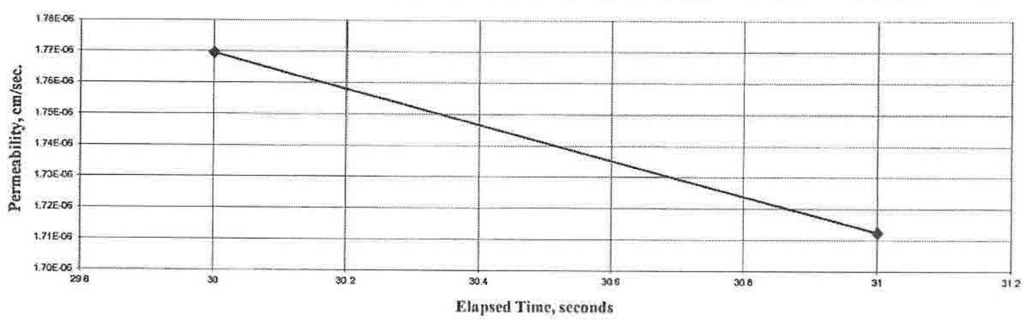
Permeability, cm/sec.: 1.74E-06
Chamber Pressure, psi: 70.0
Back Pressure, psi: 60.0
Consolidation Pressure: 10.0
"B" Coefficient: 95%

Average Hydraulic Gradient: 15.89
Initial Hydraulic Gradient: 15.9
Final Hydraulic Gradient: 15.9
Burrett Area (cm^2): 0.03
Permeant Liquid Used: Tap Water

TEST SAMPLE DATA

Before Test		After Test	
Specimen Height, cm:	7.98	*Specimen Height, cm:*	7.87
Specimen Diameter, cm:	4.80	*Specimen Diameter, cm:*	4.85
Dry Unit Weight, pcf:	99.3	*Dry Unit Weight, pcf:*	99.5
Moisture Content, %:	25.5	*Moisture Content, %:*	25.3
Specific Gravity, Assumed:	2.70	*Specific Gravity (assumed):*	2.70
Percent Saturation:	96.5	*Final Saturation, %:*	98.4





Test Method: ASTM D5084 Method F

PROJECT NUMBER:	20-239	October 6, 2020

GULF SHORE EXPLORATION AND TESTING

3362 Fitzgerald Road
Rancho Cordova, CA 95742
Phone: (916) 939-4117
Fax: (916) 635-4215

12883.01

WallaceKuhl & ASSOCIATES

HYDRAULIC CONDUCTIVITY TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/2020
Date Submitted 09/25/2020

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RA
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D6@0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83114-173476.

EVALUATION FOR SOIL CORROSION

Soil pH	6.48		
Minimum Resistivity	0.54 ohm-cm (x1000)		
Chloride	323.0 ppm	00.03230	%
Sulfate	167.4 ppm	00.01674	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES	**CORROSION TEST RESULTS** GREEN ISLAND ROAD LOGISTICS CENTER American Canyon, California	FIGURE	A15
		DRAWN BY	RWO
		CHECKED BY	JRY
		PROJECT MGR	GHG
		DATE	11/2020
		WKA NO.	12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/2020
Date Submitted 09/25/2020

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D6@0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83114-173477.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	179.4	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES	**CORROSION TEST RESULTS** GREEN ISLAND ROAD LOGISTICS CENTER American Canyon, California	FIGURE	A1
		DRAWN BY	RWO
		CHECKED BY	JRY
		PROJECT MGR	GHG
		DATE	11/2020
		WKA NO. 12883.01	

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/2020
Date Submitted 09/25/2020

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D14@0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83114-173478.

EVALUATION FOR SOIL CORROSION

Soil pH	6.89	
Minimum Resistivity	0.22 ohm-cm (x1000)	
Chloride	758.8 ppm	00.07588 %
Sulfate	821.2 ppm	00.08212 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A1
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/2020
Date Submitted 09/25/2020

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RA
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D14@0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83114-173479.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	783.1	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO. 12883.01	

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/2020
Date Submitted 09/25/2020

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D22@0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83114-173480.

EVALUATION FOR SOIL CORROSION

Soil pH	5.41	
Minimum Resistivity	0.88 ohm-cm (x1000)	
Chloride	61.6 ppm	00.00616 %
Sulfate	185.2 ppm	00.01852 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES	CORROSION TEST RESULTS GREEN ISLAND ROAD LOGISTICS CENTER American Canyon, California	FIGURE A

DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/2020
Date Submitted 09/25/2020

To: Joey Ybarra
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D22@0-5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83114-173481.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	194.2	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A2
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020

WKA NO. 12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2020
Date Submitted 10/05/2020

To: Kylie Lim
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D33 @ 0-5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83178-173600.

EVALUATION FOR SOIL CORROSION

Soil pH	5.78			
Minimum Resistivity	0.38	ohm-cm (x1000)		
Chloride	379.7	ppm	00.03797	%
Sulfate	443.5	ppm	00.04435	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES	CORROSION TEST RESULTS GREEN ISLAND ROAD LOGISTICS CENTER American Canyon, California	FIGURE A2
		DRAWN BY RWO
		CHECKED BY JRY
		PROJECT MGR GHG
		DATE 11/2020
		WKA NO. 12883.01

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2020
Date Submitted 10/05/2020

To: Kylie Lim
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RA
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D33 @ 0-5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83178-173601.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	493.8	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A22
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2020
Date Submitted 10/05/2020

To: Kylie Lim
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RA
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D40 @ 0-4 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83178-173602.

EVALUATION FOR SOIL CORROSION

Soil pH	6.77	
Minimum Resistivity	0.64 ohm-cm (x1000)	
Chloride	374.8 ppm	00.03748 %
Sulfate	295.9 ppm	00.02959 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES	CORROSION TEST RESULTS GREEN ISLAND ROAD LOGISTICS CENTER American Canyon, California	FIGURE A2
		DRAWN BY RWO
		CHECKED BY JRY
		PROJECT MGR GHG
		DATE 11/2020
		WKA NO. 12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2020
Date Submitted 10/05/2020

To: Kylie Lim
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D40 @ 0-4 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83178-173603.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	313.5	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

WallaceKuhl & ASSOCIATES	**CORROSION TEST RESULTS** GREEN ISLAND ROAD LOGISTICS CENTER American Canyon, California	FIGURE A2 DRAWN BY RWO CHECKED BY JRY PROJECT MGR GHG DATE 11/2020 WKA NO. 12883.01

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2020
Date Submitted 10/05/2020

To: Kylie Lim
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D49 @ 0-5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83178-173604.

EVALUATION FOR SOIL CORROSION

Soil pH	6.72	
Minimum Resistivity	1.07 ohm-cm (x1000)	
Chloride	9.0 ppm	00.00090 %
Sulfate	3.0 ppm	00.00030 %

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

WallaceKuhl & ASSOCIATES

CORROSION TEST RESULTS

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A25
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/09/2020
Date Submitted 10/05/2020

To: Kylie Lim
Wallace-Kuhl & Assoc.
3050 Industrial Blvd
West Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12883.01 Site ID : D49 @ 0-5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 83178-173605.

Extractable Sulfate in Water

Type of TEST	Result	Units
Sulfate-SO4	3.1	mg/kg

METHODS
ASTM D-516m from sat.paste extract-reported based on dry wt.

CORROSION TEST RESULTS

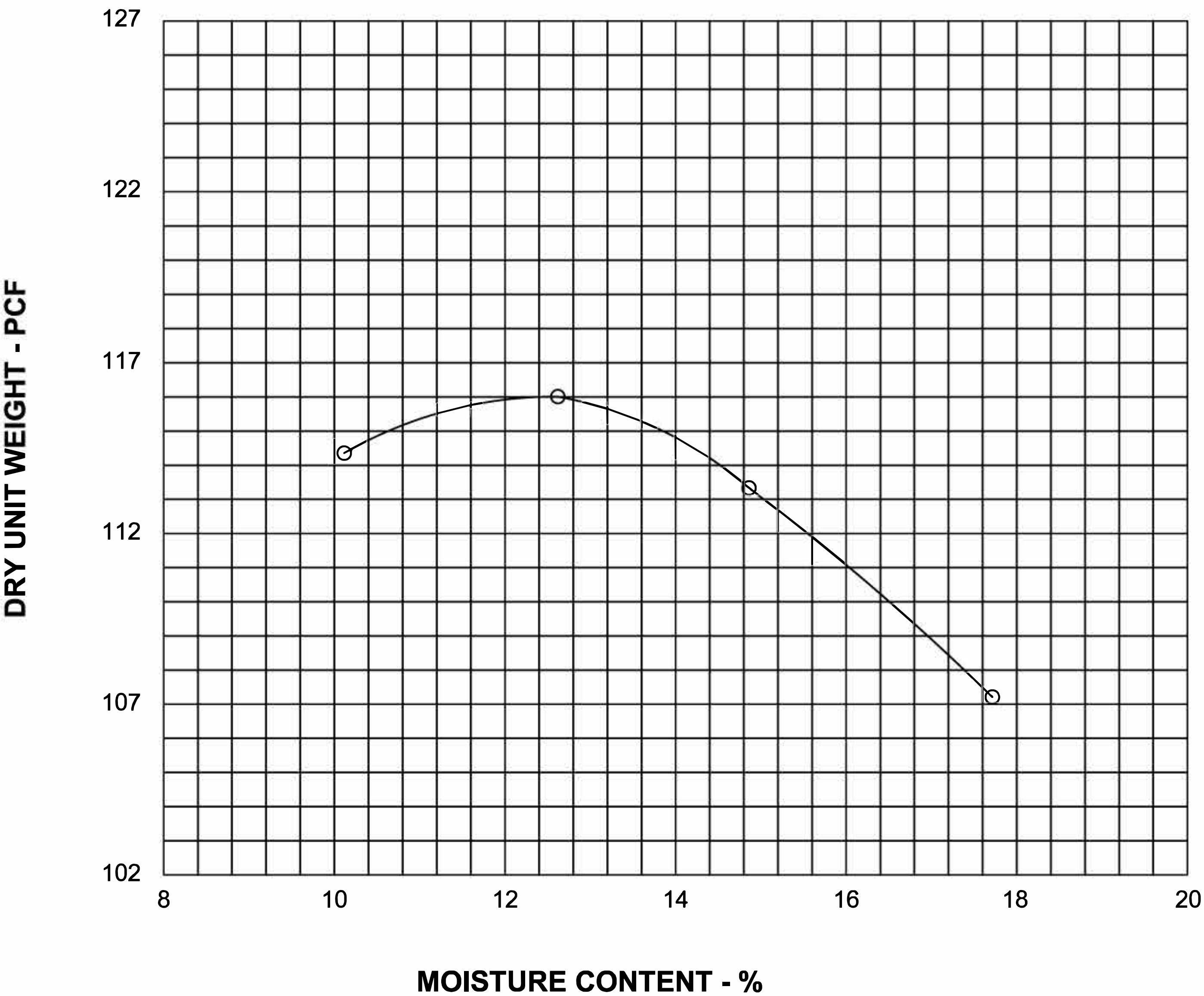
GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

WallaceKuhl & ASSOCIATES

FIGURE	A26
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

MOISTURE-DENSITY RELATION



Material: Dark reddish brown, sandy lean clay

Location: D15 (0' - 5')

Test Method: ASTM D1557 "A"

Maximum Dry Unit Weight - PCF: 116.0

Optimum Moisture - %: 12.4

WallaceKuhl & ASSOCIATES

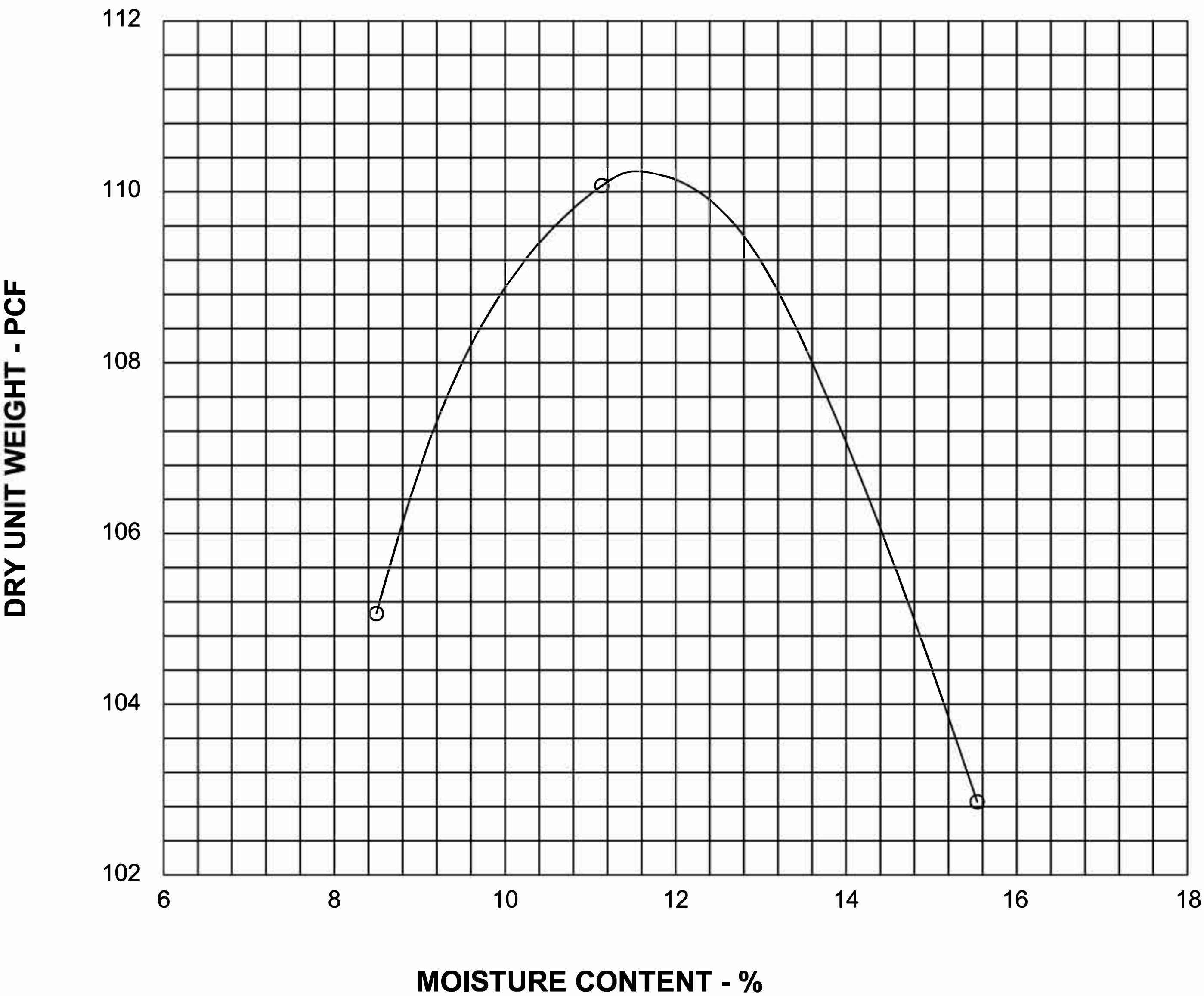
COMPACTION CURVE

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A27
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

MOISTURE-DENSITY RELATION



Material: Light brown, silty lean clay

Location: D38 (0' - 4')

Test Method: ASTM D1557 "A"

Maximum Dry Unit Weight - PCF: 110.5

Optimum Moisture - %: 11.6

WallaceKuhl & ASSOCIATES

COMPACTION CURVE

GREEN ISLAND ROAD LOGISTICS CENTER

American Canyon, California

FIGURE	A28
DRAWN BY	RWO
CHECKED BY	JRY
PROJECT MGR	GHG
DATE	11/2020
WKA NO.	12883.01

APPENDIX B
Guide Earthwork Specifications

[illegible]

GUIDE EARTHWORK SPECIFICATIONS

GREEN ISLAND ROAD LOGISTICS CENTER

[illegible]

[illegible]

[illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

5 [illegible]

[illegible]

[illegible] [illegible]

2[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] M[illegible]D[illegible]2[illegible] [illegible]

[illegible] M[illegible] [illegible]

d[illegible] [illegible] Dr[illegible] [illegible] 5[illegible] [illegible]

[illegible] M[illegible]D[illegible] [illegible] G[illegible] [illegible]

[illegible]

[illegible]	[illegible] M[illegible]D[illegible]	R[illegible]
[illegible]	[illegible]25 [illegible]r [illegible]2[illegible]	H[illegible] D[illegible] [illegible]55[illegible] M[illegible]
[illegible]	[illegible]25	[illegible] 5[illegible] 2[illegible]
[illegible]R[illegible] d[illegible]	[illegible]	[illegible]

2[illegible] dr[illegible] [illegible]5[illegible] [illegible]

[illegible] Gr[illegible]d[illegible]

[illegible]	[illegible]
[illegible]	[illegible]

[illegible] rd[illegible] G[illegible] [illegible]

[illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible]

[illegible] [illegible]

[illegible]

2 [illegible]

[illegible] [illegible]

[illegible] [illegible]

[illegible] [illegible]

d. [illegible]

[illegible] [illegible]

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[illegible] [illegible]

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[illegible] [illegible]

[illegible] [illegible]

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[illegible]

[illegible]

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[illegible]

[illegible]

[illegible]

[illegible]	[illegible]
[illegible]2"	[illegible]
[illegible]"	[illegible]5
[illegible]	[illegible]

[illegible]

[illegible]

[illegible]

d[illegible] [illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

3.5 [illegible]

A. [illegible] **Clay soils shall not be used in fills within the upper twelve inches (12”) of the final concrete foundation slabs or exterior flatwork subgrade, unless the lime-treatment alternative include in the Geotechnical Engineering Report is selected.** [illegible]

B. [illegible]

[illegible]

A. [illegible]

B. [illegible]

[illegible]

A. [illegible]

[illegible] [illegible]

[illegible] [illegible]

d[illegible] [illegible]

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	[illegible]
PROJECT NAME	[illegible]
LOCATION	[illegible]
CLIENT	[illegible]
PROJECT NUMBER	[illegible]
DATE	[illegible]

CORNERSTONE EARTH GROUP

Type of Services	**Geological Fault Investigation**
	New Sanitary Sewer Main
Project Name	**Giovannoni Property**
Location	**Devlin Road** **American Canyon, California**
Client	**Orchard Partners, LLC**
Client Address	**3697 Mt. Diablo Boulevard, Suite 200** **Lafayette, California**
Project Number	**582-4-1**
Date	**November 2, 2017**

PROFESSIONAL GEOLOGIST
RYAN A. McKEE
PG-9448
EXP. 4-30-2018
STATE OF CALIFORNIA

Prepared by Ryan A. McKee, P.G.

REGISTERED GEOLOGIST
CRAIG S. HARWOOD
EXP. 03.31.18
C.E.G. No. 2275
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Prepared by Craig Harwood, C.E.G.

Nicholas S. Devlin, P.E.

CORNERSTONE
EARTH GROUP

TABLE OF CONTENTS

CORNERSTONE
EARTH GROUP

Type of Services	**Geologic Fault Investigation**
Project Name	**Giovannoni Property**
Location	**Devlin Road**
	American Canyon, California

SECTION 1: INTRODUCTION

[illegible]

- [illegible] 25, 2[illegible]

- [illegible]

1.1 PROJECT DESCRIPTION

[illegible]

1.2 SCOPE OF SERVICES

[illegible] 22, 2[illegible]

[illegible] and the Property Owner's representatives to [illegible]

1.3 GROUND WATER EXPLORATION PROGRAM

1.4 FAULT INVESTIGATION EXPLORATION PROGRAM

under the supervision of Cornerstone's Certified Engineering Geologist.

1.5 ENVIRONMENTAL SERVICES

[illegible]

SECTION 2: REGIONAL SETTING

2.1 REGIONAL GEOLOGIC SETTING

[illegible]

[illegible] Calaveras Faults and others (see the "Faulting" section of this report) [illegible]

2.2 LOCAL GEOLOGY

[illegible] 7.5' [illegible] fan deposits ("Qf") [illegible] Late Pleistocene fan deposits ("Qpf") and Early to middle Pleistocene [illegible] fan or terrace deposits ("Qoa"). [illegible] The Qf unit is described as; "[illegible]

[illegible] “Late [illegible] age is indicated by slight dissection and/or the development of alfisols.” [illegible]

[illegible] described as “Fluvial gravel, sand, silt, and clay”. [illegible] in the “Faulting” section of this report [illegible]

2.3 REGIONAL SEISMICITY

[illegible]

[illegible]

Table 1: Approximate Fault Distances

Fault Name	Distance (miles)	Distance (kilometers)
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]

[illegible]

[illegible]

SECTION 3: SITE CONDITIONS

3.1 RECENT HISTORY

[illegible]

[illegible]

[illegible]

3.2 SURFACE DESCRIPTION/GEOMORPHOLOGY

[illegible]

[illegible] the “[illegible]” section of this report [illegible]

3.3 GROUND WATER INVESTIGATION

[illegible]

[illegible]

[illegible]

CORNERSTONE EARTH GROUP

[illegible]

[illegible] the "[illegible]ulting" section of this [illegible]

[illegible]

Table 2: Depth to Ground Water

Boring Number	Corresponding Trench	Depth to Ground Water (feet)	Ground Water Elevation[1] (feet)	Depth of Boring
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible]

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULTING

[illegible]

[illegible]

[illegible]

[illegible]

> “The scarps along this reach align with right [illegible]

[illegible] .” Bryant noted the [illegible] ”

[illegible] DMG [illegible] (“AP Zone”) trends [illegible]

[illegible] (“SNE”) [illegible]

[illegible]

[illegible] rupture didn't jump the gap [illegible] "releasing [illegible]" [illegible]

4.1.1 Proposed Sewer Line

[illegible]

4.2 FAULT SURFACE RUPTURE

[illegible] evaluations, faults are considered "active" if they display evidence of movement within Holocene time (the last 11,000 years), and "potentially active" if they display evidence of movement within [illegible] the "Mitigating Fault Surface Ruptures" section of this report [illegible]

4.2.1 Previous Consultant's Studies

[illegible] locate and/or "clear"[1] [illegible] nt's investigation sites. [illegible]

[1] i.e., prove lack of evidence for fault surface traces trending through the investigation area.

Myers stated: “No prominent slickensid

, or to “clear” building envelopes of

[illegible] the previous consultant's investigations performed within the ar[illegible]

4.2.2 Current Fault Investigation

[illegible] Owner's [illegible]

[illegible] ity's reviewing geologist (Mr. James [illegible]

[illegible] Cornerstone's Certified Engineering Geologist. Trenches were exca[illegible] re Bezore's mapping indicates the surficial deposits

extended through an area where Bezore's map shows four geologic units (which include from

An arcuate subsidiary trace (as in a "flower structure") with down

4.2.3 Encountered Geologic Units in Exploratory Trenches

Unit 1 –

Unit 2 –

Unit 3 –

Unit 4 –

[illegible]

[illegible] “marker [illegible]”. [illegible]

4.2.4 Field Investigation Summary

[illegible]

SECTION 5: CONCLUSIONS

5.1 SUMMARY

[illegible]

5.2 MITIGATING FAULT SURFACE RUPTURE

As already noted, the identified “habitable building exclusion zone shown

SECTION 6: LIMITATIONS

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible] construction as a result of misuse or misinterpretation of Cornerstone's report by others.

[illegible]

SECTION 7: REFERENCES

[illegible]

[illegible]

[illegible]

Bailey Scientific, 1991, Geologic Hazards Investigation, Struble Property, Parcel “B”, Green [illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible] 7.5′ Quadrangle, Napa County, California: A digital database: California Geological Survey, [illegible]

[illegible]

California Division of Mines and Geology (2008), “Guidelines for Evaluating and Mitigating

Geologic map of the Napa 7.5′ Quadrangle, Napa County, California: A digital database:

.5′ Quadrang

CORNERSTONE EARTH GROUP

of Jehovah's Witnesses, Commerce & Hanna Drives, American Canyon, California:

referenced in

revised

CORNERSTONE EARTH GROUP

https://dx.doi.org/10.3133/ofr20141249

Road, Napa County Assessor's Parcel No. 059

[illegible] Assessor's Parcel No. 059 [illegible]

[illegible] located on Napa County Tax Assessor's Parcel No. [illegible]

[illegible]

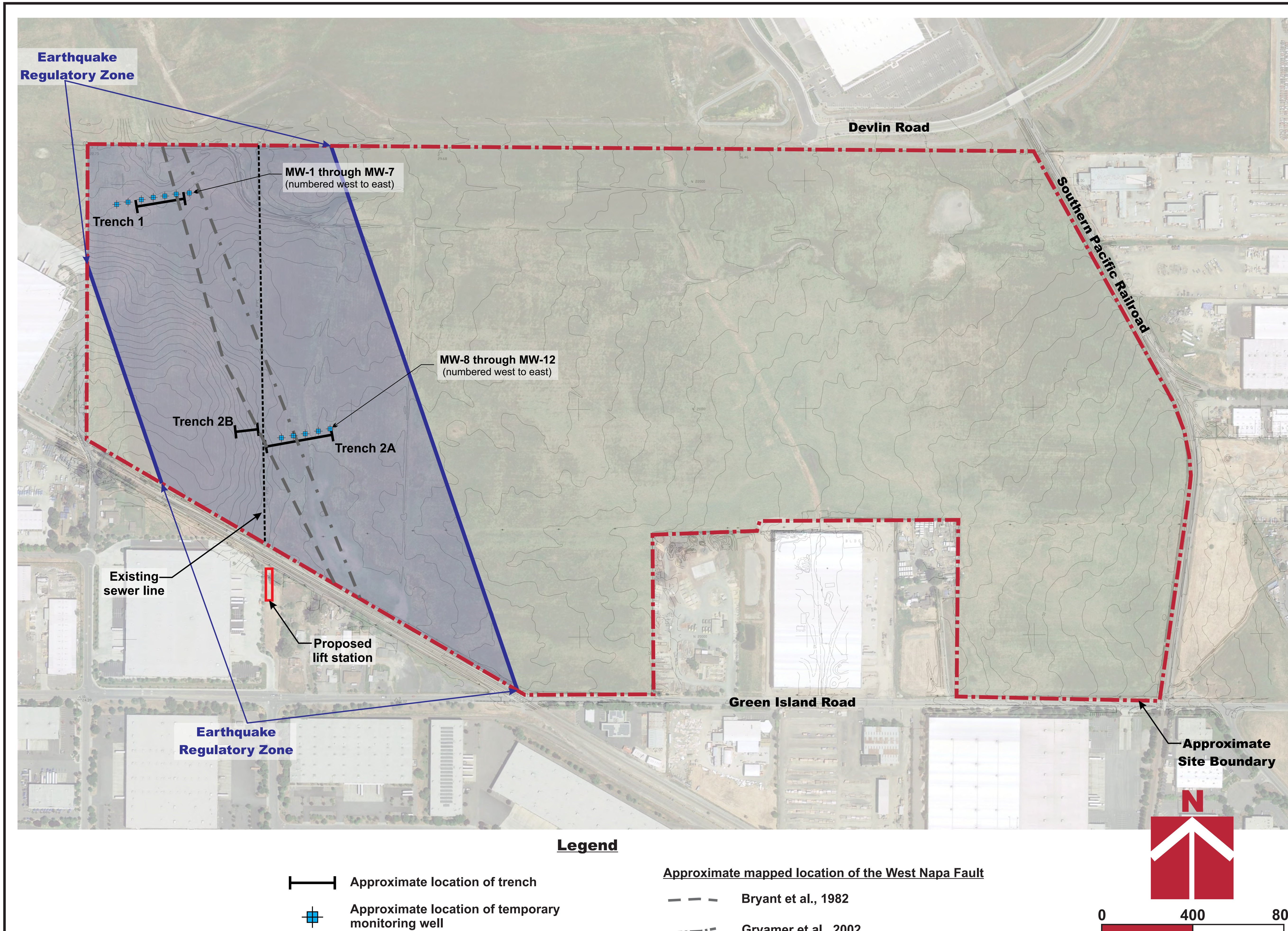
Geologic map of the Sonoma 7.5′ quadrangle, Sonoma and Napa counties, California: A digital

Wagner, D.L. and Gutierrez, C.I., 2010, Preliminary Geologic Map of the Napa 30' X 60'

Aerial Photographs:

CORNERSTONE EARTH GROUP

Vicinity Map	Project Number 582-4-1
Giovannoni Property American Canyon, CA	Figure Number Figure 1
	Date October 2017 / Drawn By RRN



Base by Google Earth, dated 5/20/2017
Overlay: Topographical Map, undated

Project Number 582-4-1

Figure Number Figure 2

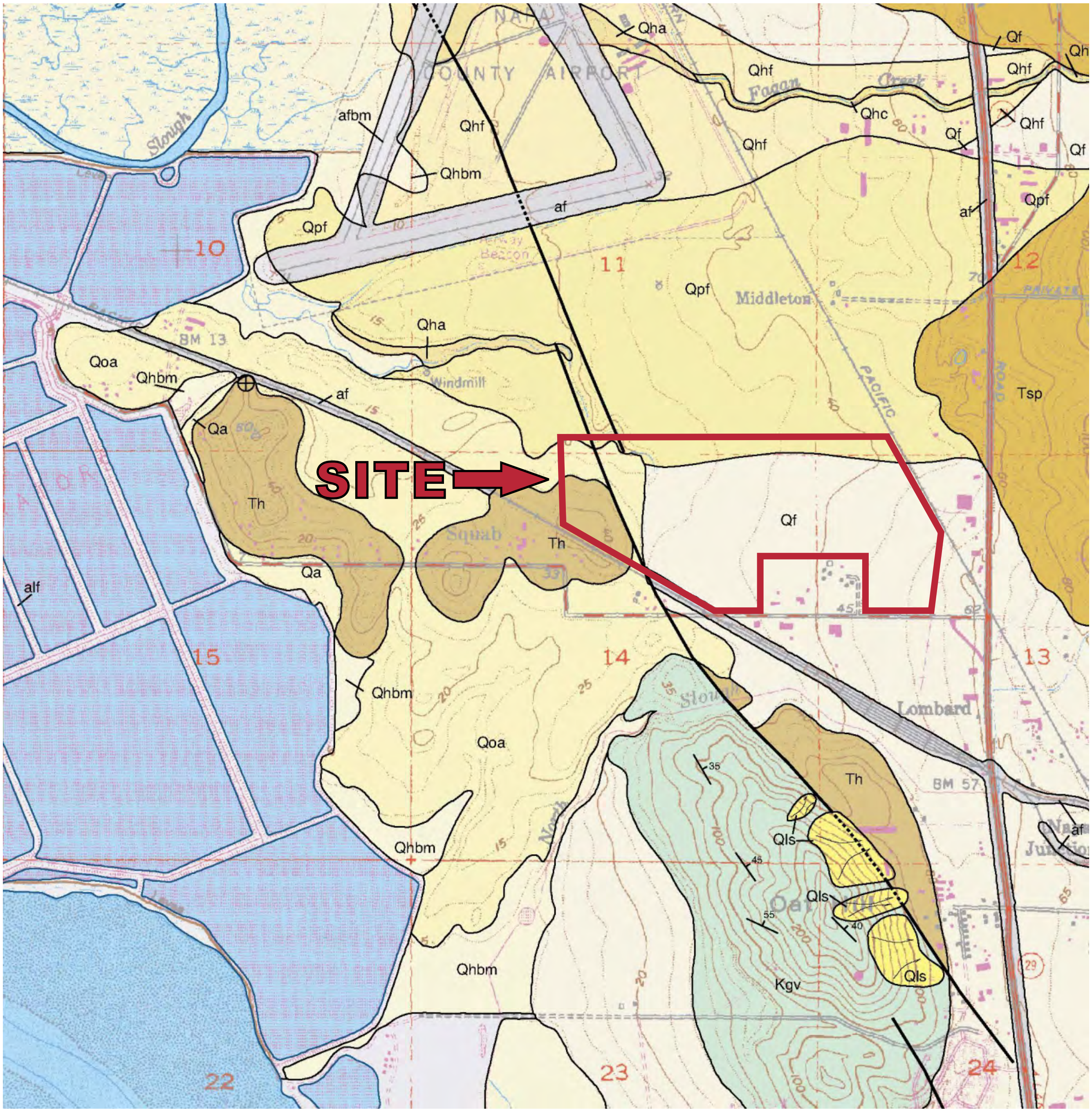
Date October 2017

Drawn By RRN

Site Plan

Giovannoni Property
American Canyon, CA

CORNERSTONE EARTH GROUP



GEOLOGIC MAP OF THE CUTTINGS WHARF 7.5' QUADRANGLE
NAPA AND SOLANO COUNTIES, CALIFORNIA: A DIGITAL DATABASE
by Stephen Bezore, Carolyn E. Randolph-Loar, and Robert C. Witter

Geologic Units

af	Artificial fill
Qha	Holocene alluvium, undifferentiated
Qf	Late Pleistocene to Holocene fan deposits
Qpf	Late Pleistocene fan deposits
Qoa	Early to middle Pleistocene fan or terrace deposits
Th	Huichica Formation (Pliocene) Fluvial gravel, sand, silt, and clay

Explanation

Contact- dashed where approximate, dotted where concealed

Fault - solid where accurately located, dashed where approximately located; short dash where inferred; dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

CORNERSTONE EARTH GROUP

Vicinity Geologic Map	Project Number 582-4-1
Giovannoni Property **American Canyon, CA**	Figure Number Figure 3
	Date October 2017
	Drawn By RRN

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200		? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000		?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000 – 4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by USGS, California Quaternary Faults, Cooperator California Geological Survey

Project Number 582-4-1
Figure Number Figure 4
Date October 2017
Drawn By RRN

Regional Fault Map

Giovannoni Property
American Canyon, CA

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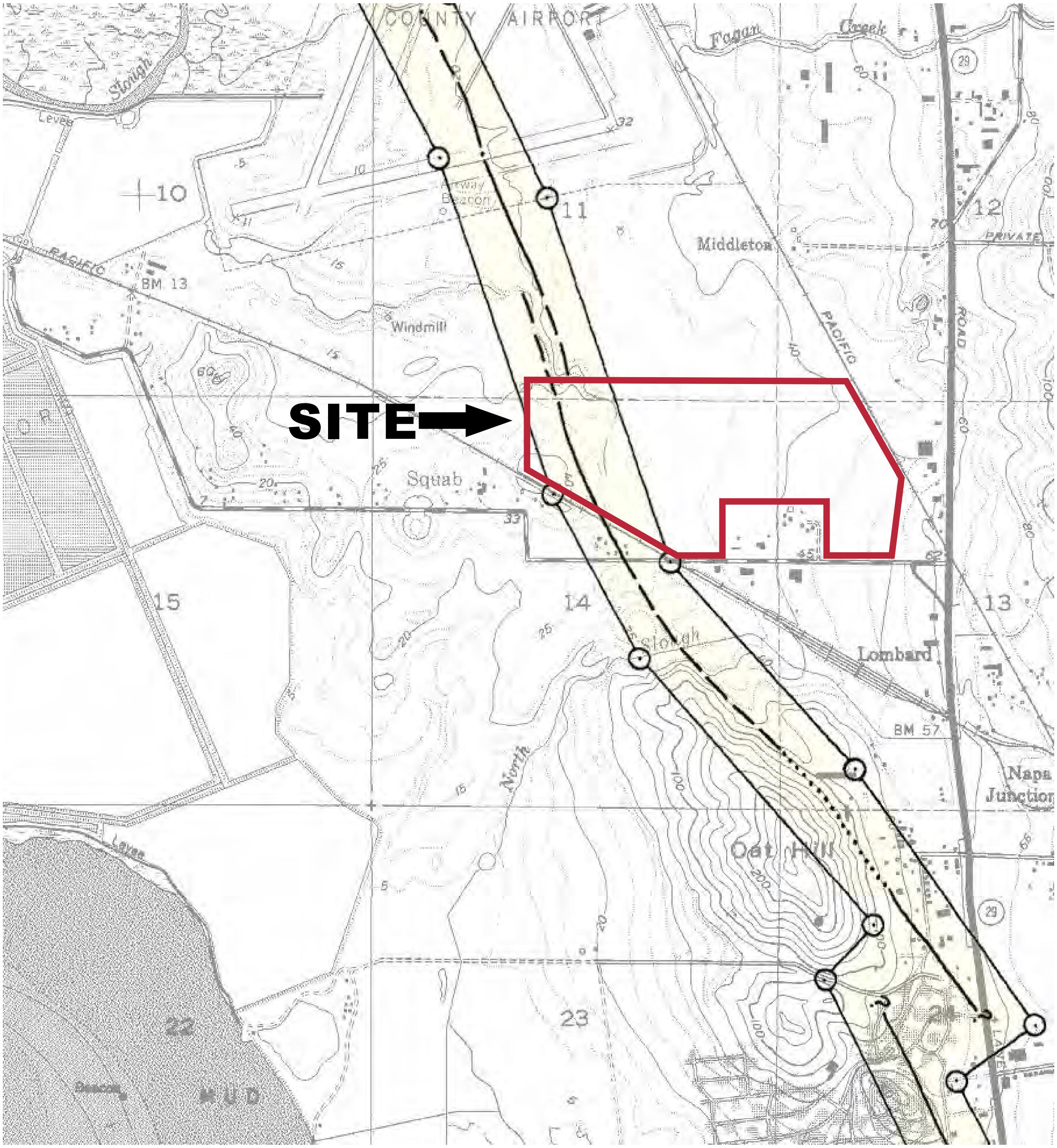


MAP EXPLANATION

Potentially Active Faults

1906 C — Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect encircled turning points so as to define special studies zone segments.

Seaward projection of zone boundary.

Base: State of California, Special Studies Zones, Cutting Wharf Quadrangle

N

0 2000 4000

APPROXIMATE SCALE (FEET)

CORNERSTONE EARTH GROUP

AP Earthquake Regulatory Zone Map

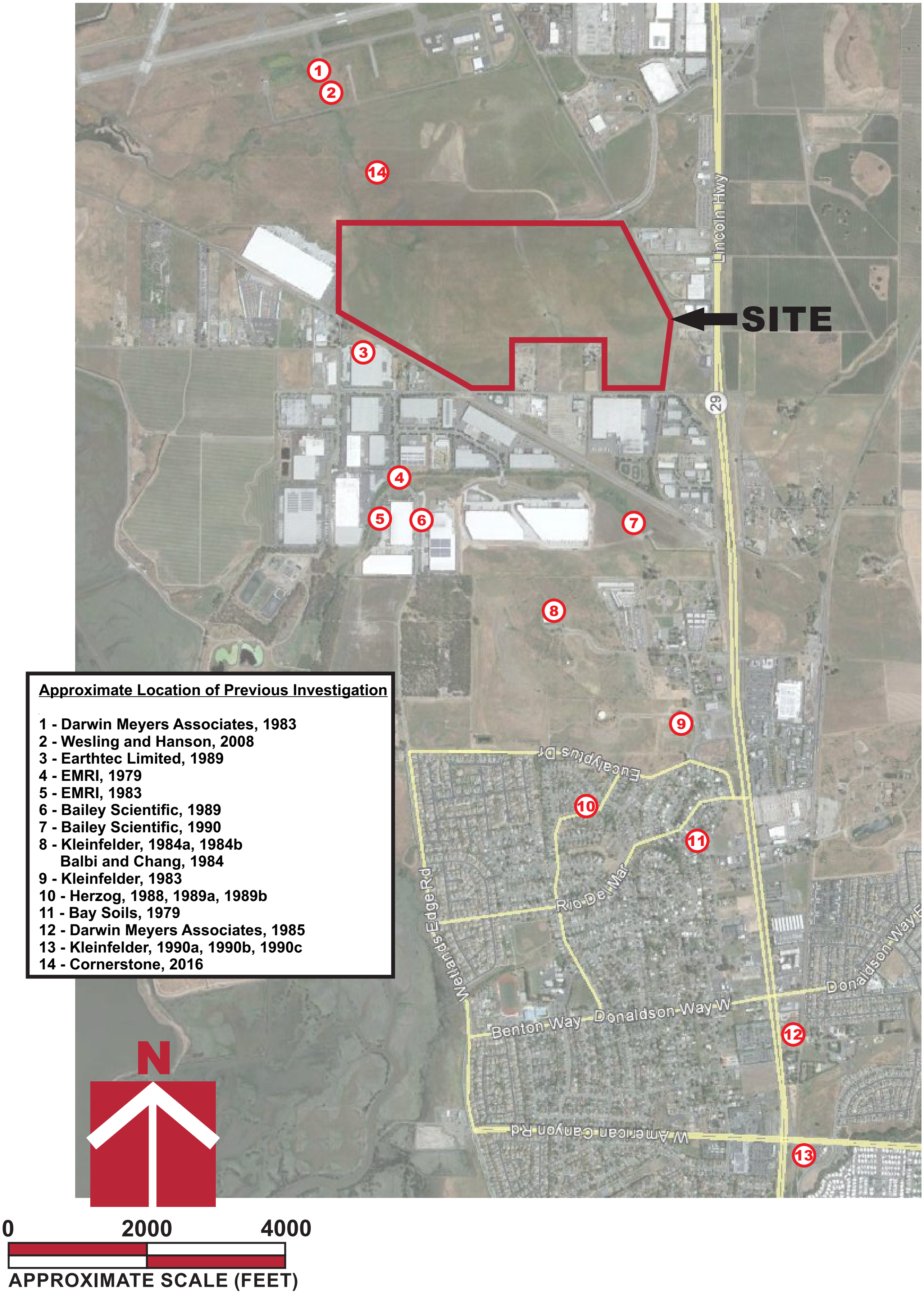
Giovannoni Property
American Canyon, CA

Project Number: 582-4-1

Figure Number: Figure 5

Date: October 2017

Drawn By: RRN



Base by Google Earth, dated 5/26/2014

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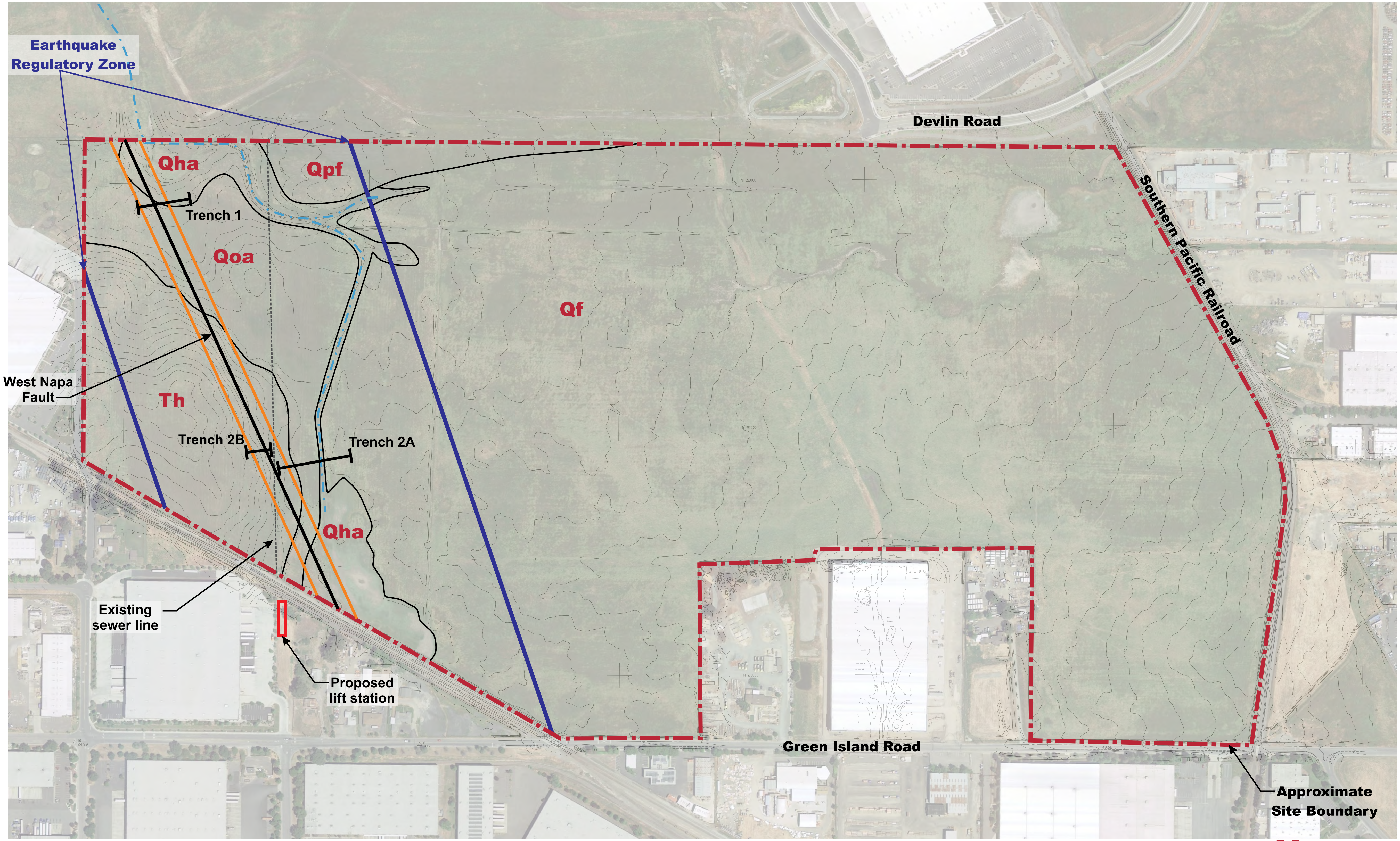
Previous Local Investigations within 2¼ Miles

Giovannoni Property
American Canyon, CA

Project Number	582-4-1
Figure Number	Figure 6
Date	October 2017
Drawn By	RRN



Geologic Units

Qha	Holocene alluvium, undifferentiated
Qf	Late Pleistocene to Holocene fan deposits
Qpf	Late Pleistocene fan deposits
Qoa	Early to middle Pleistocene fan or terrace deposits
Th	Huichica Formation (Pliocene) Fluvial gravel, sand, silt, and clay

Legend

- Approximate location of trench
- Approximate Earthquake Regulatory Zone
- Approximate location of seasonal unnamed creek
- Approximate location of West Napa Fault (this investigation)
- 50 ft setback

N

0 400 800
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 5/20/2017
Overlay: Topographical Map, undated

Project Number 582-4-1
Figure Number Figure 7
Drawn By RRN
Date October 2017

Fault Investigation

Giovannoni Property
American Canyon, CA

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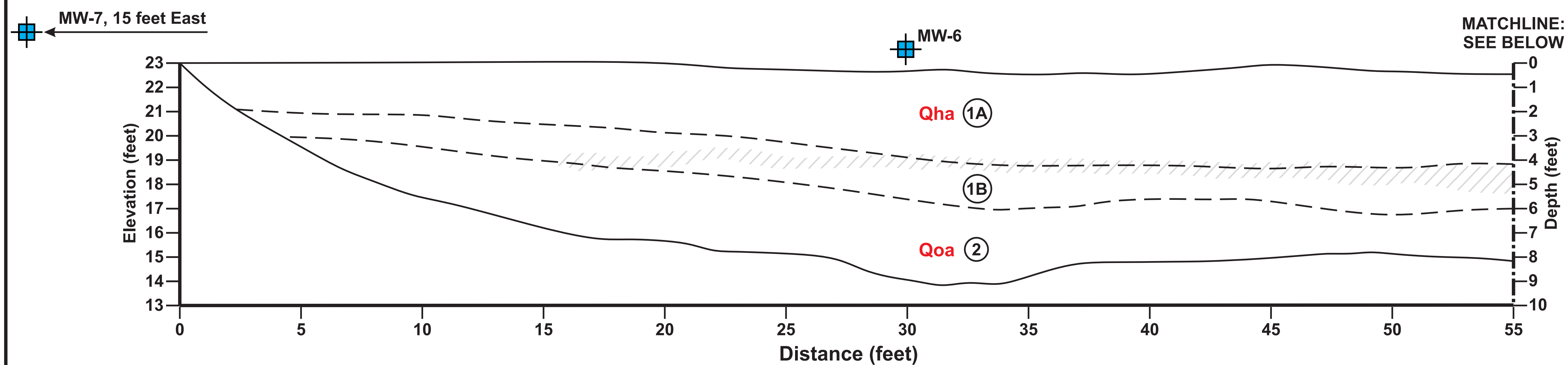
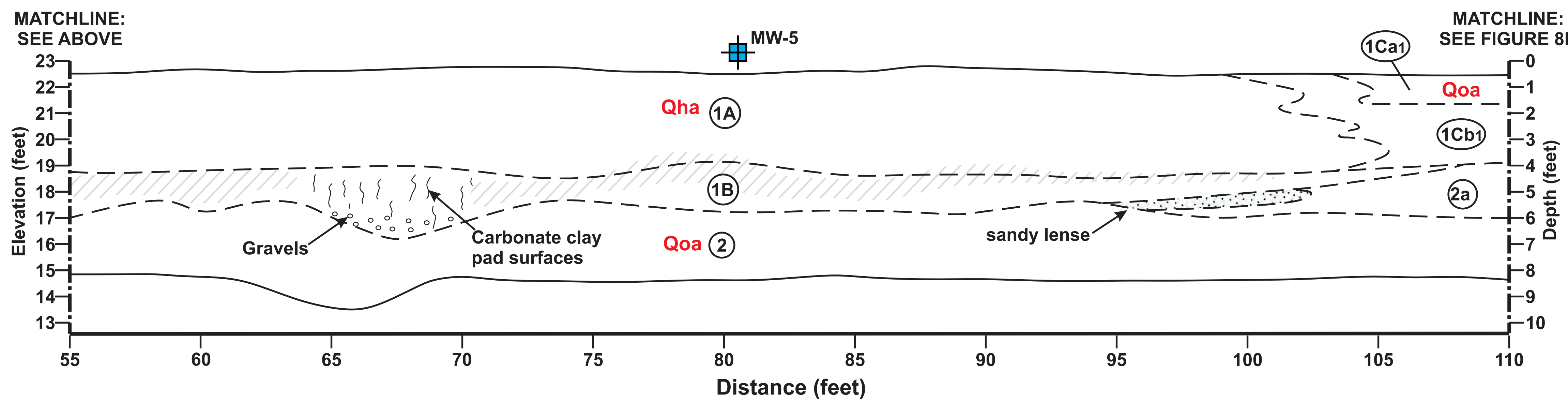
Trench 1

(View Looking South)
1"=5' H:V

S 80 W





Disseminated carbonate (I & II)

Unit 1 – Holocene soils

Qha **1A** **Clay [CL/CH]:**
stiff to very stiff, moist, dark brown to brown [10YR 3-2/1] with fine sand and occasional fine to coarse gravels. Block and prismatic A- and B-horizon soils.

1B **Sandy clay [CL]:**
stiff, moist, brown to gray brown [2.5YR 4-3/2] with fine to medium sand and occasional fine sub-rounded gravels. Weak to moderate blocky B-horizon soils.

Qoa **1Ca1** Dark gray [10YR 4/1] clay with fine sands. A-horizon soils with moderately granular to weak blocky structure.

1Cb1 Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.

Unit 2 – Holocene to Upper Pleistocene

Qoa **2** **Clayey sand and sandy clay with gravel [SC]:**
dense and stiff, moist to wet, light brown to gray [2.5Y 6/2-4/3] fine to medium sands and clays.

Project Number 582-4-1
Figure Number Figure 8A
Date October 2017
Drawn By RRN

Trench 1

Giovannoni Property
American Canyon, CA

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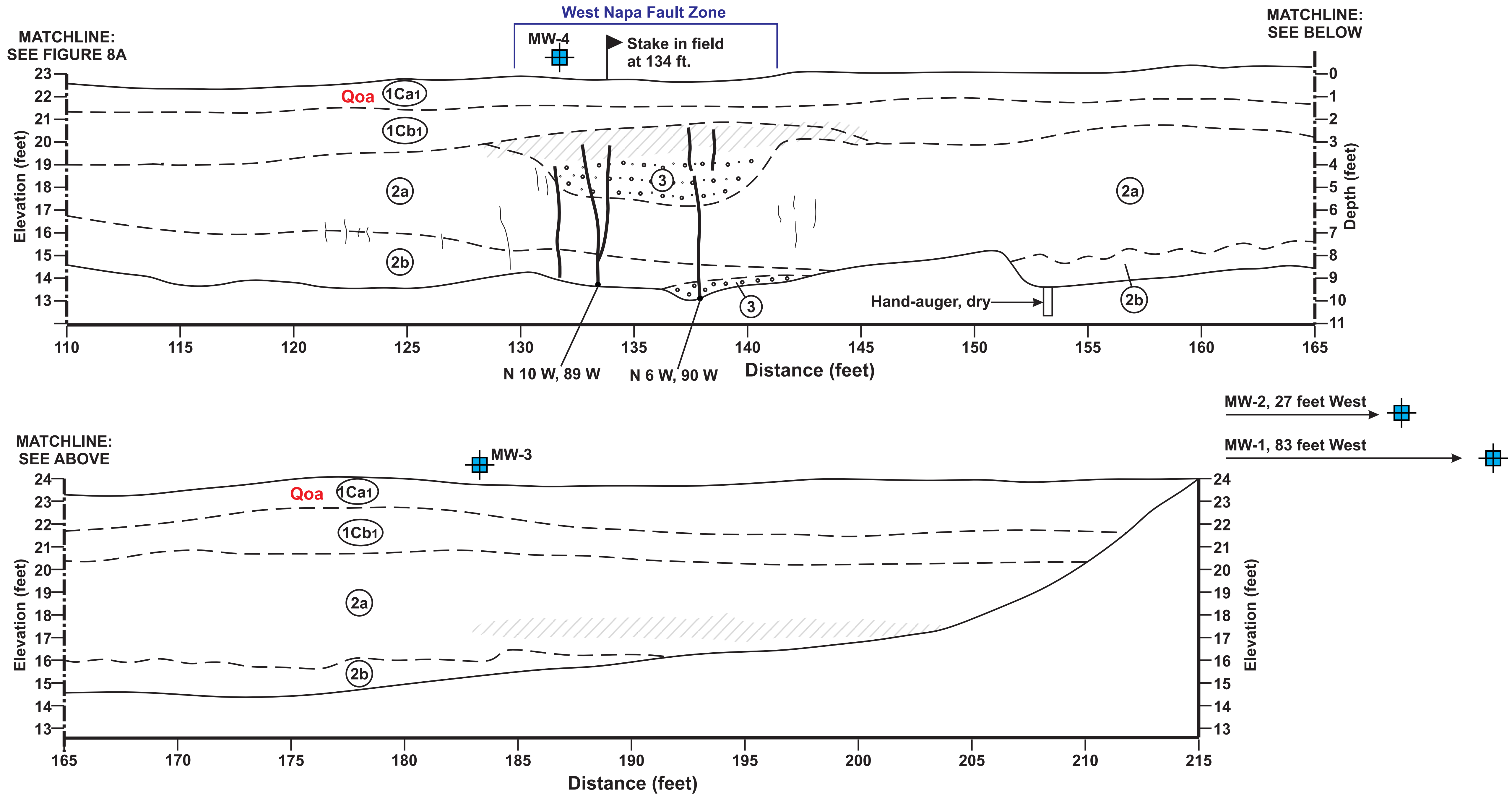
Trench 1 (continued)

(View Looking South)
1"=5' H:V

S 88 W



Unit 1 – Holocene soils

Disseminated carbonate (I & II)

Qoa	1Ca1	Dark gray [10YR 4/1] clay with fine sands. A-horizon soils with moderately granular to weak blocky structure.
	1Cb1	Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.

Unit 2 – Holocene to Upper Pleistocene

Qoa	2a	**Sandy clay [SC]:** stiff, moist to wet, light brown [2.5Y 6/3] with fine to medium sands.
	2b	**Clayed sands [CL]:** dense, moist to wet, light brown [2.5Y 5/2] with occasional sub-angular to sub rounded gravels. Orange mottles [10YR 5-4/4] common above the saturated zone.

Unit 3 – sandy channel deposits

Qoa	3	**Clayey sand with gravels [SC]:** dense, moist, light brown to brown [10YR 4/2-3/4] with sub-angular to sub-rounded shale and chert.

Project Number 582-4-1

Figure Number Figure 8B

Drawn By RRN

Date October 2017

Trench 1 (continued)

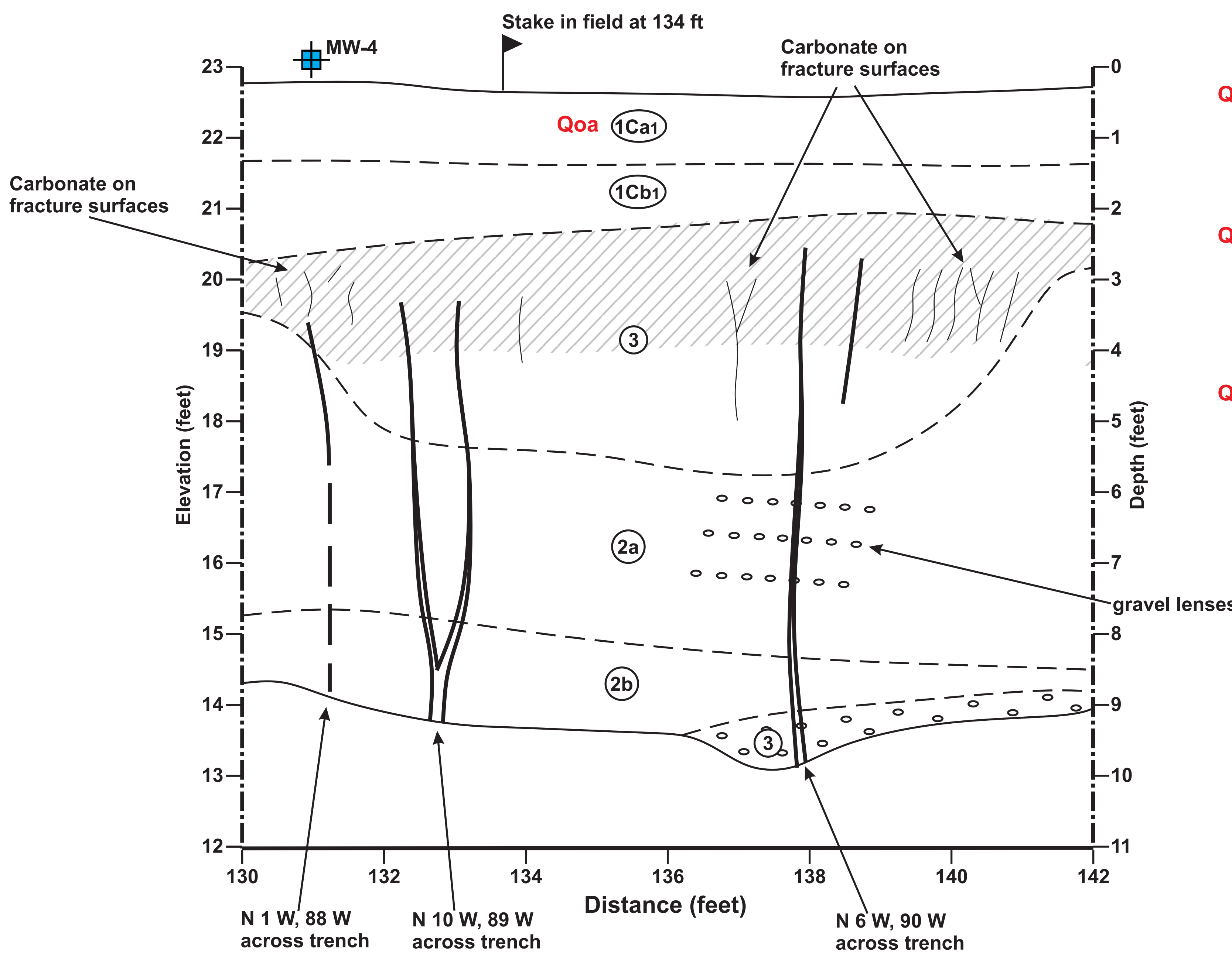
Giovannoni Property
American Canyon, CA

CORNERSTONE EARTH GROUP

Detail of Trench 1 (130' to 142')

(View Looking South)
1"=5' H:V

S 88 W



Unit 1 – Holocene soils

Qoa **1Ca1** Dark gray [10YR 4/1] clay with fine sands. A-horizon soils with moderately granular to weak blocky structure.

1Cb1 Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.

Unit 2 – Holocene to Upper Pleistocene

Qoa **2a** **Sandy clay [SC]:** stiff, moist to wet, light brown [2.5Y 6/3] with fine to medium sands.

2b **Clayed sands [CL]:** dense, moist to wet, light brown [2.5Y 5/2] with occasional sub-angular to sub rounded gravels. Orange mottles [10YR 5-4/4] common above the saturated zone.

Unit 3 – sandy channel deposits

Qoa **3** **Clayey sand with gravels [SC]:** dense, moist, light brown to brown [10YR 4/2-3/4] with sub-angular to sub-rounded shale and chert.

Disseminated carbonate (I & II)

Project Number 582-4-1

Figure Number Figure 8C

Drawn By RRN

Date October 2017

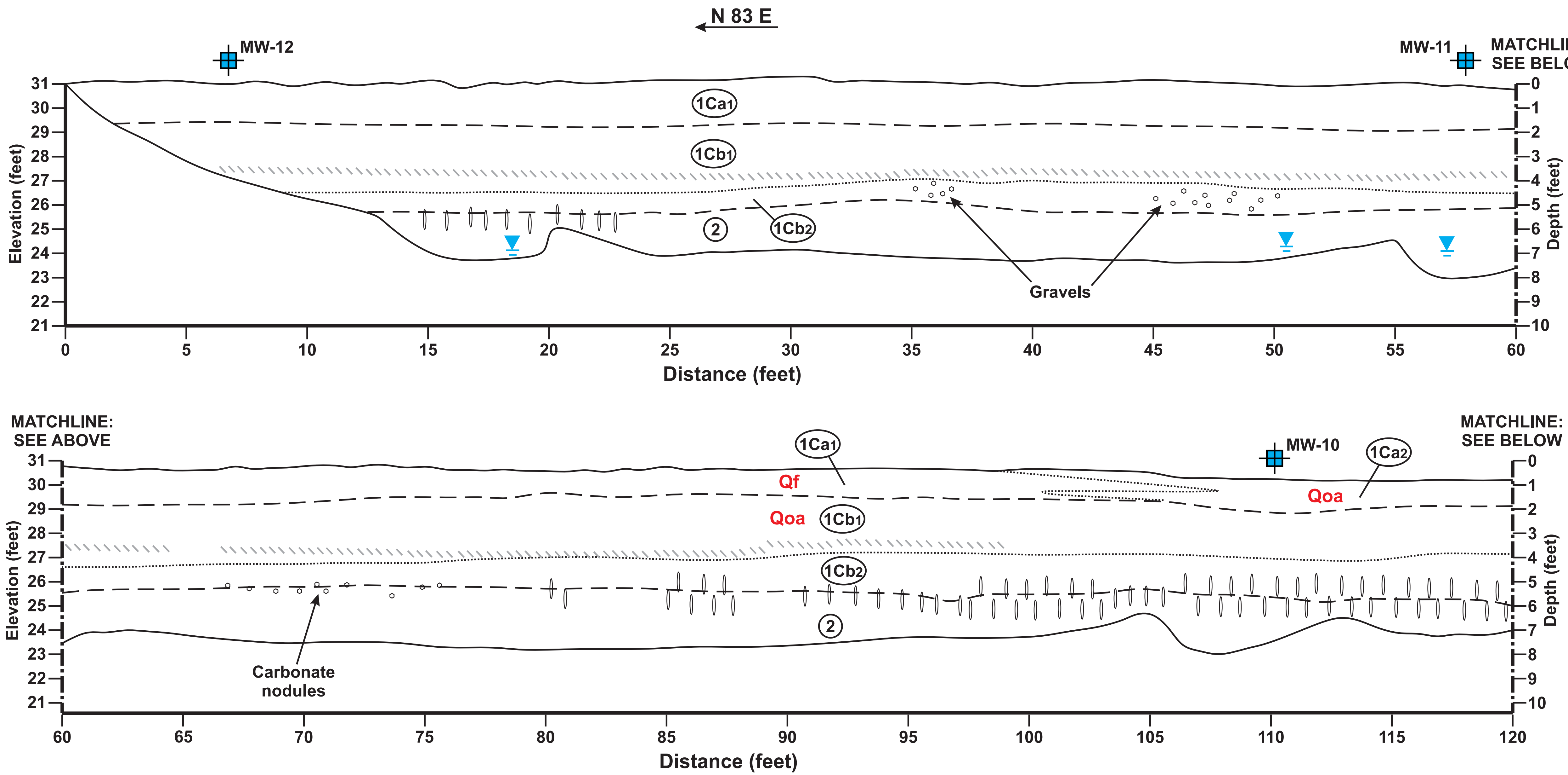
Detail of Trench 1 (130' to 142')

Giovannoni Property
American Canyon, CA

CORNERSTONE EARTH GROUP

Trench 2A

(View Looking South)
1"=5' H:V



Hematite concretions

Disseminated carbonate and carbonate concretions (I, II, & III)

Unit 1 – Holocene soils

Qf	1Ca1	Dark gray [10YR 4/1] clay with fine sands. A-horizon soils with moderately granular to weak blocky structure.
Qoa	1Ca2	Dark gray [5YR 4/1] clay with fine sands and occasional well-rounded gravels at base of horizon. A-horizon soils with massive to moderately granular structure.
	1Cb1	Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.
	1Cb2	Brown to gray brown [10YR 4/3 – 5/2] clay with fine sands. B-horizon soils with moderate blocky structure.

Unit 2 – Holocene to Upper Pleistocene

Qoa	2	**Clayey sand and sandy clay with gravel [SC]:** dense and stiff, moist to wet, light brown to gray [2.5Y 6/2-4/3] fine to medium sands and clays.

Project Number 582-4-1

Figure Number Figure 9A

Drawn By RRN

Date October 2017

Trench 2A

Giovannoni Property
American Canyon, CA

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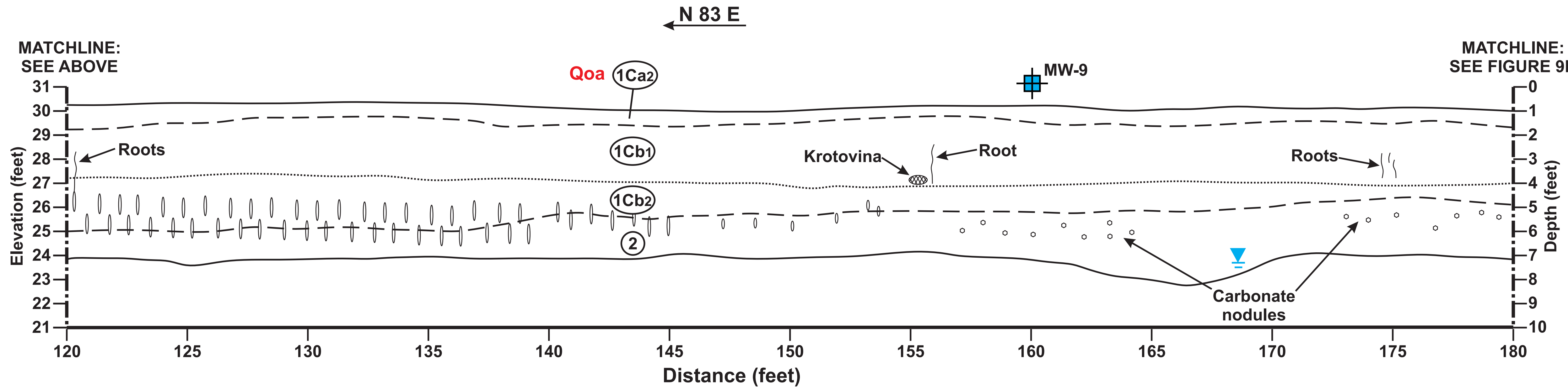
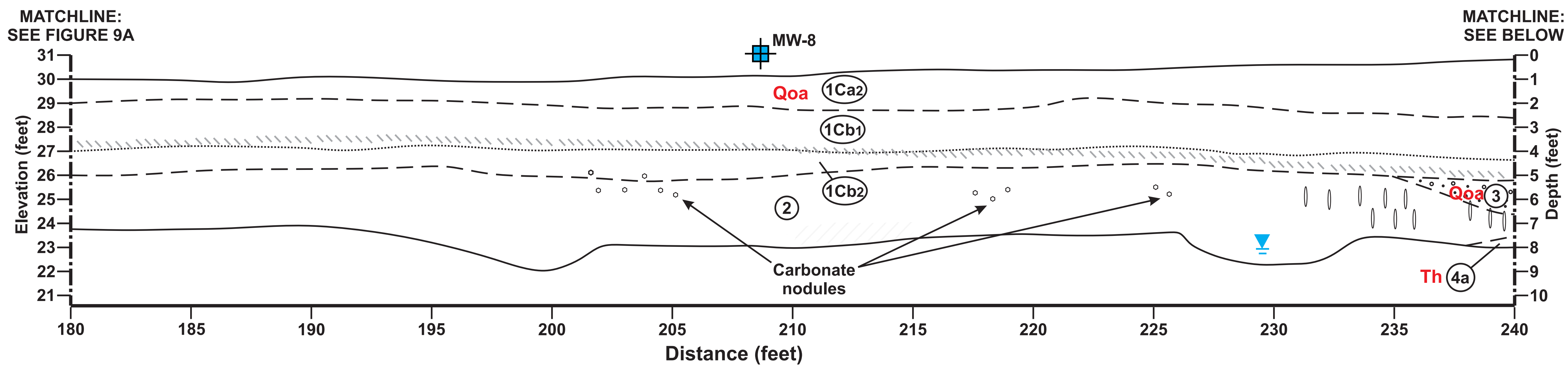
Trench 2A (continued)

(View Looking South)
1"=5' H:V





Disseminated carbonate (I & II)

Hematite concretions

Disseminated carbonate and carbonate concretions (I, II, & III)

Unit 1 – Holocene soils

Qoa **1Ca2** Dark gray [5YR 4/1] clay with fine sands and occasional well-rounded gravels at base of horizon. A-horizon soils with massive to moderately granular structure.

1Cb1 Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.

1Cb2 Brown to gray brown [10YR 4/3 – 5/2] clay with fine sands. B-horizon soils with moderate blocky structure.

Unit 2 – Holocene to Upper Pleistocene

Qoa **2** **Clayey sand and sandy clay with gravel [SC]:** dense and stiff, moist to wet, light brown to gray [2.5Y 6/2-4/3] fine to medium sands and clays.

Unit 3 – sandy channel deposits

Qoa **3** **Clayey sand with gravels [SC]:** dense, moist, light brown to brown [10YR 4/2-3/4] with sub-angular to sub-rounded shale and chert.

Unit 4 - Huichica Formation (Pliocene)

Th **4a** **Sandy clay [CL]:** stiff to very stiff, moist, clay with fine sands, yellowish brown [10YR 5/6], brown [7.5YR 4-5/4-6] and reddish brown [2.5YR 6/3]. Bedding present between sandier and clayey packets of sediment.

Project Number 582-4-1

Figure Number Figure 9B

Drawn By RRN

Date October 2017

Trench 2A (continued)

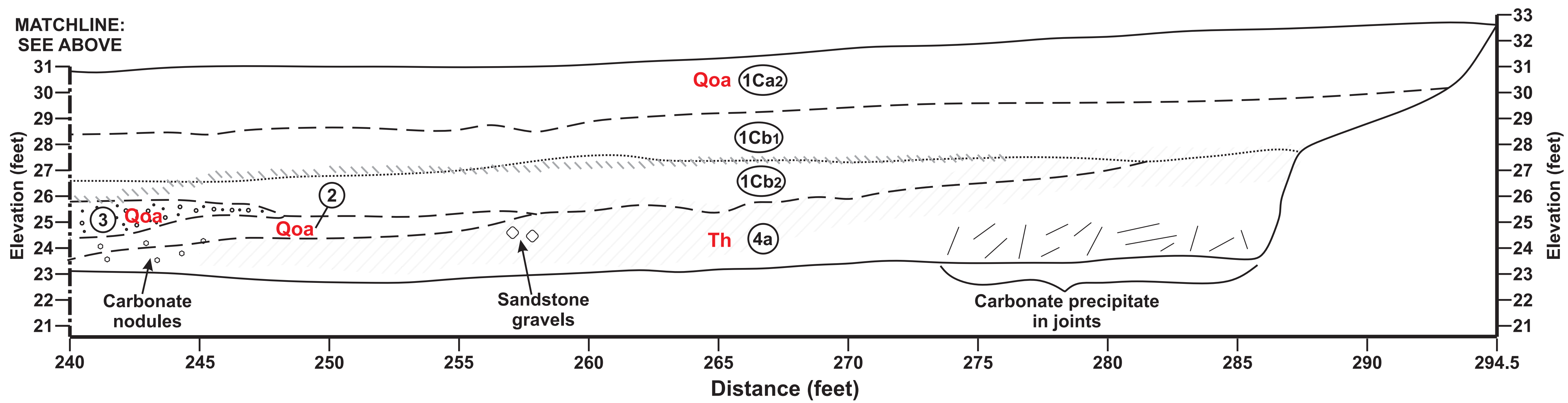
Giovannoni Property
American Canyon, CA

CORNERSTONE EARTH GROUP

Trench 2A (continued)

(View Looking South)
1"=5' H:V

N 83 E



Disseminated carbonate (I & II)

Hematite concretions

Unit 1 – Holocene soils

Qoa **1Cb1** Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.

1Cb2 Brown to gray brown [10YR 4/3 – 5/2] clay with fine sands. B-horizon soils with moderate blocky structure.

Unit 2 – Holocene to Upper Pleistocene

Qoa **2** **Clayey sand and sandy clay with gravel [SC]:** dense and stiff, moist to wet, light brown to gray [2.5Y 6/2-4/3] fine to medium sands and clays.

Unit 3 – sandy channel deposits

Qoa **3** **Clayey sand with gravels [SC]:** dense, moist, light brown to brown [10YR 4/2-3/4] with sub-angular to sub-rounded shale and chert.

Unit 4 - Huichica Formation (Pliocene)

Th **4a** **Sandy clay [CL]:** stiff to very stiff, moist, clay with fine sands, yellowish brown [10YR 5/6], brown [7.5YR 4-5/4-6] and reddish brown [2.5YR 6/3]. Bedding present between sandier and clayey packets of sediment.

Project Number 582-4-1

Figure Number Figure 9C

Date October 2017

Drawn By RRN

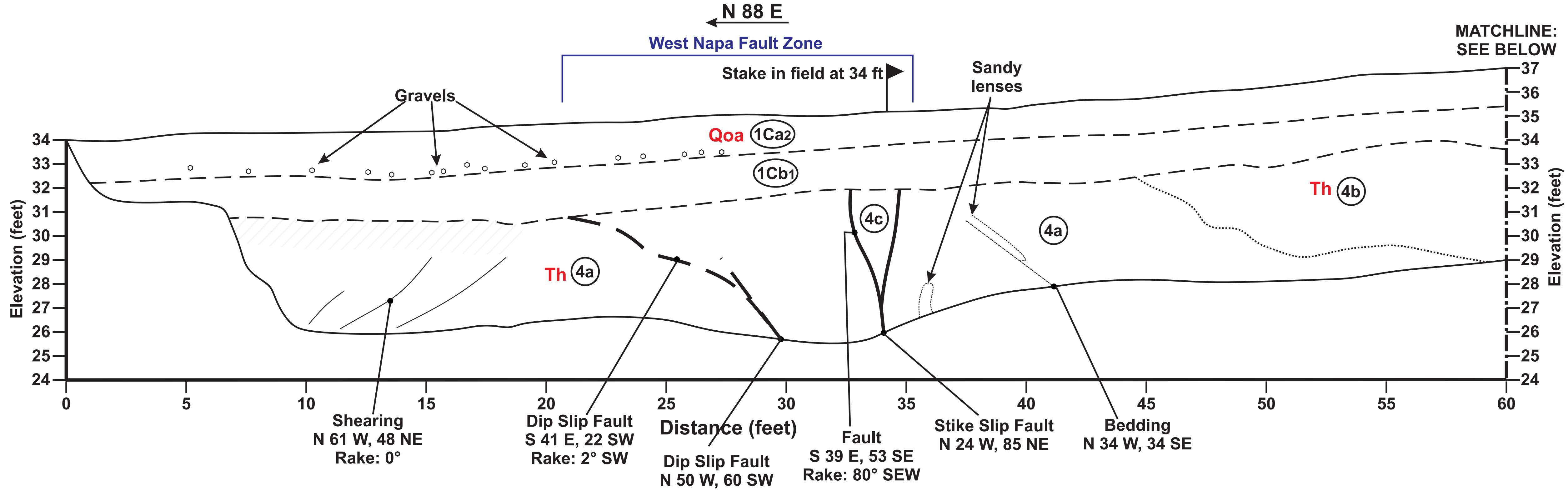
Trench 2A (continued)

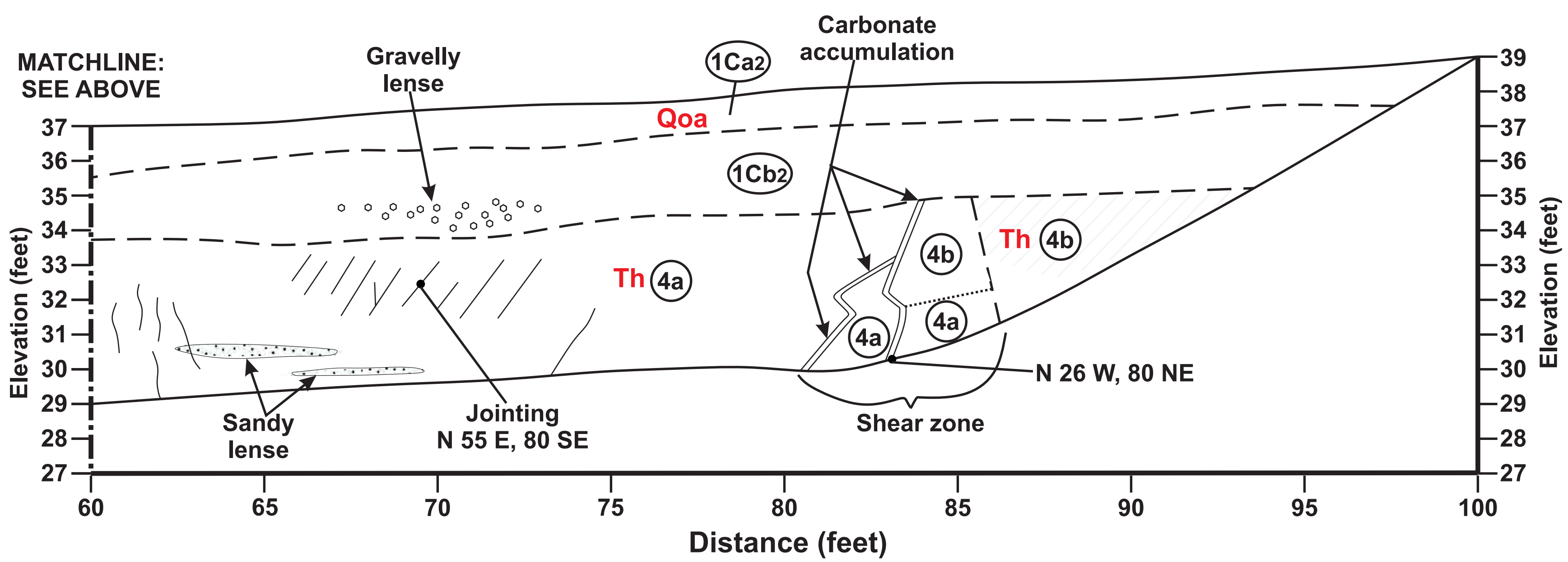
Giovannoni Property
American Canyon, CA

CORNERSTONE EARTH GROUP

Trench 2B

(View Looking South, north wall cleaned, log is reversed)
1"=5' H:V





Unit 1 – Holocene soils

Qoa **1Ca2** Dark gray [5YR 4/1] clay with fine sands and occasional well-rounded gravels at base of horizon. A-horizon soils with massive to moderately granular structure.

1Cb1 Brown [10YR 4/3] clay with fine sands and occasional fine sub-angular gravels. B-horizon soils with strong blocky to prismatic structure.

Unit 4 - Huichica Formation (Pliocene)

Th **4a** **Sandy clay [CL]:** stiff to very stiff, moist, clay with fine sands, yellowish brown [10YR 5/6], brown [7.5YR 4-5/4-6] and reddish brown [2.5YR 6/3]. Bedding present between sandier and clayey packets of sediment.

4b **Clayey sands with gravels [SC]:** very dense, moist, yellowish red to reddish brown [5YR 4-5/4-6].

4c **Clay with silt [CL]:** medium stiff, moist, olive yellow [2.5Y 6/6], clay with silt and some fine to coarse angular to sub-angular gravel and fine carbonate nodules. Carbonate accumulations formed on the vertical margins where the slide planes of the fault were observed. Fault gouge.

Disseminated carbonate (I & II)

Project Number 582-4-1

Figure Number Figure 9D

Date October 2017

Drawn By RRN

Trench 2B

Giovannoni Property
American Canyon, CA

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CORNERSTONE EARTH GROUP

APPENDIX A : SEISMIC DESIGN AND ASSESSMENT GUIDLINES

Catalog No. L51927

Technology for Energy Pipelines

Guidelines for the Seismic Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines

Contract PR-268-9823

Prepared for the
Pipeline Design, Construction & Operations Technical Committee

of
Pipeline Research Council International, Inc.

Prepared by the following Research Agencies:

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Publication Date:
October 1, 2004

Pipeline Research Council International Catalog No. L51927
Price:

PRCI Reports are Published by **Technical Toolboxes, Inc.**

TECHNICAL TOOLBOXES

3801 Kirby Drive, Suite 340
Houston, Texas 77098
Tel: 713-630-0505
Fax: 713-630-0560
Email: info@ttoolboxes.com

Acknowledgements

This report was prepared for the PRCI Design, Construction, and Operations Technical Committee. Ad hoc steering group members participating on this project over its five-year duration include the following:

Lynn A. Christensen
Colorado Interstate Gas Company

Richard W. Gailing (Chair)
Southern California Gas Company

Alan G. Glover
TransCanada PipeLines Ltd.

Stanley R. Kitt
Foothills Pipe Lines, Ltd.

Martin McLamb
BP

Moness Rizkalla
Visitless Integrity Assessment Ltd.

Joe Zhou
TransCanada PipeLines Ltd.

This project would not have been possible without the financial contributions of Southern California Gas Company and the authors greatly appreciate their support. We also recognize M. McLamb and J. Zhou for their contributions to refine the material on strain acceptance criteria and M. Rizkalla for his valuable review comments on early versions of the guidelines. We are especially thankful to R. Gailing for his leadership in support of the project and his thorough review of the various drafts of these guidelines.

The authors are indebted to the individuals contracted to serve as expert reviewers during the development of these guidelines. Their comments and suggestions were valuable in refining the content and organization of these guidelines.

Jean M.E. Audibert (2000 review)
Fugro-McClelland Marine Geosciences, Inc.

Michael J. Rosenfeld (1999 review)
Kiefner and Associates, Inc.

James D. Hart (2000 review)
SSD, Inc.

Robert W. Warke (1999 review)
Southwest Research Institute, Inc.
(Currently at Le Tourneau University)

William A. Lettis (2000 review)
William A. Lettis and Associates, Inc.

Thomas J.E. Zimmerman (1999 and 2000 review)
Center for Engineering Research, Ltd.

David W. Murray (1999 review)
D.W. Murray Engineering, Ltd.

Finally, we are grateful to E.L. Von Rosenberg, Consulting Engineer, Materials and Welding Technology, for his valuable assistance in improving the guidance related to specifications of pipe material and welding procedures provided in the commentary.

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Introduction

Project PR-268-9823 was initiated in February 1998 by PRCI to provide current seismic guidelines for the design and assessment of natural gas transmission pipelines. These guidelines were refined using two rounds of review by outside technical experts in 1999 and 2000. The PRCI ad hoc steering group for the project also provided regular input regarding the scope and technical content for these guidelines. A decision was made in late 2000 to expand the scope to liquid hydrocarbon pipelines (crude oil and refined products) based upon the identical analytical treatment of seismic design and assessment of these types of pipelines.

Much of the current seismic practice can be traced to research conducted in the 1970s to develop design criteria and procedures for the Trans-Alaska oil pipeline. Many of the procedures in these guidelines have been implemented, in one form or another, by the major natural gas pipeline operators in California. The recognition of credible seismic hazards in other parts of the United States has been the primary driver for extending current practices in California to the rest of the United States. The experience in designing pipelines in seismically active regions of the United States has also provided a basis for specifying pipeline seismic design practices in other parts of the world.

The first comprehensive set of seismic guidelines for oil and gas pipeline systems was published in 1984 by the American Society of Civil Engineers[1]. These guidelines were the product of a collaborative effort among pipeline operators, consultants, and researchers. The 1984 ASCE guidelines have become accepted as a de-facto standard for seismic design within the oil and gas industry. Since their publication in 1984, considerable advancements have been made in seismic hazard definition and the understanding of pipeline response to earthquake hazards.

The present guidelines are intended to be an update of the 1984 ASCE guidelines relating to buried pipelines transporting natural gas and liquid hydrocarbons. To that end, preparation of these guidelines attempts to take full advantage of recent research findings with respect to soil loading on buried pipelines, acceptable strain-based pipeline limit states, and current analysis tools. To the extent possible, these guidelines have been stated in a concise manner with minimal discussion so as to keep the main text clear and concise. More in-depth discussions of the bases for recommendations in these guidelines are provided in a comprehensive Commentary at the end of this document. These guidelines are the product of considerable peer review by experts within the pipeline industry, key investigators at various research organizations, and consultants providing seismic design and assessment services to natural gas pipeline operators.

There continue to be advancements in understanding pipeline behavior and seismic hazard definition. It is anticipated that these guidelines will be updated as ongoing research is incorporated into actual practice.

[1] Nyman, D.J. (ed.), 1984. *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems*, American Society of Civil Engineers.

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GUIDELINES FOR THE SEISMIC DESIGN AND ASSESSMENT OF NATURAL GAS AND LIQUID HYDROCARBON PIPELINES

PART I

GUIDELINES AND RECOMMENDED PROCEDURES

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Table of Contents

1.0 GENERAL PROVISIONS

These guidelines provide recommended procedures and methods for the assessment of new and existing natural gas and liquid hydrocarbon pipelines subjected to seismic-related loading conditions. These guidelines are believed to be representative of, and consistent with, current practices for seismic hazard mitigation for these pipelines.

The specific procedures contained in these guidelines are related to determining pipeline response to predefined seismic hazards (e.g., liquefaction, fault displacement, slope instability). The process of defining these hazards is not part of the scope of these guidelines, although considerable guidance is given on typical practice and simplified approaches for estimating the potential severity of seismic hazards. The suitability of the methods described in these guidelines to define seismic hazards should be confirmed by specialists with appropriate knowledge and expertise.

1.1 Application

These guidelines are limited to onshore and offshore buried steel pipelines fabricated with full-penetration girth welds. Although the procedures of these guidelines may be extended to other applications (e.g., low-pressure distribution mains), the special considerations that apply to buried piping systems with conditions such as numerous tie-ins of distribution facilities and potential fatigue effects of large cyclic pressure fluctuations may occur during operation are not addressed.

1.2 Process

The general process for performing a design or an assessment of a buried pipeline is illustrated by the flow chart in Figure 1.1. As indicated in the flow chart, application of the procedures in these guidelines may require input from experts in specialized technical disciplines, particularly in the areas of seismology, geology, soil mechanics, and materials and welding technology. The content of these guidelines is limited to guidance regarding typical practices in these areas as they relate to the seismic assessment of buried pipelines. This information is also valuable to facilitate interaction among pipeline engineers and individuals having the requisite specialized expertise.

Most of the approaches for quantifying seismic hazards presented in these guidelines are based upon empirical relationships developed from past earthquake observations. As a result, each such approach is limited to the particular conditions characteristic of the earthquake data used to develop the underlying empirical relationship. Understanding the relative importance of these limits and rational approaches for implementing modifications for site-specific conditions is the primary reason for relying on individuals with special technical expertise.

The procedures in this document assume that the performance criteria and critical pipeline locations have been identified. The level of required seismic performance, as defined by the annual probability of occurrence of an unacceptable condition, is a decision that needs to be made on a case-by-case basis considering public safety, environmental impacts, operational requirements, economic consequences, regulatory requirements and corporate policies. Factors to be considered when establishing performance requirements are discussed in Appendix A.

The state of knowledge in the field of pipeline response does not allow the definition of well-defined post-yield strain limits associated with some levels of pipeline performance. This is particularly true for cases where detailed knowledge of the pipeline material properties is lacking and the performance level is associated with maintaining pipeline pressure integrity. Considerable uncertainty also exists in defining seismic hazards. In these guidelines, the specification of partial safety factors associated with a rigorous reliability analysis has not been attempted. While such an approach may be desirable for specific pipelines, it is not considered practical for general application and, therefore, is not provided in these guidelines. The philosophy adopted in these guidelines is to consider conservative estimates of seismic hazards that are likely to occur. The likelihood of experiencing a seismic hazard is assumed to be equal to the acceptable probability of not achieving the performance goals for the pipeline. Pipeline response to the seismic hazard defined in this manner is assumed to utilize estimates of pipeline strength and strain capacity that are equal to or slightly more conservative than the median. The determination of what constitutes a "conservative" hazard definition and "median" pipeline strength and strain capacity is based largely upon judgment and procedures implemented in many pipeline projects and approved by various regulatory and oversight agencies over the past two decades.

These guidelines address the site-specific evaluation of a pipeline to a particular seismic ground displacement hazard. For a single pipeline, the number of site-specific conditions to be evaluated is typically based upon an assessment of conditions within 200 m to 300 m of the pipeline right-of-way. For transmission systems providing multiple feeds to large metropolitan regions, an alternate approach may be to first identify critical pipeline segments within the system. The critical segments are those portions of the system necessary to maintain life safety, avoid environmental damage, and provide reliable service. Critical segments of the pipeline may also include locations where options are not available for rapidly reconfiguring the system to isolate earthquake damage and reroute flow to maintain service. Identification of seismic hazards would then focus on the critical pipeline segments within the system. For large transmission systems in areas with numerous seismic hazards, this approach has the advantage of significantly reducing the level of effort for quantifying seismic hazards. A drawback of this approach is that damage that might occur to less critical portions of the pipeline system is not identified. Of course, when examining a single pipeline for the level of reliable service, the entire pipeline becomes a critical segment as damage anywhere along the pipeline can lead to service interruption.

1.3 Alternate Procedures

These guidelines do not preclude the use of alternate procedures for determining pipeline performance when subjected to seismic-induced load conditions. Alternative methods may be equally acceptable if based upon sound engineering and rational analysis satisfying the intent of provisions given in these guidelines. In most cases, alternative procedures are justified and feasible if they accommodate the use of site- or project-specific conditions such as those listed below:

1. Site-specific seismic hazard definition based upon more detailed knowledge of the tectonic and geologic setting in which the pipeline is operated;
2. Site-specific assessment of geotechnical conditions, e.g., potential liquefaction areas;
3. Improved characterization of pipe-soil interaction based upon field testing of in situ soil conditions or laboratory tests representative of in situ conditions; and
4. Determination of alternative pipeline limit-state criteria based upon laboratory tests representative of the full range of expected pipeline cross-sections, materials, applied load conditions and deformation states.

Alternative analysis methods must be capable of correctly capturing non-linear soil behavior, the influence of large pipeline deformations on computed pipeline strains, and post-yield or post-buckling pipeline strength characteristics.

1.4 Required Information

The analysis procedures contained in these guidelines require the following engineering information.

[illegible]

1. Outside diameter and wall thickness of the pipe, elbows or induction bends;
2. Stress-strain relationships representative of the pipe material;
3. Toughness properties of the pipe, girth weld, seam weld, and weld heat-affected zone;
4. External coating specifications (as it relates to the pipe-soil friction interface) and thickness and type of insulation or shielding material (if any);
5. Maximum allowable operating pressure;
6. Design temperature differential between installed and operating conditions;
7. Pipeline alignment details (plan, profile, and location of fittings); and
8. For existing pipelines, reduced strain limits based upon type of girth-weld, expected material and weld defects in the girth weld and seam weld, and corrosion condition over a length of pipeline experiencing strains resulting from permanent ground displacement.

[illegible]2 [illegible]r [illegible]

1. Depth of soil cover over a length of pipeline experiencing strains from permanent ground displacement;
2. Backfill specifications;
3. Depth to water table;
4. Water depth for offshore pipelines in the area of potential permanent ground displacement;
5. Soil strength parameters over a length of pipeline experiencing strains from permanent ground displacement and to a depth equal to the bottom of the pipe trench. The needed parameters include:
    a. Total unit weight
    b. Internal friction angle
    c. Cohesion

[illegible] Gr[illegible]d D[illegible]r[illegible] [illegible]H[illegible]rd D[illegible]

1. Expected amount and dominant direction of permanent ground deformation (usually defined at the ground surface);
2. Length of pipeline exposed to permanent ground deformation;
3. Variation of permanent ground deformation with depth, if any, along the length of the pipeline under consideration;
4. Variation in the direction of permanent ground deformation; and
5. Uncertainty in the location of the fault with respect to the pipeline crossing.

1.5 Acronyms

APE	annual probability of exceedance
API	American Petroleum Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing and Materials
CPT	cone penetrometer test
EPS	expanded polystyrene
HAZ	heat affected zone
NPS	nominal pipe size (NPS 12 corresponds to 12-inch and 323 mm pipe)
SMYS	specified minimum yield strength

SPT	standard penetration test
TGD	transient ground displacement
XPS	extruded polystyrene

1.6 Nomenclature

To the extent possible, the nomenclature in these guidelines matches the nomenclature in equations from various reference sources. In several cases, this leads to duplication in nomenclature. Since many terms are well established in various technical disciplines, the potential for confusion in the use of unfamiliar terms is considered a more significant drawback than duplication.

A_c	critical acceleration to trigger seismic slope instability
AD	average surface fault displacement in a vertical plane parallel to fault strike
c	cohesion representative of the soil backfill
c	apparent wave propagation velocity for estimating strains from wave propagation
c'	effective cohesive strength of slope material for landslide assessment
C_B	borehole diameter correction (assume 1.0 if unknown)
C_E	hammer energy correction ($C_E = E_{eff}/60$; assume 1.0 if unknown)
C_N	overburden correction ($\sqrt{P_a / \sigma_{ve}} \leq 2.0$)
C_Q	correction factor for assessing liquefaction using CPT test data
C_R	rod length correction ($C_R = 0.75 + 0.25(L_{rod} - 4)$; $0.75 \leq C_R \leq 1.0$)
CRR	cyclic resistance ratio for assessing liquefaction
C_S	sample liner correction (1.2 if no liner; if unknown, assume 1.0)
CSR	cyclic stress ratio for assessing liquefaction
d_s	average depth to top of seismogenic rupture zone in attenuation relationship
D	outside pipe diameter
D	depth to basement rock
D'	ovalization parameter
$D50_{15}$	mean soil particle diameter for saturated, liquefiable soils with $(N_1)_{60}$ values less than 15
D_{bore}	SPT bore hole diameter
D_F	total fault displacement

D_{FS}	component of fault displacement in a vertical plane parallel to the fault strike (quantity provided by the Wells and Coppersmith empirical fault displacement relationship)
D_{min}	minimum pipe diameter from assumed ovaling
D_r	relative density, %
D_w	depth to water table
E	random error term used in ground shaking attenuation relationships
E	modulus of elasticity
E_{eff}	percentage of hammer energy delivered to sampling rod in SPT test
f	pipe coating factor for estimating interface friction angle from internal friction angle
f_s	sleeve friction measured in a CPT conducted in accordance with ASTM D-3441
F	attenuation modification factor (0 for normal or strike-slip faulting, 1 for reverse or thrust faulting, if type of faulting unknown, use 0.5)
FFH	free face height for lateral spread displacement equation, m
FFL	distance from toe of free face, m
F_{15}	percentage of material passing through a #200 sieve from a sample of liquefiable soil with an $(N_1)_{60}$ value less than 15
F_ℓ	factor of safety against liquefaction, CRR/CSR
FC	fines content of soil as measured by percentage of a sample passing a #200 sieve
g	acceleration due to gravity
h	thickness of potential sliding soil mass
H	depth from the ground surface to the centerline of a buried pipeline
H	rupture depth, km, in attenuation relationship
H_B	depth to bottom of fault, km
H_F	horizontal component of fault displacement parallel to fault strike
H_S	depth to top of seismogenic zone, km
H_T	depth to top of fault, km
$I_{0.5}$	index parameter in CPT liquefaction assessment procedure computed for $n = 0.5$
I_1	index parameter in CPT liquefaction assessment procedure computed for $n = 1.0$

I_n	index parameter in CPT liquefaction assessment procedure computed for a specific value of *n*
K_c	fines content correction factor for liquefaction assessment using CPT data
K_o	coefficient of earth pressure at rest
K_σ	overburden correction factor to account for overburden stresses greater than 100 kPa
L_{anchor}	length of burial sufficient to develop yield in the pipe under relative axial soil displacement
L_D	estimated slope displacement for assessing whether or not large landslide movements are possible
L_{rod}	length of rod connected to SPT sampler, m
LSD	mean lateral spread displacement
M	earthquake moment magnitude
MD	maximum surface fault displacement in a vertical plane parallel fault strike
MSF	magnitude scaling factor for computing cyclic resistance factor
n	factor in CPT liquefaction assessment related to fines content of soil
N	SPT blow counts collected in accordance with ASTM D-1586
$(N_1)_{60}$	standard penetration resistance normalized to an overburden pressure of 1 tsf (96 kPa) and a hammer energy efficiency ratio of 60%
$(N_1)_{60FC}$	$(N_1)_{60}$ corrected for fined content of the soil
N_c	soil bearing capacity factor for vertically downward loading in clay
N_{ch}	soil bearing capacity factor for horizontal loading in clay
N_{cv}	soil bearing capacity factor for vertically upward loading in clay
N_q	soil bearing capacity factor for vertically downward loading in sand
N_{qh}	soil bearing capacity factor for horizontal loading in sand
N_{qv}	soil bearing capacity factor for vertically upward loading in sand
N_γ	soil bearing capacity factor for vertically downward loading in sand
p	internal pipeline pressure
p_y	internal pipeline pressure that produces a hoop stress equal to σ_y
P_a	reference pressure (100 kPa ≈ 14.5 psi ≈ 1 tsf)
P_{cr}	predicted collapse pressure from external pressure
P_e	elastic buckling pressure
P_{ext}	net external pressure on pipe (total external pressure - internal pressure)
P_{fr}	average annual probability of fault displacement

P_g	average annual probability associated with exceeding specified performance goal for the pipeline (e.g., 0.001 annual probability for loss of pressure integrity)
P_u	maximum lateral soil load caused by pipe lateral movement relative to the surrounding soil
P_y	axial load in a pipeline corresponding to a uniform tensile stress equal to the SMYS
PGA	peak ground acceleration at surface
q_c	tip resistance measured in a CPT conducted in accordance with ASTM D-3441
q_{c1N}	q_c normalized to an overburden pressure of P_a
$(q_{c1N})_{FC}$	q_{c1N} corrected to account for fines content
Q_d	maximum lateral soil load caused by pipe vertically downward movement relative to the surrounding soil
Q_u	maximum lateral soil load caused by pipe vertically upward movement relative to the surrounding soil
r	amount of rattle-space for culvert mitigation concept
r_d	cyclic stress reduction coefficient for assessing liquefaction using SPT data
R	earthquake source distance parameter (variously defined in equations)
R_{fp}	fault displacement reduction factor accounting for probability of fault displacement
R_S	closest distance to seismogenic rupture surface, km
R^*	effective distance from site to earthquake epicenter, km, used in lateral spread displacement equation
RLD	fault rupture length at depth
sr	fault slip rate
S	percentage of surface slope for estimating lateral spread displacement
S_{HR}	attenuation relationship surficial soil correction parameter (0 for soft rock, alluvium and firm soil, and 1 for hard rock)
S_{SR}	attenuation relationship surficial soil correction parameter (0 for hard rock, alluvium and firm soil, and 1 for soft rock)
SRL	surface rupture length, km
t	pipe wall thickness
t_u	maximum soil force on pipeline from relative axial movement
T_{15}	thickness of saturated, liquefiable, soils with $(N_1)_{60}$ values less than 15
T_F	component of fault displacement perpendicular to fault strike

V_F	component of vertical fault displacement
V_{max}	maximum ground velocity from ground shaking
W	free face ratio, expressed as percent, for estimating lateral spread displacement
W	expected down-dip fault width, km
z	depth below ground of SPT or CPT test measurement
Z_T	factor in attenuation relationship, 0 for interface event, 1 for intra-slab event in attenuation relationship
α	fault dip angle, degrees
α	coefficient for determining fines correction factor in liquefaction assessment
α	adhesion factor for estimating axial pipeline soil load
α_ε	ground strain coefficient for computing axial strains from wave propagation
α_κ	ground strain coefficient for computing bending strains from wave propagation
β	angle of regional stress azimuth for estimating fault displacement, degrees
β	coefficient for determining fines correction factor in liquefaction assessment
δ	soil interface friction angle
δ_o	initial ovalization of pipe cross-section
Δ	girth weld offset used to compute compression strain limit
Δ_d	vertical downward relative pipeline displacement caused by liquefaction
Δ_p	relative displacement between pipe and soil in lateral direction necessary to develop P_u
Δ_{qd}	relative displacement between pipe and soil in vertically downward direction necessary to develop Q_d
Δ_{qu}	relative displacement between pipe and soil in vertically upward direction necessary to develop Q_u
Δ_t	relative displacement between pipe and soil in axial direction necessary to develop T_u
Δ_u	vertical upward relative pipeline displacement caused by liquefaction
ε_{cl}	longitudinal compression strain limit for load-controlled conditions
ε_{co}	longitudinal compression strain limit for continued operation
ε_{cp}	longitudinal compression strain limit for pressure integrity
ε_{cr}	critical longitudinal compression strain
ε_{cr-p}	critical longitudinal compression strain for conditions with external pressure

ε_{tl}	longitudinal tension strain limit for load-controlled conditions
ε_g	ground strain related to seismic wave propagation
ε_{to}	longitudinal tension strain limit for continued operation
ε_{tp}	longitudinal tension strain limit for pressure integrity
ε_v	volumetric strain experienced by soil as the result of liquefaction
ϕ	soil internal friction angle
ϕ'	effective angle of internal friction for slope stability assessment
Φ	standard normal probability function
γ	total unit weight of soil
$\bar{\gamma}$	effective (or submerged) unit weight of soil
γ_{max}	maximum soil shear strain
γ_w	unit weight of water
κ_g	ground curvature from seismic wave propagation
λ	ratio of pore pressure to overburden stress
σ	standard deviation
σ_t	axial stress in pipeline from thermal differential
σ_y	pipe material yield stress
σ_{vo}	total soil overburden pressure
σ'_{vo}	effective soil overburden pressure
σ_{AD}	standard deviation of average fault displacement
σ_{MD}	standard deviation of maximum fault displacement
θ	slope angle

1.7 References

For clarity, references have been largely omitted from the Procedures section of these guidelines. References to the basis documents used in these guidelines are noted in the Commentary section along with a complete reference listing.

1.8 Units

Seismic design and assessment of buried pipelines invariably crosses multiple disciplines with an equal variability in the degree to which English and metric units are adopted. As many of the equations in these guidelines are adopted from other sources, the original unit system has been maintained resulting in a mix of English and metric units within these guidelines. The majority of equations in these guidelines can be used with any consistent set of units. Exceptions are noted and typically relate to empirical relationships or equations fitted to published curves to facilitate calculations using spreadsheets or other computer-based applications.

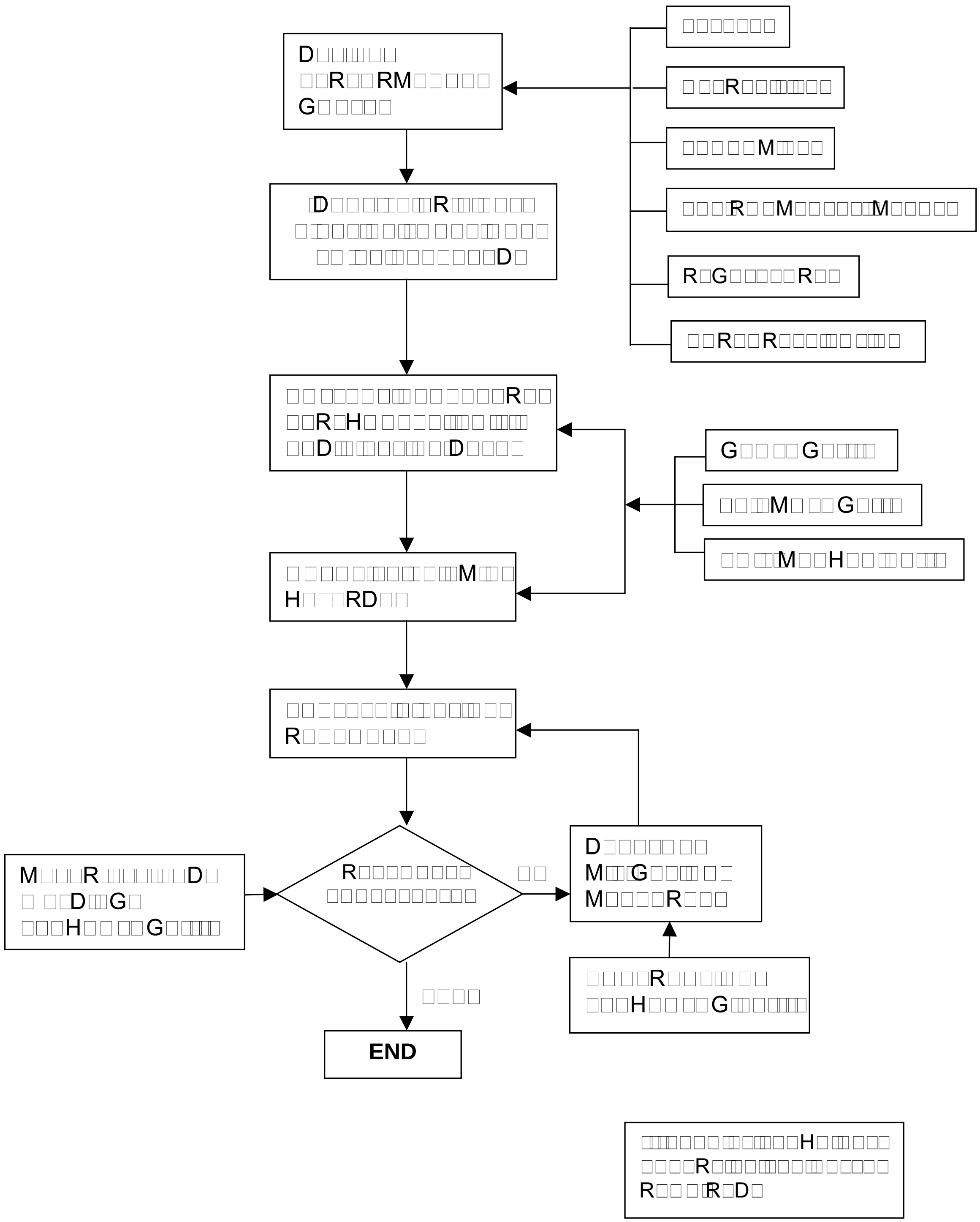

Figure 1.1: Seismic Design and Assessment Process for Assessing Seismic Hazards for Buried Natural Gas and Liquid Hydrocarbon Pipelines

2.0 QUANTIFYING SEISMIC HAZARDS

Credible seismic hazards for buried steel pressure pipelines are generally limited to those that produce permanent deformation of the ground along the pipeline alignment or large transient ground displacements that result in significant permanent deformation following an earthquake. These hazards primarily include surface faulting, liquefaction, liquefaction-induced lateral spread movement, landslides, and near-surface settlement.

Seismic wave propagation is also a consideration for buried pipelines, but it generally does not have a serious effect on buried welded steel pipelines. Transient ground deformation generally has not been a problem, except in a few special situations that are discussed in Section 2.7.

This section presents the methods and approaches commonly used to define seismic hazards for buried pipelines. Detailed coverage of the theory and development of procedures for collecting data and quantifying seismic hazards is beyond the scope of these guidelines. The information provided relies heavily on project experience and recent advances identifying and characterizing seismic hazards. It is recommended that the definition of seismic hazards be performed by individuals experienced with the methods described in this section and possessing the requisite background in seismology, geology, and soil mechanics.

The discussion of seismic hazards is organized by category and is presented in accordance with the development of parameters to define the hazard. The focus is on direct seismic hazards as opposed to potential indirect hazards from such causes as debris impact (e.g., rockfall impact) or collateral damage resulting from earthquake damage to adjacent pipelines in a common right-of-way or utility corridor.

The definition of potential seismic hazards is often developed during a separate geotechnical or geologic investigation. In lieu of such investigations, the procedures provided in this section can be followed to provide an estimate of the amount and likelihood of ground movement. These procedures are best used as a tool to determine if a pipeline is at significant risk from seismic hazards. The procedures also provide a means to identify the need for, and scope of, a more rigorous geologic or geotechnical investigation.

2.1 Surface Faulting

Surface fault rupture is an important consideration for buried pipelines, because pipelines crossing fault zones must deform longitudinally and in flexure to accommodate ground displacement. If a pipeline crosses an active fault, it is necessary to delineate its location, orientation, width of fault rupture zone, and the amount and direction of potential fault displacement.

The term “active fault” implies a judgment on the part of a geologist that the fault could experience surface rupture within the time frame established by the performance goals for the pipeline (e.g., an average annual probability of exceedance less than 1x10^{-3}). Generally, a fault is considered active if it can be demonstrated to have displaced the ground surface during the Holocene epoch (i.e., within the past 11,000 years).

For some faults, a characteristic earthquake and associated recurrence interval may have been established through field studies and historical seismicity. For such cases, the expected fault displacement may be estimated as the cumulative slip between characteristic earthquakes (i.e., the slip rate per year times the recurrence interval). Otherwise, it is more meaningful to rely on empirical relationships such as those described in Section 2.1.1.

Some faults experience aseismic creep in addition to surface rupture produced by an earthquake. For pipeline design, the total fault movement, including creep, is important since both lead to deformation of the pipeline. The effects of creep need to be considered separately because the slow rate of displacement is consistent with drained soil loading conditions while fault displacement occurs rapidly under undrained conditions. This impacts the determination of soil loads on the pipeline as discussed in Section 4.

It is recommended that fault displacement estimates for the design or assessment of pipeline response be approached as a three-stage process:

1. Estimate the expected fault displacement of an active fault;

2. Adjust the estimate of expected fault displacement based upon the consequences of pipeline damage and pipeline performance objectives to obtain the design fault displacement; and

3. Convert the design fault displacement to orthogonal displacement components relative to the fault (i.e., vertical component, horizontal component parallel to the fault strike, and horizontal displacement perpendicular to the fault strike).

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The most common method for estimating surface fault displacements relies on observations of fault displacements during past earthquakes. Wells and Coppersmith (1994) developed a series of empirical relationships that relate moment magnitude, *M*, surface rupture length, subsurface rupture length, down-dip rupture width, rupture area, and maximum and average displacement per event. The most commonly used Wells and Coppersmith relationships that are used when other information is lacking provide maximum and average fault displacements (MD and AD) as function of fault surface rupture length or moment magnitude.

Estimating MD and AD as a function of surface rupture length is generally preferred because the determination of fault displacement is based directly upon the geologic attributes of the fault. Using earthquake moment magnitude to estimate MD and AD is more indirect because it is based upon a two-stage correlation of moment magnitude to fault parameters (typically either surface rupture length or seismogenic rupture area) and fault displacement to magnitude. However, relating fault displacement to earthquake magnitude is preferable for situations where earthquake magnitude has been previously established by other geologic investigations.

For estimating fault displacement (MD or AD) as a function of surface rupture length (SRL), the following regression formulae from Wells and Coppersmith (1994) apply:

$$\log(MD) = -1.38 + 1.02 \cdot \log(SRL) \qquad [\log(\sigma_{MD}) = 0.41] \qquad (2\text{-}1a)$$

$$\log(AD) = -1.43 + 0.88 \cdot \log(SRL) \qquad [\log(\sigma_{AD}) = 0.31] \qquad (2\text{-}1b)$$

where:

AD = Average surface fault displacement, m

MD = Maximum surface fault displacement, m

SRL = Surface rupture length, km. The surface rupture length is the maximum length of the fault segment crossing the pipeline that, by reasonable and qualified judgment, can be expected to rupture during an earthquake.

σ_{MD} = Standard deviation of maximum displacement regression

σ_{AD} = Standard deviation of average displacement regression

The data sets for the two regression equations listed above include displacement data for all types of slip, i.e., strike-slip, normal and reverse faulting. The displacements estimated by the relationships of Wells and Coppersmith (1994) is the vector sum of the horizontal and vertical slip components along the fault strike. The implications of this definition on determining three-dimensional components of fault displacement are discussed further in Section 2.1.2.

Wells and Coppersmith (1994) also provide relationships for estimating maximum and average fault displacements as a function of moment magnitude (M). These relationships are provided below.

$$\log(MD) = -5.46 + 0.82 \cdot M \qquad [\log(\sigma_{MD}) = 0.42] \qquad (2\text{-}2a)$$

$$\log(AD) = -4.80 + 0.69 \cdot M \qquad [\log(\sigma_{AD}) = 0.36] \qquad (2\text{-}2b)$$

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As a final step in defining fault displacement, the design fault displacement determined in Section 2.1.2 must be transformed into orthogonal components of fault displacement. This is necessary only if fault displacement is estimated using equations (2-1) or (2-2), since these expressions only provide the net displacement in a vertical plane perpendicular to the strike of the fault. The transformation to orthogonal displacement components relative to the fault strike is illustrated in Figure 2.1. The following equations provide the transformation:

Total fault displacement, D_F:

$$D_F = \frac{D_{FS}}{\sqrt{\sin^2\alpha + \left(\cos^2\alpha\right)\left(\cos^2\beta\right)}} \tag{2-3a}$$

Vertical displacement, V_F:

$$V_F = D_F \sin\alpha \tag{2-3b}$$

Horizontal displacement parallel to the fault strike, H_F:

$$H_F = D_F \cos\alpha \cos\beta \tag{2-3c}$$

Transverse horizontal displacement perpendicular to the fault strike, T_F:

$$T_F = D_F \cos\alpha \sin\beta \tag{2-3d}$$

Apparent dip angle, α:

$$\tan\alpha = \tan\delta \sin\beta \tag{2-3e}$$

where:

D_{FS} = fault displacement in a vertical plane parallel to the fault strike

δ = fault dip angle corresponding to "average" dip angle inherent in equations (2-1) and (2-2) (see Commentary). Lacking other information, the dip angle can be taken as 75° for strike slip faults, 60° for normal slip faults, and 45° for reverse or thrust faults.

β = horizontal angle between fault strike and regional stress azimuth. For faults that are predominantly strike slip, this angle is 0°, and for faults that are predominantly reverse or normal faults, this angle will be 90°. Other faults will have some combination of strike and reverse/normal slip, with an angle, β, between 0° and 90°.

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Determination of a design fault displacement is based upon estimates of expected average and maximum fault displacements described in Section 2.1.1. Design fault displacements are determined based upon consideration of the performance requirements for the pipeline and the consequences for loss of pipeline pressure integrity. Two modifications are recommended for determining design fault displacements, one to account for the consequences of loss of pipeline pressure integrity and another to account for faults with a low likelihood of occurrence relative to the pipeline performance criteria.

2.1.3.1 Accounting for Consequences of Loss of Pressure Integrity

It is preferable that design fault displacements be based upon the uncertainty in fault displacement and site-specific consideration of the potential risk (likelihood of a particular type of damage and consequences) associated with earthquake-related pipeline damage. In lieu of such considerations, the following design factors are recommended to account for the consequences of loss of pipeline pressure integrity and the expected recurrence interval for fault displacement:

1. Consider the mean maximum fault displacement for natural gas pipelines and flammable or explosive petroleum products pipelines in Location Class 4 areas (ASME B31.8).
2. Consider two-thirds of the mean maximum fault displacement for natural gas pipelines and flammable or explosive petroleum products pipelines in Location Class 3 areas (ASME B31.8).
3. Consider two-thirds of the mean maximum fault displacement for petroleum products pipelines located in environmentally sensitive areas.
4. Other natural gas or liquid hydrocarbon pipelines in other areas than above should be designed for the mean average fault displacement.

Prior to use, the above guidelines should be carefully reviewed and evaluated by the pipeline owner or its representatives for consistency with project objectives and regulatory requirements.

2.1.3.2 Accounting for Expected Occurrence of Fault Displacement

The time frame for defining a fault to be active is typically an order of magnitude greater than the typical performance goals for a pipeline (e.g., 11,000 years for the Holocene epoch versus a pipeline performance goal of a 1,000-year average return period for experiencing loss of pressure integrity). To account for this, it is recommended that the expected fault displacements be reduced when the estimated likelihood of fault displacement is less than the acceptable probability of not achieving pipeline performance goals. A simple correction factor based upon the ratio of the probability of fault displacement and pipeline performance goal is recommended as defined by equation (2-4):

$$R_{fp} = \frac{P_{fr}}{2P_g} \qquad (2\text{-}4)$$

where:

R_{fp} = reduction factor accounting for probability of fault displacement

P_{fr} = average annual probability of fault displacement

P_g = average annual probability associated specified performance goal (e.g., $1\text{x}10^{-3}$ mean annual probability for loss of pressure integrity)

If fault displacements are determined by other means than equation (2-1) or (2-2), appropriate steps should be taken to determine orthogonal components in the vertical, horizontal and transverse directions.

2.2 Peak Ground Acceleration

Estimates of peak ground acceleration (PGA) are required for quantifying permanent ground displacement hazards related to liquefaction, lateral spread movement, and landslide movement. Ground shaking is estimated using empirical attenuation equations representing the decrease in ground motion with increasing distance from the fault rupture plane. There are many attenuation relationships available, and many are developed to represent specific types of earthquakes and specific geologic and tectonic environments.

Peak ground acceleration can be obtained from the results of probabilistic hazard analyses or deterministically based upon a postulated earthquake on a known fault. Probabilistic hazard analyses typically form the basis for building code earthquake hazard maps. The details of how probabilistic hazard analyses are performed is a specialized field and is beyond the scope of these guidelines. However, a summary of the general procedures is provided in Appendix C to familiarize the reader with the general concepts.

Deterministic estimates of peak ground acceleration are relatively straightforward to apply provided an attenuation relationship is selected that is appropriate for the earthquake mechanism and tectonic and geologic characteristics of the area.

A deterministic approach to estimating peak ground acceleration is required for assessing other hazards related to liquefaction and some approaches for assessing slope instability. Where probabilistic hazard analyses are available, it may be possible to infer a magnitude and acceleration value from the details of the calculation of the probabilistic hazard. Selecting a suitable combination of magnitude and acceleration from a probabilistic hazard estimate should be performed by individuals familiar with the generation of the probabilistic hazard and the seismicity of the region.

The two relationships presented in Sections 2.2.1 and 2.2.2 are provided to illustrate some of the general characteristics and differences in attenuation relationships. These relationships were developed for near-source earthquakes (western United States) and subduction zone (Cascadia region) earthquakes. It is ***strongly*** recommended that users seek out specific attenuation relationships applicable for the region in which the pipeline is located.

Both attenuation relationships contain an error term, E, that represents a factor that can be used in conjunction with a standard normal probability table to estimate the probability of exceeding the mean of $\ln(PGA)$ represented by the equation. For example, for a mean estimate, the standard normal probability, $\Phi(x)$ is 0.5 (50%) and the value of x is 0. If the probability of exceedance is desired to be less than 16%, $\Phi(x)$ is 0.84, x is 1.0 and the value of $\ln(PGA)$ is increased by E which is equivalent to multiplying the PGA by e^E.

Other percentile levels may be established by selecting values of $\Phi(x)$ consistent with the percentile associated with the desired probability of non-exceedance.

The attenuation relationships also illustrate the potential variability in the amount and types of information needed for their application. This is particularly true for definitions of source-to-site distance. The near source earthquake relationship requires more descriptive information on the fault plane and the portions of the fault capable of generating earthquake ground motions (the seismogenic part of the fault) in order to compute this distance. There is also a difference in the number of parameters used to characterize the site.

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$$\begin{aligned}\ln(PGA) = -3.512 + 0.904M - 1.328\ln\sqrt{R_S^2 + \left[0.149e^{0.647M}\right]^2} \\ + \left[1.125 - 0.112\ln(R_S) - 0.0957M\right]F + \left[0.440 - 0.171\ln(R_S)\right]S_{SR} \\ + \left[0.405 - 0.222\ln(R_S)\right]S_{HR} + f_A(D) + E\Phi^{-1}(x)\end{aligned} \qquad (2\text{-}5)$$

where:

PGA	=	peak ground acceleration, g
M	=	moment magnitude
F	=	0 for normal and strike-slip faulting
	=	1 for reverse or thrust faulting
	=	0.5 if type of faulting unknown
R_S	=	closest distance to seismogenic rupture, km
	=	$\sqrt{R^2 + d_S^2}$
R	=	horizontal projection of closest site to seismogenic rupture distance, km
d_S	=	average depth to the top of the seismogenic rupture zone, km
	=	$\frac{1}{2}\left[H_B + H_T + W\sin(\alpha)\right] \geq H_S$
H_B	=	depth to bottom of fault, km
H_S	=	depth to top of seismogenic part of the crust, km
H_T		depth to top of fault, km
W	=	expected down-dip fault width, km
	=	$10^{-1.01+0.32M}$
α	=	fault dip angle

$f_A(D)$ = 0 if D ≥ 1 km, otherwise

= $\left\{\left[0.405-0.22\ln(R_S)\right]-\left[0.440-0.171\ln(R_S)\right]S_{SR}\right\}(1-D)(1-S_{SR})$

D = depth to basement rock, km

E = random error term = $0.889-0.691M$

Φ = standard normal probability function

x = cumulative probability of actual PGA being less than or equal to PGA from the attenuation relationship

S_{SR} = 0 for hard rock, alluvium, and firm soil

= 1 for soft rock

S_{HR} = 1 for hard rock

= 0 for soft rock, alluvium, and firm soil

For generic rock sites, S_{SR} = 1, S_{HR} = 0, and D = 1 km.

For generic soil sites, S_{SR} = 0, S_{HR} = 0, and D = 5 km.

2.2.2

$$\ln(PGA)=0.2418+1.414M-2.2.552\ln\left(R+1.7818e^{0.554M}\right)$$
$$+0.00607H+0.3846Z_T+E\Phi^{-1}(x) \quad \text{for rock} \qquad (2\text{-}6a)$$

$$\ln(PGA)=-0.6687+1.438M-2.329\ln\left(R+1.097e^{0.617M}\right)$$
$$+0.00648H+0.3643Z_T+E\Phi^{-1}(x) \quad \text{for soil} \qquad (2\text{-}6b)$$

where:

H = rupture depth, km

R = closest distance to fault rupture, km

Z_T = 0 for interface event, 1 for intra-slab event

E = $1.45-0.1M$ but not greater than 0.65

2.3 Liquefaction

Liquefaction hazards to pipelines include flotation, sinking, and general ground settlement from dissipation of excess pore water pressure. Pipeline flotation or sinking requires the pipe to be located below the ground water table within a zone of liquefiable soil. Ground settlement occurs when a liquefiable soil layer beneath a layer of competent soil densifies. If the pipeline is located in the layer of competent soil near the surface, it will be subjected to displacement associated with subsidence of the ground.

Quantification of liquefaction hazards requires site-specific information on the subsurface soil properties and peak ground acceleration. Soils information can most commonly be obtained through subsurface investigations using the standard penetration test (SPT) and cone penetration test (CPT). These investigations should be performed by personnel experienced in the interpretation of soil boring logs and liquefaction assessment. Parameters used in the assessment of liquefaction potential are summarized in Table 2.1.

Liquefaction also may cause lateral spreads or flow slides. These hazards are discussed in Sections 2.4 and 2.5.

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Liquefaction is a phenomenon primarily limited to loose saturated sand deposits. Normalized SPT blow counts (defined in Section 2.3.1.1) greater than 30 can be taken as an indication that the soil densities are sufficiently high to prevent liquefaction. In addition, work primarily from Chinese investigators, suggests that some saturated clayey soils may also be susceptible to significant loss of strength similar to liquefaction. The criteria for identifying potentially sensitive clayey soils are as follows:

1. Percent of grain size finer than 0.005 mm is less than 15%.
2. Liquid limit is less than 35.
3. Water content is greater than 90% of the liquid limit.

The information in Table 2.1 is required for performing a liquefaction assessment for a particular subsurface location where SPT or CPT data are collected. In addition, it is necessary to know if sample liners were used and the energy efficiency of the hammer used during collection of SPT data.

Both the SPT and CPT methods for assessing liquefaction potential compare the resistance to liquefaction, measured by the cyclic resistance ratio (CRR), to the cyclic stress ratio (CSR). Liquefaction is likely when CSR is greater than CRR. The cyclic stress ratio is primarily a function of the peak ground acceleration (PGA) and the in situ soil stress. The equation for computing CSR is the same for evaluating liquefaction potential using SPT or CPT data.

$$CSR = 0.65 \cdot PGA \cdot \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) \cdot r_d \qquad (2\text{-}7)$$

where:

PGA = peak ground acceleration, g

r_d = $\dfrac{1-0.4113\sqrt{z}+0.04052z+0.001753z^{1.5}}{1-0.4177\sqrt{z}+0.05729z-0.006205z^{1.5}+0.001210z^2}$

z = depth of interest (z must be in meters for computing r_d)

σ_{vo} = total overburden soil pressure at test location = γz

σ'_{vo} = effective soil overburden pressure = $\gamma z - \gamma_w(z - D_w)$

D_w = depth to water table

γ = unit weight of soil

γ_w = unit weight of water

2.3.1.1 Assessment of Liquefaction Using SPT Data

The *CRR* is computed as follows:

for $x \geq 3$,

$$CRR = \frac{MSF \cdot K_\sigma (4.8(10)^{-2} - 4.721(10)^{-3}x + 6.136(10)^{-4}x^2 - 1.673(10)^{-5}x^3)}{1 - 1.248(10)^{-1}x + 9.578(10)^{-3}x^2 - 3.285(10)^{-4}x^3 + 3.714(10)^{-6}x^4} \qquad \text{(2-8a)}$$

for $x < 3$,

$$CRR = 0.05 \cdot MSF \cdot K_\sigma \qquad \text{(2-8b)}$$

where:

x = $(N_1)_{60FC}$, the normalized SPT data corrected for fines content

MSF = magnitude scaling factor for approximately a 32% chance of liquefaction

= $10^{3.74}/M^{4.33}$

K_σ = overburden pressure correction factor for $\sigma'_{vo} > P_a$

= 1 for $\sigma'_{vo} \leq P_a$

= $0.514 + \dfrac{0.897}{\left(\sigma'_{vo}/P_a\right)} - \dfrac{0.411}{\left(\sigma'_{vo}/P_a\right)^2}$ for $P_a < \sigma'_{vo} < 10P_a$

= 0.6 for $\sigma'_{vo} \geq 10P_a$

P_a = reference pressure (100 kPa ´ 14.5 psi ´ 1 tsf)

The expression for *CRR* is based largely upon past earthquake observations for earthquakes with magnitudes of approximately 7.5. This expression is depicted graphically in Figure 2.2 as a function of $(N_1)_{60FC}$ for *MSF* and K_σ equal to 1.0.

Two steps are used to correct the measured SPT data. First, the measured SPT value, *N*, is corrected using equation (2-9) to account for specific test procedures and normalized to an effective overburden pressure equal to P_a. The second correction accounts for the greater liquefaction resistance of soils with larger quantities of fine-grained material. In this case, the fines content, *FC*, is determined as the percentage of material passing through a #200 sieve (0.074 mm opening). The normalized SPT value, $(N_1)_{60}$, is computed as follows:

$$(N_1)_{60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \tag{2-9}$$

where:

$(N_1)_{60}$ = SPT value normalized to P_a and 60% hammer energy efficiency

N = measured SPT

C_N = overburden correction = $\sqrt{P_a / \sigma'_{vo}} \le 2.0$

C_E = hammer energy correction

= $E_{eff}/60$ (assume 1.0 if unknown)

E_{eff} = hammer energy efficiency, percentage

C_B = borehole diameter correction (assume 1.0 if unknown)

C_R = rod length correction

= $0.75 + 0.04(L_{rod} - 4)$ $(0.75 \le C_R \le 1.0)$

L_{rod} = length of rod in SPT test (L_{rod} must be in meters for computing C_R)

C_S = sample liner correction (1.2 if no liner; assume 1.0 if unknown)

The following expression corrects the $(N_1)_{60}$ value for the effect of fines content, *FC*:

$$(N_1)_{60FC} = \alpha + \beta (N_1)_{60} \tag{2-10}$$

where α and β are defined as follows:

Fines Content, *FC*	α	β
5% or less	0	1.0
Greater than 5% to less than 35%	$e^{1.76-\frac{190}{FC^2}}$	$0.99+\frac{FC^{1.5}}{1000}$
35% or greater	5	1.2

2.3.1.2 Assessment of Liquefaction Using CPT Data

Data collected from cone penetrometer tests (CPT) need to be corrected for normalized test conditions and the fines content of the soil. Since no samples are collected in CPTs, the fines content correction relies on inferred conditions from the ratio of measured tip bearing, q_c, and friction value, f_s. The equation for computing *CRR* from CPT data is given below:

For $(q_{c1N})_{FC} < 50$

$$CRR = \left[\frac{0.833 \cdot (q_{c1N})_{FC}}{1000} + 0.05\right] MSF \tag{2-11a}$$

For $50 \le (q_{c1N})_{FC} < 160$

$$CRR = \left[93\left(\frac{(q_{c1N})_{FC}}{1000}\right)^3 + 0.08\right] MSF \tag{2-11b}$$

where:

$(q_{c1N})_{FC}$ = tip resistance normalized to P_a and corrected for fines content

The measured tip resistance is normalized to an effective overburden pressure equal to P_a and corrected for fines content. These corrections are dependent upon the value of I_n, a parameter related to the granular nature of the soil.

$$I_n = \sqrt{(3.47 - \log Q_n)^2 + (1.22 + \log F)^2} \tag{2-12}$$

$$Q_n = \frac{(q_c - \sigma_{vo})}{100}\left(\frac{100}{\sigma'_{vo}}\right)^n \tag{2-13}$$

$$F = \left[\frac{f_s}{q_c - \sigma_{vo}}\right] 100\% \tag{2-14}$$

where:

q_c = CPT tip bearing pressure

f_s = CPT side friction pressure

The value of n used in equations (2-13) and (2-14) is, in turn, dependent upon the resulting value of I_n. Determination of n requires computing I_n for values of n equal to 0.5 ($I_{0.5}$) and 0.1 (I_1). The value of n is then determined as follows:

n = 1.0 if I_1 greater than 2.6 (clayey)

n = 0.5 if I_1 and $I_{0.5}$ less than 2.6 (granular)

n = 0.7 if I_1 less than 2.6 and $I_{0.5}$ greater than 2.6 (silty)

The normalized CPT tip resistance, $(q_{c1N})_{FC}$, is then computed as shown below:

$$(q_{c1N})_{FC} = K_c C_Q \left(\frac{q_c}{P_a} \right) \qquad (2\text{-}15)$$

where:

K_c = 1.0 for $I_n \leq 1.64$

= $-17.88 + 33.75 I_n - 21.63 I_n^2 + 5.581 I_n^3 - 0.403 I_n^4$ for $I_n > 1.64$

$$C_Q = \left(\frac{P_a}{\sigma'_{vo}} \right)^n$$

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Pipeline flotation is a displacement-controlled phenomenon for onshore pipelines. The amount of displacement is limited by the distance between the bottom of the pipe and the water table or the depth of soil cover plus about 1/3 the pipe diameter for cases where the water table is at the ground surface. This is an upper limit assuming that the liquefied state is maintained for a long time. Otherwise, the displacement is further limited by the velocity at which the pipeline moves through the liquefied material and the “drag” as the pipe moves through the liquefied soil. Offshore pipelines or pipelines crossing bodies of water will tend to float if they are not at least neutrally buoyant with respect to the unit weight of the liquefied soil.

Pipelines that are negatively buoyant with respect to the unit weight of the liquefied soil are subject to sinking if they are located within a liquefiable soil deposit. The amount of sinking is limited by the distance between the bottom of the pipe and the depth to non-liquefiable soil. This is an upper limit assuming that the liquefied state is maintained for a long time. Otherwise, the displacement is further limited by the velocity at which the pipeline moves through the liquefied material similar to flotation.

The length of pipeline exposed to liquefaction is dependent upon the subsurface extent of liquefiable soil and the alignment of the pipeline through the zone of liquefaction. Determining the length of exposed pipeline can be made through subsurface investigations at regular intervals along the pipeline. Alternatively, the extent of exposure may be estimated based upon obvious changes in surface geology (e.g., a transition from deltaic deposits to rock or glacial till).

2[illegible] D[illegible] [illegible]R[illegible]d[illegible]d[illegible]d[illegible]

Liquefaction-induced settlements are credible hazards to pipelines located within liquefiable layers of soil. For the more typical onshore case where the pipeline is located above the water table, liquefaction-induced settlement is a credible hazard if the liquefiable layer is close enough to the surface to produce settlement in near-surface soil layers. One means to determine this possibility is to compare the thickness of the liquefied layer and the depth to the liquefied layer. The screening curves in Figure 2.3 are based upon observations of where surface settlements have been sufficient to cause structural damage in past earthquakes. Since buildings are generally more susceptible to damage from settlement, the use of Figure 2.3 can be used as a conservative screening guideline for buried pipelines.

The amount of liquefaction-induced settlement can be estimated for each sandy soil layer using Figure 2.4. For each soil layer, the amount of volumetric strain is taken from Figure 2.4 based upon the factor of safety against liquefaction and the relative density of the soil or the corresponding corrected blowcount, $(N_1)_{60FC}$. The total amount of settlement is estimated by multiplying the volumetric strain by the thickness of each soil layer below the invert of the pipeline. Utilization of Figure 2.4 is made by computing the factor of safety against liquefaction, F_ℓ, as given by the ratio of *CRR* to *CSR*.

2.4 Lateral Spread Movement

The most appropriate method to estimate lateral spread movement is to rely on computer-based approaches that account for the actual strength properties of the liquefied soil. These programs typically are limited to two-dimensional representations of the soil. They have the advantage of being able to quantify variation in the magnitude and direction of soil displacements. Examples of two commonly used programs are SOILSTRESS and DESRA, both developed at the University of British Columbia. In addition, there are many research-oriented programs and special purpose programs used within the geotechnical community.

In lieu of computer analyses, an estimate of lateral spread movement can be obtained using empirical relationships developed from lateral spread and soil data collected in past earthquakes in the United States and Japan (Youd et al., 2002), as shown below.

For sloping ground conditions (Figure 2.5):

$$\begin{aligned} Log(LSD) = {} & -16.213 + 1.532M - 1.406 LogR^{*} \\ & -0.012R + 0.338 LogS + 0.540 LogT_{15} \\ & +3.413 Log(100 - F_{15}) - 0.795 Log(D50_{15} + 0.1) \end{aligned} \tag{2-16a}$$

For free face conditions (Figure 2.6):

$$
\begin{aligned}
Log(LSD) = &-16.713 + 1.532M - 1.406LogR^{*} \\
&-0.012R + 0.592LogW + 0.540LogT_{15} \\
&+3.413Log(100 - F_{15}) - 0.795Log(D50_{15} + 0.1)
\end{aligned}
\qquad (2\text{-}16b)
$$

where:

LSD	=	horizontal lateral spread movement (m)
M	=	earthquake moment magnitude [$6.0 < M < 8.0$]
R	=	epicentral distance, km
R^*	=	$R + R_o$
R_o	=	$10^{0.89M-5.64}$
S	=	ground slope (%) [$0.1 < S < 6.0$]
W	=	free face ratio (%) = 100(*FFH/FFL*) [$1 < W < 20$]
FFH	=	height of free face
FFL	=	distance from base of free face
T_{15}	=	thickness (m) of saturated cohesionless soils with $(N_1)_{60} < 15$ [$1 < T_{15} < 15$]
$(N_1)_{60}$	=	standard penetration blowcount normalized to P_a and 60% driving efficiency
F_{15}	=	average fines content (%) in T_{15} [$0 < F_{15} < 50$]
$D50_{15}$	=	average median particle size (mm) in T_{15} [$0 < D50_{15} < 50$]

The limits associated with the above variables represent the range of values in the Bartlett and Youd data set. The simplified approach may result in significant error if site-specific conditions deviate significantly from these limits. Note that the range of $(N_1)_{60}$ is limited to less than 15. Available data indicate that lateral spread displacement does not occur in sediments with $(N_1)_{60}$ greater than 15.

Variation in ground displacement away from a free face can be estimated by computing the displacement using the free face equation (2-16b) with increasing values of *FFL* (i.e., decreasing free face ratio, *W*). This is relatively straightforward because there is only one term dependent upon the free face ratio.

The foregoing methods for estimating permanent ground movement relate to maximum displacements in a lateral spread. For the evaluation of pipeline behavior, it is necessary to estimate a transverse distribution of the pattern of ground deformation within the soil mass that moves across or along a buried pipeline. Considerations for defining the pattern of ground displacement are provided in Section 4.5.

2.5 Landslides

Strong ground shaking during earthquakes may trigger landslides in a variety of geologic and topographic settings as depicted in Figure 2.7. The potential threat to pipeline performance is a function of the following:

1. The amount of landslide displacement;
2. The depth of the landslide relative to the depth of the pipeline;
3. The type of ground displacement associated with the landslide movement (i.e., block-type (coherent) movement or disrupted movement); and
4. The direction of landslide movement relative to the pipeline.

An approximate estimate of the magnitude of triggered landslides can be made using the approach sometimes adopted for regional identification of the extent of landslide hazards. The basis of the recommended approach is the sliding block analysis approach developed by Newmark (1965). The approach is applicable to both disruptive and coherent slides. Disruptive slides typically are shallow and are broken during movement into small blocks, rock fragments, or soil particles. Coherent slides typically are deeper seated movements of intact blocks of soil or rock. For both types of slides, a hazard to the pipeline exists only if the pipeline is above the interface of the potential slide zone.

The recommended approach for identifying the potential for triggered landslides requires an estimate of the critical acceleration, A_c, that will trigger down-slope displacement for the type of slide and material. The critical acceleration can be computed using the following equation developed for the assumption of a constant slope and material strength:

$$A_c = \frac{c'}{\gamma \cdot h} + (1-\lambda)\tan\phi' \cos\theta - \sin\theta \tag{2-17}$$

where:

h	=	thickness of the potential landslide
θ	=	slope steepness angle
c'	=	effective cohesion of the slope material under dynamic conditions
ϕ'	=	effective friction angle of the slope material under dynamic conditions
λ	=	ratio of the pore pressure to overburden stress = $\gamma_w(h - D_w)/\gamma h \leq 0.6$
D_w	=	depth to water table
γ	=	unit weight of soil
γ_w	=	unit weight of water

Equation (2-17) can be used to plot values of A_c as a function of slope angle for various values of c', ϕ', h, γ, and D_w.

An example of a plot of A_c versus slope angle is illustrated in Figure 2.8. The curves in Figure 2.8 are based upon common slope material properties listed in Table 2.2 and assume h = 10 ft and γ = 100 pcf. The two groups of curves in Figure 2.8 correspond to completely saturated conditions (i.e., the water table is at the ground surface) or unsaturated conditions (i.e. the water table depth is greater than h).

The computation of slope displacement generally requires the double integration of earthquake ground accelerations larger than the critical acceleration, A_c, that cause the forces driving slope displacement to exceed the resisting forces. This approach implies that one or more ground motion time histories representative of the site under consideration are available. As a simpler alternative, empirical relationships that envelop the displacements computed for a range of time histories are available for estimating slope displacement as a function of peak ground acceleration, *PGA*.

The amount of landslide movement, L_D, can be estimated using the following bounding relationships.

For very firm ground or rock conditions

$$L_D = 1.7 \cdot PGA \cdot (1 - \frac{A_c}{PGA}) \cdot \left(\frac{PGA}{A_c}\right)^2 \qquad \text{(2-17a)}$$

For soil conditions

$$L_D = 3.0 \cdot PGA \cdot (1 - \frac{A_c}{PGA}) \cdot \left(\frac{PGA}{A_c}\right)^2 \qquad L_D \text{ in inches for Group C} \qquad \text{(2-17b)}$$

where:

A_c = critical acceleration (% of gravity) from Figure 2.8

Triggered landslides should be considered possible if L_D is greater than 1 inch (25 mm) for disruptive landslides or 4 inches (100 mm) for coherent landslides. If the possibility of a triggered landslide exists, site-specific investigations should be performed to quantify the magnitude, extent, and likelihood of landslide movements.

The evaluation of pipeline behavior in landslide areas requires the definition of a displacement pattern for the displaced soil mass as shown in Section 4.5.

2.6 Seismic Wave Propagation

Body waves, including compression waves and shear waves, propagate radially from the source of earthquake energy release (hypocenter) into the surrounding rock and soil medium. Compression waves cause axial compression and tension strains in the ground in a radial direction away from the hypocenter. Shear waves cause shear strains in the ground perpendicular to these radial lines. When the compression waves and shear waves are reflected by interaction with the ground surface, surface waves (Love waves and Rayleigh waves) are generated. Except at very large distances from the epicenter, the magnitudes of surface waves are much less than body waves.

A pipeline buried in soil that is subject to the passage of these ground waves will incur longitudinal and bending strains as it conforms to the associated ground strains. In most cases, these strains are relatively small, and welded pipelines typically do not incur damage. Propagating seismic waves also give rise to hoop membrane strains and shearing strains in buried pipelines, but these strains are even smaller and may be neglected.

A relatively simple method based upon an approach developed by Newmark (1965) can be used to obtain an upper-bound estimate of ground strain due to a propagating seismic wave with a constant shape (sine wave). The maximum ground strain, ε_g, is given by:

$$\varepsilon_g = \frac{V_{\max}}{\alpha_\varepsilon c} \tag{2-19}$$

where:

V_{max} = Maximum horizontal ground velocity in the direction of wave propagation

c = Apparent propagation speed of seismic wave (i.e., the component of wave speed parallel to the pipeline under consideration)

α_ε = Ground strain coefficient corresponding to the most critical angle of incidence and type of seismic wave

= 1.0 for compression and Rayleigh waves (ASCE, 1984)

= 2.0 for shear waves (ASCE, 1984)

The apparent propagation velocity is the inverse of the "slowness," a term used in seismology. The slowness is a function of the source-to-site geometry and the wave propagation velocity along the entire path of propagation. Tables of earthquake wave travel times will exist wherever detailed earthquake location work has been performed. An apparent propagation velocity of 2 km/sec can generally be taken as a lower bound estimate to provide a maximum estimate of ground strain.

If there is no slippage of the pipeline relative to the surrounding soil, then the maximum axial strain in the pipeline is equal to maximum ground strain. The ground strain coefficient corrects this ground strain for the effects of pipe orientation relative to the particular wave motions. If slippage occurs, then pipeline longitudinal strains will be less than the ground strain; hence the assumption that pipeline strain is equal to ground strain is a conservative upper-bound.

The development of the maximum ground strain in the pipeline is dependent upon whether or not there is sufficient pipeline length and a sufficiently long wave length over which to establish equilibrium without longitudinal slipping of the pipeline within the soil. For very soft soils, the assumption of no slipping may lead to a significant overestimate of the strains that can be transferred from the ground to the pipeline.

The maximum ground curvature, κ_g, is the second derivative of the transverse displacement and is given by:

$$\kappa_g = \frac{PGA}{\alpha_\kappa c^2} \tag{2-20}$$

where:

α_κ = Ground curvature coefficient corresponding to the most critical angle of incidence and type of seismic wave

= 1.6 for compression waves (ASCE, 1984)

= 1.0 for shear and Rayleigh waves (ASCE, 1984)

The bending strains calculated by equation (2-20) are of small magnitude. Therefore, as a rule, the effect of curvature associated with seismic wave propagation is neglected.

For buried piping or pipeline segments with significant bends, the strain associated with seismic wave propagation will be influenced by both the virtual anchorage caused by the bend and the local flexibility of the pipe bend. An analysis approach that accounts for these conditions for buried pipelines subjected to thermal strain is provided in Appendix VII of ASME/ANSI B31.1. The same analysis approach may be suitable for determining the effects of seismic wave propagation by equating the seismic strains to thermal strains.

2.7 Transient Ground Deformation

Large transient ground deformations, associated with shear failure of weak soils or liquefaction of subsurface soil layers, have been identified as a potential cause of pipeline damage in recent earthquakes. Two conditions for transient ground deformation (TGD) have been identified based upon investigations following the 1989 Loma Prieta and 1994 Northridge earthquakes in California.

One condition susceptible to TGD is a site close to the earthquake epicenter (typically within 20 km) that is characterized by a narrow valley filled with weak soil overlying much stronger soil deposits and bounded by steep side walls. Such valleys are susceptible to direct lateral loading as the surrounding side walls experience long period displacement pulses associated with the directivity of earthquake rupture (i.e., earthquake "fling"). During the earthquake, the motions imparted to the soft soil by the surrounding side walls of the valley are sufficient to generate shear failure in the weak soil. This shear failure decouples the soft soil from the motions of the surrounding side walls and can lead to large transient relative motions at the interface between the weak soil and the stronger soil deposits at depth.

The second condition susceptible to TGD is a site experiencing liquefaction in soil confined by non-liquefiable material. In this case, liquefaction acts to decouple the soil response from that of the surrounding non-liquefiable material leading to the potential for large transient relative motions at the boundaries between liquefied and competent soils.

There are no established procedures for identifying and quantifying TGD. Techniques that have been used to identify TGD are limited to studies to identify the cause of failures in older pipelines not typical of current construction practices (i.e., girth welds with severe defects, pipelines with large-angle mitre joints, or pipelines with corrosion defects). The few past studies performed have used detailed subsurface information and analyses incorporating actual earthquake time histories from nearby ground motion recording stations.

Table 2.1 Information Required for Performing Liquefaction Assessment

	Information Required	Notation
SPT Assessment	SPT blow counts collected in accordance with ASTM D-1586	N
	SPT bore hole diameter	D_{bore}
	Length of rod connected to SPT sampler	L_{rod}
	Percentage of hammer energy delivered to sampling rod in SPT	E_{eff}
	Fines content of soil as measured by % of a sample passing a #200 sieve	FC
CPT Assessment	CPT tip resistance in accordance with ASTM D-3441	q_c
	CPT sleeve friction in accordance with ASTM D-3441	f_s
SPT and CPT Assessment	Depth to water table at the time of the SPT or CPT	D_w
	Earthquake moment magnitude	M
	Peak ground acceleration at surface	PGA
	Unit weight of soil	γ
	Depth below ground of test measurement	z

Table 2.2 Slope Material Strengths for Three Common Slope Materials

LABLE IN FIGURE 2.8	SLOPE MATERIAL / SOIL CONDITION	Effective Cohesion, c' (psf)	Effective Friction Angle ϕ'
A	Strongly cemented rocks, crystalline rock, and well-cemented sandstone	300	0
B	Weakly cemented rocks, sandy soil, and poorly cemented sandstone	0	35°
C	Argillaceous rocks, clayey soil, and shale	0	20°

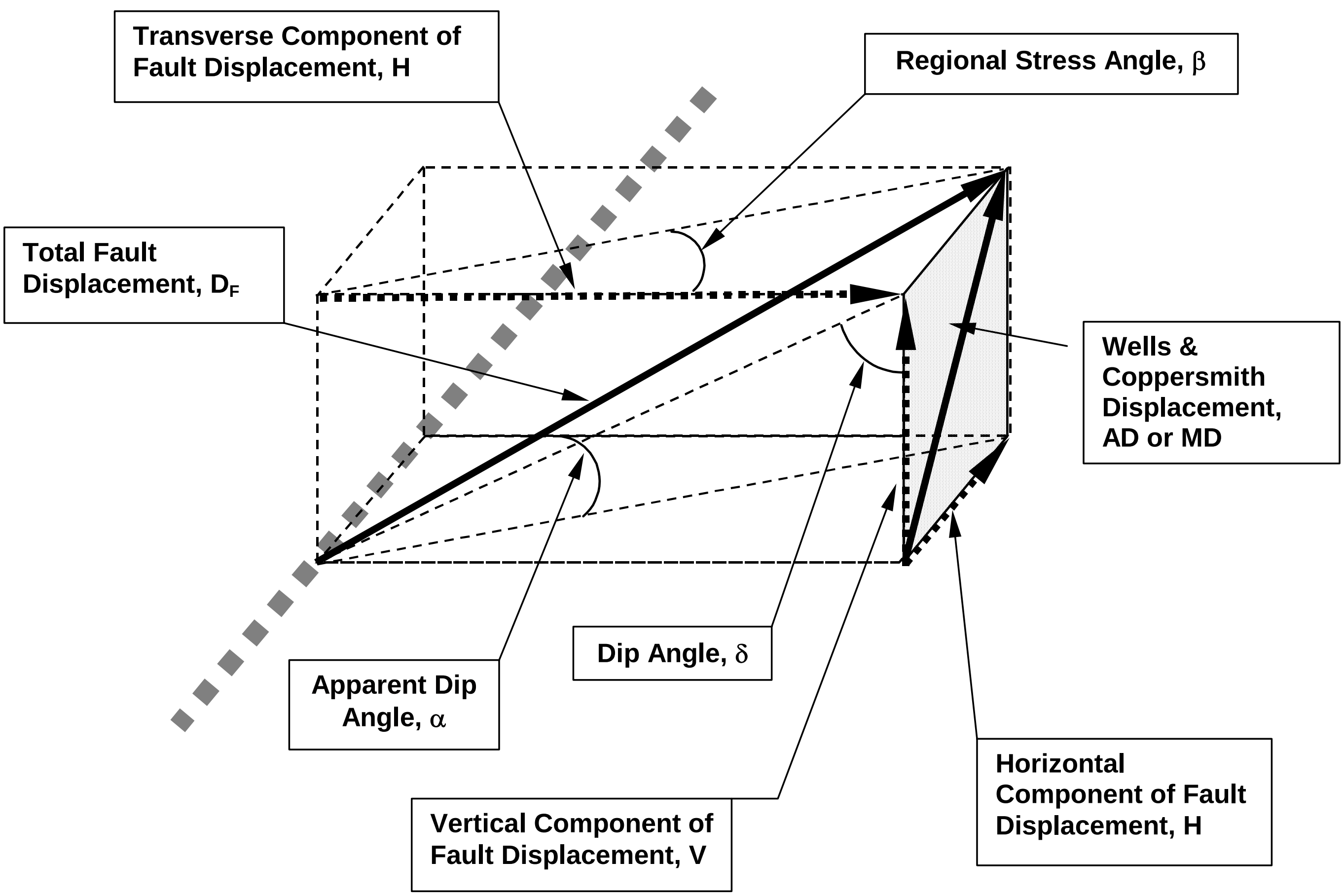


Figure 2.1 Determination of Components of Actual Fault Displacement Based Upon Fault Dip Angle and Regional Stress Azimuth

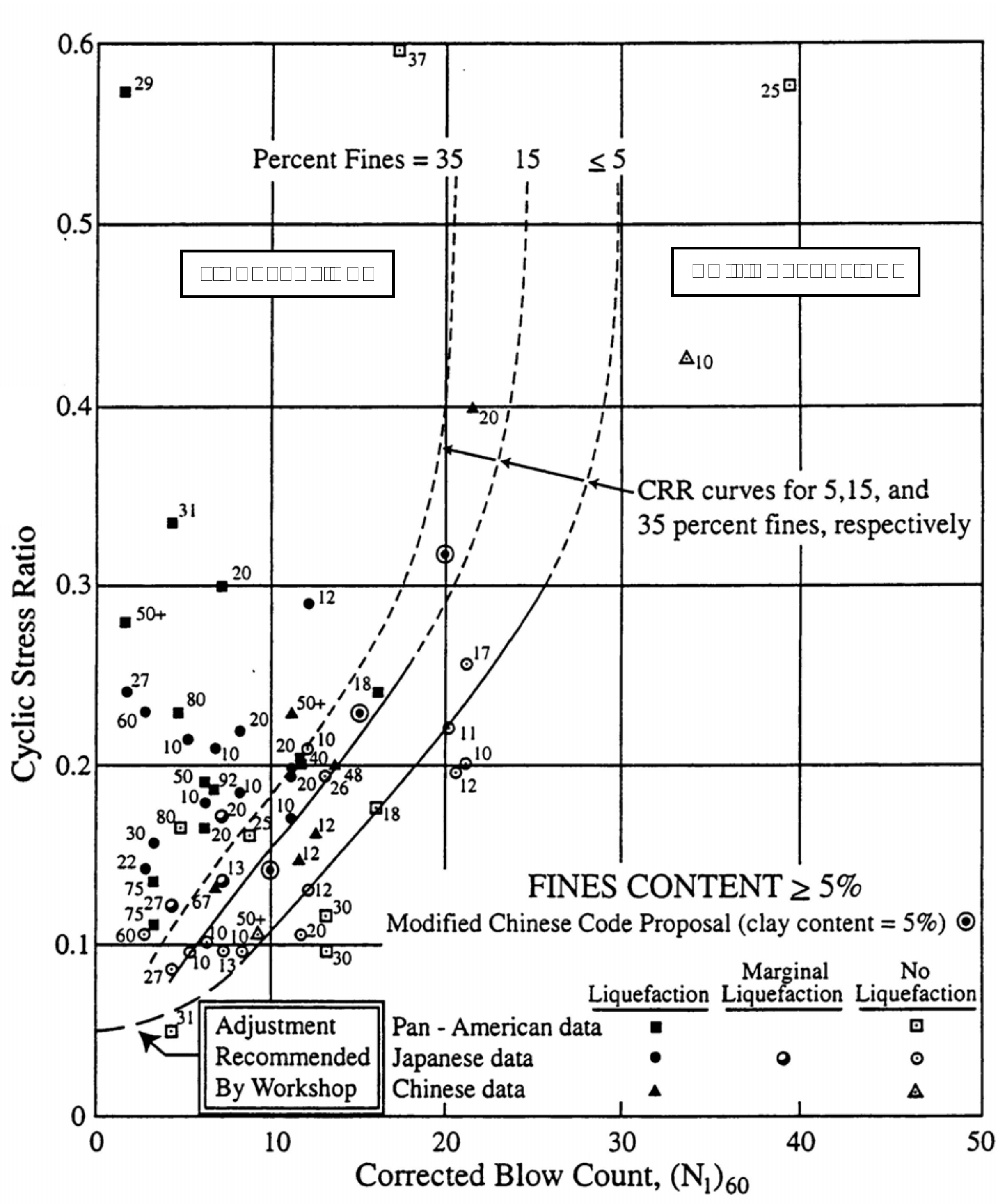


Figure 2.2 CRR Required to Prevent Liquefaction vs. *$(N_1)_{60FC}$* (from Youd and Idriss, 1997)

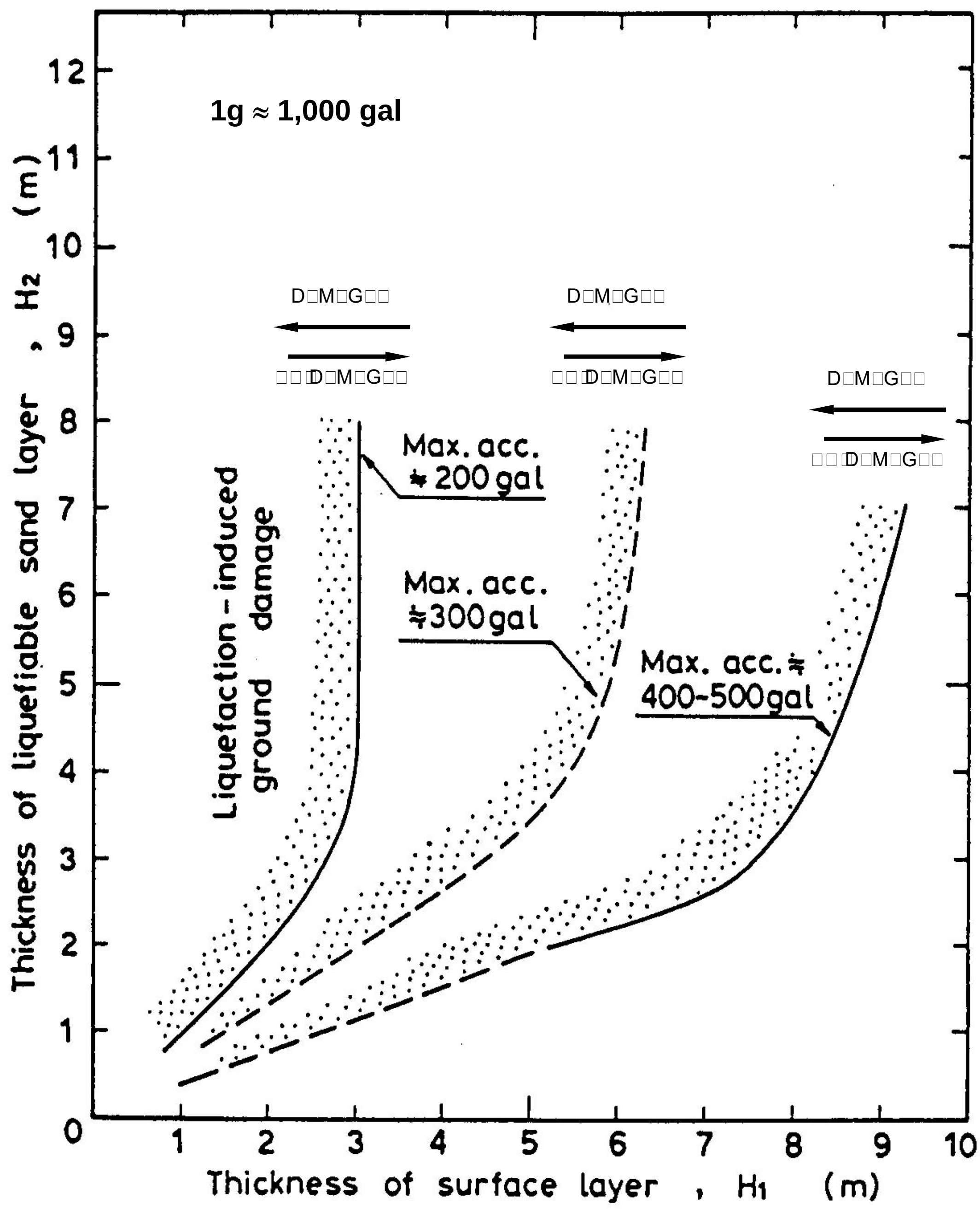


Figure 2.3 Screening Chart for Determining whether Consolidation of Liquefied Soil is a Credible Seismic Hazard (from Ishihara, 1985)

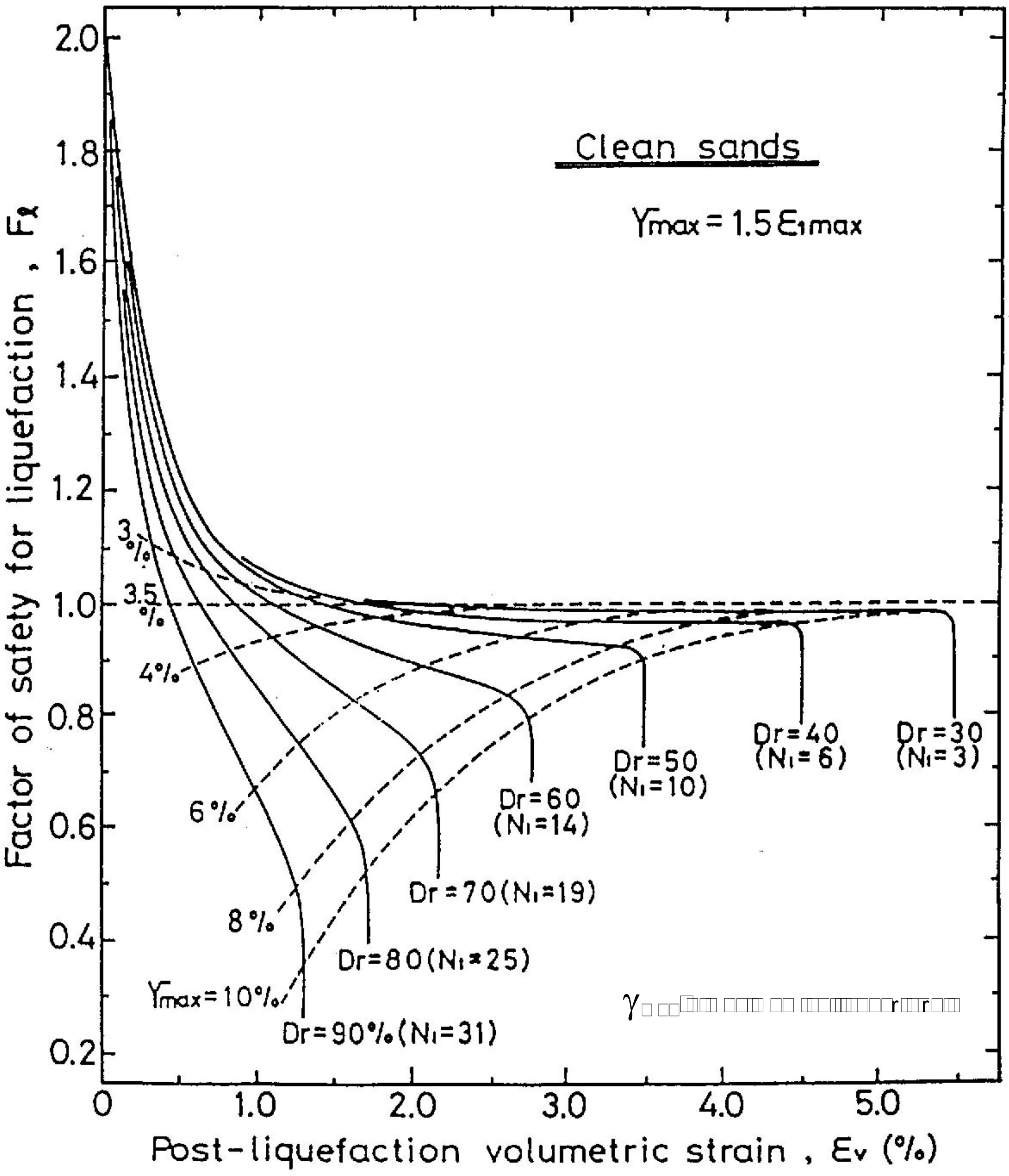


Figure 2.4 Chart to Estimate Earthquake-Related Volumetric Strain in Saturated Clean Sand Soil Deposits (from Ishihara, 1990)

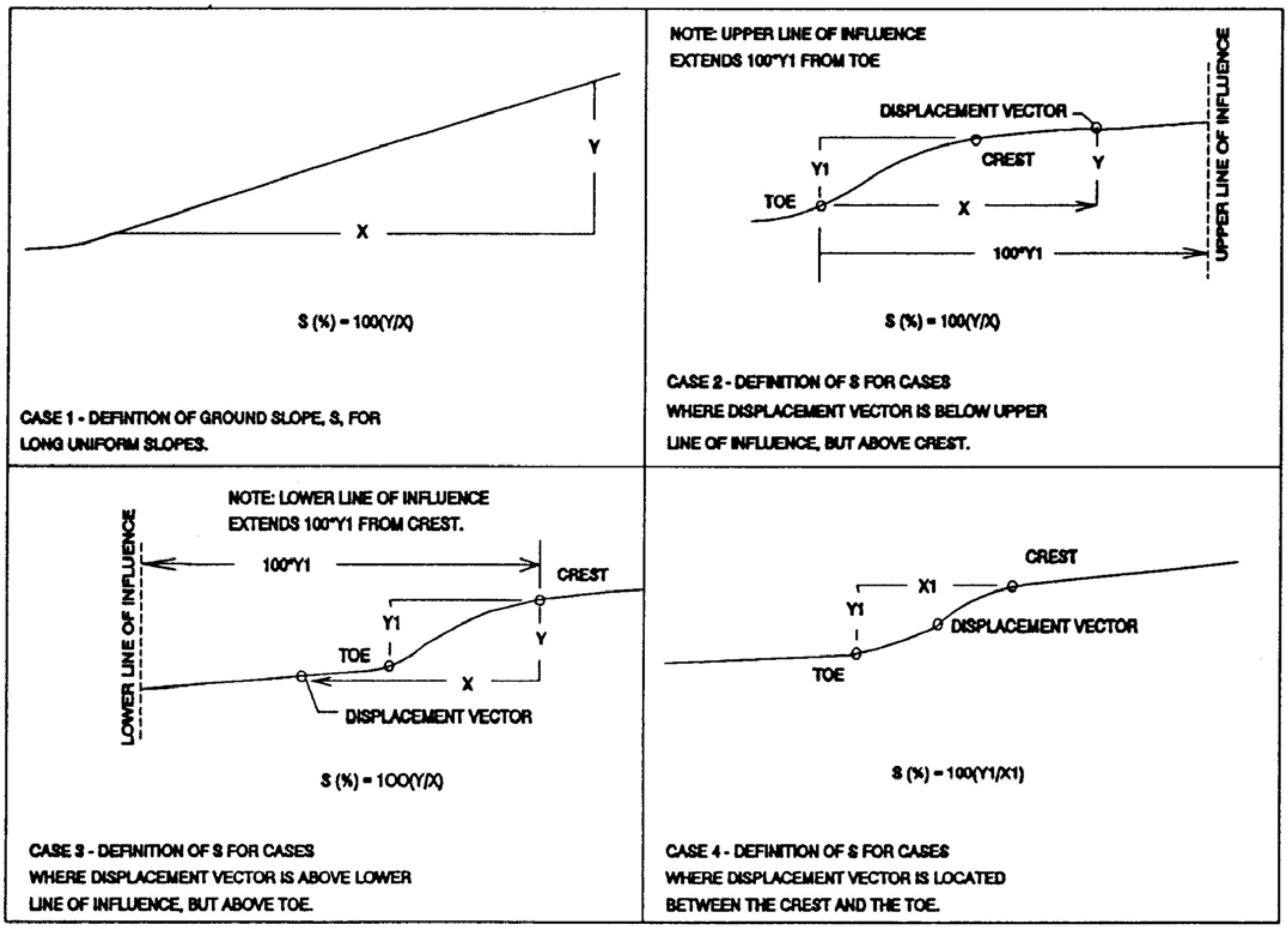
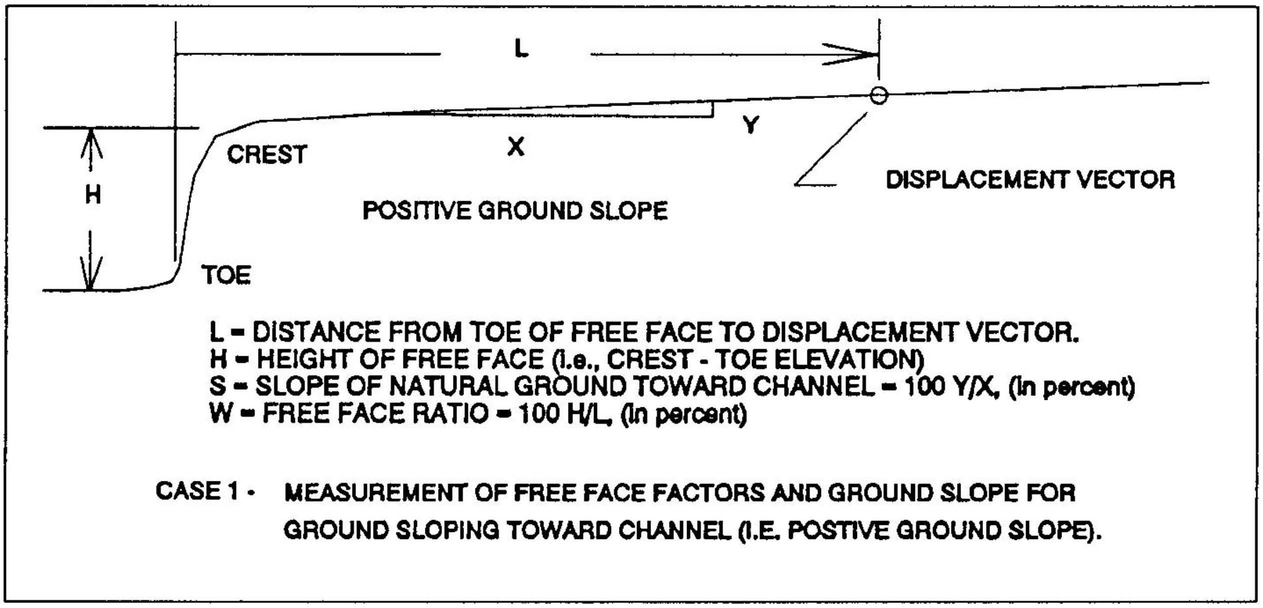


Figure 2.5 Geometry Assumed for Estimating Lateral Spread Displacement for Ground Slope Conditions (from Bartlett and Youd, 1992)



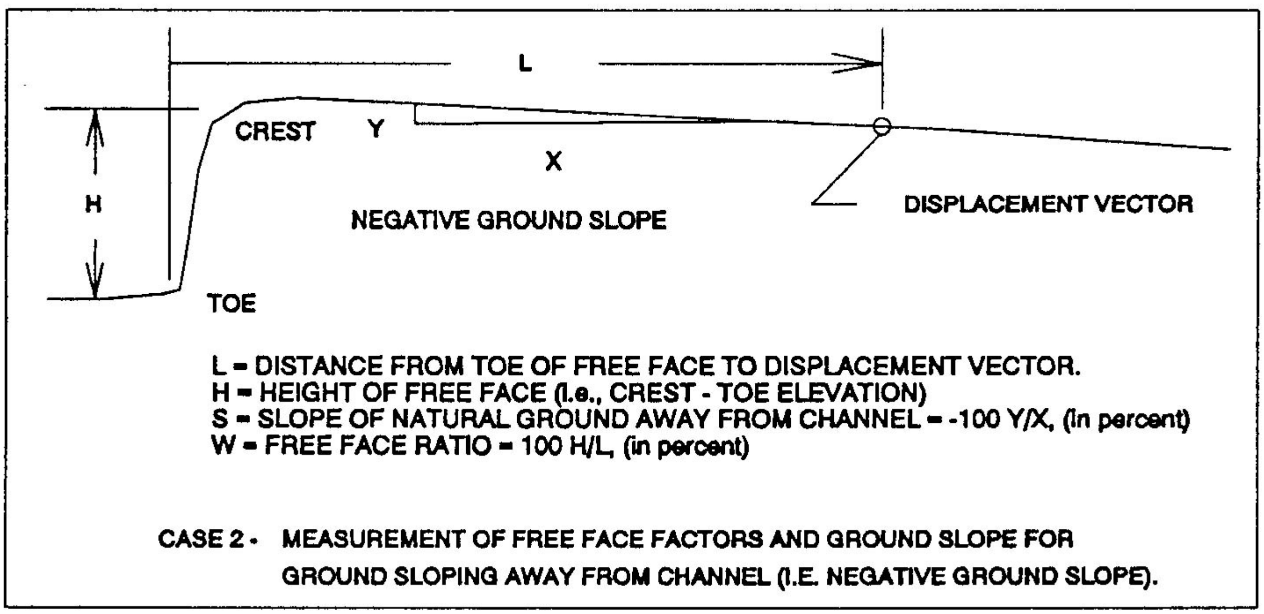


Figure 2.6 Geometry Assumed for Estimating Lateral Spread Displacement for Free Face Conditions (from Bartlett and Youd, 1992)

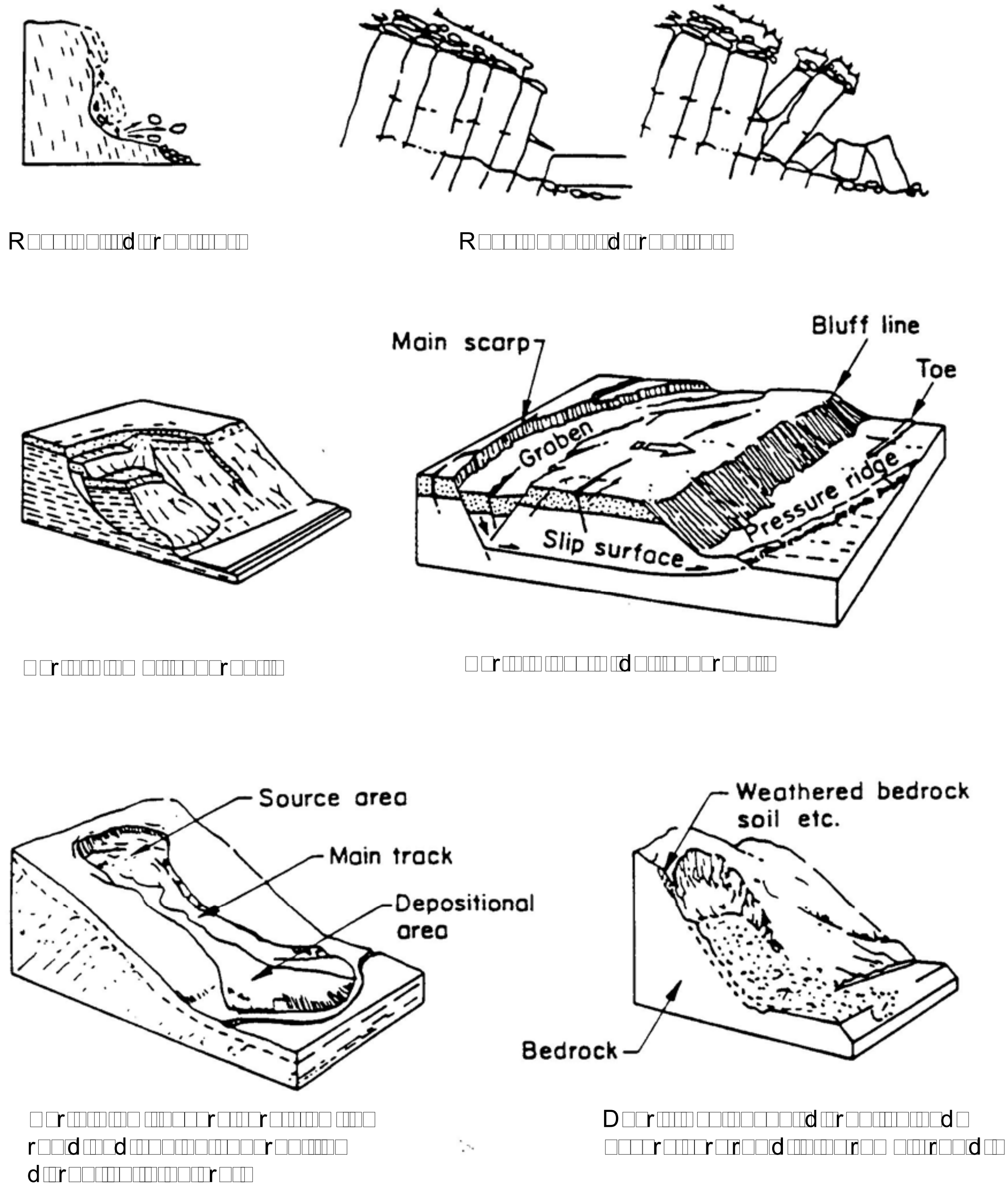


Figure 2.7 Examples of Various Types of Landslides (Varnes, 1978)

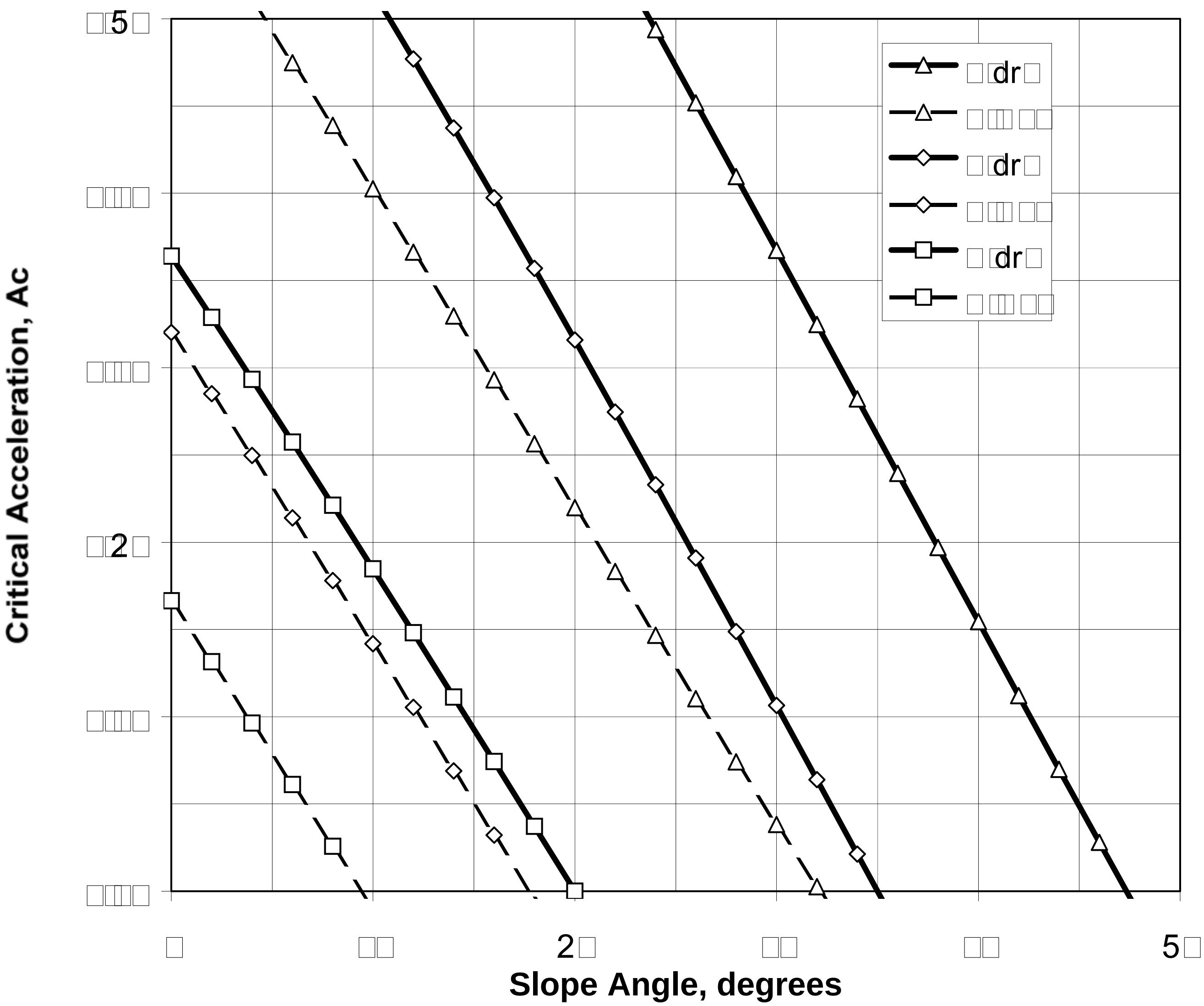


Figure 2.8 Critical Acceleration for Constant Slope and Soil Strength

3.0 PIPELINE PERFORMANCE CRITERIA

The most common type of loads generated on a buried pipeline by seismic hazards is generated by ground movement relative to the pipeline. The pipeline sees no further soil load once it has deformed sufficiently to match the ground movement. This type of loading is commonly referred to as displacement-controlled. Guidance is provided for two performance goals associated with displacement-controlled conditions, maintaining strains to a level that normal operations can continue and maintaining the pressure integrity of the pipeline.

Pipelines buried with minimal soil cover or in relatively weak soils and subjected to high axial loads as a result of ground movement may experience upheaval buckling. While initial pipeline response is displacement controlled, upheaval of the pipeline can lead to some strain relaxation and a related concentration of compression strain in the vicinity of pipeline upheaval. The amount of strain concentration that can occur is limited by the axial soil restraint on the portion of the pipeline that remains buried. This type of behavior is referred to as displacement-limited. Development of significant pipeline compression sufficient to cause upheaval buckling should be avoided by proper orientation of the pipeline relative to the direction of ground movement. Where upheaval buckling can not be avoided, the potential for pipe strains to concentrate needs to be investigated. Such an investigations will typically consider the bending moment capacity of the pipeline, the length of pipeline that might release strain into the portion pipeline undergoing upheaval displacement, and changes in soil restraint as a function of pipeline upheaval displacement. Full treatment of upheaval buckling is beyond the scope of these guidelines.

In some cases, a seismic hazard results in a condition where loads act continuously on the pipeline. This type of loading is commonly referred to as load-controlled. Pipeline strains for load-controlled loading conditions must remain below the strain associated with reduced load carrying capacity of the pipeline. Guidance is also provided for strain limits associated with load-controlled conditions.

All of the strain limit equations in this section are based upon recommendations from previous testing programs. In nearly all cases, strains were measured over a length of pipe, often referred to as the gauge length, equal to 0.5 to 2.0 pipe diameters. For tests that produced wrinkles or buckling in the pipe wall, the gauge length encompassed the wrinkle and buckle. Therefore, the strain limits in this section are applicable to analyses that can resolve global pipe response over a distance of one pipe diameter or less. More refined analytical representation of the pipe (e.g., use of shell elements) will provide information on the strains at a location of local pipe wall distortion. Alternate strain criteria need to be developed to relate pipe performance to these localized strains.

Pipeline performance criteria are defined in terms of allowable longitudinal compression and tension strains for onshore pipelines. The strain criteria assume that an analysis of pipeline response will be performed using finite element techniques and bilinear representation of soil loads as described in Section 4.0. The criteria defined in this section are not applicable to analyses based upon approximations of pipeline response carried out using hand calculations that cannot resolve pipeline strains within a length of approximately one diameter.

3.1 New Pipelines and Pipelines Compatible with Modern Construction Practices

The strain limits provided in this section are applicable to pipelines that meet the following conditions:

1. Pipe material and fabrication equivalent to that specified in API 5L
2. Joint connections using full-penetration butt welds in portions of the pipeline expected to experience stresses near or above yield (e.g., no belled-end welded joints, slip joints, lap-welded joints, sleeves)
3. Weld quality consistent with current welding procedures (e.g., API 1104)
4. Weld strength matching or exceeding the expected actual pipe strength
5. Pipe, weld, and heat-affected zone toughness:

   (a) Average Charpy energy greater than 40 J (30 ft-lb), or

   (b) Average CTOD toughness greater than 0.20 mm.

6. Ratio of yield strength to tensile strength (Y/T) less than 0.92

Strain limits for computed longitudinal compression and tension strains (hereafter referred to as simply compression and tension strains) are provided for three conditions:

1. Strains resulting from imposed ground displacements with the goal of maintaining pressure integrity of the pipeline with a high likelihood of repairs to the pipeline following the earthquake;
2. Strains resulting from imposed ground displacements with the goal of maintaining normal operation of the pipeline with a low likelihood of repairs to the pipeline following the earthquake; and
3. Strains resulting from load conditions in which the displacement of the pipeline is not limited by ground displacement.

3.1.1 Performance Goal: Maintain Pressure Integrity

The performance goal, “maintain pressure integrity,” accepts significant ovalization and possible initiation of wrinkle formation in the body of the pipe. With this performance goal, post-earthquake response should include provisions to reduce operating pressure and perform site investigations to identify indications that permanent ground deformation has occurred. Replacement of the damaged portion of the pipeline may be necessary to resume normal operations.

Longitudinal Compression Strain Limit

$$\varepsilon_{cp} = 1.76\frac{t}{D} \quad \leq \quad \text{the longitudinal tension strain limit} \qquad (3\text{-}1)$$

Longitudinal Tension Strain Limit

$$\varepsilon_{tp} = 0.02 \text{ to } 0.04 \quad \text{(see Commentary)} \qquad (3\text{-}2)$$

[illegible]

The performance goal, “maintain normal operability,” provides a high level of confidence of no significant pipeline damage. The expression in equation (3-1) is valid for diameter to pipe wall thickness ratios of 100 or less and provides a term to account for an assumed level of pipe ovaling.

Longitudinal Compression Strain Limit

Recommended relationships for allowable longitudinal compression strains for continued operation are provided for two types of typical pipeline stress-strain curves: (1) those exhibiting typical rounded shapes at yield and (2) those that exhibit a distinct yield plateau.

Rounded Stress-Strain Curve with Plain Pipe Imperfection

$$\varepsilon_{co} = 0.437\left(\frac{t}{D}\right)^{1.72}\left[1-0.892\left(\frac{p}{p_y}\right)\right]^{-1}\left(\frac{E}{\sigma_y}\right)^{0.70}\left(1.09-\left(\frac{\Delta}{t}\right)^{0.086}\right) \qquad (3\text{-}3a)$$

Rounded Stress-Strain Curve with Pipe End Offset at Girth Weld

$$\varepsilon_{co} = 0.056\left(\frac{t}{D}\right)^{1.59}\left[1-0.868\left(\frac{p}{p_y}\right)\right]^{-1}\left(\frac{E}{\sigma_y}\right)^{0.85}\left(1.27-\left(\frac{imp}{t}\right)^{0.15}\right) \qquad (3\text{-}3b)$$

Distinct Yield Plateau with Plain Pipe Imperfection

$$\varepsilon_{co} = 1.06\left(\frac{t}{D}\right)^{2}\left[1-0.50\left(\frac{p}{p_y}\right)\right]^{-1}\left(\frac{E}{\sigma_y}\right)^{0.70}\left(1.10-\left(\frac{\Delta}{t}\right)^{0.09}\right) \qquad (3\text{-}3c)$$

Rounded Stress-Strain Curve with Pipe End Offset at Girth Weld

$$\varepsilon_{co} = 0.404\left(\frac{t}{D}\right)^{2}\left[1-0.906\left(\frac{p}{p_y}\right)\right]^{-1}\left(\frac{E}{\sigma_y}\right)^{0.80}\left(1.12-\left(\frac{imp}{t}\right)^{0.15}\right) \qquad (3\text{-}3d)$$

where:

t	=	pipe wall thickness
D	=	pipe outside diameter
p	=	internal pressure
p_y	=	internal pressure to produce hoop stress equal to yield stress
σ_y	=	yield stress
E	=	modulus of elasticity of pipe material
imp	=	blister-type initial imperfection expressed as a percentage of wall thickness
Δ	=	offset between pipe joints at girth weld

Longitudinal Tension Strain Limit

$$\varepsilon_{to} = 0.01 \text{ to } 0.02 \quad \text{(see Commentary)} \qquad (3\text{-}4)$$

□□□□□ □□□d□□ □□r□□□d□□ □□d□□□□□

The primary earthquake hazards to buried pipelines result in load conditions that are controlled by the ability of the pipeline to conform to the displacement of the ground (displacement-controlled). It is possible that situations can arise in which the loading on the pipeline is not related to permanent ground displacement. For these conditions, pipeline strength is of primary importance. The strain criteria for load-controlled conditions are based upon preventing formation of local wrinkling of the pipe wall and preventing large post-yield tension strains.

Longitudinal Compression Strain Limit:

$$\varepsilon_{cl} = 0.75\varepsilon_{co} \geq 2.42\left(\frac{t}{D}\right)^{1.59} \qquad (3\text{-}5)$$

Longitudinal Tension Strain Limit:

$$\varepsilon_{tl} = 0.005 \qquad (3\text{-}6)$$

Note that the maximum tension strain limit is equal to the nominal yield strain for pipe materials consistent with API 5L. Given this, the use of the pipe yield stress as a controlling acceptance criterion is also acceptable. If compression strain limits are determined to be less than 0.005, the allowable compression stress limit should be established based upon the stress-strain properties of the pipe material.

3.2 Considerations for Offshore Pipelines

Offshore pipelines that are simply laid on the sea floor with minimum embedment, so as not to be constrained by burial in the soil, can generally accommodate significant levels of deformation without damage. Sections of a pipeline that are trenched into the sea floor will exhibit seismic response characteristics similar to those of onshore pipelines. Both trenched and untrenched pipelines are vulnerable to hazards similar to onshore pipelines including submarine landslides and liquefaction-induced lateral spreads, flow failures, and debris flows. Like onshore debris flows, submarine debris flows can move very quickly and may present a significant load-controlled design condition for an offshore pipeline. In addition, offshore pipelines may be subjected to turbidity currents related to submarine ground failure.

Analysis of offshore pipelines proceeds in the same manner as for onshore pipelines. Assessment of computed pipeline strains needs to address the effects of external pressure to reduce the available compression bending strain. For offshore pipelines, the compression longitudinal bending strain limit is given by equation (3-7):

$$\varepsilon_{cr-p} = \varepsilon_{cr}\left(g - \frac{P_{ext}}{P_{cr}}\right) \tag{3-7}$$

where:

ε_{cr-p} = longitudinal compression strain for condition with external pressure

ε_{cr} = limiting compression strain from Section 3.1

P_{ext} = net external pressure = total external pressure – internal pressure

$P_{cr} = \dfrac{P_e \cdot P_y}{\sqrt{P_e^2 + P_y^2}}$ = predicted collapse pressure

$P_e = 2.2 \cdot E\left(\dfrac{t}{D}\right)^3$ = elastic buckling pressure with Poisson's ratio = 0.3

$P_y = \dfrac{2 \cdot \sigma_y \cdot t}{D}$ = pressure to produce cross-section yielding

$$g = \frac{\sqrt{1+\dfrac{P_y}{P_e}}}{\sqrt{\dfrac{1}{\left(\sqrt{1+\dfrac{\delta_o^2 D^2}{t^2}}-\dfrac{\delta_o D}{t}\right)^2}+\left(\dfrac{P_y}{P_e}\right)^2}} \leq 1$$

δ_o = initial ovalization

3.3 Pipelines Not Compatible with Current Construction Practices

Older pipelines may not conform to the attributes listed in Section 3.1, particularly with respect to pipe toughness and absence of corrosion or weld defects. These conditions may be accounted for in the assessment by appropriate reduction of the allowable strain limits in Sections 3.1.1 through 3.1.3.

Some older pipelines were constructed with a variety of girth weld details with oxyacetylene or arc welding techniques that predated modern day controls on filler material and flux composition and power supply, and lacked good quality control. Bell-and-spigot or bell-bell-chill-ring details are examples of older girth weld details. In addition, older pipelines may have been fabricated in manner that would not be permitted under current codes with such details as mitre bends, wrinkle bends, hot and cold field bends to accommodate changes in alignment. The strain capacity of these types of joints and conditions is highly variable and generally low compared to modern construction. Strain acceptance criteria for these types of pipelines need to be developed on a case-by-case basis. Acceptable approaches for establishing strain criteria include full-scale testing, finite element shell analyses, wide-plate testing, and fracture mechanics evaluations.

More rigorous approaches to establishing allowable strains based upon pipeline inspections and fracture mechanics principles are strongly encouraged. Selecting methods appropriate for specific pipe and load conditions is an area of active research that is not sufficiently developed for incorporation into general seismic guidelines.

4.0 PIPELINE ANALYSIS PROCEDURES

Finite element analysis of pipeline response to imposed ground movements is the preferred means to assess pipeline response using the provisions of these guidelines, particularly considering the advent of high-speed, desktop computers and the proliferation of relatively inexpensive, user-friendly finite element analysis software. The recommended approach is to represent the pipeline with pipe or beam elements and represent the soil loading on the pipeline with discrete spring elements. This approach is illustrated in Figure 4.1.

There are no restrictions on the analysis software that can be used as long as it is capable of capturing the non-linear effects of non-linear soil springs, user-defined stress-strain curves for the pipe material, and large changes in pipeline geometry. The definition of soil springs assumes that the spring forces always act in the axial, horizontal, and vertical directions relative to the pipeline. To maintain this condition for situations where ground and pipeline movements are large, the analysis model should be capable of defining the soil spring displacements relative to the local pipeline coordinate system. The analysis should be capable of accounting for the effects of internal pressure because the hoop stress from internal pressure reduces the longitudinal stress at which the material will yield. It is also very beneficial if the analysis software has the capability to provide computed strains at multiple locations around the circumference of the elements used to represent the pipeline.

4.1 Pipe Element Definition

The pipe element length in regions where the pipe strain is expected to exceed the yield strain (typically at abrupt transitions in ground displacement or locations with abrupt changes in soil restraint such as elbows) generally should not exceed one pipe diameter. Multiple straight or curved pipeline elements can be used to model elbows and bends in the pipeline alignment. The maximum length of pipeline elements used to model elbows should not represent more than a 15° angular change (e.g., at least 6 elements to model a 90° elbow). It is preferable to use multiple curved pipe elements at bends and elbows if they are available. In most cases, the differences in results between using curved pipe elements versus many straight pipe elements are not significant.

4.2 Pipe Stress-Strain Definition

Nonlinear material representation in analysis software is typically based upon a definition of a uniaxial engineering stress-strain curve that is converted to a true stress-strain curve within the software application. Users typically have several options to choose from regarding the modeling of plasticity. For the analysis methodology adopted in these guidelines, bilinear or multilinear isotropic hardening rules based upon a von Mises yield criterion are adequate. Alternate approaches to modeling plastic behavior can also be adopted provided they have a sound basis in engineering mechanics.

4.3 Soil Spring Definition

Soil loading on the pipeline is represented by discrete nonlinear springs as illustrated in Figure 4.1. The maximum soil spring forces and associated relative displacements necessary to develop these forces are computed using the equations given in the sections below.

Soil properties representative of the backfill should be used to compute axial soil spring forces. Other soil spring forces should generally be based upon the native soil properties. Backfill soil properties are appropriate for computing horizontal and upward vertical soil spring forces only when it can be demonstrated that the extent of pipeline movement relative to the surrounding backfill soil is not influenced by the soils outside the pipe trench.

Although tests have indicated that the maximum soil force on the pipeline decreases at large relative displacements, these guidelines are based upon the assumption that the soil force is constant once it reaches the maximum value. The dimension for the maximum soil spring force is force per unit length of pipeline. The equations below are based upon buried pipelines in uniform soil conditions.

For deeply buried pipelines with variable soil properties between the ground surface and the pipeline depth, the equations below may not be representative of the true soil loading conditions. Guidance on how to proceed with variable soil conditions is provided in the Commentary.

Horizontal soil loads on offshore pipelines resting on the sea floor increase more gradually with displacement due to the formation of a mound of soil in front of the pipeline. Determination of the soil spring characteristics for this condition requires special treatment by experienced practitioners and is not covered in these guidelines.

The expressions for maximum soil spring force are based upon laboratory and field experimental investigations on pipeline response, as well as general geotechnical approaches for related structures such as piles, embedded anchor plates, and strip footings. Several of the equations have been derived to fit published curves to facilitate their use in spreadsheets or other computer-based applications.

[illegible]

$$T_u = \pi D \alpha c + \pi D H \bar{\gamma} \left(\frac{1 + K_o}{2} \right) \tan(\delta) \tag{4-1}$$

where:

D = pipe outside diameter

c = soil cohesion representative of the soil backfill

H = depth to pipe centerline

$\bar{\gamma}$ = effective unit weight of soil

K_o = coefficient of pressure at rest

α = adhesion factor, a curve fit to plots of recommended values is

$$\alpha = 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1} \quad \text{where c is in ksf or kPa/50}$$

δ = interface angle of friction for pipe and soil = $f\phi$

ϕ = internal friction angle of the soil

f = coating dependent factor relating the internal friction angle of the soil to the friction angle at the soil-pipe interface

Representative values of f for various types of external pipe coatings are provided below.

PIPE COATING	***f***
Concrete	1.0
Coal Tar	0.9
Rough Steel	0.8
Smooth Steel	0.7
Fusion Bonded Epoxy	0.6
Polyethylene	0.6

Δ_t = displacement at T_u

= 0.1 inches (3 mm) for dense sand

= 0.2 inches (5 mm) for loose sand

= 0.3 inches (8 mm) for stiff clay

= 0.4 inches (10 mm) for soft clay

[illegible]

$$P_u = N_{ch}cD + N_{qh}\bar{\gamma}HD \le Q_d \tag{4-2}$$

where:

N_{ch} = horizontal bearing capacity factor for clay (0 for c = 0)

N_{qh} = horizontal bearing capacity factor (0 for ϕ = 0°)

The expressions below for N_{ch} and N_{qh} are closed-form fits to published empirical (plotted) results (see Commentary).

N_{ch} = horizontal bearing capacity factor for clay (0 for c = 0)

$$= a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} \le 9$$

N_{qh} = horizontal bearing capacity factors for sand (0 for $\phi = 0°$)

$$= a + b(x) + c(x^2) + d(x^3) + e(x^4)$$

Factor	ϕ	x	a	b	c	d	e
N_{ch}	0°	H/D	6.752	0.065	-11.063	7.119	--
N_{qh}	20°	H/D	2.399	0.439	-0.03	$1.059(10)^{-3}$	$-1.754(10)^{-5}$
N_{qh}	25°	H/D	3.332	0.839	-0.090	$5.606(10)^{-3}$	$-1.319(10)^{-4}$
N_{qh}	30°	H/D	4.565	1.234	-0.089	$4.275(10)^{-3}$	$-9.159(10)^{-5}$
N_{qh}	35°	H/D	6.816	2.019	-0.146	$7.651(10)^{-3}$	$-1.683(10)^{-4}$
N_{qh}	40°	H/D	10.959	1.783	0.045	$-5.425(10)^{-3}$	$-1.153(10)^{-4}$
N_{qh}	45°	H/D	17.658	3.309	0.048	$-6.443(10)^{-3}$	$-1.299(10)^{-4}$

N_{qh} can be interpolated for intermediate values of ϕ between 20° and 45°.

Δ_p = displacement at P_u

$$= 0.04\left(H + \frac{D}{2}\right) \le 0.10D \text{ to } 0.15D$$

[illegible]

The equations for determining upward vertical soil spring forces are based upon small-scale laboratory tests and theoretical models. For this reason, the applicability of the equations is limited to relatively shallow burial depths, as expressed as the ratio of the depth to pipe centerline to the pipe diameter (*H/D*). Conditions in which the *H/D* ratio is greater than the limit provided below require case-specific geotechnical guidance on the magnitude of soil spring force and the relative displacement necessary to develop this force.

$$Q_u = N_{cv} cD + N_{qv} \bar{\gamma} HD \qquad (4\text{-}3)$$

where:

N_{cv} = vertical uplift factor for clay (0 for c = 0)

N_{qv} = vertical uplift factor for sand (0 for $\phi = 0°$)

$$N_{cv} = 2\left(\frac{H}{D}\right) \leq 10 \qquad \text{applicable for } \left(\frac{H}{D}\right) \leq 10$$

$$N_{qv} = \tan(\phi)\left(\frac{\phi}{44}\right)\left(\frac{H}{D}\right) \leq N_q \qquad \text{(see Section 4.3.4 for definition of } N_q\text{)}$$

Δ_{qu} = displacement at Q_u

= $0.01H$ to $0.02H$ for dense to loose sands $\leq 0.1D$

= $0.1H$ to $0.2H$ for stiff to soft clays $\leq 0.2D$

[illegible]

$$Q_d = N_c cD + N_q \bar{\gamma} HD + N_\gamma \gamma \frac{D^2}{2} \tag{4-4}$$

where:

N_c, N_q, N_γ = bearing capacity factors

$$N_c = \cot(\phi + 0.001)\left[e^{\pi \tan(\phi + 0.001)} \tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right]$$

$$N_q = e^{\pi \tan(\phi)} \tan^2\left(45 + \frac{\phi}{2}\right)$$

$N_\gamma = e^{(0.18\phi - 2.5)}$ (this is a curve fit to plotted values of N_γ)

γ = total unit weight of soil

Δ_{qd} = displacement at Q_d

= $0.1D$ for granular soils

= $0.2D$ for cohesive soils

4.4 Extent of Pipeline Model

The length of pipeline modeled outside of the zone of applied ground movement is dependent upon the specific pipeline alignment. The length of pipeline needs to be sufficient to assure that small elastic deformations outside the area of high pipeline strains do not significantly change the magnitude of the highest computed longitudinal strains. An estimate of the extent of the model for which the pipe can be considered anchored to the soil, L_{anchor}, can be computed using the following formula:

$$L_{anchor} = \frac{\pi D t \sigma_y}{T_u} \tag{4-5}$$

The length L_{anchor} is the distance necessary for the axial soil force, T_u, to generate axial yield in the pipeline. Termination of pipelines within a zone of ground deformation should force the pipeline to move with the ground. For example, analysis of straight pipelines crossing faults will have one end of the pipeline model fixed at a distance L_{anchor} from the fault and the other end displaced an amount consistent with the fault movement.

For pipelines that do not have a straight alignment outside of the zone of ground movement (e.g., side bends, over bends, sag bends), a virtual anchor location may exist at a distance considerably less than that estimated by equation (4-6). This actual point at which the pipeline is anchored needs to be considered on a case-by-case basis. If the location of anchorage is not readily apparent, the modeling can include the actual pipe configuration for a pipe length equal to L_{anchor}. Any anchorage provided by the pipe configuration (e.g., bends, elbows, changes in soil cover) will then be captured in the analysis.

4.5 Representation of Applied Ground Movement

Ground movements representative of the displacement patterns and amplitudes for lateral spreads or landslides are applied to the base of the soil spring elements. The ground deformations should be specified based upon estimates of relative ground movement at the depth of the pipeline. Ideally, the expected pattern of ground displacement should be determined on the basis of geotechnical field investigations, but such may not be feasible in all cases considering the spatial extent of pipelines and the potential for numerous hazard areas.

It is always conservative to assume that the ground displacement occurs abruptly. Abrupt ground displacements have been observed at surface faults, as well as the head of landslides and lateral spreads. In some locations, it may be possible to infer future ground displacement patterns from past earthquake displacements. For locations with little earthquake experience to aid in the determination of ground deformation pattern, an abrupt offset cannot be ruled out without the assistance of a specialist in geology or geotechnical engineering.

4.6 Sensitivity Analyses

The selection of material properties, soil strengths, and ground movement patterns is an inherently uncertain process. Generally, additional analyses should be performed to provide information on the sensitivity of the computed strain levels to changes in input parameters when the level of conservatism associated with the input parameters is not well understood. This information can be used to better define the “best estimate” of pipe response and provide information that can be used to assess the level of confidence in the expected pipeline performance. The range of variation to be used in these sensitivity analyses generally can be estimated in conjunction with the selection of baseline parameters and any geotechnical investigations of soil properties and ground deformations. Unless other information is available to determine the amount of variation, the following are suggestions for examining the sensitivity of the results obtained by analysis:

1. Upper-bound estimates of pipe material strength (as opposed to specified minimum values);

2. Variation in soil strength to capture the reasonable range of upper-bound and lower-bound ranges;

3. Increased applied ground displacement (this can normally be done at the time of the analysis); and

4. Modifications in the ground displacement transitions at boundaries with zones of potential ground movement.

The above variations typically result in a range of acceptable ground displacements or a range of strains for a particular ground displacement value. If a balanced range of parameters is selected, the “best estimate” of pipe response can typically be estimated from the resulting range in pipeline response determined from the variation in input parameters.

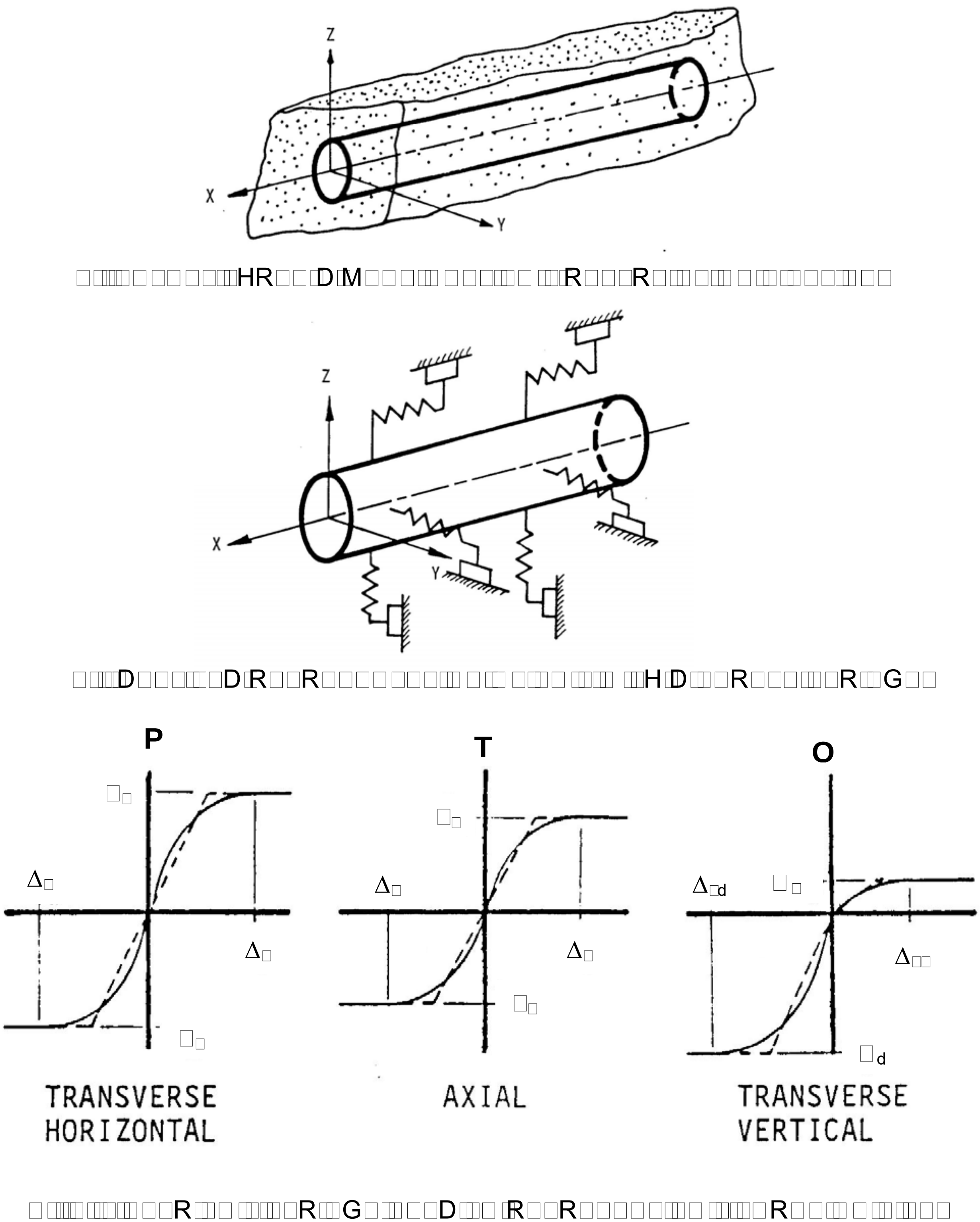


Figure 4.1 Pipeline Modeling Approach

5.0 MITIGATION OPTIONS

There are several options to improve post-earthquake conditions if the pipeline response is found to exceed the acceptance criteria defined in 3.0. Selection of a particular approach is dependent upon considerations that vary with pipeline location, expected failure mode, potential for collateral damage, risk acceptance philosophy, and estimated mitigation costs. Mitigation options fall into one of four categories: (1) relocate the pipeline to avoid the hazard, (2) modify/design the pipe construction to reduce the soil loads on the pipeline, (3) modify the pipeline configuration to increase its ability to resist ground displacement, and (4) modify post-earthquake response procedures to address the consequences of pipeline damage. Relocation to avoid the hazard requires no further explanation. The following discussion focuses on pipeline installation changes and operational measures to improve pipeline performance.

5.1 Modifying Pipeline Loading Conditions

Soil loads on buried pipelines can be reduced in several ways. The most common approach is to minimize the strength of the soils surrounding the pipeline or the frictional characteristics of the pipeline. The alignment of the pipeline can also be modified to change the orientation and distribution of soil loads with respect to the pipeline. Finally, provisions can be made to isolate the pipeline from relative soil movement. Potential options for implementing changes to modify the soil loading on buried pipelines are summarized below. Many of the options have limited applicability because of restrictions related to right-of-way access, the need to avoid existing subsurface structures and utilities, or the compaction requirements associated with various types of land use. Urban environments are particularly restrictive with respect to the feasibility of mitigation options to improve pipeline response.

5[illegible] R[illegible]d[illegible]d[illegible]M[illegible]r[illegible]r[illegible]r[illegible]

The capacity of a buried pipeline to withstand ground displacements can be improved by minimizing the longitudinal, lateral, and uplift soil resistance to pipe movements.

5.1.1.1 Loose Granular Backfill

A practical means for achieving minimum soil restraint is to bury the pipeline in a shallow trench filled with a loose granular backfill. As depicted in Figure 5.1, the trench walls should be sloped at an angle of about 30° to 45° for horizontal ground displacement components and about 60° for vertical ground displacement components. This trench geometry will allow the soil to fail within the backfill material rather than in the higher strength, undisturbed soil outside the trench.

For typical pipeline trench conditions, loose granular backfills (sand or gravel) will offer less resistance to pipe movement than compacted cohesive backfill materials (clay or silty clay). A granular material with an angle of internal friction of 35° or less is recommended. To satisfy an angle of internal friction of 35° or less, the backfill material should be a well-graded granular material with 100 percent of the aggregate less than one inch (25 mm) in diameter. The material should be obtained from a natural well-graded fluvial deposit; crushed rock is not acceptable. The backfill should be moderately compacted to a relative density, D_r, of 66 percent or less if achievable.

The assumption of any particular soil condition and the development of spring restraint properties must be consistent with field conditions. In particular, for horizontal relative ground displacement, the logarithmic spiral failure surface (depicted in Figure 5.2) must be enveloped by the limits of the excavated pipe trench that is backfilled with the selected material. Similarly, for vertical relative ground displacements, the upward breakout must occur within the designated backfill. If these trench excavation and backfill requirements are not satisfied, soil parameters applicable to the hybrid situation of in situ soils and trench backfill must be considered in the development of soil restraints for the pipeline, and these restraint properties are typically much higher than for loose granular backfill.

5.1.1.2 Locating Pipeline at or Above Grade

Lateral soil loads can be greatly reduced by placing the pipeline on the ground surface or on aboveground supports. Typcially this is done by attaching sliding shoes to the pipeline that bear on structural steel members tied to the ground or mounted in an aboveground configuration (e.g., aboveground segments of the Trans-Alaska pipeline). Teflon, or other low-friction materials, can be incorporated into the construrction of the sliding shoes to improve the ability of the pipeline to accommodate ground displacement by sliding laterally. Several options for such a modification are shown in Figure 5.3. Where soil cover is required to protect the pipeline from third-party damage, an earthen berm can be used in lieu of burial. Locating pipelines above ground is rarely a practical solution outside of controlled access areas or very remote regions. However, in some cases, it is feasible to construct a sliding support configuration at the bottom of an open trench. The top of the open trench is will generally be covered (e.g., steel grating) to protect the pipeline from vandalism and reduce the impacts on surface activities.

5.1.1.3 Low-Friction Coating or Protective Wrapping

Axial soil friction loads can be further reduced over what is achieved by loose backfill by the use of smooth, hard, low-friction coatings. Larger reductions in axial soil friction can be obtained by creating a preferred slip surface other than the pipe-soil interface. One means of achieving this is to use two separate layers of geosynthetic wrapping as shown in Figure 5.4a. This type of installation forces axial slip to occur at the interface between the two layers of geotextile fabric and can reduce the interface friction angle used to calculate maximum axial soil spring force to less than 10°. In practice, the reduction in axial soil friction force obtained from a double geotextile fabric wrapping should be verified by field tests under soil and backfill conditions that replicate the actual installation. The effects of aging on the low friction interface should also be evaluated.

5.1.1.4 Geosynthetic Lining of Sloped Trench Walls

Additional reduction in horizontal soil load over what can be achieved using loose granular backfill in conjunction with sloped trench walls (see 5.1.1.1) is possible by lining the walls of the trapezoidal trench with two layers of geosynthetic fabric. The two layers of geosynthetic fabric create a low-friction failure surface in lieu of the logarithmic spiral failure surface that would be developed in the backfill material. The load necessary to overcome the friction between the two layers of geosynthetic fabric is much less than that required to develop a shear failure in the backfill soil.

5.1.1.5 Replacing Soil with Geofoam

Geofoam materials offer a means to reduce axial, lateral, and upward vertical soil loads. Geofoam is a rigid cellular plastic foam of either expanded polystyrene (XPS) or extruded polystyrene (EPS). Geofoam has been used extensively in northern Europe for subgrade insulation in regions susceptible to frost heave. Another usage of geofoam in Europe and the U.S. is as low-density fill for construction over weak soils. One common application is to use geofoam as fill for bridge approaches and abutments. Geofoam varies in weight from about 160 N/m^3 (1 lb/ft^3) to 470 N/m^3 (3 lb/ft^3). Compressive strength of XPS is generally less than EPS although the compressive strength of both increases with density. The typical range of compressive strengths is 140 kPa to 240 kPa (20 psi to 35 psi) for EPS and 200 kPa to 500 kPa (30 psi to 75 psi) for XPS.

Replacing soil above the pipeline with geofoam reduces axial friction force by effectively reducing overburden stresses acting normal to the pipeline. Care must be taken to maintain a proper balance between limiting pipeline restraint for ground movement, yet providing sufficient restraint to prevent upheaval buckling of straight pipe and excessive bending stress at pipe bends due to operating load conditions.

Lateral loads/restraints on buried pipelines also can be reduced by using geofoam to replace much of the backfill soil as shown in Figure 5.4b. This load reduction is achieved due to two factors. First, the boundary between soil and geofoam forms a failure surface that is weaker than that corresponding to the logarithmic spiral failure surface depicted in Figure 5.2. Second, the weight of material displaced along the soil-geofoam boundary is much less. The shearing force can be reduced even further by placing loose sand between the geofoam and the native soil or through the use of dual layers of geosynthetic fabric at the interface.

The use of geofoam has some advantages in urban settings because the compressive strength of the geofoam is sufficient to handle light traffic loads. Potential significant drawback to geofoam applications include increased vulnerability to applied external loads, need to prevent contact with gasoline and other organic fluids or vapors, high flammability, and cost.

5.1.1.6 Use of Crushable Material to Limit Maximum Loads on the Pipeline

Controlled strength material around the pipeline can be used to limit the lateral loads that can be exerted normal to the pipe wall. Geofoam and cellular concrete are two materials well-suited for this purpose since they can experience large compression strain under near-constant compression load. Cellular concrete is a mixture of sand, cement, and water to which a foaming agent or polystyrene beads are added to create small air pockets. The use of crushable material as a mitigation measure is typically only a practical consideration for fault crossings in rock or rock-like materials where very large lateral soil-spring forces can be generated and excavating a trapezoidal trench that can be filled with loose granular material is difficult or impractical. Controlled-strength materials improve pipeline response by allowing the pipe to bend in a more gradual manner to accommodate the imposed ground displacement.

The distance from the pipeline that is to be filled with controlled-strength material is largely governed by the amount of compressive irrecoverable strain that can be accommodated before the material begins to exhibit much higher compressive strength. This strain level is commonly referred to as the lock-up strain. The lock-up strain for cellular concrete can vary from 15% to 35%. The lock-up strain for EPS or XPS geofoam can vary from 25% to more than 50%.

5[illegible]2[illegible] [illegible]r[illegible]d[illegible]

The capacity of a buried pipeline to withstand ground displacement components can be improved by maximizing the distance from the deformation zone (fault rupture, landslide, lateral spread, etc.) to points of virtual anchorage, typically side bends, overbends, and sagbends. Sharp bends, tees, branch fittings, valves, etc. also will have a tendency to anchor the pipeline against axial movement and should be avoided within or near a zone of ground displacement. Good design practice is to provide a straight segment of pipeline as long as practical through and beyond the ground displacement zone to maximize the length of pipeline available to distribute strain.

5[illegible] [illegible]r[illegible]Gr[illegible]d[illegible]D[illegible]

Soil loads on buried pipelines are the result of relative movement between the pipe and the surrounding soil. This relative movement can be minimized or eliminated by providing space around the pipeline that is greater than the relative movement associated with poor pipeline response. Isolating the pipeline from ground displacement is generally practical only when the ground displacements are relatively small, the length of pipeline requiring isolation is relatively short, or the pipeline diameter is relatively small.

Mitigation concepts using culverts allow the pipeline to respond in a manner similar to an on-grade condition. The term culvert refers to any buried structure built partially or completely around the pipeline to provide an unobstructed space for the pipeline to deform in a direction transverse to its axial alignment. Three conceptual culvert configurations are illustrated in Figure 5.5. Culvert concepts are essentially specialized casings and the same problems that can arise for cased pipelines generally apply to culverts. In addition, caution is needed to assure that axial loads from thermal changes or internal pressure do not lead to buckling of the pipe within the culvert. Axial buckling can typically be prevented by incorporating bends or expansion loops in the pipeline.

5.2 Modify Pipeline Configuration

In some cases, minor modifications to proposed or existing pipeline configurations can greatly improve performance. Such modifications include increasing the pipe wall thickness, increasing the strength and toughness of the pipe material, and replacing sharp bends and elbows with induction bends or gradual pipeline field bends.

Increasing the pipe wall thickness increases the allowable longitudinal compression strain and increases the bending and axial strength of the pipeline relative to the soil. Avoiding sharp bends and elbows can minimize the degree to which the pipeline must accommodate the ground displacement through local bending.

Isolation valves, automatic or remotely controlled, may be provided on each side of the zones of ground displacement to mitigate the consequences of possible pipeline ruptures. It should be recognized that the use of automatic or remotely-controlled valves on liquid lines also involves consideration of the effects of hydraulic transients to avoid having a rapid shutdown result in risks from possible overpressurization more significant than the risks from the seismic hazards.

5.3 Modify Emergency Response Procedures

Knowing that a pipeline is susceptible to earthquake damage and planning for the damage in the event an earthquake occurs is often sufficient to assure that the performance criteria for the overall system are met. Planning for pipeline earthquake damage is not fundamentally different than planning for damage from any other source. However, normal emergency response procedures are typically inadequate for dealing with post-earthquake recovery, because there may be multiple emergencies occurring simultaneously at a time when the general civil infrastructure is damaged (e.g., roads, bridges, electric power, communication) and access to the pipeline or coordination of inspection and repair efforts may be impeded.

Knowledge of the conditions under which emergency response procedures will be implemented is vital. A partial list of some conditions to consider is provided below:

1. Normal electric power may be lost for several hours to several days.
2. Water and sanitary services may be lost for several hours to weeks.
3. There likely will be reduced demand for product.
4. Telephone services, wire and wireless, may be interrupted or heavily congested.
5. Highway bridges and over passes may be damaged, resulting in road closure.
6. Traffic may be congested due to road damage and emergency activity.
7. Personnel may be unavailable due to property damage, injury, or family needs.

8. Maintenance and construction contractors may be unavailable due to increased demand for their services or damage to their facilities.

9. Repair crews may be hampered by food and fuel shortages and interruption of electric power, water, and sanitary services.

10. Damage to adjacent facilities may hamper access to pipeline repair areas.

11. Replacement pipe, fittings, and valves may be unavailable in the quantities required.

Planning for rapid earthquake response needs to be coordinated with local and regional government authorities, as well as key customers. Identifying potential post-earthquake response needs with local and regional governments can minimize delays in accessing damaged portions of the system and problems that may arise with securing a work site. Also, governmental agencies can provide important information on the earthquake hazard and expected levels of earthquake damage to buildings and services such as electricity, water, sewer, and transportation. Making customers aware of the potential severity and duration of service interruption can allow them to incorporate the information into their own emergency response planning.

Finally, it is not sufficient to simply have an earthquake response plan. Because of the infrequent nature of earthquakes, regular earthquake simulation exercises are necessary to maintain personnel readiness and identify potential planning deficiencies. These exercises should be coordinated with local and regional planning exercises to identify coordination issues and take full advantage of current information on earthquake hazards and other earthquake damage that could potentially jeopardize a rapid response to pipeline damage.

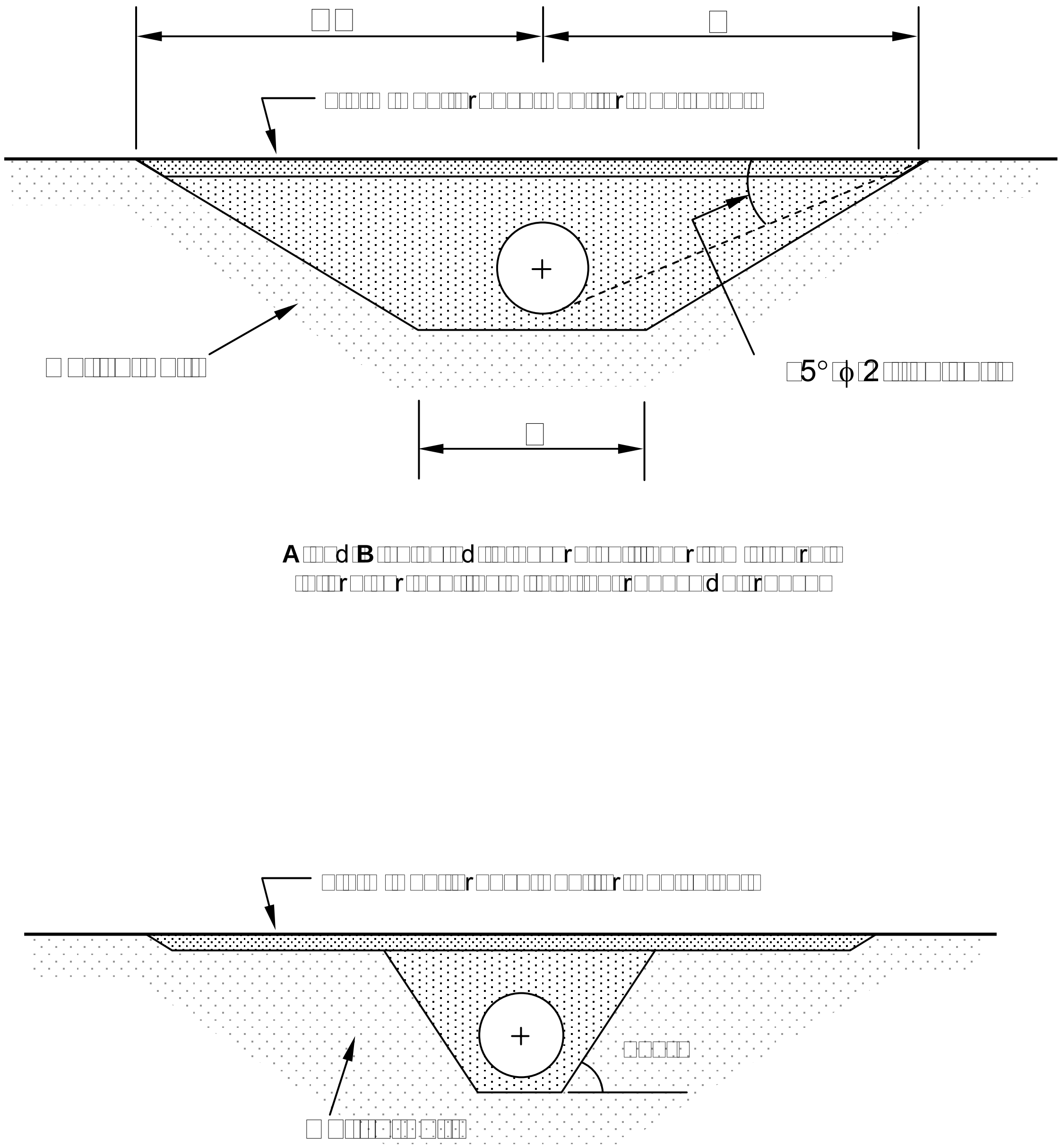


Figure 5.1 Use of Loose Granular Backfill to Reduce Soil Loads

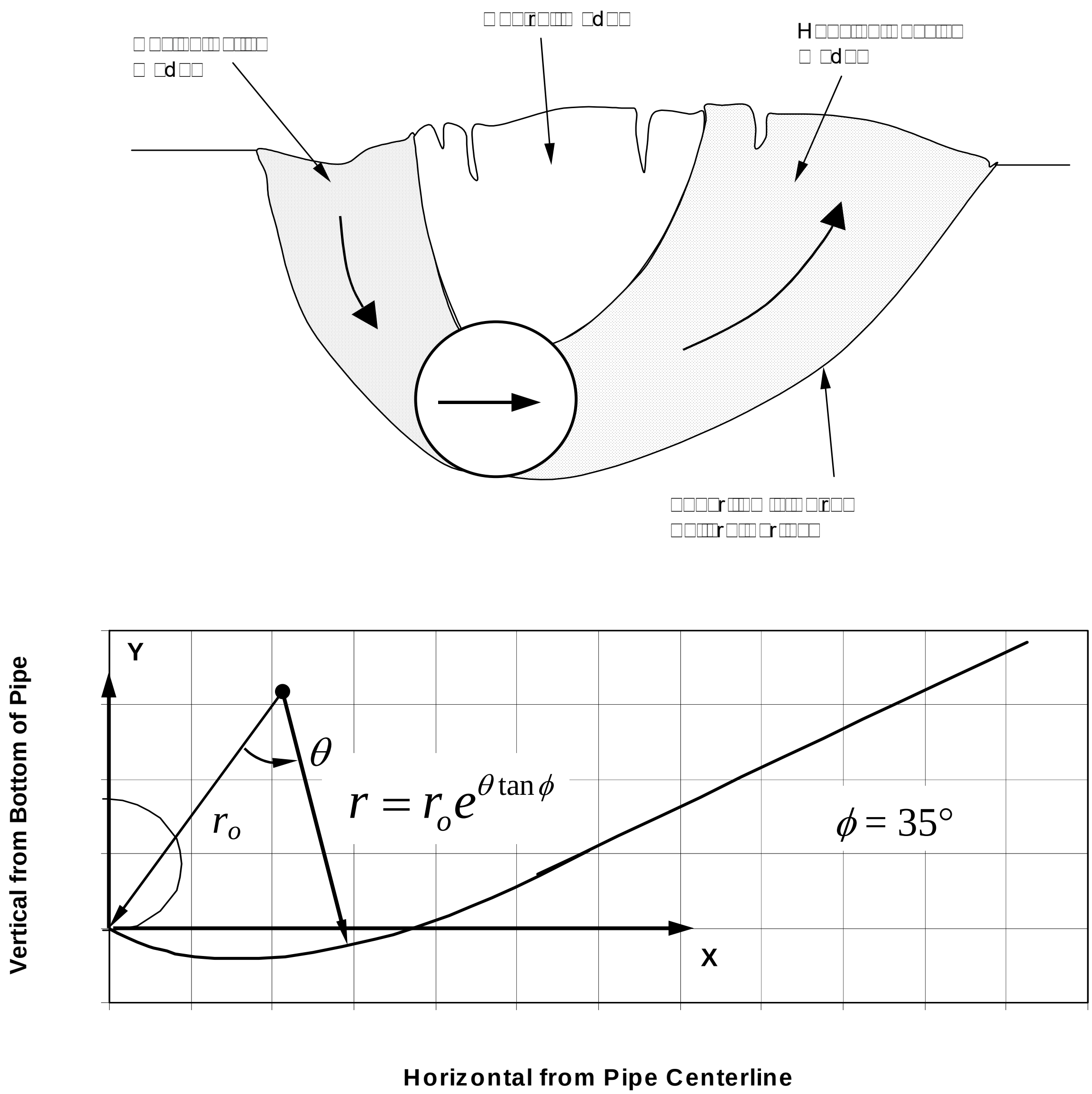


Figure 5.2 Theoretical Logarithmic Spiral Failure Surface

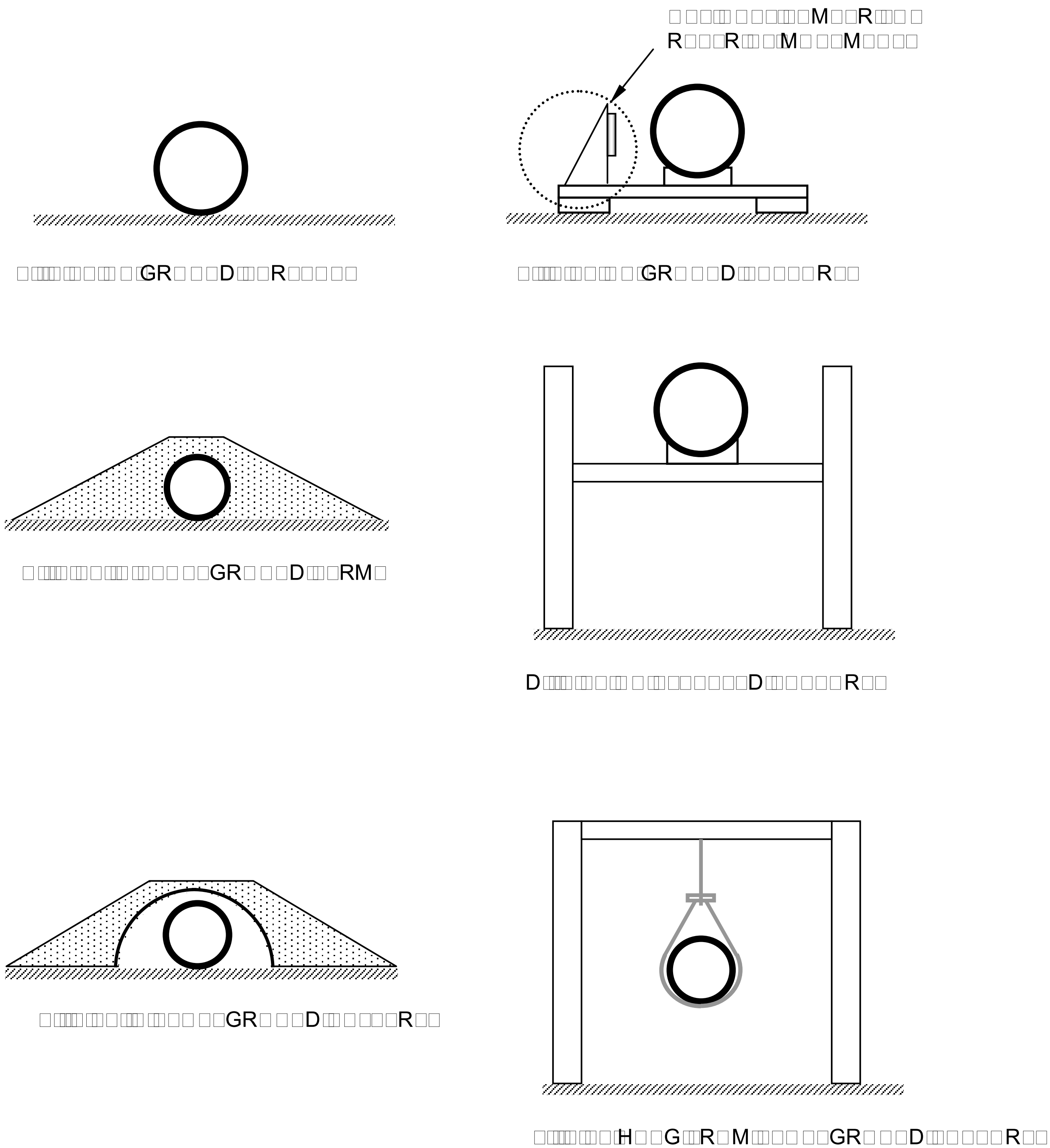
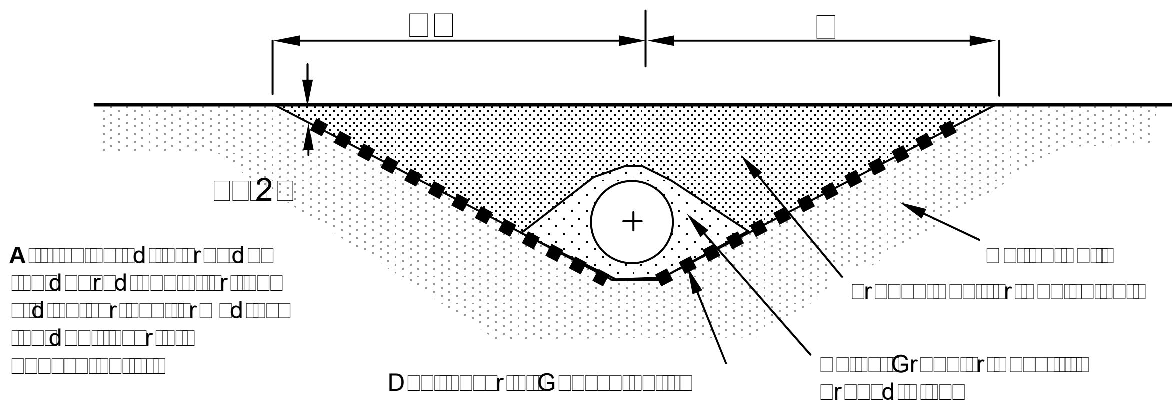


Figure 5.3 Aboveground Mitigation Concepts



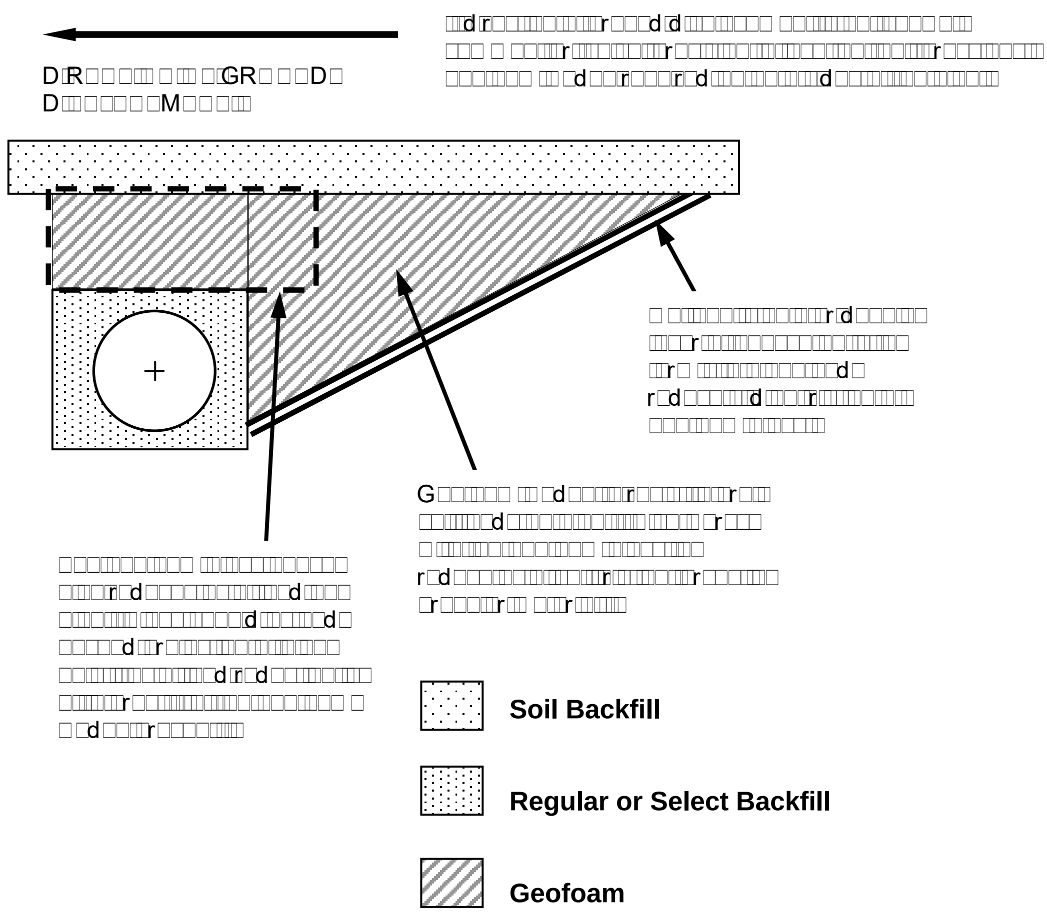


Figure 5.4 Application of Geosynthetic Geofoam Materials to Reduce Soil Loads

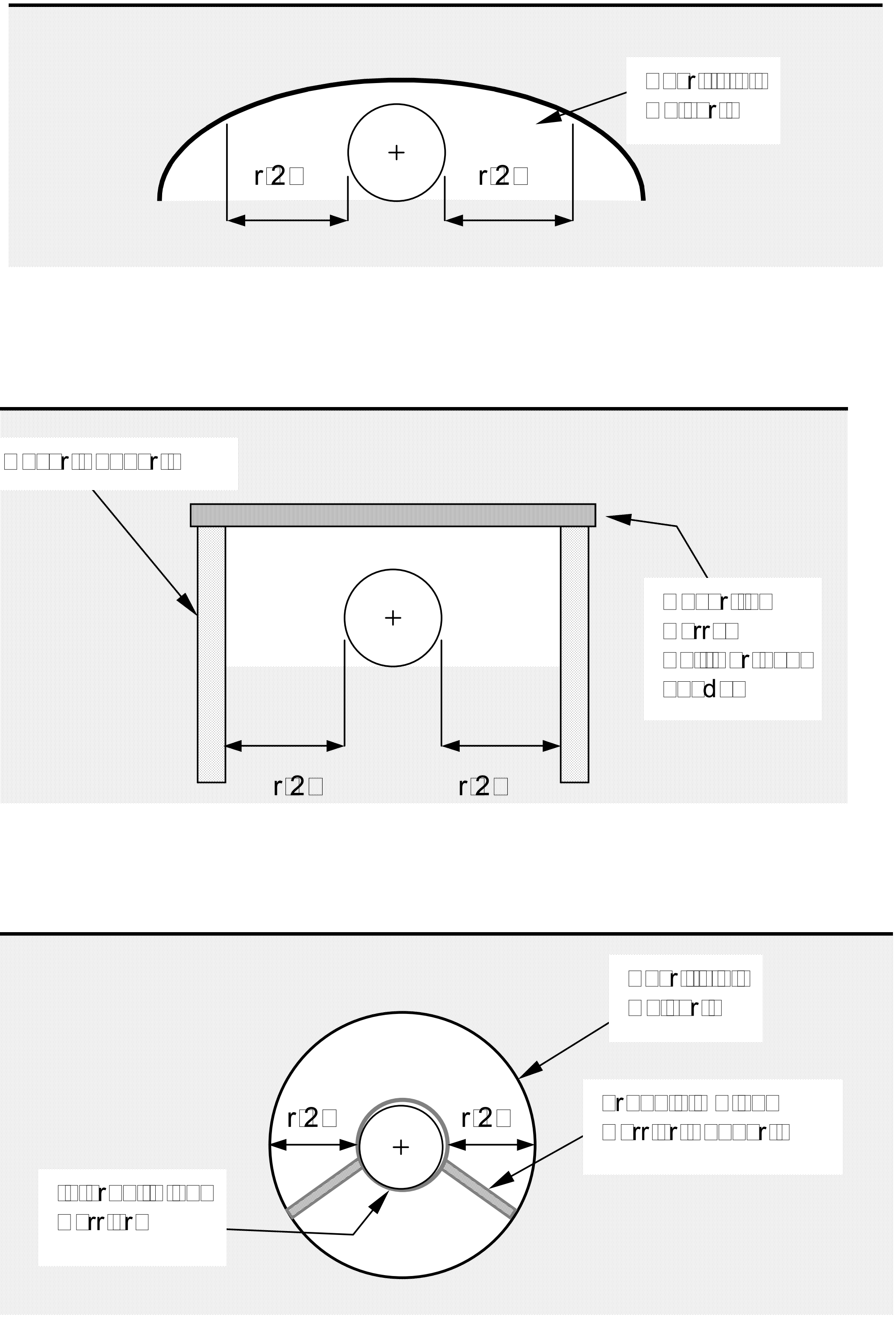


Figure 5.5 Culvert Concepts to Isolate Pipelines from Relative Ground Displacement
(Ground displacement reduced by “rattlespace”, r)

GUIDELINES FOR THE SEISMIC DESIGN AND ASSESSMENT OF NATURAL GAS AND LIQUID HYDROCARBON PIPELINES

PART II

COMMENTARY

Table of Contents

C1.0 GENERAL PROVISIONS

The presentation of these guidelines utilizes a format in which a concise set of general procedures is provided separate from more detailed discussion. This Commentary is meant to provide the necessary background and discussion in order to properly implement the recommendations contained in Part I: Guidelines and Recommended Procedures. This format was selected to provide easy reference for users once they have familiarized themselves with the entire guidelines document.

C1.1 Application

These guidelines should not be interpreted as a set of minimum requirements for a particular application. Instead, the goal is to provide consistency among the definition of seismic hazard, analytical methods, and acceptance criteria. Modifications to the level of analysis or the definition of seismic hazard will necessarily lead to a change in the acceptance criteria.

The acceptance criteria in these procedures are based upon consideration of steel pipe with butt-welded pipe joints. This has been the typical configuration for natural gas and liquid hydrocarbon transmission pipelines for the past 40 to 50 years. These guidelines do not cover piping configurations that include appurtenances such as valves, meters, filters, branch line connections, flange connections and other configurations with deformation limits considerably different from butt-welded steel pipe.

C1.2 Process

The procedures in these guidelines are consistent with the philosophy that the earthquake hazards are defined in a conservative fashion, and the subsequent pipeline assessments are based upon more liberal strain acceptance criteria. With this philosophy in mind, the performance goal for a pipeline defines the expected occurrence rate of the earthquake hazard.

Defining the target performance goal for assessing pipeline response to seismic hazards is a fundamental and critical task. It is incumbent on the pipeline owner to determine an appropriate level of seismic risk considering public safety, service reliability to customers, regulatory agency approval, potential property and environmental damage, and protection of capital investment. The definition of performance goal should be in terms of an acceptable median annual probability for experiencing an unacceptable level of pipeline performance (e.g., continued function or pressure integrity).

Identification and quantification of seismic hazards requires expertise in the fields of geology and seismology. In addition, quantification of the soil properties necessary for the evaluation of pipeline response requires expertise in the field of geotechnical engineering. It is assumed that sufficient information on the seismic hazard and soil conditions is available for use with these procedures. In most cases, estimates of soil strength parameters will require subsurface site investigations and laboratory tests of soil samples.

While the scope of these procedures is directly applicable to earthquake-generated hazards, the procedures can be extended to other non-seismic hazards that produce permanent ground movements along a pipeline alignment such as slope movement and ground settlement provided such hazards are defined in a similar fashion (i.e., the hazards are directly related to the target performance goal). These procedures may also be extended to other pipeline systems (e.g., chemical, water) provided adequate consideration is given to the operational characteristics that differ significantly from natural gas and liquid hydrocarbon pipelines. Such characteristics may include alternate pipe material and alternate pipe construction methods.

The approach in these guidelines differs from typical practices embodied in building codes that rely on partial safety factors to achieve a target level of reliability. The simplified approach in these guidelines is justified by the current inability to reasonably quantify the uncertainty in estimating various seismic hazards, soil loading parameters, and pipeline performance under specific strain levels. For most building codes, conservatism in design methods and specification of safety factors are specifically intended to result in a probability of not meeting a performance objective (e.g., building collapse) that is less than the probability of experiencing the design seismic ground motions. As an example, U.S. building codes are intended to achieve a mean annual probability of building collapse from earthquakes no greater than 4×10^{-4}. The building design is based upon earthquake ground motions with a mean annual probability of exceedance of approximately 1×10^{-3} to 2×10^{-3}. Thus, use of these guidelines with seismic hazard definitions based upon design values specified in building codes will not generally result in the same level of performance intended for buildings. Instead, to achieve a level of reliability that approximates that for buildings, these guidelines require that the annual probability of exceeding seismic hazards be the same as the probability of not meeting the building performance objective. Using the above example for the U.S. leads to defining the mean annual probability of exceedance for the seismic hazard as 4×10^{-4}.

In actual practice, it is more typical for performance goals to be defined by mean annual probabilities of 1×10^{-3} to 2×10^{-3} for experiencing loss of pressure integrity from a seismic event. Adopting lower performance goals compared to what are intended for new buildings is justified considering the relatively low potential for significant safety consequences from damage to natural gas and liquid hydrocarbon pipelines compared to large occupied buildings.

In some cases, pipeline operators may wish to achieve a very high level of reliability for especially critical pipelines. The approach in these guidelines is to introduce additional conservatism by considering less frequent and more severe earthquake hazards. Users are cautioned that this approach may not be appropriate when performance goals dictate that earthquake hazards with mean annual probabilities of exceedance are considerably less than 4×10^{-4}. Caution is required because of the fact that the majority of the available methods for estimating seismic ground displacements are empirical and based upon a limited set of historical observations. Because of this, extrapolation to extremely rare seismic events can be highly imprecise. If it is necessary to achieve extraordinarily high levels of reliability, the necessary conservatism needs to be achieved through a combination of a low likelihood of experiencing a seismic hazard and a high likelihood (higher than considered in these guidelines) of achieving specific pipeline capacity without compromising pipe performance (e.g., maintaining operability or pressure integrity). Establishing appropriate levels of conservatism in hazard definition and pipeline capacity will typically require a more formal formulation incorporating state-of-the-art methods that are beyond the scope of these guidelines.

C1.3 Alternate Procedures

The relationships for soil loads and pipeline limit states are based upon information that can be obtained readily through available construction records or standard geotechnical investigations. Field and laboratory tests of actual pipelines and soil conditions are generally much better representations of actual behavior. However, specific tests are not typically performed because of the relatively high costs and difficulty in extrapolating data from a few locations to represent conditions present over long sections of the pipeline alignment. For these reasons, the procedures have been formulated based upon what is believed to be readily available information.

Nonlinear, large-displacement finite element analysis, with the pipeline represented by beam-like elements with nonlinear stress-strain properties, is the recommended approach. In some cases, bounding analyses using simplified calculation procedures may be sufficient to assess pipeline response. It is also possible to perform more complex finite element analyses with the pipeline represented by shell-elements. Such refined analyses are not prohibited by these guidelines and have been shown to provide good correlation with pipeline behavior in controlled tests. However, it should be recognized that the methods described in these guidelines for determining soil restraint boundary conditions do not apply to finite element models represented by shell elements. Furthermore, it is doubtful that shell element soil restraint properties could be developed without extensive, full-scale test programs. Consequently, the use of shell element models for the pipeline should be restricted to investigations of localized response of the pipeline where non-linear deformations (e.g., wrinkling, buckle formation) are expected.

C1.4 Required Information

The information requirements listed are based upon data necessary to implement the procedures in these guidelines. The sources of this information may include mill tests, laboratory tests, geotechnical investigation, and seismic hazard assessment.

C2.0 QUANTIFYING SEISMIC HAZARDS

Seismic hazards affecting buried pipelines are divided into two categories: permanent ground deformations (surface faulting, lateral spread movement, subsidence, landslide) and dynamic strains from wave propagation.

The response of buried pipelines to inertial shaking effects (i.e., the pipeline vibrating within the soil) are not considered in these guidelines, as past experience and prior studies show that these do not have a significant effect on pipeline stress or strain (Nelson and Baron, 1981; Parnes and Weidlinger, 1979).

C2.1 Surface Faulting

Faulting is the deformation associated with the relative displacement of adjacent parts of the earth's crust. Fault displacements can occur rapidly during an earthquake (duration on the order of one to several seconds). In addition, relatively minor displacements may accumulate gradually over many years as aseismic creep.

Faulting that results in surface rupture is an important consideration for buried onshore pipelines, because pipelines crossing fault zones must deform longitudinally and in flexure to accommodate ground surface offsets. For offshore pipelines that are simply laid on top of the seafloor, fault movements are generally of little, if any, consequence. Due to the unrestrained pipeline configuration, strains induced by fault movement will be only a small fraction of those that would be developed in a buried pipeline with similar dimensional characteristics and properties. It is possible that offshore faulting could produce a vertical offset that would cause a segment of the pipeline to be elevated above the seafloor for a short distance, and, hence, cause concern for vortex-induced oscillations from water currents. It is not practical to attempt to design for such a remote possibility in advance, particularly considering that fatigue damage to the pipeline would not occur for some time, probably years. For offshore pipelines that are intentionally buried (typical for pipelines in less than 100 feet of water) or are expected to become buried as a result of sea bottom sediment transport processes, the treatment of offshore pipelines mirrors that for onshore pipelines.

Faults are classified on the basis of their direction of movement with respect to the ground surface. The various types or classifications of fault movement are illustrated schematically in Figure C2.1. A strike-slip fault is one in which the predominant component of ground movement is horizontal displacement. If the movement of one side of the fault when viewed from the other side is to the right, the fault is called a right-lateral strike-slip fault. When the movement is to the left, the fault is called a left-lateral strike-slip fault. Normal-slip and reverse-slip faults are those in which the overlying side moves downward and upward, respectively, in relation to the underlying side of the fault. In many cases, faults exhibit a combination of strike-slip and normal or reverse-slip movements.

The effect of right-lateral strike-slip movement on a buried pipeline is illustrated in Figure C2.2. For some faults, the maximum credible surface movements can be as large as 10 meters for strike-slip faults and 5 meters for normal-slip faults. In some cases, ground movements for a particular fault may occur over a number of closely-spaced parallel fault traces or splays within a fault zone, and consideration must be given to the total as well as the individual movements within the zone.

The evaluation or design of a pipeline crossing a fault generally requires the following information:

1. Location of the pipeline-fault intersection and error bounds on location (e.g., ±50 meters).

2. Expected allocation of fault slip among multiple fault splays, if applicable.

3. Angle of intersection of the fault with the pipeline alignment at the point of crossing.

4. Type of slip expected (e.g., strike-slip, normal-slip or reverse-slip).

5. Width of the zone of disturbed ground. For strike-slip faults, the zone of ground distortion will be on the order of 2 to 5 meters. For normal or reverse faults, the principal zone of surface disturbance is typically on the order of 5 to 15 meters. For reverse faults, secondary surface cracking and heaving may occur over a distance of as much as 200 meters on the hanging wall block of the fault.

6. Fault dip angle.

7. Design fault displacement (defined as orthogonal components of displacement in three-dimensional space).

Generally, a fault is considered active if it can be demonstrated to have displaced the surface of the ground during the Holocene Epoch, i.e., within the past 11,000 years. This, of course, assumes that faults with evidence of such historic movement will be the ones that move again. This has not always been the case. The 1999 Hector Mine earthquake in southern California occurred on a fault that was shown on fault maps to be “inactive,” albeit some geologists believe that the fault rupture was closely tied to an adjacent active fault system.

In some jurisdictions, fault activity may be defined by regulatory bodies. In California, the Alquist-Priolo Special Studies Zones Act of 1972 provides special regulations regarding land uses near faults that are “well defined” and “sufficiently active.” In this context, well-defined implies that the fault can be detected by a trained geologist as a physical feature, at or just below the ground surface. “Sufficiently active” is defined as having evidence of Holocene surface displacement along one or more segments.

Unfortunately, there are no reliable means for predicting the probability of future fault displacements. The basic mechanisms that govern such phenomena are measured in terms of thousands of years, i.e., orders of magnitude greater than the design life of the pipeline. Consequently, for the evaluation or design of pipeline crossings of active surface faults, it is necessary to utilize judgment based upon experience and consideration of the best available data.

The preferred method for estimating fault displacement is to rely on historical data on fault displacement or the findings from site-specific studies that may include examination of topographic features, observations of geomorphic features, or trenching across the fault to identify historical evidence of past fault displacement. For most pipelines, this preferred approach is only practical where site-specific studies have previously been performed for the purposes of refining regional seismic hazard estimates. In lieu of site-specific information, estimates of fault displacement often rely on empirical methods tempered by the judgments of individuals experienced in their development and limitations.

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A number of investigators have compiled statistics regarding historic fault displacements as a function of earthquake magnitude, fault rupture length, and other related parameters. The most recent compilation was done by Wells and Coppersmith (1994) using a worldwide data base of source parameters for 421 historical earthquakes. In their study, Wells and Coppersmith developed a series of empirical relationships among moment magnitude, surface rupture length, subsurface rupture length, down-dip rupture width, rupture area, and maximum and average displacement per event.

The data sets for the two regression equations given in these guidelines include displacement data for all types of slip, i.e., strike-slip, normal-slip and reverse-slip faulting. Regression equations that are unique for particular slip types are also available in Wells and Coppersmith, but the correlation is unsatisfactory for reverse faults, probably due to the difficulty in making reliable field observation of reverse-slip movements. Wells and Coppersmith (1994) recommended using the regressions for an aggregate of "all" types of slip.

The surface rupture length is not necessarily the entire length of the fault trace visible on the surface but, rather, it is the maximum length of the fault segment crossing the pipeline that, by reasonable and qualified judgment, can be expected to rupture during an earthquake. Such judgments should be made by geologists with knowledge of regional seismotectonics and with reference to site-specific information and studies. As an upperbound estimate of fault displacement, it may be appropriate to assume that the fault can rupture along its entire length. However, this could lead to unnecessarily conservative design displacements.

In regions where probabilistic or deterministic ground shaking hazard maps have been developed, information on fault length may be readily available (e.g., consider the fault map data provided for use with the 2000 International Building Code in the U.S.) Other information sources include the U.S. Geological Survey (U.S.), Canadian Geological Survey, Pacific Geological Survey (British Columbia, Canada), and the California

Geologic Survey. In the absence of published information, it may be necessary to conduct geologic field studies in order to estimate fault length. Surface expression of fault movement can be considered unlikely for earthquakes with moment magnitudes less than 6.0.

The potential variability in the empirical expressions for fault displacement can be illustrated by a sample calculation. Using equations (2-1a) and (2-1b) for a surface rupture length of 75 km yields mean maximum and mean average fault displacements of 3.4 m and 1.7 m, respectively. Using equations (2-2a) and (2-2b) with a moment magnitude of 7.5, yields mean maximum and mean average fault displacements of 4.9 and 2.4 m, respectively. For the case of a fault with a 75 km surface rupture, the maximum and average fault displacements within one standard deviation of the mean are 1.3 m to 8.7 m and 1.8 m to 7.6 m, respectively. The range is somewhat larger for fault displacements estimated based upon earthquake magnitude. Thus the empirical formulae produce a range of fault displacements that vary by a factor of about 4 to 7 in order to capture the actual fault displacement slightly better than half of the time. For this reason, projects that pose an extreme threat to safety or the environment (e.g., large LNG facilities, nuclear power plants) typically require site-specific investigations to better define fault displacement. Site-specific investigations may include trenching, soil borings, and paleoseismic studies to establish the approximate dates of prior fault displacement and the amount of displacement that has occurred in past earthquakes. For most pipeline projects, however, this level of effort is not justified by the potential consequences of earthquake damage.

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For the design of a pipeline crossing a fault, the relative displacement of the soil block on one side of the fault must be known in a three-dimensional sense relative to the soil block on the other side. The user should be aware that it is not uncommon for geologists to define fault displacement in terms of the resultant two-dimensional displacement vector in a vertical plane parallel to the strike of the fault. In other words, the fault displacement is the *resultant* of the observed vertical offset of the ground surface and horizontal displacement parallel to the fault.

If the fault is known to be predominantly strike-slip, it usually is sufficient to assume that the resultant displacement will be parallel to the fault strike. However, in the case of reverse faults, the nature of the fault displacement may be highly variable due to an irregular (nonlinear) surface trace such that the displacement at the pipeline crossing may be a combination of horizontal, vertical and compressional-extensional displacement components relative to the strike of the fault.

Fault displacement estimated using the Wells and Coppersmith regression equations is the projection of a resultant fault displacement on a vertical plane that intersects the fault strike at the surface. The database for the regression equations does not include the vertical and horizontal components of displacement in the vertical plane that intersects the strike, nor does it include the transverse component of reverse-slip or normal-slip components of displacement perpendicular to the fault strike.

Empirical approaches, other than those developed by Wells and Coppersmith, are available for estimating fault displacement. These guidelines do not prevent the use of these approaches but encourage the use of alternate relationships that may be more appropriate for specific regions or fault systems.

The recommended approach to determine three-dimensional fault displacement components from fault displacement defined in a manner similar to the Wells and Coppersmith regression equations requires some estimate of the fault geometry and regional seismic stress regime as defined by (1) the angle of intersection between regional stress azimuth and fault strike, β, and (2) the fault dip angle, δ.

The displacement of a hypothetical fault displacement is shown in Figure C2.3. The net displacement in a vertical plane parallel to the fault strike is the resultant of two of the three orthogonal components of fault displacement, i.e., the horizontal component of slip, H_F, parallel to and the vertical component, V_F, perpendicular to the strike of the fault. The individual horizontal and vertical displacement components, H_F and V_F, as well as the transverse component of fault displacement perpendicular to the fault strike, T_F, are unknown.

If it is assumed that the total fault displacement, D_{FS}, is in the direction of the regional stress azimuth, the total fault displacement and its constituent orthogonal displacement components can be determined from the geometry of right triangles. The resulting relations for the three orthogonal components are given in equations (2-3b), (2-3c) and (2-3d) in these guidelines.

The values for the fault dip angle provided with equation (2-3) are approximate values that are believed to be generally representative of the database used in the development of the Wells and Coppersmith empirical relationships. It is possible that empirical relationships developed for specific seismic regions are available. In this case, the generic fault dip angles provided with equation (2-3) may not be appropriate. In general, the determination of fault movement components is best left to an experienced geologist having knowledge of local plate tectonics and the regional stress regime in the earth's crust.

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Recommended design fault displacements are based upon the potential consequences associated with pipeline damage and defined in terms of the expected maximum or average fault displacement consistent with the probability of exceeding pipeline performance criteria established by the pipeline owner. The process for determining design fault displacement consists of three basic steps:

1. Determine the performance requirements for the pipeline in terms of an average annual probability of exceeding strains associated with maintaining normal operability or pressure integrity;

2. Determine the average and maximum fault displacement consistent with the probability established in Step 1; and

3. Determine design fault displacement values by modifying the fault displacements in Step 2 based upon the consequences associated with the pipeline not meeting its performance objectives, the average probability of the occurrence of fault displacement, and the acceptable annual probability for the pipeline exceeding its performance objectives.

Pipeline performance requirements are established by the pipeline owner and will typically vary depending upon the factors discussed in Section C1.2.

The maximum fault displacement that occurs during a seismic event typically occurs along a short length of the fault rupture, with lesser amounts of movement along most of the fault rupture length. For faults with no information from historical rupture patterns or detailed fault studies, estimating the distribution of maximum displacements along the fault rupture length is extremely uncertain. Since that maximum is generally considerably greater than the average displacement (about double), it is reasonable to use a value smaller than the maximum credible displacement when developing design values for engineering design at active fault crossings.

Determination of fault displacements consistent with the performance goals of the pipeline requires considerable judgment, particularly for faults where the episodic nature of earthquake activity has not been investigated. Reduction of fault displacement estimates for design are warranted when the average probability of surface rupture is less than the performance goals of the pipeline. The approach adopted in these guidelines is to simply factor the fault displacement estimates by the ratio of the acceptable probability for the pipeline not meeting its performance goal by twice the average probability associated with the fault displacement estimate. The factor of 2 on the average probability of fault displacement can be reduced if there is good information on fault occurrence based upon historical records or fault trenching investigations.

The approach for estimating design fault displacement in these guidelines implicitly accepts the risk associated with uncertainties in understanding fault behavior. Some of these uncertainties include the following:

1. Actual displacement on a fault may be considerably greater than that produced by earthquakes of similar magnitude.

2. The likely fault displacement, based upon estimates of fault slip rate and expected earthquake magnitude (the most common approach), is not a reliable means to assess the difference in the probability of fault displacement in the near future (i.e., 50-year to 100-year time frame).

C2.2 Peak Ground Acceleration

Estimates of peak ground acceleration (PGA) are necessary in order to estimate potential seismic hazards related to liquefaction, lateral spread movement, and slope stability. Peak ground accelerations are estimated using empirical attenuation equations that relate the dissipation of earthquake energy with distance from the source of the earthquake. Attenuation relationships utilize data recorded from strong motion accelerographs. The acceleration time history data are typically segregated according to the local site conditions on which the accelerograph is founded, such as hard rock, soft rock, firm soil, and soft soil. In addition, many recent attenuation relationships segregate the data by the type of faulting (e.g., strike-slip, normal-slip, reverse-slip). Attenuation relationships also have been developed to represent spectral accelerations at various frequencies. However, only PGA is of importance for use in these guidelines.

All attenuation relationships require as input the earthquake magnitude and a measure of distance from the earthquake source. The selection of a particular measure of magnitude (e.g., moment magnitude, surface wave magnitude, and local magnitude) and distance (e.g., epicentral, hypocentral, and shortest distance to fault plane) is largely a matter of personal preference on the part of the individual developing the relationship. Figure C2.4 provides a definition of earthquake source parameters often incorporated into attenuation relationships.

There is typically a great deal of scatter in the recorded ground motion data for a particular distance from the earthquake source. Directivity associated with the orientation of fault rupture is believed to be a significant factor in the observed scatter. Another factor in the amount of observed scatter is local interaction of seismic waves as they propagate to a single site from multiple locations along the fault rupture surface. This scatter introduces uncertainty in the attenuation relationships that is represented by the random-error term E. The random-error term is typically a function of earthquake magnitude.

Setting E to zero results in a best estimate of the fit of the attenuation equation to the data. The random-error term is used to estimate the probability that accelerations might be different than that estimated from the best-estimate equation. In application, the random-error term is multiplied by a factor representing the number of standard deviations from the mean producing the desired standard normal probability value. Thus, using 1(E) produces an estimate of PGA that envelops 84% of the data, 1.65(E) produces an estimate of PGA that envelopes 95% of the data, and 2(E) produces an estimate of PGA that envelops 98% of the data.

Some measure of the precision with which attenuation relationships estimate PGA can be gained by examining the factors representing ±25% variation from the mean. This corresponds to the factors on the mean that would provide a range of accelerations that have a 50% chance of occurring. A mean plus 25% estimate corresponds to an estimate that envelops 75% of the data or is 0.68 standard deviations from the mean. A mean minus 25% estimate corresponds to an estimate that envelops 25% of the data or is –0.68

standard deviations from the mean. For the two attenuation relationships provided, the random-error terms for a magnitude 7 earthquake are 0.41 for the near-source earthquake equation and 0.75 for the Cascadia earthquake. The corresponding factors on the mean estimate to give a 50% chance of experiencing the estimated PGA are computed as follows:

Near-Source: Factor = $e^{0.68(0.41)} = 1.32$

Cascadia: Factor = $e^{0.68(0.75)} = 1.67$

Assuming the PGA estimate was 0.5 *g*, the range of PGA values having a 50% chance of occurrence is 0.38 *g* to 0.66 *g* for the near-source earthquake and 0.30 *g* to 0.84 *g* for the Cascadia earthquake. This example illustrates the wide variability typically encountered in estimating seismic hazards. It is also clear that stating estimates of PGA within increments of 0.05 *g* or greater is generally sufficient.

Since attenuation relationships are developed from recorded ground motion data, users should attempt to identify attenuation relationships developed for the specific region of interest. If such relationships cannot be identified, the selected attenuation relationship should be based upon records from regions that are tectonically and geologically similar. Often, multiple attenuation relationships may be identified as appropriate for a particular location. Where possible, it is recommended that peak ground acceleration be estimated as the average of at least three representative attenuation relationships.

A good source of information for attenuation relationships applicable to the United States is Seismological Research Letters (SSA, 1997). This reference provides a collection of recently developed attenuation relationships and is a starting point for identifying attenuation relationships for locations other than the west coast of the United States. For other parts of the world, it is generally necessary to identify applicable attenuation relationships from past earthquake investigations, seismic hazard analyses, or accepted practice in past estimates of seismic ground motion hazards for the region.

C2.3 Liquefaction

Liquefaction is the transformation of a saturated cohesionless soil from a solid to a more liquid state as a result of increased pore-water pressure and concomitant loss of shear strength. Loss of shear strength gives rise to bearing failures and large deformations in surface structures founded on liquefied soil. Liquefaction often leads to the formation of sand boils, mud volcanoes, fissures and other channels through which water and sediments are ejected. These ejections cause volume loss resulting in differential settlement even though no significant lateral movement occurs.

Liquefaction is a phenomenon that is generally limited to saturated sandy soils that have been deposited in the Holocene (the past 11,000 years) and most commonly to fluvial sediments deposited in the past 1,500 years. These characteristics typically limit consideration of liquefaction to locations where the water table is less than 15 m below the ground surface. Liquefaction is most likely to occur within a kilometer of bodies of water (e.g., shorelines, streams, drainage channels) and within deltaic deposits where the water table is typically shallow.

Existing maps and databases pertaining to surficial geology should be used to identify areas with soils of depositional characteristics and geologic age that are known to have liquefied in previous earthquakes. The procedures proposed by Youd and Perkins (1978) should be followed. Table C2.1 (Youd and Perkins, 1978) summarizes the liquefaction susceptibility of various deposits according to geologic age. For most pipeline projects, it is only necessary to consider sedimentary deposits with high or very high liquefaction susceptibility.

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Collecting information necessary to assess the potential hazards from liquefaction requires site-specific subsurface data. Unless adequate subsurface data are available from past projects at the site of interest, site investigations consisting of subsurface sampling, standard penetration tests, cone penetrometer tests, and laboratory tests are required.

Assessing liquefaction potential is based upon both peak ground acceleration and earthquake magnitude. Estimating peak ground acceleration at the site of interest can be performed using probabilistic or deterministic approaches. The deterministic approach is based upon selecting a magnitude for a particular fault and using attenuation relationships that relate peak ground acceleration to magnitude, distance, local soil conditions, and type of earthquake faulting. The probabilistic approach is similar to the deterministic approach only in the sense that attenuation relationships are used to relate peak ground acceleration to earthquake magnitude, distance, and type of earthquake faulting.

In a probabilistic approach, the effects of numerous earthquake sources, each with different rates of occurrence for earthquakes of various magnitudes, are included. Additional information on probabilistic seismic hazard estimation is provided in Appendix C. The results of a probabilistic approach for estimating PGA are typically represented in maps of PGA contours for different annual probabilities of exceedance (Figure C2.5) or site-specific curves representing the variation of PGA with annual probability of exceedance (Figure C2.6). For liquefaction assessment, it is necessary to examine the contribution of earthquake magnitudes and distances to the ground shaking hazard. This is termed deaggregation of the hazard. The deaggregation of the hazard is often represented by plots of magnitude and distance contributions such as those shown in Figure C2.7. There are no established rules for selecting an appropriate magnitude and distance from a deaggregation of the probabilistic hazard analysis. Three common approaches are to select either the mean contribution, estimates enveloping mean plus one standard deviation (84%) of the contributions, or a magnitude and distance combination that has the largest contribution to the probabilistic hazard. These approaches are illustrated in Figure C2.7.

Assessment of potential liquefaction hazards requires site-specific information on the density and grain-size distribution of subsurface soil layers. As a general consideration, site-specific investigations should be performed at the types of areas listed in Table C2.1 as having moderate to very high risk of liquefaction where the peak ground acceleration is expected to be greater than about 0.15 g.

The procedures presented for assessing liquefaction are based upon the findings of a 1997 workshop (Youd and Idriss, 1997). The proceedings of this workshop provide an excellent summary of the state-of-practice in assessing liquefaction hazards. Two changes in the manner in which liquefaction is assessed resulting from the workshop deserve mention. First, the workshop participants agreed to modify the hand drawn curve developed by Seed and Idriss (1982) to reflect a minimum *CRR* value of 0.05. The modified curve for which the equations in these guidelines are developed is shown in Figure C2.8. Second, the workshop participants agreed that the magnitude scaling factors originally developed by Seed (Seed and Idriss, 1982) were far too conservative for earthquakes with magnitudes less than 7. Although specific magnitude scaling factors were not recommended by the workshop participants, the recommended range of magnitude scaling factors represent an increase of 50% to 100% in the *CRR*.

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Vertical pipeline movement resulting from buoyancy forces has not been a significant hazard to buried onshore pipelines in past earthquakes. The combination of limited displacement potential and the length of pipeline typically exposed to liquefaction generally prevents the development of significant pipeline strains.

Onshore pipelines located above the water table are not subjected to relative buoyancy forces. Pipelines located beneath the water table in liquefaction-susceptible soils can experience vertically upward or downward displacements depending upon the weight of the pipeline with coating and contents, the weight of liquefied soil displaced by the pipeline, and the residual shear strength of the liquefied soil. The time over which the soils remain in a liquefied state is a key variable to determining the amount of vertical movement. No studies to quantify the duration of liquefaction and its effect on movements caused by differential buoyancy could be identified. As a first approximation, the duration of liquefaction can be assumed to be equal to the duration of ground motion sufficient to cause liquefaction, another poorly defined parameter. The procedure in these guidelines is limited to a means to identify the maximum potential movement of the pipeline.

Practically, relatively extreme conditions of buoyancy mismatch appear to be necessary to produce relative pipe movement. This is illustrated by the following example.

Consider an NPS 24 pipe with a wall thickness of 9.5 mm (0.375 inches). The weight of the pipe is 1.42 kN/m (97 lb/ft) and the effective density of the pipe is 4.9 kN/m^3 (31 lb/ft^3). If the total unit weight of liquefied soil is 17.4 kN/m^3 (110 lb/ft^3), the net buoyancy force on the pipe is 3.62 kN/m (248 lb/ft) in the upward direction. It is assumed that the depth to the centerline of the pipe is 2 m (6.56 ft), the water table is at the ground surface, and the residual shear strength of the liquefied soil is 5% of the effective stress (typical range is 1% to 10%). These assumptions lead to a residual shear strength of only 0.976 kPa (20.4 psf) and a soil resistance to vertical movement of 12.9 kN/m (884 lb/ft) based upon the approaches in Section 4.3.3. Since the soil resistance is about 3.6 times the upward buoyancy force, no movement would be expected.

Given this example, it is expected that vertical movements from pipeline buoyancy will only be a significant hazard for large diameter pipelines within soils expected to exhibit unusually low residual shear strength in a liquefied state. Calculations, similar to those in the above example, can be used to gauge the potential for significant pipeline vertical movement from buoyancy forces.

Evaluating displacements from relative buoyant forces for underwater pipelines can be treated in the same fashion as onshore pipelines. However, buoyancy is typically not an issue for offshore pipelines or inland water crossings as they are required to be negatively buoyant to remain stable on the sea floor. Achieving negative buoyancy is typically done by coating the pipeline with concrete or using discrete weights attached to the pipeline. If buoyancy is an issue, additional conservatism may be warranted to account for the fact that upward movement sufficient to uncover the pipeline will expose it to potential hazards related to flow-induced oscillation and impact with debris carried by water currents (e.g., exposed river crossings).

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Although a methodology is presented for estimating the amount of settlement related to liquefaction, assessing pipeline performance also requires an estimate of the extent of liquefaction along the pipeline alignment. Estimating the extent of soil layers susceptible to liquefaction can best be determined from subsurface profiles produced from boring log data. Borings may be drilled at regular intervals along the pipeline alignment to identify the limits of liquefaction. In some locations, detailed maps may be available describing surficial soil deposits and their depositional environments. These may be used for preliminary screening to identify locations more prone to liquefaction.

It is possible that potentially liquefiable deposits extend for several hundred meters along the pipeline. As it is unlikely, based upon historical observations, that the pipeline will experience uniform liquefaction over large distances, some alternate approach is needed to define the extent of a pipeline subjected to relative settlement. This requires the judgment of individuals familiar with the local geology and performing liquefaction assessments.

The graphical methods presented are based upon both observational data (Figure 2.2 from Ishihara, 1985) and laboratory tests (Figure 2.3 from Ishihara, 1990). Both graphs are based upon sands with a fines content less than 5% ("clean" sands). For engineering purposes, this factor can often be ignored as the determination of what constitutes a liquefiable layer and the factor of safety against liquefaction incorporates the presence of fines content. Applying the charts for other soil types generally overestimates settlement since the presence of fines would tend to fill some voids and reduce the amount of soil grain reorganization responsible for liquefaction-induced settlement.

C2.4 Lateral Spread Movement

Lateral spreads involve the horizontal movement of competent surficial soils due to liquefaction of an underlying deposit (see Figure C2.9). Lateral spreads can be especially destructive to buried pipelines, albeit the degree of damage depends on the magnitude and extent of ground movement and the configuration of the pipeline. Offshore, liquefaction requires special treatment as the amount and extent of ground movements associated with liquefaction can be much greater than for onshore installations.

Lateral spread movement is the most common and one of the most severe earthquake hazards for buried pipelines. Techniques for estimating the magnitude of lateral spread movement have only been developed in the past 15 years.

The simplified approach developed by Youd et al. (1999) based upon multiple linear regression of data from past earthquake observations represents the best available method for estimating lateral spread movement outside of explicit finite element techniques. This approach is a modification of a similar method developed by Bartlett and Youd (1992). The 1999 modifications included data from the 1995 Kobe, Japan earthquake and corrected some of the data records in the 1992 investigation. The expected variability in the simplified approach can be assessed by comparing the predicted lateral spread displacements with the observed displacements. This is provided in Figure C2.10 where it can be seen that nearly all data points are within a factor of two of the predicted displacement.

One weakness of the Youd et al. (1999) approach is its inability to provide information on the extent of lateral spread movement and the variation of ground movement with depth and within the body of the lateral spread. Because of the importance of the spatial variation in ground displacement along the pipeline, the recommended approach for estimating lateral spread movements is to employ an analysis program based upon a finite-element, finite-displacement, or discrete-element representation of the soil. Some examples of programs that have been used in past pipeline projects include DESRA-2 (Lee and Finn, 1978), TENSI-M (Larkin and Marsh, 1982), and SOILSTRESS. Many geotechnical engineering firms also maintain proprietary in-house analysis programs. A comparison between the results of the Bartlett and Youd appoach and the results from a SOILSTRESS analysis are shown in Figure C2.11 for a hypothetical river crossing. In the analyses represented in Figure C2.11, the Bartlett and Youd approach was used to benchmark the maximum surface displacements estimated by the SOILSTRESS analysis. With this benchmark established, the SOILSTRESS results are then used to determine the variation with respect to depth below the ground surface for relative ground movements normal to the river.

C2.5 Landslides

Landslide activity most commonly triggered by earthquake shaking includes rock falls, disrupted soil slides, rock slides, soil slumps, soil block slides, and soil avalanches (Varnes, 1978). Several common types of landslides are illustrated in Figure C2.12. Except for rock avalanches, locations susceptible to earthquake triggered landslides are most often associated with areas of past landslide activity. The most significant landslide hazards that can affect buried pipelines are slumps, shallow slides, and deep slides. Slides can occur onshore on relatively steep slopes or offshore in soft sediments on relatively shallow slopes. Landslides may develop into debris flows and impart large dynamic loads on the pipeline. For offshore pipelines, turbidity currents created by the slide may also lead to significant loads on the pipeline.

Slumps and shallow slides are caused primarily by inertial forces, but are often assisted by densification of loose soil or liquefaction of underlying sediments. These movements occur mostly along the margins of embankments, cut-and-fill slopes, and slopes with relatively shallow cover in hilly or mountainous terrain.

Deep slides involving significant components of translation and rotation of a soil mass often develop catastrophically and affect large areas. A landslide frequently causes underthrusting in soils near the base of its slope, so that substantial compression and bending may be transferred to pipelines located there.

Areas of past landslide activity can be identified through reconnaissance of the pipeline alignment and stereo aerial photographs or sidescan sonar for offshore pipelines. Earthquake-triggered landslides typically occur during the earthquake. If the earthquake forces are insufficient to completely develop a failure plane, earthquake-related landslides may occur several days following the earthquake due to reduced resistance to slide movement.

Reconnaissance of a pipeline alignment can be used to identify and estimate the relative activity of existing slides. The findings from the site reconnaissance can be used to modify new pipeline alignments to avoid many potential landslide hazards. Determining the amount of expected deformation requires a site-specific determination of the existing soil strength properties and an assessment of the factor of safety under seismic loading conditions.

In lieu of a detailed assessment of landslide potential, an approximate estimate of potential landslide movement can be made based upon the existing slope and a general description of the near-surface material. This approximate approach is implemented in these guidelines and is to be used as a screening tool to determine the potential for slide movement.

The approach to estimating the potential severity of landslides is based upon bounding relationships for sliding-block displacement developed by Newmark (1965) and estimated levels of acceleration associated with slope instability developed by Wilson and Keefer (1985). The approach to estimating landslide movement developed by

Wilson and Keefer utilizes two relationships based upon Arias intensity, one for the acceleration levels associated with 2 cm or 10 cm of landslide movement and the other for the variation of acceleration as a function of earthquake magnitude.

Wilson and Keefer adopted the use of Arias intensity as a means to avoid explicit calculation of movement using multiple time histories and the methodology developed by Newmark. The Wilson and Keefer approach is not considered generally applicable because of the lack of validation of the Arias intensity relationships for a large number of historical earthquakes.

To provide a general indication of the potential magnitude of landslide movements, a bounding empirical relationship developed by Newmark is used. The analytical model used by Newmark is quite simple. A block resting on a sloping surface is prevented from sliding by the friction force between the block and the surface. Motion of the block is produced when acceleration of the block in the downslope direction exceeds the frictional resistance reduced by the component of block weight in the downslope direction. The factor of safety against block sliding is equal to the resistance to sliding divided by the acceleration acting on the block. The sliding resistance in the downslope direction is the frictional resistance minus the component of block weight in the downslope direction. The upslope movement of the block can occur if the acceleration in the upslope direction exceeds the sum of the friction resistance and the component of block weight in the downslope direction. For the simplified approach used in these guidelines, the Newmark model is identical to the model assumed by Wilson and Keefer. In fact, Wilson and Keefer used explicit analyses using Newmark's approach to develop their approach.

The duration of motion is limited to the time period over which the acceleration exceeds the friction force. Newmark analyzed the movement of a hypothetical block using 5 earthquake time histories normalized to a peak ground acceleration of 0.5 *g*. These analyses did not consider vertical motions and assumed the horizontal earthquake acceleration acted in a direction parallel to the sliding surface. He then examined the fit of several possible analytical expressions to the data, as illustrated in Figure C2.13, and developed several bounding relationships based upon ground velocity, peak ground acceleration and the factor of safety. The equations for estimating landslide displacement in equation (2-18) is based upon an assumption of peak ground velocity equal to 36 and 48 inches per second per *g* (91 and 122 cm per second per *g*) for rock and soil sites, respectively. The bounding displacement equation used to derive equation (2-18) is given by equation (C2-1).

$$D = \frac{V^2}{2gN}\left(1 - \frac{N}{PGA}\right)\frac{PGA}{N} \tag{C2-1}$$

where:

- D = computed displacement
- V = peak ground velocity
- PGA = peak ground acceleration

N = resistance coefficient = A_c

g = acceleration of gravity

Rearranging the terms and substituting A_c for N and V as $x(PGA)$ gives:

$$D = \frac{x^2 PGA^2}{2gA_c}\left(\frac{PGA}{A_c} - 1\right) \qquad \text{(C2-2)}$$

Equation C2-2 depends on having a velocity ratio, x, the ratio of the peak ground velocity to PGA. Several investigators have proposed such relationships and representative values are provided in Table C2.2. Equations (2-15) and (2-16) result from substituting values of 36 inches per second per g (0.91 meters per second per g) for rock and 48 inches per second per g (1.22 meters per second per g) for the velocity ratio, x.

The displacements necessary to initiate coherent and disruptive slides (10 cm and 2 cm, respectively) are adopted from Wilson and Keefer. If the displacements computed using the relationships in these guidelines exceed these threshold values, the potential for landslides exist, and additional geotechnical investigation is warranted. Such investigations typically involve borings to identify different soil deposits, sample retrieval for ascertaining soil strength properties, and slope stability calculations to estimate the amount of landslide movement.

A pseudostatic analysis of slope stability is often used in the assessment of slope movements. The pseudostatic analysis should account for the strength properties of the slope material expected during the earthquake. Pseudostatic analyses represent the effects of an earthquake by applying static horizontal and/or vertical accelerations to a potentially unstable mass of soil. The inertial forces induced by these pseudostatic accelerations increase the driving forces and may decrease the resisting forces acting on the soil. Pseudostatic analyses are not appropriate for soils that build up large pore pressures or show more than about 15% degradation of strength due to earthquake shaking. Stability is expressed in terms of a pseudostatic factor of safety calculated by limit equilibrium procedures. Selection of an appropriate pseudostatic acceleration requires great care; values considerably smaller than the peak acceleration of the sliding mass are usually used.

If conditions exist where such strength loss may occur, it is generally necessary to perform a local site response analysis in order to obtain an estimate of the shear strains produced and the appropriate soil strength for slope stability analysis. Inertial instabilities are most commonly analyzed by pseudostatic, sliding block (Newmark, 1965), or stress-deformation analyses. The Makdisi-Seed approach, based upon the results of sliding-block analyses, is also used frequently.

The pseudostatic acceleration required to bring a slope to the point of incipient failure is known as the yield acceleration. If earthquake-induced accelerations in a slope momentarily exceed the yield acceleration, the unstable soil will momentarily accelerate relative to the material beneath it. Sliding-block analyses can be used to calculate the amount of displacement that occurs. The total displacement depends on the amount by which the yield acceleration is exceeded (a function of the ground motion amplitude), the time over which the yield acceleration is exceeded (a function of the frequency content of the ground motion), and the number of times the yield acceleration is exceeded (a function of ground motion duration). Given the highly variable nature of ground motion characteristics, computed displacements can be quite variable. The Makdisi-Seed procedure is based upon sliding-block analyses of earth dams and embankments. Knowing the fundamental period of vibration of the dam/embankment and the yield acceleration of the slope, simple charts can be used to estimate earthquake-induced permanent displacements.

It is important to note that the factor of safety based upon equivalent static slope stability assessment can be substantially less than 1.0 before significant sliding displacement occurs. Thus, concluding that large slope movements are highly probable because static slope analyses indicate that the slope cannot withstand the PGA often greatly overestimates the likelihood that a significant landslide hazard exists.

Several commercial software packages are available that greatly simplify the application of sliding-block analysis. Stress-deformation analyses have been used to estimate permanent deformations caused by inertial instabilities. Strain potential and stiffness reduction approaches allow estimation of permanent deformations from relatively simple analyses; their estimates are highly approximate. Although the computational effort is dramatically increased, permanent deformations can be analyzed more rigorously using nonlinear finite-element techniques. As the accuracy of constitutive models for soils improve, the use of nonlinear finite-element analyses is likely to increase. Quantification of slope stability under static and dynamic conditions with analytic techniques for estimating potential slope movements requires special expertise and should be carried out by individuals experienced with the local geologic conditions.

A slope stability analysis is only one part of a comprehensive evaluation of slope stability. Prior to the analysis, detailed information on geologic, hydrologic, topographic, geometric, and material characteristics must be obtained. The accuracy of the analysis will be only as good as the accuracy of this information.

C2.6 Seismic Wave Propagation

Wave propagation strains are related to the incoherency between seismic ground motions at two locations along a linear structure. Well-constructed buried natural gas and liquid hydrocarbon pipelines in good condition generally have not been affected by seismic wave propagation. This is borne out by the lack of a single reported case of failure of ductile, full penetration welded natural gas or liquid hydrocarbon pipeline attributable to wave propagation alone. Recent earthquake experience (Honegger, 1999a) has indicated that wave propagation is a credible earthquake hazard for pipelines only in cases of extremely poor quality girth welds or corrosion defects subjected to very high levels of seismic ground motion. In both instances, these guidelines recommend a case-specific assessment of the longitudinal strain to failure. Such an assessment may require removal of sufficient girth weld samples to develop a statistical description of strain capacity based upon laboratory tests. Other approaches may involve detailed non-destructive examination, fracture mechanics models, and laboratory tests of a limited number of pipeline specimens. In either case, the details of such case-specific investigations are currently under development in the industry and are beyond the scope of these guidelines.

The only instance where seismic wave propagation has been linked to pipeline is a case study of the 1985 Michoacan (Mexico City) earthquake. The welded pipeline was constructed of 42-inch x 0.313-inch wall API 5L X42 pipe. The reported damage was compressive wrinkling at intervals of approximately 500 feet and occurred in soft clay deposits. The pipeline was influenced by unusual seismic and geologic conditions. Because of the long distance separating Mexico City and the earthquake epicenter (nearly 250 miles), the predominant ground shaking was the result of Rayleigh waves with a relatively high period of incoming waves (2 to 3 seconds). Site amplification contributed to a high peak particle velocity, measured at 35 cm/sec, and the soft lake sediments were characterized by a wave propagation velocity as low as 40 to 100 m/sec. The combination of high particle and low wave propagation velocities would have promoted high ground strains, and the long period would have promoted a relatively large development length for mobilizing shear resistance between the pipe and surrounding soil. O'Rourke (1988) concluded that calculated pipe strains were sufficient to conclude that traveling ground waves were responsible for the damage.

For these reasons, the conclusions of O'Rourke (1988) can, at best, only be considered applicable to geologic settings similar to those from the case study, i.e., very soft deep sediment deposits prone to harmonic excitation by Rayleigh waves. The soil deposits near Mexico City and the tectonic regime surrounding Mexico are unique in this respect. Furthermore, estimating wave propagation strains for any region of similar geology for which strong motion earthquake records do not exist requires an attenuation relationship to estimate ground motions associated with Rayleigh waves, which do not exist.

Although the potential for damage to welded steel pipelines due to seismic wave propagation is generally low, it is occasionally necessary to compute the stress or strain in a pipeline caused by seismic waves. A procedure for estimating apparent wave propagation velocity was first proposed by Newmark (1965). The basis for estimating wave propagation strains is illustrated in Figure C2.14. Earthquake body waves (compression and shear waves) emanate from the earthquake source region as a continuous wave front. Ground shaking at the surface results from the propagation of this wave front up through the near-surface soils. At two locations, A and B, the apparent propagation of body waves is related to the difference in arrival times of the wave front at these locations. The arrival time is a function of the source-to-site distance for A and B and the propagation velocity along the path between A and B. The straight propagation paths shown in Figure C2.14 are a simplification, as the actual propagation paths are likely to be curved as a result of passage through strata of varying density. On the surface, the seismic wave front appears to propagate along the surface at a velocity that is closely related to the propagation of the seismic waves at depth (e.g., 3 km to 30 km). The approach proposed by Newmark (1965) leads to an estimate of maximum ground strain between any two locations on the surface that is proportional to the ground velocity generated by the response of surface soils to the input motion divided by the apparent velocity of the seismic wave front relative to the surface. In this model, locations directly above the earthquake source zone experience virtually no strain as they experience near-simultaneous arrival of the seismic wave front which translates to a near infinite apparent velocity.

There are some cases that are outside the scope of these guidelines that occasionally merit special consideration. These include transitions between very stiff and very soft soils, penetration into buried valve enclosures, branch connections, pipe fittings, and valves. In addition, pipelines with significant weld flaws, corrosion defects or not connected with full-penetration butt welds (e.g., compression couplings, bell and spigot joints) may also be vulnerable to damage from seismic wave propagation. In such cases, an assessment of the strain capacity of the pipe and comparison with the magnitude of wave propagation strains can be used to judge the expected performance of the pipeline.

C2.7 Transient Ground Deformation

Another potential seismic hazard not related to permanent ground deformation is transient ground deformation (TGD). Transient ground deformations are produced when local site conditions allow weaker surface soils to respond differently from adjacent near-surface soils that are considerably stronger and can possibly damage buried pipelines. This type of behavior has been observed repeatedly in past earthquakes where pipeline damage appears to be more prevalent near the boundaries between strong and weak soils.

Transient ground deformation has been identified as a possible mode of local soil response that can lead to pipeline damage. One of the first investigations to identify transient ground deformation as a cause of pipeline damage was undertaken by O'Rourke (O’Rourke, 1991, Pease and O’Rourke, 1997) in a study of water and sewer pipeline damage in the Marina District of San Francisco following the 1989 Loma Prieta earthquake. O’Rourke postulated that liquefaction in the Marina District caused the hydraulic fill soils to be decoupled from the response of the underlying stiff soils and respond more like a sliding mass.

Following the 1994 Northridge earthquake, investigations into the causes of pipeline damage in Potrero Canyon identified a similar decoupling mechanism as a probable cause of the ground cracking patterns and locations of pipeline damage (Honegger, 2000). The underlying mechanism in Potrero Canyon was postulated to be shear failure within the weak canyon soils that were subjected to a velocity pulse of more than 1 m/s and a period of 5 to 6 seconds. The magnitude and long period of the velocity pulse is attributed to the proximity of Potrero Canyon to the projection of the caU.S.tive fault plane. This type of near-field motion has been recorded in past earthquakes and is commonly referred to as earthquake “fling.”

There are no widely accepted procedures for identifying sites susceptible to TGD or estimating potential TGD displacements. A large part of the difficulty lies in the inability to characterize near-field ground motions and the characterization of the strength properties of soils undergoing variable states of liquefaction. The following guidance for identifying sites potentially susceptible to TGD are provided based upon the sparse amount of field data:

1. Presence of a relatively uniform layer of soil with low shear strength (or high potential for liquefaction).
2. Presence of firm soil or rock adjacent to, or confining, deposits of weak or liquefiable soils.
3. Soft soil sites located in the near-field of a potential earthquake with a likelihood for experiencing ground motions characteristic of earthquake “fling”.

Techniques for modeling TGD include two-dimensional finite element analysis and simplified sliding-block analysis. Representative models are illustrated in Figure C2.15. Finite element methods are complicated by the uncertainty associated with modeling slip along the soil failure plane and, in some cases, the need to account for separation at the boundary between stiff and soft soils. Sliding-block analogies face similar impediments with respect to defining a limiting value of shear strength for the soft soil and the sliding friction associated with the decoupled soil mass. Both methods require an explicit input ground motion time history that adds an additional layer of uncertainty in the modeling process.

At this time (2001), assessment of TGD remains a research topic. Unless there is compelling evidence to suspect that TGD displacements exceed a few centimeters, consideration of TGD is believed to be unnecessary for new pipelines or pipelines constructed in a manner consistent with modern construction. This position is supported by the fact that there is no evidence that TGD has been a factor in damage to such pipelines.

Older pipelines with potential weld or corrosion defects or not constructed with butt-welded pipe joints are at greater risk of damage from TGD. Given the lack of an accepted method for estimating TGD displacements, it does not seem prudent to devote large resources to estimating TGD for potentially susceptible pipelines. Instead, TGD damage occurring at unknown locations should be recognized as a possible consequence of the earthquake, and the need to potentially respond to TGD damage should be incorporated into earthquake response plans.

Type of Deposit	General Distribution of Cohesionless Sediments in Deposits	Likelihood that Cohesionless Sediments (saturated) would be Susceptible to Liquefaction (by age of deposit)			
		Less than 500 years	Holocene	Pleistocene	Pre-Pleistocene
Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	N/A	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial til	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	Unknown	Unknown
Residual soils	Rare	Low	Low	Very low	Very low
Selka	Locally variable	High	Moderate	Low	Very low
Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Estuarine	Locally variable	High	Moderate	Low	Very low
Beach: high wave energy	Widespread	Moderate	Low	Very low	Very low
Beach: low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore-shore	Locally variable	High	Moderate	Low	Very low
Artificial					
Uncompacted fill	Variable	Very high	N/A	N/A	N/A
Compacted fill	Variable	Low	N/A	N/A	N/A

Table C2.1 Liquefaction Susceptibility Screening Table
(from Youd and Perkins, 1978)

Material	V/PGA (inches/s-g)	V/PGA (cm/s-g)
Rock	26	66
Stiff Soil	57	144
Cohesionless Soil	55	140
Alluvium	48	122

Table C2.2 Velocity Ratios for Various Types of Surface Soils (from O'Rourke and El Hamdi, 1988)

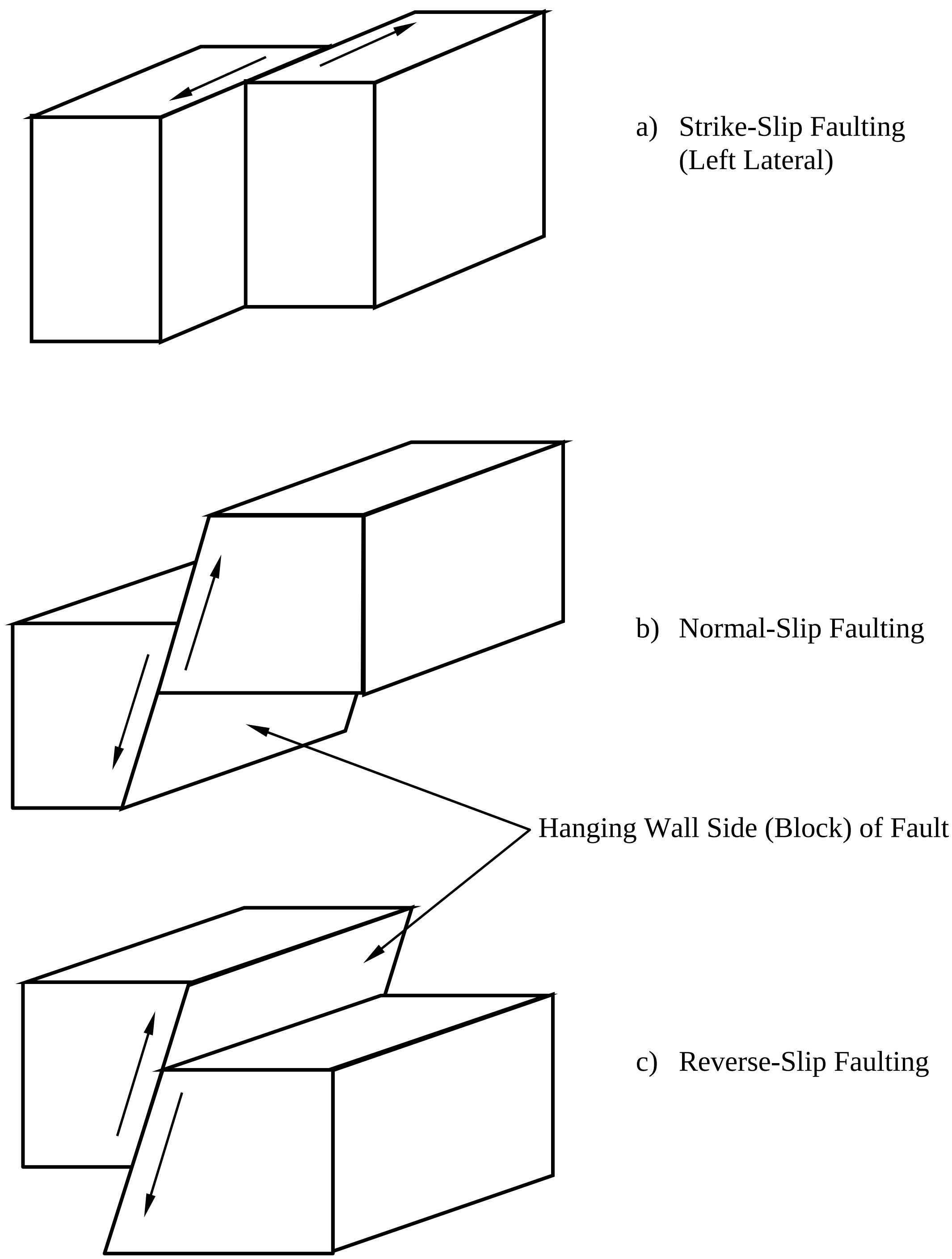


Figure C2.1 Fault Movement Classification

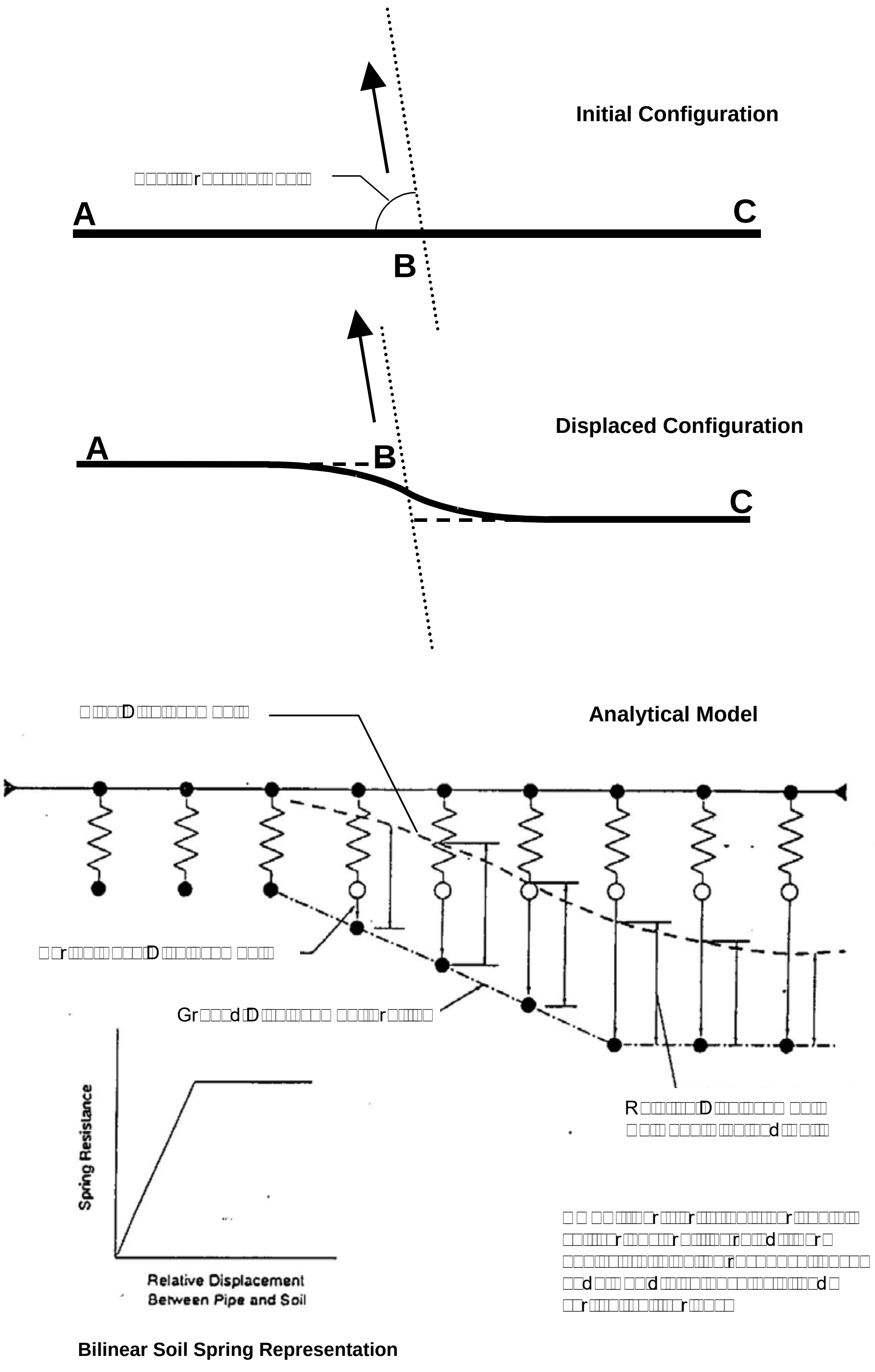


Figure C2.2 Effect of Right Lateral Strike-Slip Fault Movement on a Buried Pipeline

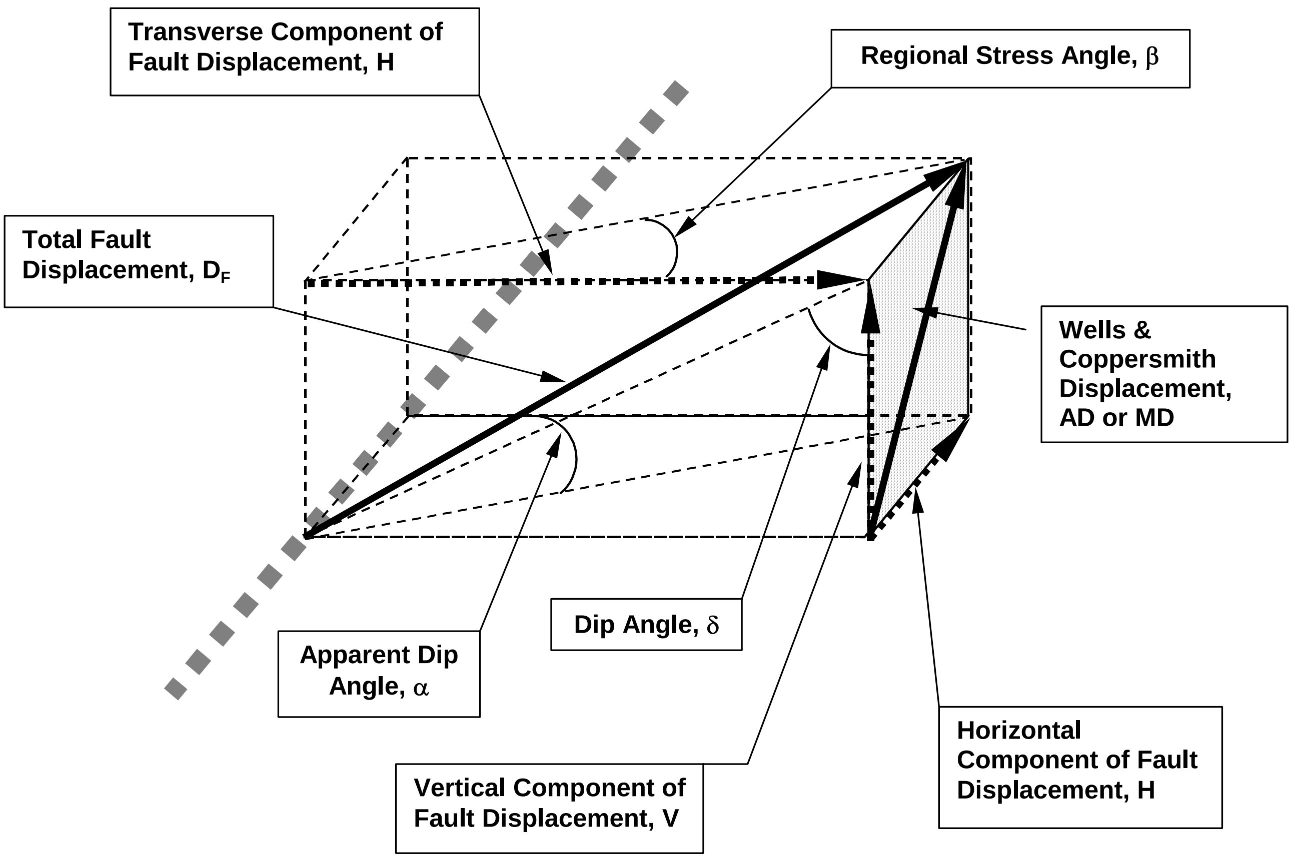


Figure C2.3 Orthogonal Components of Fault Displacement Based Upon Estimates Using Wells & Coppersmith (1994) Empirical Equations

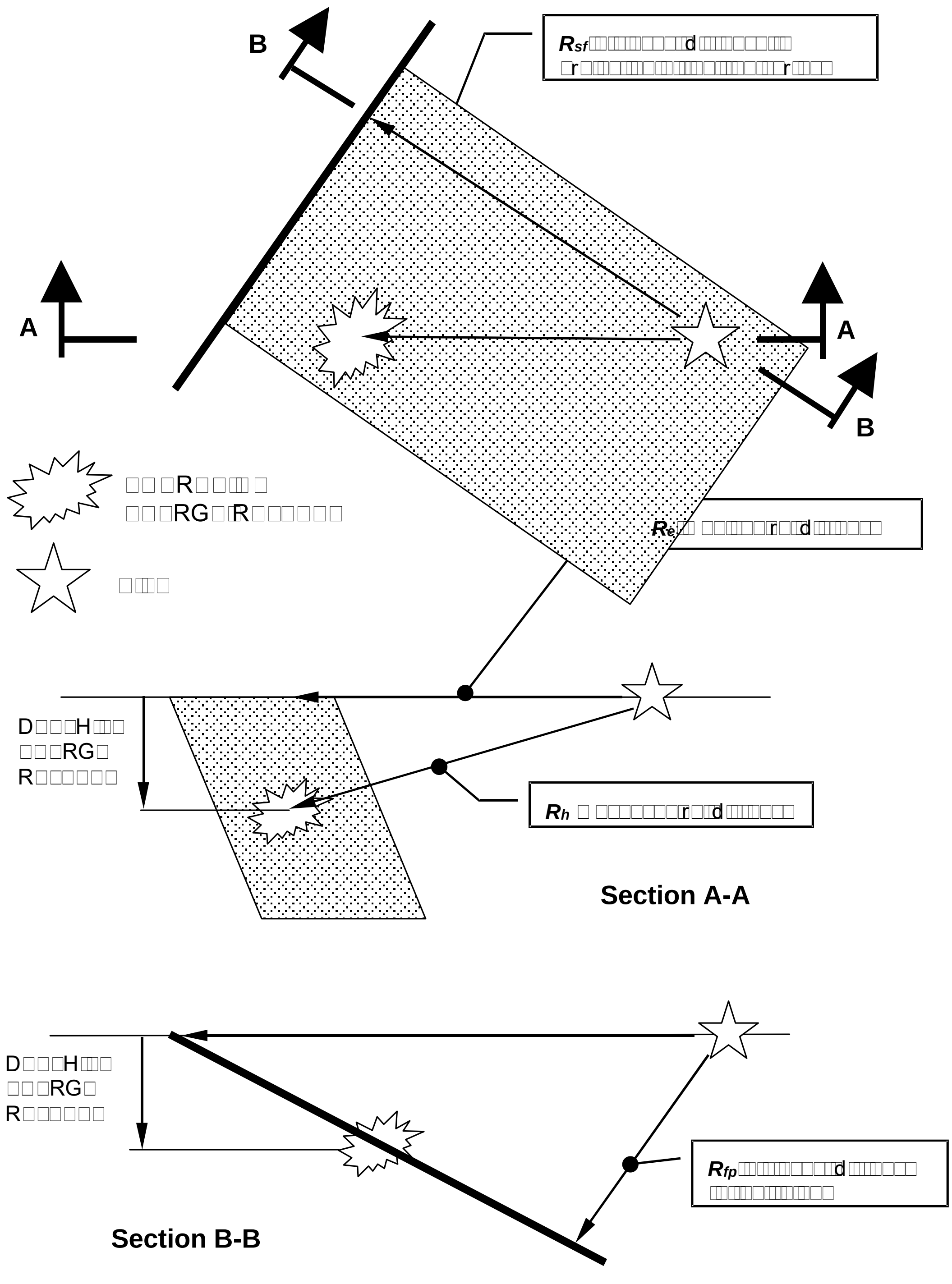
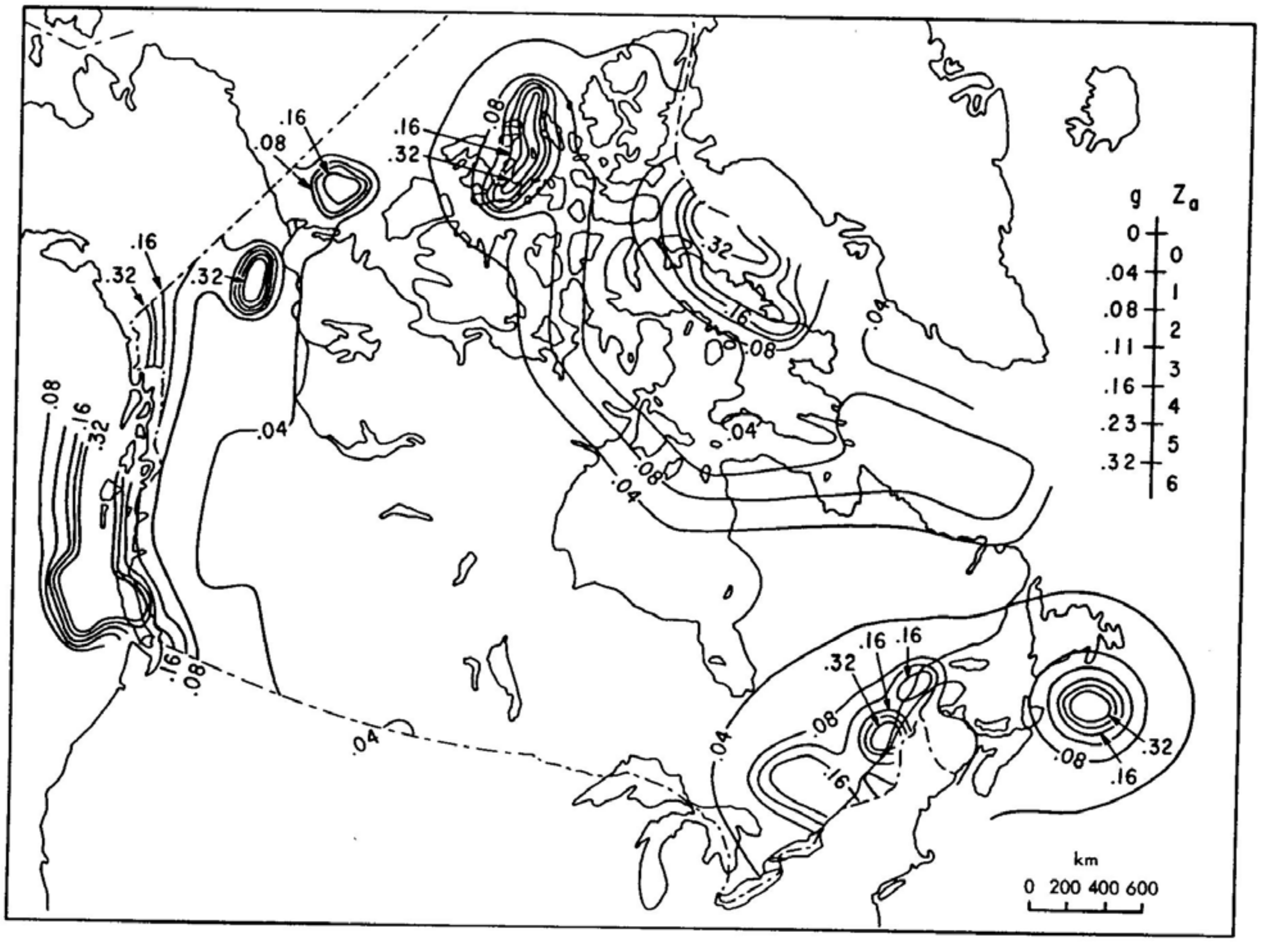


Figure C2.4 Various Definitions of Earthquake Parameters Adopted in Attenuation Relationships



(from the Supplement to the National Building Code of Canada (1990) for
PGA with a 10% probability of exceedance in 50 years)

Figure C2.5 Example of Mapped Representation of Probabilistic Ground Motion

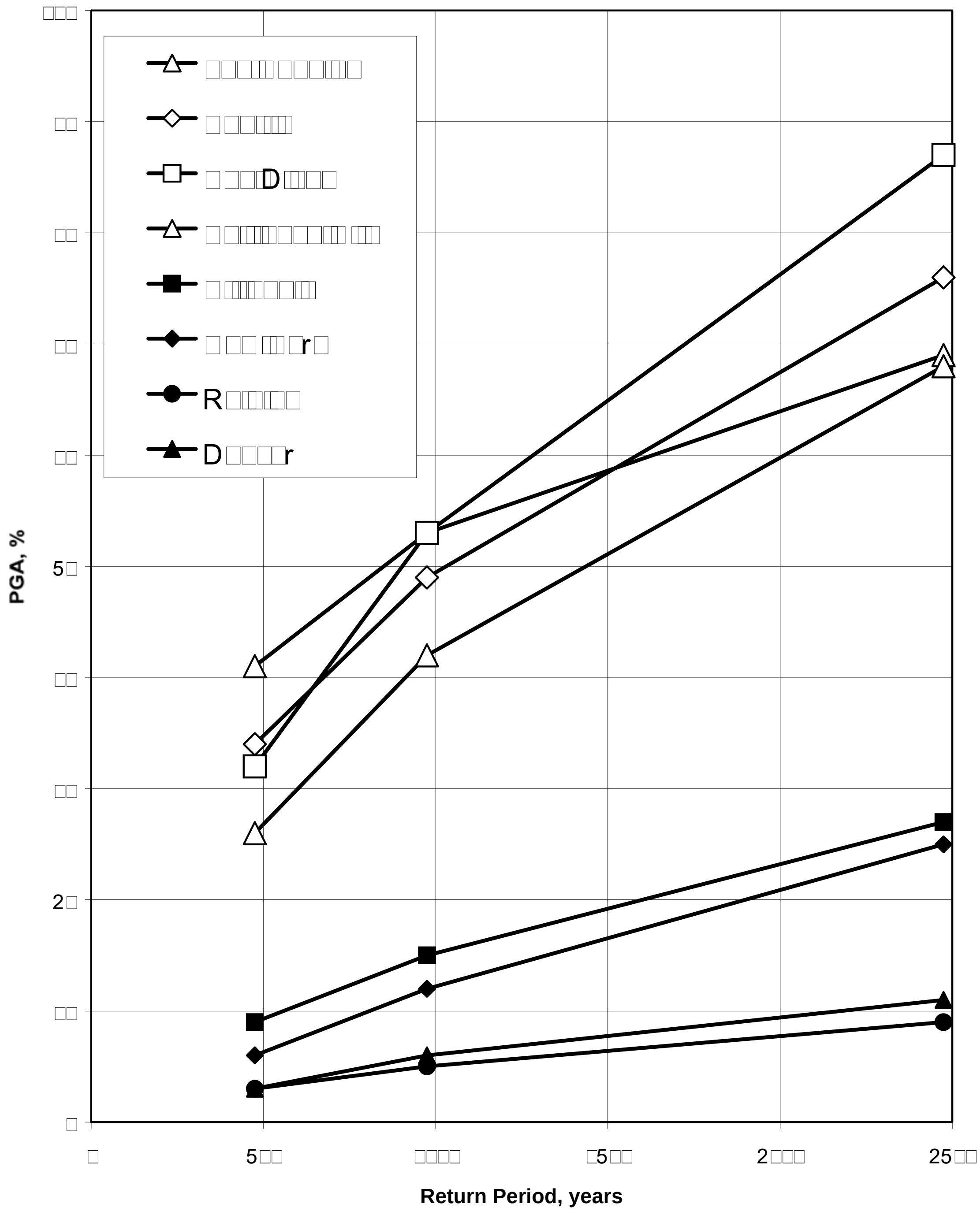
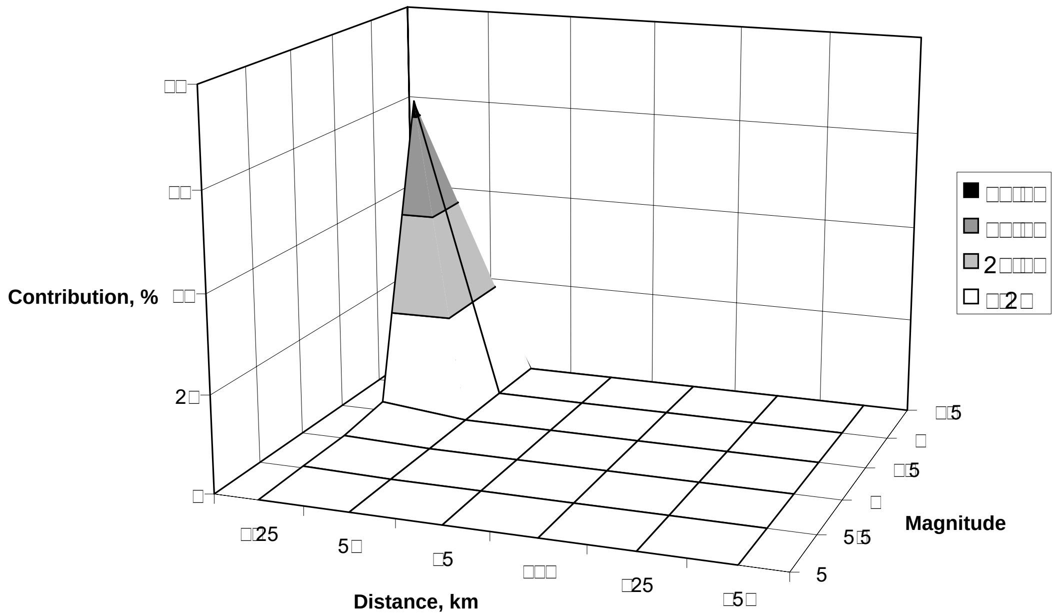


Figure C2.6 Example Hazard Curve Representation of Probabilistic Ground Motion



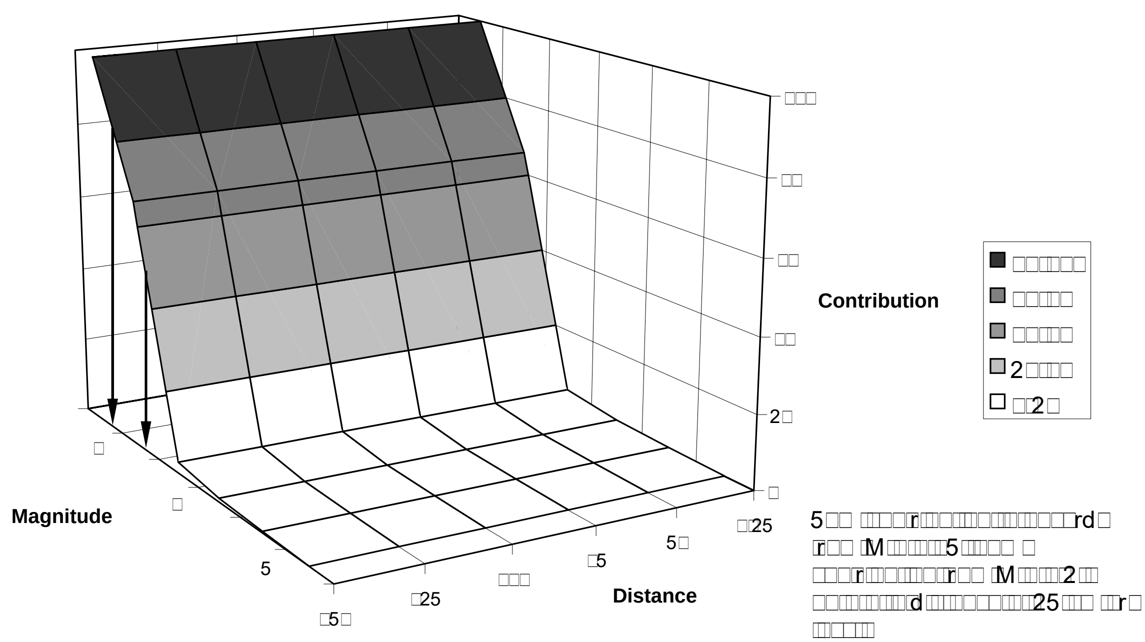


Figure C2.7 Example of Extraction of Deaggregated Hazard Information from the 2,500-year Firm Rock PGA Hazard at Salt Lake City

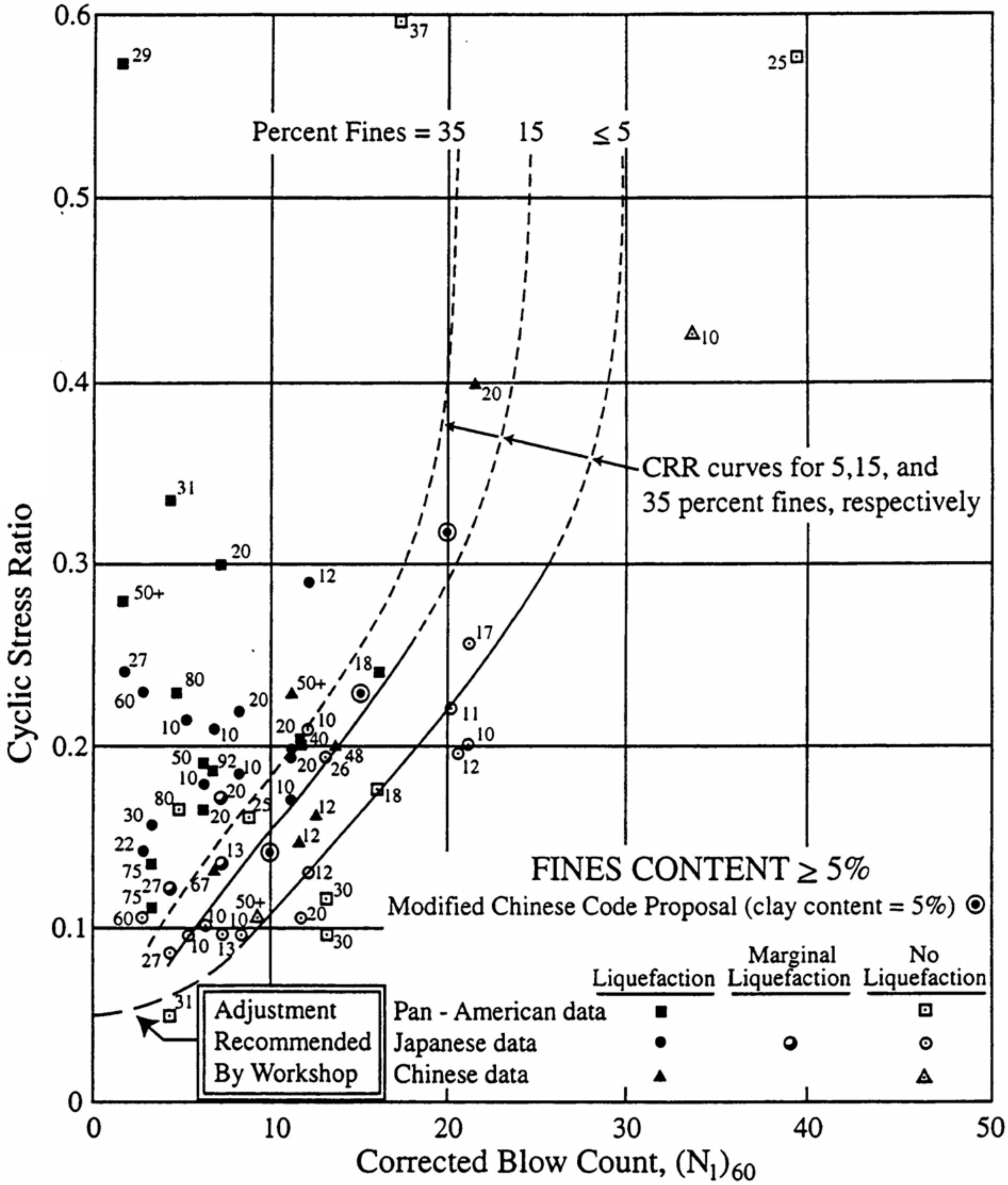
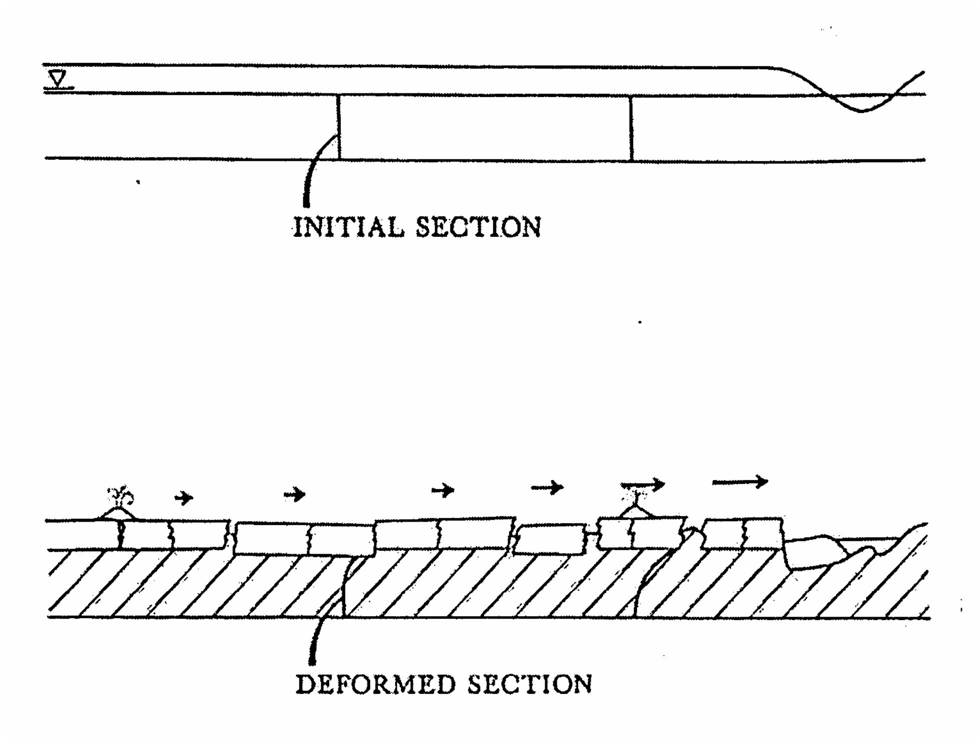
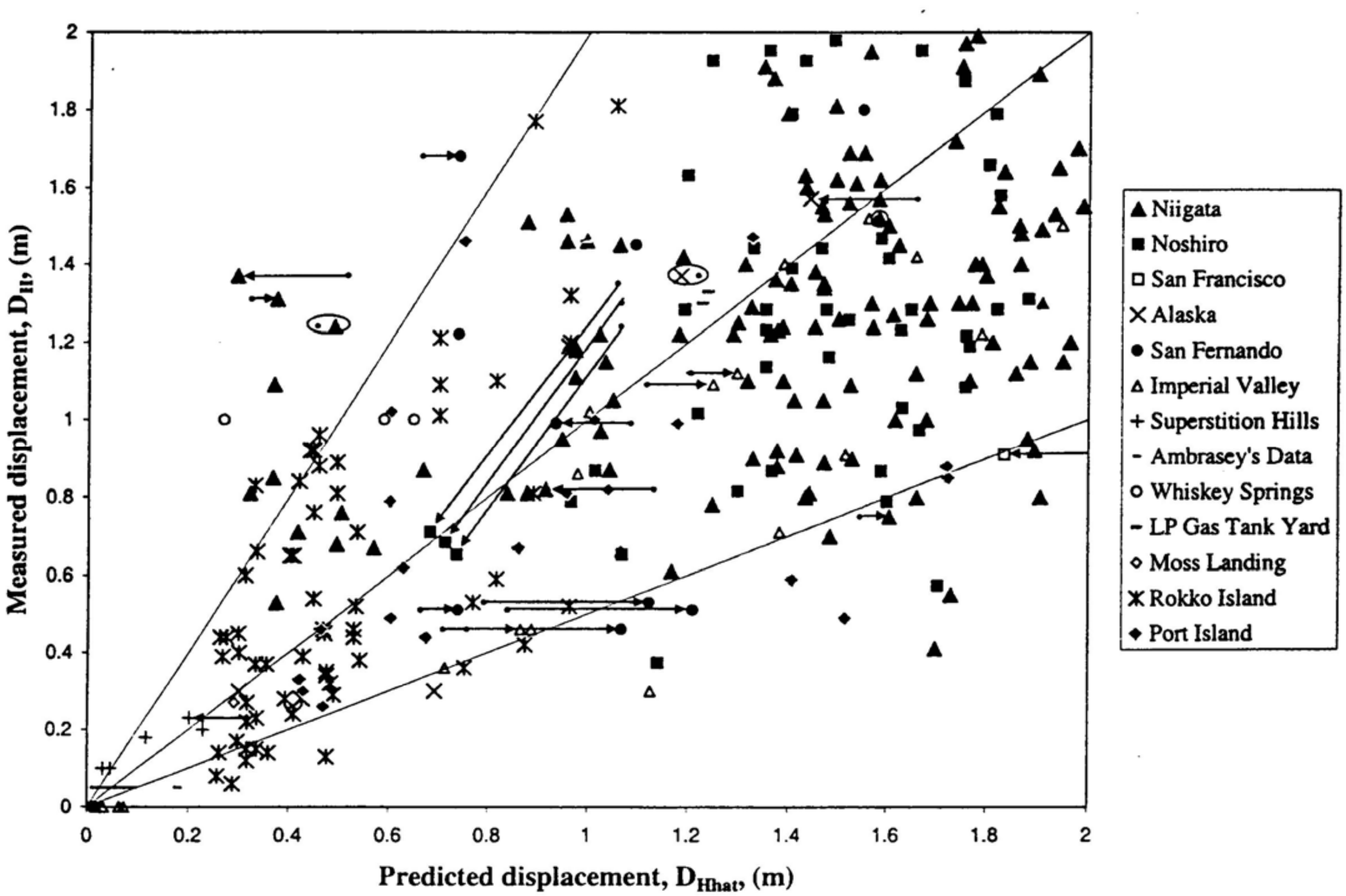


Figure C2.8 Liquefaction CSR Curve (from Youd and Idriss, 1997)



Liquefaction occurs in the cross-hatched zone. The surface layer moves laterally down the mild slope or toward a free breaking up into blocks which may jostle back and forth and settle unevenly during the spreading process.

Figure C2.9 Schematic Representation of a Lateral Spread (from Youd, 1984)



(arrows reflect changes from approach of Bartlett and Youd (1992) for selected data points)

Figure C2.10 Comparison of Actual Lateral Spread Displacements with Estimates Using Youd et al., 1999 (from Youd et al., 1999)

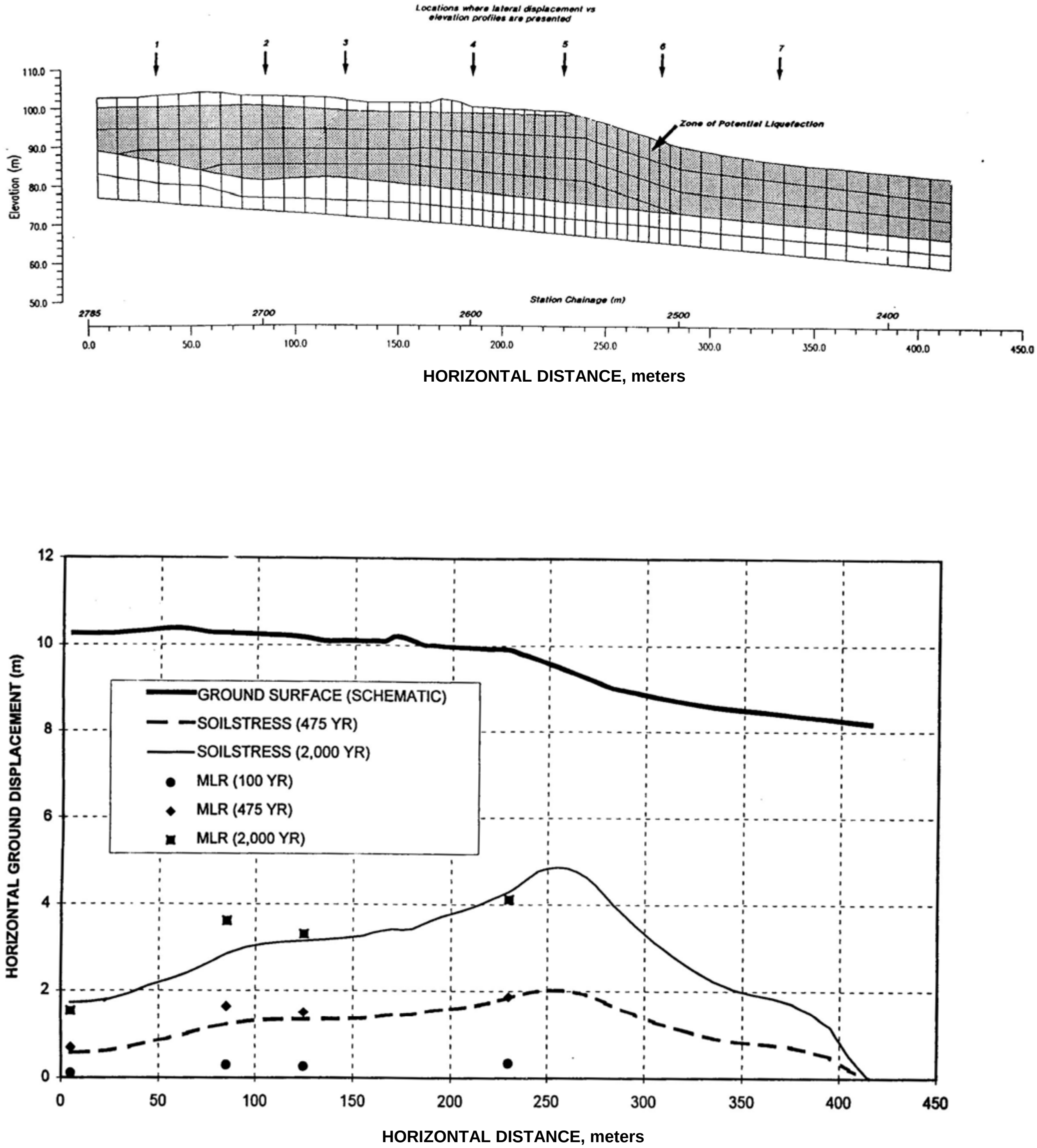


Figure C2.11 Sample Comparison of Bartlett and Youd Method with SOILSTRESS (Bartlett and Youd (1992) results denoted by “MLR”)

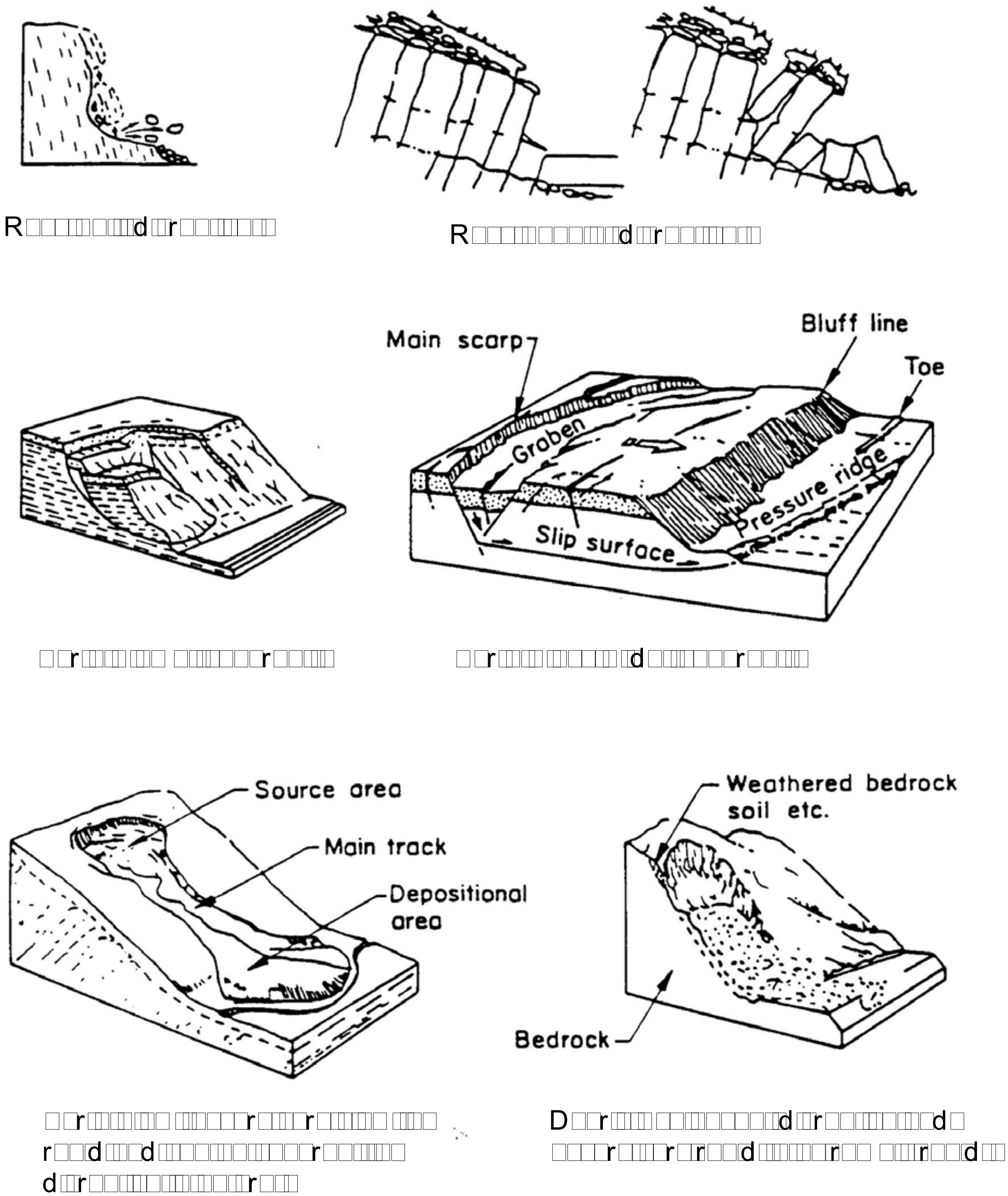
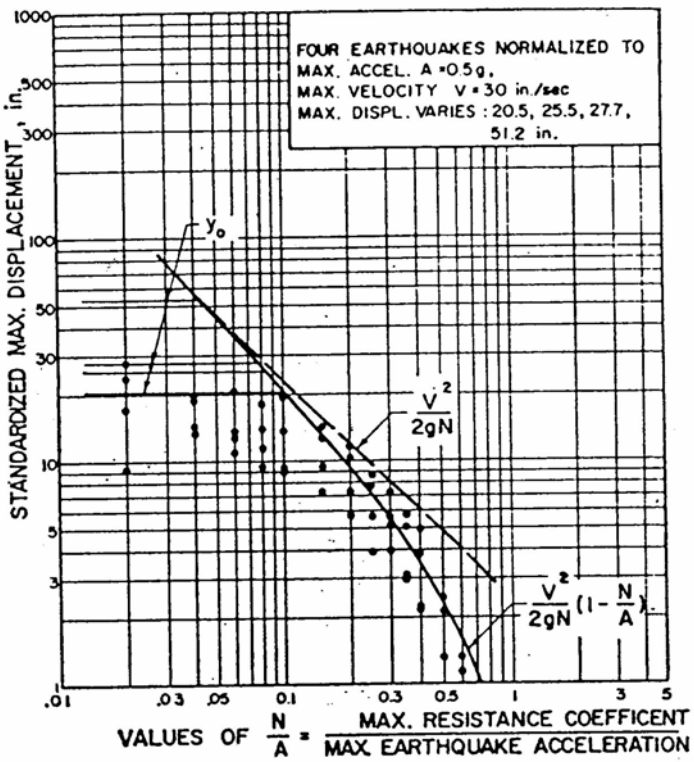
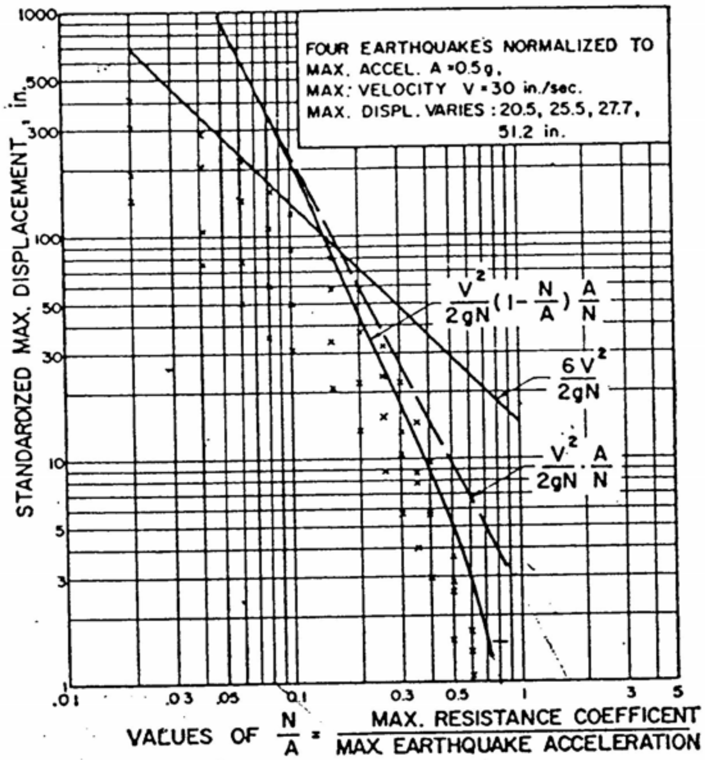


Figure C2.12 Various Types of Landslides (after Varnes, 1978)

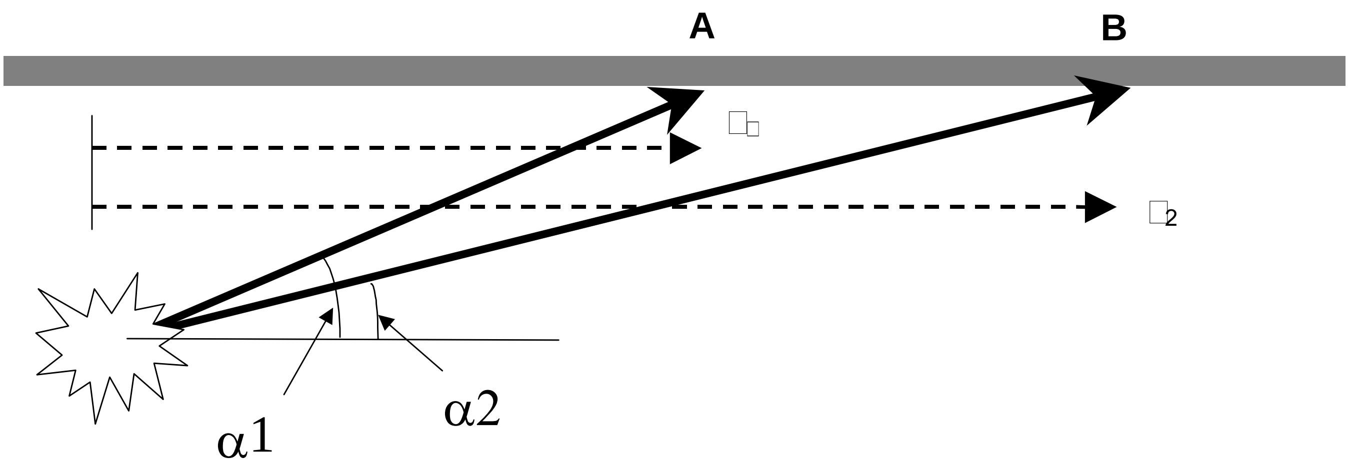


Displacement from normalized earthquakes for a sliding block with symmetrical resistance (horizontal sliding plane)



Displacement from normalized earthquakes for a sliding block with unsymmetrical resistance (inclined sliding plane)

Figure C2.13 Generic Sliding-block Relationships Using Actual Earthquake Records Scaled to a PGA of 0.5 *g* (from Newmark, 1965)

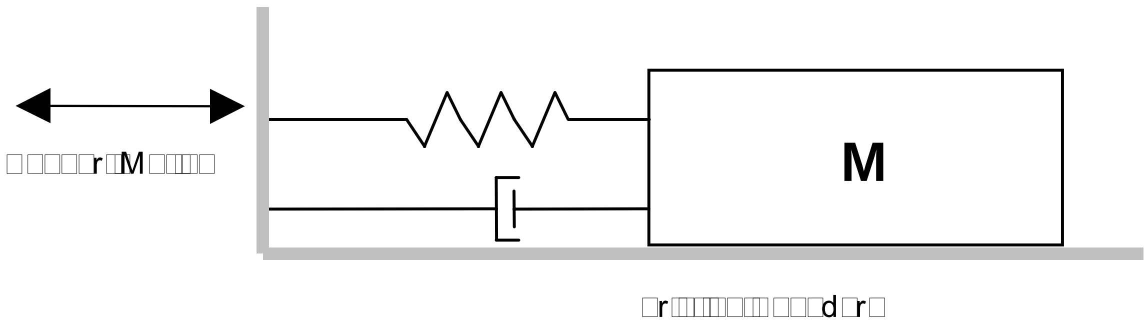


$$t_1 = \frac{L_1}{c \cdot \cos(\alpha 1)}$$

$$t_2 = \frac{L_2}{c \cdot \cos(\alpha 2)}$$

$$c_{apparent} = \frac{c\left(L_2 - L_1\right)}{\left[\frac{L_2}{\cos(\alpha 2)} - \frac{L_1}{\cos(\alpha 1)}\right]}$$

Figure C2.14 Wave Propagation Paths Leading to Incoherent Seismic Ground Strains



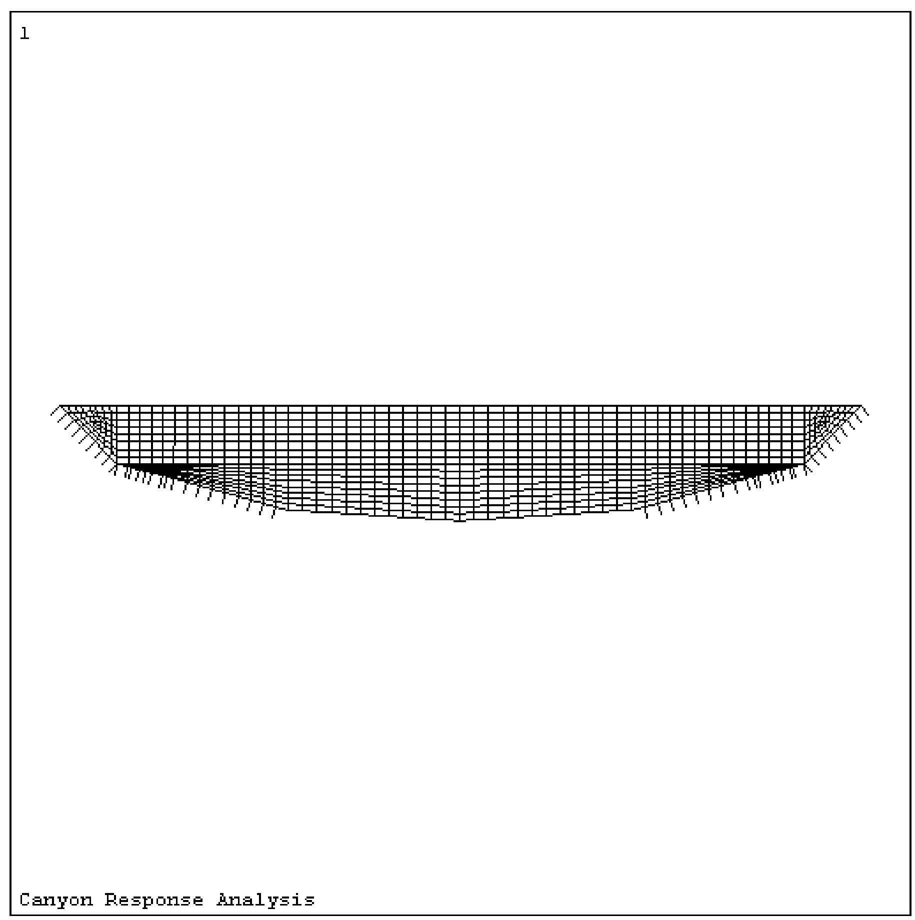


Figure C2.15 Modeling Techniques with Potential Application to Estimating TGD

C3.0 PIPELINE PERFORMANCE CRITERIA

Determining limiting pipeline strains associated with varying levels of performance is an ongoing area of investigation within the pipeline industry. While there have been several notable projects using full-scale tests and detailed finite element analyses, the differences in load conditions, measurement techniques, and project goals make direct comparison difficult. As a result, defining appropriate relationships for seismic assessment of pipeline response is largely based upon available test data reported in the open literature and the collective judgment of the authors and reviewers contributing to these guidelines.

C3.1 New Pipelines and Pipelines Compatible with Current Construction Practice

There is experience in the pipeline industry of strain based design. Most of this is generally related to offshore applications such as pipe reeling for installation, high temperature, high pressure pipelines that can buckle due to high axial loading, and deepwater pipelines. Much of this experience relates to "load-controlled" situations on small diameter, low *D/t* (diameter to wall thickness), thick-walled pipes. In these situations, there is a high likelihood that the pipe will experience the design strain (in the range of 2% nominal strain[1]), thus validating the methodologies used. However, the strains in these applications may not be as large as those expected for seismic design. There are situations where pipelines have survived much larger strains, as well as instances where pipelines have failed at much lower strains.

The selection and specification of both pipe and welding for a strain-based design pipeline system includes consideration of a number of factors that may not be adequately addressed in a conventional stress-based design, which is thoroughly addressed in various codes and standards such as ASME B31.4, B31.8 and API 1104. Considerable research efforts in the pipeline industry have been directed at understanding the behavior of pipe at high strains, with this effort increasing over the last few years with more focus on strain-based design. Although the industry now has a vastly improved understanding of high strain behavior, significant advancements continue to be made.

The overall strain capacity of a pipe is dependent upon a number of factors:

1. ***Compressive Strain***:
    a. The properties of the pipe material including the shape of the stress-strain curve, the ratio of yield strength to tensile strength (Y/T), and uniform elongation strain (ε_{UL});

[1] Nominal strain is the strain that the pipe "globally" experiences over a certain gauge length (anywhere between 0.5 to 2 diameters). However, locally the strains can be much higher due to small geometric imperfections, and once a pipe starts to wrinkle and locally buckle, the strains that can be experienced in the vicinity of the buckle can be many times that of the nominal strain.

b. The *D/t* ratio, with a greater wall thickness having a higher strain capacity for the same pipe diameter (lower *D/t* ratio); and

c. Geometric anomalies associated with out of roundness (ovality) and wall thickness variations need to be considered for single pipe joints and at girth welds, where they can introduce wall thickness step changes when joints are butted together. The effect of geometric anomalies is to reduce the strain to initiate pipe wall buckling.

2. ***Tensile Strain:*** The tensile strain capacity depends upon the pipe material properties mentioned above, as well as the following factors related to the resistance of girth welds to the presence of welding defects:

a. Increased wall thickness reduce the effects of a loss of cross-section from weld defects and increases the amount of material available to resist crack growth;

b. Weld properties (yield strength, ultimate strength, and toughness) that meet or exceed the pipe metal reduce the potential for flaw growth; and

c. Weld quality specifications and inspection requirements increase the likelihood that girth welds will achieve the required properties and defect tolerances.

Key aspects of the above issues are briefly discussed below:

Pipe Steel Properties: The pipe properties of interest for both compressive and tensile strain capacities are the longitudinal properties. Circumferential properties are typically only of interest for assuring that the material is not anisotropic.

1. The <u>stress-strain curve</u> should be monotonically increasing, and should contain no plateau regions in the post yield portion of the curve. A high yield strength for the grade is not necessarily a benefit in strain-based assessments. It is good practice to obtain several stress-strain curves from the mill from full thickness, longitudinal specimens for the pipe.

2. The <u>Y/T</u> and <u>ε_{UL}</u> influence strain capacity with increases in strain capacity generally obtained by lower Y/T and higher ε_{UL}. Both of these values vary with pipe grade and the pipe toughness with increases in the grade and toughness generally resulting in higher Y/T ratios and lower ε_{UL} and lower strain capacities (i.e., X65 will generally have a higher strain capacity than X80). The required value of Y/T and ε_{UL} depends upon each application and the required strain capacity. Values of Y/T and ε_{UL} quoted in pipe material specifications need to be considered carefully, as “standard” values used in conventional specifications may not be sufficient. Specifying unusually low values of Y/T and high values of ε_{UL} may significantly increase costs for manufacturing the

limited amount of pipe that might be needed for seismic design and result in additional costs for testing to show that the pipe meets specifications. Some pipeline projects have taken steps to segregate pipe by heat and, therefore, Y/T properties. They then use the high Y/T pipe at high strain areas.

3. Weld heat affected zone (HAZ) properties are an important consideration as there is a tendency of certain low alloy materials to soften in the HAZ. This softening reduces the original pipe yield strength near the weld, leaving stronger material on either side of the zone. Once yielding occurs in the HAZ, all of the plastic strain will accumulate in this narrow zone, rapidly exhausting the available ductility and leading to plastic instability. Careful specification of the carbon level and the presence of secondary hardening alloys in the pipe steel is necessary to minimize the potential for softening in the HAZ.

Pipe Geometric Properties:

1. Wall thickness considerations include both the absolute thickness and variability. A thicker pipe wall increases both tension and compression strain capacities. This results in an additional benefit from using lower grade pipe as a lower grade pipe will require a thicker pipe wall. For example, if X65 is used for seismic design instead of X80, the strain capacity is increased because of both the lower grade (lower Y/T and higher ε_{UL}) and thicker wall (lower D/t). Careful consideration needs to be given to the allowable wall thickness tolerances in the pipe specifications as geometric misalignment decreases the compression strain for initiation of wrinkling (a concern for strain levels associated with continued operation). There is a trade-off between minimizing the geometric misalignment across the girth weld and the extra cost associated with having tighter tolerances.

2. Control of tolerances on pipe ovality, like control of pipe wall thickness, is important because ovality can reduce the compression strain for initiation of pipe wrinkling in a pipe joint. Furthermore, ovality of the ends of the pipe can result in geometric misalignment of the pipe ends at the girth weld that reduces the compression strain for initiation of pipe wrinkling in the same way as wall thickness variation. There is also a trade-off between minimizing pipe ovality and the extra cost associated with having tighter tolerances.

Welding Materials and Procedures: The presence of small imperfections or flaws in the weld deposit, and particularly those having a planar geometry and located near the root or cap of the weld, severely degrade the ability of the material to uniformly distribute plastic strain. Flaw extension in either a brittle or ductile mode from continuing strain at the extremities of these flaws leads rapidly to rupture. Because the imperfections cannot be entirely eliminated, a weld metal having slightly overmatching yield strength should be selected to minimize the opportunity for over-strain of the surrounding material. The degree to which the weld metal will overmatch the pipe metal must account for the possibility that the actual pipe strength may be significantly greater than the specified minimum values. The need to achieve overmatching weld strength is another reason to consider the use of lower grade pipe steels in designing the pipeline to resist ground movement. A welding process should be chosen to minimize both the number and size of the imperfections. In the absence of highly skilled manual pipeline welders, the welding procedure should consider the use of mechanized GMAW or TIG processes.

Quality Control Over the Welding Process: The experienced-based workmanship requirements (flaw acceptance criteria) contained in the stress-based welding codes such as API 1104 or ISO 13847 may not be adequate for strain-based designs. New acceptance limits (generally smaller and fewer allowable flaw sizes) may need to be calculated based upon fracture mechanics technology. The fracture mechanics calculations used to determine weld acceptance are semi-empirical and require estimates of several parameters, particularly those related to flaw geometry, flaw location, material strength, and material toughness. Unlike approaches aimed at fitness-for-purpose assessments, the approach in these guidelines will not generally require the same high level of reliability that the pipeline will be capable of withstanding a particular tension strain for the reasons noted in Section C3.1.2. The level of required reliability needs to be considered when making decisions on the appropriate level of conservatism associated with the many assumptions necessary to establishing strain limits using fracture-mechanics approaches.

The new and smaller allowable flaw size presupposes that there will be an inspection system in place capable of detecting and sizing any flaws remaining in the weld. It is well established that flaw height and the distance to the pipe surface are the critical parameters. Conventional radiographic techniques are not suitable for either of these measurements and the use of a mechanized ultrasonic procedure such as that commonly used for offshore lay barges is recommended. Accuracies of about 0.040 inches are achievable for flaw height measurement in pipe with wall thickness less than about one inch. The net effect of the special welding procedure and the extensive post-weld inspection may lead to a high weld rejection rate, thereby reducing productivity and increasing cost per weld. However, this should be economically practicable considering the relatively few segments of the pipeline where seismically induced permanent ground displacements may occur.

Achieving strain limits as high as 2% to 4% may not be possible under certain circumstances, particularly for the assessment of existing pipelines with flaws or defects or where the use of high-grade pipe steel reduces the likelihood that girth welds will have strength and toughness characteristics superior to the pipe steel such that welds develop the full strength and strain capacity of the pipe steel. Specific criteria for assessing the strain capacity of girth welds is beyond the scope of these guidelines but techniques for performing such assessments are well-established. Two of the more common approaches to qualify pipe material and welding specifications for a specific project application depend upon either curved wide plate testing of weld specimens that conform to project specifications or theoretical fracture mechanics models.

It is noted that there may be additional proprietary data that would lead to an alternate basis for establishing pipeline strain acceptance criteria. Users of these guidelines may elect to review such additional data prior to establishing strain criteria for specific projects. As noted in Section C1.2, users that need to demonstrate levels of seismic reliability significantly higher than has been accepted in typical practice will likely need to adopt more conservative strain acceptance criteria than provided in these guidelines.

[illegible]

Failure in a strain-based design is normally taken to mean loss of the pressure boundary integrity. The failure can result from axial tensile overload or excessive bending. Bending involves both tension and compression strains with failure frequently occurring on the compression side of the bend as a result of a buckle in the pipe wall followed by a wrinkle. This leads to gross tension strain in local areas of the wrinkle. Most failures involve first the initiation of a partial thickness crack followed by brittle or ductile crack propagation through the wall. The latter may also involve out-of-plane ductile tearing.

Longitudinal Compression Strain Limit

In compression, local instabilities such as wrinkling can develop at strains much less than the allowable tension strain limits. Wrinkling of the pipeline wall does not, in itself, constitute a failure condition. While wrinkling may also lead to a significant reduction in pipeline moment capacity, this is not a concern for displacement-controlled conditions, as the pipeline deformation is not related to an applied load.

Under sustained loading conditions, further compressive shortening would be expected to concentrate at points of initial wrinkling. Typically, the initiation of compressive wrinkling occurs in the range of 0.3% to 0.6% strain for most large diameter pipes, or about one-tenth the strain level noted above for tension. Loss of pressure boundary integrity is associated with strains far greater than that associated with the initiation of compressive wrinkling for moderate *D/t* ranges.

In recommending longitudinal compression strain limits, the clear evidence of substantial compression strain capacity prior to rupture was tempered by the concerns regarding establishing recommendations on the maximum compression strain capacity that are much greater than past practice for seismic load applications within the pipeline industry (e.g., ASCE, 1984). The following discussion describes some of the test data used in assessing the suitability of equation (3-1).

Pipe tests performed by Mohareb et al. (1994) focused on conditions where the axial load was constant (five of the seven analysis cases utilized constant axial load). The axial force applied in the pipe tests by Mohareb et al. corresponded to a 45°C temperature differential, a tension force equivalent to the force necessary to counteract axial shortening from the Poisson effect of internal pressure, and a compressive force to counteract the tension produced by the closed-end conditions of the test specimens.

Ghodsi et al. (1994) repeated the tests of Mohareb et al., but include a girth weld in the center of the test specimen to assess the impact of a girth weld on initiation of wrinkling. The testing performed by Ghodsi et al. demonstrated that the presence of a girth weld significantly reduces the strain associated with a visual indication of pipe wrinkling. However, the reduction in the strain that could be reached in the tests was not as significant as the reduction in strain at initiation of wrinkling.

Pipe tests at the Center for Engineering Research (C-FER) in Edmonton, Alberta, Canada, have also been performed to examine post-wrinkling behavior. The details of the tests are proprietary but some results, including a relationships for compression strain limits for X70 steel, have been published by Zimmerman et al. (1995). The Zimmerman et al. relationship is reportedly based upon limiting maximum computed tension strain in the crest of the buckle formed after wrinkling to 10%.

Another testing program at the University of Alberta was carried out to investigate strains associated with the initiation of wrinkling (Dorey et al., 2001). These tests were performed on X70 pipe with a *D/t* ratio of 92 and internal pressures producing hoop stresses ranging from 0% to 80% of the specified minimum yield stress. These tests are relevant for comparing with equation (3-1) since they were taken past the point of wrinkle formation and development of maximum pipe moment capacity. Dorey et al. recommend several relationships for estimating the longitudinal strain in the pipe at the point of maximum moment. These relationships account for imperfections in the pipe wall and at the girth weld and also consider two typical shapes for the stress-strain curve of the pipe material.

Data from the available papers and test reports for the above tests are plotted in Figure C3.1 with strains associated with the end of testing plotted in Figure C3.1(a) and strains associated with initial wrinkling or maximum moment plotted in Figure C3.1(b). Several data points are plotted for the same *D/t* ratios. These data points correspond to different levels of internal pressure and net axial load applied during the testing. Internal pressure in the tests varies from zero to an internal pressure that produces hoop tensile stresses equal to 72% (Mohareb et al., 1994 and Ghodsi et al., 1994) or 80% (Zimmerman et al., 1995 and Dorey et al., 2001) of the pipe yield stress. Higher strains for the same *D/t* ratios are from tests with higher internal pressure.

The potential conservatism in equation (3-1) is clearly evident in Figure C3.1(a) by the fact that the maximum local compression strains imposed by laboratory tests with moderate to high internal pressure are typically at least two times greater than those given by equation (3-1). The only exceptions to this are the C-FER data. The smaller strains for the C-FER data may be a result of the presentation of limited data in the public-domain paper, the lack of interest in testing well beyond maximum moment, or the physical limitations of the testing configuration. It should be noted that all of the tests used to generate the data in Figure C3.1 were carried out without the pipe losing pressure integrity; hence, the actual safety margin against loss of pressure integrity is indeterminate.

Longitudinal Tension Strain Limit

Determining an appropriate tension strain limit will vary with welding specifications and inspection criteria and needs to be done on a project-by-project basis. The likelihood of achieving a particular strain capacity need not be unnecessarily restrictive. It is the intent of these guidelines that any acceptable strain criteria used on a particular project should be based upon the concepts of achieving a balanced seismic design as discussed in Appendix A. Estimates of the probability of pipeline failure at the specified strain limits in the range of 10% to 30% are judged acceptable for achieving a balanced seismic design in these guidelines. This level of weld failure, which may seem high compared to other applications such as fitness for purpose assessments, is justified in these guidelines where the likelihood of seismic hazard is the same as the acceptable likelihood experiencing seismic damage. Other factors supporting a higher acceptable probability of a weld not meeting a particular strain level are related to the low probability of a severe undetected flaw at a location of high strain.

It is expected that the allowable longitudinal tension strains will range from 2% to 4% for pressure integrity when evaluating pipeline response to permanent ground deformation. Longitudinal tensile strains of 3% to 5% have been adopted for assessing the ability of pipelines to maintain pressure integrity when subjected to earthquake-generated ground displacement for more than 20 years and are recommended in ASCE (1984). Advancements have been made in the use of fracture-mechanics approaches to estimate failure strains and the comparatively low strain capacities that often resul from fracture-mechanics approaches have raised questions regarding the suitability of historically adopted strain limits.

The magnitude of permissible computed tension or compression strain is limited to no greater than 4% based upon what is believed to be the practical limits of the ability of the analytical approach recommended in these guidelines to adequately represent pipeline behavior. Strain limits supported by additional test data, more refined analysis methods, or corroboration of the analysis approach with test or case history data are generally preferred.

Resolving differences between historic practice, test data, and fracture-mechanics approaches is complicated by several factors. Typical fracture-mechanics approaches were originally developed for applications that require a very low likelihood of failure. This low failure probability is achieved, in part, by adopting conservative assumptions with respect to the type of flaw, the location of the flaw, material toughness, and degree of weld overmatching. Thus, the strain estimates obtained by existing fracture mechanics methods are likely to be associated with a probability of failure much lower than required by the approach adopted for defining seismic hazards in these guidelines. Revising fracture-mechanics approaches to adequately match test data for strain capacities much greater than 1% and provide a statistical estimate of strain capacity is an area of active research within the pipeline industry.

The need to justify high strain capacities will vary among projects and the nature of the seismic hazards faced. In some cases, the required strain capacity necessary to withstand earthquake-generated ground movement is much less than 2% to 4% range recommended in these guidelines as generally achievable. There are also cases, particularly for high-cost projects, when a substantial increase in pipeline wall thickness required by adopting conservative fracture-mechanics based estimates of tension strain capacity may not have an economically significant impact, considering the increased wall thickness is only required at locations where earthquake ground movement is expected. Finally, ground displacements that are much more likely to occur compared to earthquake-related ground movement (e.g., frost heave, thaw settlement, active landslides, mining subsidence), generally require more conservative estimates of strain capacity to meet desired performance goals. For such cases, guidance on longitudinal strain limits is provided in Tables C3.1 through C3.3 based upon a fracture mechanics study by EMC^2 (Wang, 2003). The three tables provide three levels of longitudinal tension strain limits based upon material toughness and Y/T ratio. Comparison with test results described in the EMC^2 report indicate the tension strain limits in Tables C3.1 through C3.3 may have a mean factor of safety as high as 1.8 on strain. Users should carefully review the EMC^2 report to fully understand the basis for the recommendations in Tables C3.1 through C3.3 before adopting them for a particular project.

□□□□2□ □□r□r□ □□□□□G□□□□M□□□□□□□ □r□ □□□ □□r□□□□

The compression strain limits to maintain normal operability for displacement-controlled conditions are largely based upon judgment. The magnitude of longitudinal tension and compression strains are limited to 2%. The reasons for the 2% limit are related to concerns about the overall distortion of the pipe cross-section that might be associated with coating damage or impair the passage of internal pigs.

Longitudinal Compression Strain Limits

Longitudinal compression strains associated with buckling instability of the pipe wall are limited to the value estimated using the maximum moment expression developed by Dorey et al. (2001). Although there is evidence that strains greater than those corresponding to the maximum moment onset of wrinkling can be sustained without appreciable deformation of the pipe cross-section, a conservative approach is adopted to reduce the likelihood of conditions that might impair the long-term integrity of the pipeline. Equations (3-3a) and (3-3b) are the equations developed by Dorey et al. (2001) for offsets at the girth weld. The compression strain limit from equation (3-3a) is plotted in Figure C3.1(b) for the case of X70 pipe with a p/p_y ratio of 0.72 and a Δ/t ratio of 0.10. Also plotted in Figure C3.1(b) are the pressurized test data points corresponding to initiation of wrinkling and maximum moment from tests performed at the University of Alberta. For comparison, data assembled by Stephens et al. (1991) from a wide variety of unpressurized pipe tests are also shown to illustrate similarity with the University of Alberta results. As shown by Figure C3.1(b), the 2% limit on strains for continued operation governs for the conditions selected at *D/t* ratios less than about 57. The reasons for the 2% limit are related to concerns about the overall distortion of the pipe cross-section that might be associated with maintaining corrosion coating integrity or impair the passage of internal pigs.

Longitudinal Tension Strain Limits

Similar to the selection of tension strain limits for pressure integrity, the longitudinal tension strain limit associated with continued operation are associated with strains that can be accommodated without degrading corrosion coating or internal pig passage. The literature review for these guidelines did not identify information in the open literature relating the impact of longitudinal tension strain on the long-term integrity of the pipeline. Longitudinal tension strains of approximately 1% to 2% seem to be reasonable considering that these strains are generally less than what might be generated during field cold bending operations. Lacking additional information or project-specific test data, it is suggested that longitudinal tension strains for continued operation be taken as half of the tension strain limits for pressure integrity.

[illegible]

The loading conditions considered in these guidelines are typically governed by imposed displacement of the surrounding soil (displacement-controlled). There are some situations where the pipeline loads may not be related to displacement (load-controlled). In the ASME codes, load-controlled conditions are considered primary loads and displacement-controlled conditions are considered secondary loads. Seismic inertial loads are an example of primary loads for aboveground piping.

Examples of load-controlled conditions include flow-type ground failures and situations where ground failure removes soil restraint around a section of pipeline. Figure C3.2 illustrates pipeline configurations corresponding to these two ground displacement conditions. Other examples of load-controlled pipeline conditions not related to earthquake hazards can be found in the offshore industry where pipe laying operations can impose strains in the pipe that must be accommodated without degradation in pipeline strength. For load-controlled situations, local buckling of the pipeline can lead to progressive deformation and pipeline rupture.

In developing load-controlled strain criteria, consideration was given to establishing a uniform strain limit of 0.005, the strain at which the specified minimum yield strength is determined. For most pipeline steels historically used for gas transmission pipelines, stresses below the specified minimum yield stress (corresponding to a strain of 0.5%) do not pose a significant risk for progressive deformation. However, for compression strains, this limitation underestimates strain capacities for pipes with low *D/t* ratios and may underestimate strain capacity for pipes with very high *D/t* ratios. It is also desirable to account for the beneficial effects of internal pressure in reducing the loss of bending strength of the pipe once the maximum moment is reached. The approach adopted is to base the compression strain limits on the maximum moment relationships developed by Dorey et al. (2001) that are recommended for continued operation. For load-controlled conditions, the relationships developed by Dorey et al. (2001) are factored by 0.75 to provide a reasonable margin against exceeding the maximum moment. The Stephens et al. (1991) relationship is used as a lower limit for compression strains under load-controlled conditions as this relationship represents a 95% lower bound fit to a large database of tests on unpressurized pipes assembled by Battelle.

C3.2 Considerations for Offshore Pipelines

The strain criteria for offshore pipelines undergoing bending and external pressure is adopted from the approach of Murphey and Langner (1985). This approach was adopted because of the relatively simple forms of the governing equations. Alternate approaches such as BS 8010 or API Bulletin 5C3 were also considered. Alternate approaches can be used but the degree of variation in the resulting allowable bending strains are well within the expected variation in hazard quantification.

Tension in offshore pipelines reduces the estimated collapse pressure by reducing the hoop stress at which yielding occurs. The approach adopted in these guidelines is consistent with past recommendations (AGA, 1990) as well as recent investigations (Foeken and Gresnigt, 1998). The approach limits the effective von Mises stress to the material yield stress. For a particular value of axial stress, the hoop stress corresponding to a von Mises yield condition is estimated as follows:

$$\sigma_{yh-red} = \sigma_{yh}\left[\sqrt{1-\frac{3\sigma_a^2}{4\sigma_{yh}^2}}-\frac{\sigma_a}{2\sigma_{ya}}\right] \tag{C3-3}$$

where:

$\sigma_{yh\text{-}red}$ = reduced hoop yield stress

σ_{yh} = hoop yield stress

σ_{ah} = axial yield stress

σ_a = applied axial stress

With the simplifying assumption that the axial and hoop yield stresses are equal to the nominal yield stress, σ_y, equation (C3.3) limits the tension strain limit to less than the yield strain.

C3.3 Pipelines Not Compatible with Current Construction Practices

Pipelines not covered by the strain limits in these guidelines include pipelines with one or more of the following characteristics:

1. Alternate girth weld details
2. Welds with detectable crack defects or not acceptable by current welding criteria
3. Corrosion defects
4. Undermatched welds

Pipelines with Alternate Connections Details

Older pipelines may have been constructed with welded slip joints, bell-bell-chill-rings, or other non-butt-welded pipe joints. In addition, pipelines may have been fabricated with mechanical connections such as Dresser® or Victaulic® couplings. The specific details of the configuration of these joints can significantly reduce their strain capacity. For example, depending upon the shape of the bell for welded slip joints, local buckling under longitudinal compression may occur at strains well below the yield strain of the pipe (Tawfik and O'Rourke, 1985). The vulnerability of older pipelines with non-butt-welded pipe joints is often compounded by the fact that the quality of the welds is typically much worse than obtained by more recent construction practices.

It is not possible to define generic recommendations for establishing strain limits for non-butt-welded pipe joints. Detailed finite element analyses utilizing shell elements capable of capturing the effects of material yielding and local wall buckling is perhaps the most cost-effective approach to assess strain capacity for non-standard girth welds. If the welds are suspected of having defects, the finite element analyses will need to be combined with fracture mechanics-based assessment of strain capacity for the defective weld.

Pipelines with Corrosion or Weld Defects

Evaluating the effects of corrosion or weld defects is a potential issue for the assessment of existing pipelines. Explicit consideration of the effect of defects is limited to cases where evidence from past pipeline repairs or findings from internal inspections permit quantification of defect size and distribution. Assessing pipelines for weld and corrosion defects requires expertise in fracture mechanics and experience in characterizing defect geometry and material toughness parameters. Test data on pipe or weld toughness properties are normally not available for existing pipelines. The lack of data generally leads to removing samples of material for testing, fabrication of simulated test specimens, or estimating parameters based upon correlations with similar pipe material, chemical composition, or Charpy test data.

Procedures for estimating the strain capacity of pipelines with weld or corrosion defects is an area of ongoing research. Much of the work is related to estimating to what extent defects, in combination with relatively small amounts of induced bending, reduce the safe pipeline operating pressure. For seismic assessment purposes, the more common situation is longitudinal strains exceeding yield from axial or bending response with the pipeline operated well below its maximum permissible pressure.

There are many references that can be consulted regarding the assessment of weld and corrosion defects and non-standard joint details. Examples include ASME B31G (ASME, 1991), Roberts and Pick (1998), Rosenfeld (1996), Wang et al. (1996, 1998), Wang (2003), Warke and Amend (1997), and Warke et al. (1997).

For the majority of older pipelines, available information is often limited to pipe size, wall thickness, and material specification. In some cases, there may be coupon tensile test data but this is typically from a statistically insignificant number of pipe samples. This lack of data can severely limit the analytic tools available to estimate strain capacity. In many cases, a decision will need to be made as to whether or not it is necessary to take a pipeline out of service to remove test samples for determining mechanical properties. Information gathered from the operational history of the pipeline, limited excavation to examine the pipeline, information from field personnel on site during construction of the pipeline, and historical experience with the performance of pipe suppliers and pipeline contractors can provide sufficient information to form a judgment on the strain capacity of a pipeline suspected of having weld or corrosion defects. Extraction of specimens for testing is typically limited to cases where potential pipeline damage from a seismic hazard is judged to be extremely serious.

It is not necessary to establish a very conservative acceptable strain limit for the purposes of the seismic assessment. The typical approach to determining an acceptable strain limit for pipelines with weld or corrosion defects is to assume that the defect is located on the circumference coincident with the occurrence of maximum pipe strain. This approach can be very conservative. The combination of maximum pipe strain with a defect is dependent upon the variation in location of of ground displacement, the direction of ground displacement with respect to the pipeline alignment, and the variation in the distribution of weld defects around the circumference or the distribution of corrosion defects around the circumference and along the pipeline. It should be remembered that the approach described in these guidelines is assumed to contain sufficient conservatism in the definition of the seismic hazard. If highly conservative approaches to assess the impact of defects are applied, it is reasonable to consider reducing the conservatism in defining the seismic hazard.

Pipelines with Undermatched Girth Welds

For the purposes of these guidelines, girth welds are considered to undermatch the pipeline material if the actual weld metal yield strength and toughness do not exceed the actual values of yield strength and toughness of the pipe material. There is considerable evidence (e.g., Denys, 1992, 1993, 1994; Horsely et al., 1997) that undermatching girth welds prevent the development of gross-section yielding generally assumed to occur in the analysis of pipeline response to permanent ground displacement. If weld undermatching is suspected, tension strains for pipeline assessment should be based upon limiting pipe stresses to the yield stress of the weld unless data are available to support higher values.

Studies investigating the effect of weld undermatching on compression strains have not been identified. It seems reasonable to assume that the effect of undermatched girth welds is much less under compressive loading. Unless there is reason to believe that local yielding of the weld metal substantially reduces the occurrence of local buckling of the pipe wall, it is recommended that no adjustment is necessary to the longitudinal compression strain limits for undermatched girth welds.

It must be pointed out that undermatched welds are rare for most existing pipelines and the majority of new pipelines. Weld undermatching has only become a significant concern with the recent introduction of high-strength pipelines steels consistent with API 5L Grade X70 and above. However, without careful weld specifications, it is possible to have undermatched welds for lower strength pipe. The difficulty in achieving weld overmatching may limit the ability of higher grade pipe steels from being used when it is necessary to resist significant seismic ground movements.

Assessing the Risk of Not Meeting Performance Goals

The ability of the pipeline to achieve the desired level of performance, expressed as an annual probability estimate, should be taken as the product of the annual probability that the pipeline will experience ground displacements greater than the capacity assuming factored strain limits and the probability that the pipe defects more severe than those accounted for exist in portions of the pipeline experiencing high strain levels. This can be expressed analytically in the following manner:

$$P_G = P_{GD}\left[1-\left(1-P_D\right)^n\right] \qquad \text{(C3-2)}$$

where:

P_G = computed performance expressed as an annual probability of exceedance

P_{GD} = annual probability of exceedance for the earthquake producing the ground displacement hazard representing the pipeline capacity with reduced strain limits

P_D = probability that the reduced strain limit for any individual weld is less than expected accounting for the variability in defect distribution and methods for relating defect size to strain capacity

n = number of welds exposed to strains above the reduced strain limits

As an example, consider the following conditions:

> Analysis of a portion of a pipeline assuming nominal properties indicates that a pipeline subjected to permanent ground movement is adequate to meet the desired performance goal, defined as an annual probability of loss of pressure integrity no greater than 0.2%. The ground displacement corresponding to a 0.2% annual probability of exceedance is 2 m (6.6 ft) and is governed by longitudinal tension strain. Inspections of another portion of the pipeline built at the same time and by the same contractor indicate a potential for girth weld or corrosion defects. A reduced longitudinal tension strain capacity is determined based upon the most severe flaws identified by the prior inspection and fracture mechanics analyses. It is estimated that there is a 5% chance that the actual reduced strain capacity will be less than the determined reduced strain limit for any one weld. The reduced longitudinal tension strain capacity is exceeded for ground displacements greater than 1.25 m (4.1 ft). The analysis of pipeline response indicates that the length of pipeline exceeding the reduced strain limits is approximately 120 m at the full ground displacement of 2 m. The annual probability of exceeding 1.25 m of ground displacement is estimated to be 0.5%. The pipeline was constructed with 12-m (40-ft) joints of pipe.

The annual probability of loss of pressure integrity accounting for the potential for defects is estimated as follows:

$P_{GD} = 0.005$

$P_D = 0.05$

$n = 120 \text{ m} \div 12 \text{ m/joint} = 10$ girth welds

$$P_G = 0.005\left[1-(1-0.05)^{10}\right] = 0.002$$

The pipeline with defects can be considered as meeting the performance goal since the estimated probability of exceeding the reduced strain limit is equal to the 0.2% defined by the performance goal. While the annual probabilities of exceeding acceptable longitudinal tension strain limits are the same, the probabilities associated with a particular time interval can be different.

For a 50-year time frame, T, the probability of ground movement exceeding 2 m (i.e., not meeting the desired performance goal) is computed as follows:

$$P_{eT} = \left[1-(1-P_e)^T\right] = \left[1-(1-.002)^{50}\right] = 0.0952$$

where:

P_{eT} = probability of exceeding nominal strain limit in time T

P_e = annual probability of earthquake more severe than defined by the performance goal

T = time frame for computing P_{eT}

The above calculation may greatly overestimate the likelihood of failure in 50 years because it assumes that the pipeline has a 100% probability of failure at the strains produced by a 2-m ground displacement.

The probability that the reduced longitudinal tension strain will be exceeded in the same 50-year time frame is computed as follows:

$$\begin{aligned} P_{erT} &= \left[1-\left(1-P_{GD}\right)^{T}\right]\left[1-\left(1-P_{D}\right)^{n}\right] \\ &= \left[1-\left(1-.005\right)^{50}\right]\left[1-\left(1-.05\right)^{10}\right] \\ &= \left[0.2217\right]\left[0.4013\right] = 0.0890 \end{aligned}$$

where:

P_{erT} = probability of exceeding reduced strain limit in time T

T = time frame for computing P_{erT}

The differences in the above example probability calculations are not significant considering the level of uncertainty in estimating earthquake hazards. This is generally the case when the pipe meets the desired performance goal considering the effects of potential defects in a probabilistic manner. When it is useful to express the probability in terms of a reference time frame, basing calculations on only the annual probability of exceeding acceptable longitudinal strain capacity is generally sufficient.

Table C3.1 Level 1 Longitudinal Tension Strain Limits for Special Applications

Grade	Wall Thickness		*Y/T*	Strain Limit (%)	
□□□	□□□□□	□□□□		D□□□□□ □□2□5□□□	D□□□□□ 25□□□□
□52	□25□	□□	□□□□	□□□	□55
	□□□5	□5		□55	□□□
	□5□□	□2□□		□□□	□2□
	□□5□	□□□		2□22	□□□
	□□□□	25□□		2□□□	□□□
□□□	□25□	□□	□□5□	□□□	□5□
	□□□5	□5		□5□	□□□
	□5□□	□2□□		□□5	□□□
	□□5□	□□□		2□□□	□□□
	□□□□	25□□		2□2□	□□□
□5	□25□	□□	□□□□	□□□	□□□
	□□□5	□5		□□□	□□□
	□5□□	□2□□		□□□	□□□
	□□5□	□□□		2□□□	□5□
	□□□□	25□□		2□□□	□□2
□□□	□25□	□□	□□□□	□□5	□□□
	□□□5	□5		□□□	□□□
	□5□□	□2□□		□□2	□□□
	□□5□	□□□		2□5	□52
	□□□□	25□□		2□□□	□□□
□□□	□25□	□□	□□□□	□□□	□□5
	□□□5	□5		□2□	□□□
	□5□□	□2□□		□□□	□□□
	□□5□	□□□		□□□	□□□
	□□□□	25□□		2□□□	□□□

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2□ M□□ □□ □□ □□r□□□□r□□□□□□□□□2□□□5□□□□□d□□□r□□□□ □□r□□ □□r□□□□□□□□2□□2□□□□

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5□ □ □d□r□□□ □□□□ □r□ □□□□□r□r□□

Table C3.2 Level 2 Longitudinal Tension Strain Limits for Special Applications

Grade	Wall Thickness		*Y/T*	Strain Limit (%)	
[illegible]	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]

[illegible]

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible]
5. [illegible]
6. [illegible]

Table C3.3 Level 3 Longitudinal Tension Strain Limits for Special Applications

Grade	Wall Thickness		*Y/T*	Strain Limit (%)	
[illegible]	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]
	[illegible]	[illegible]		[illegible]	[illegible]

[illegible]

[illegible]

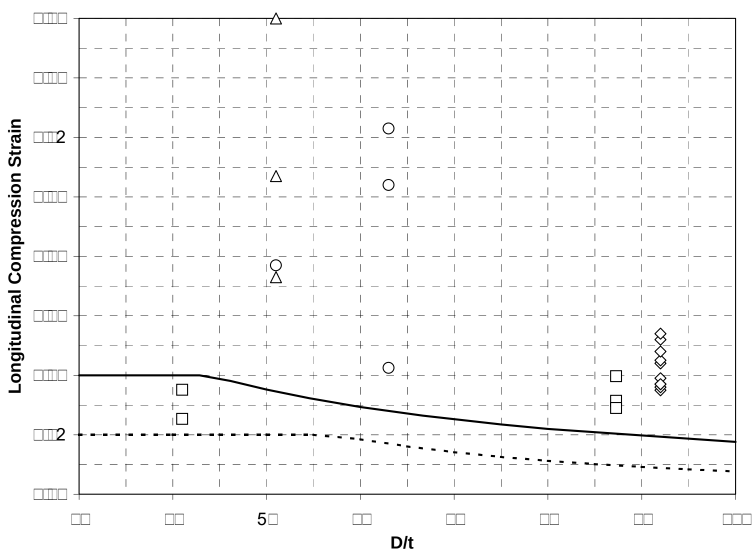
2. [illegible]

[illegible]

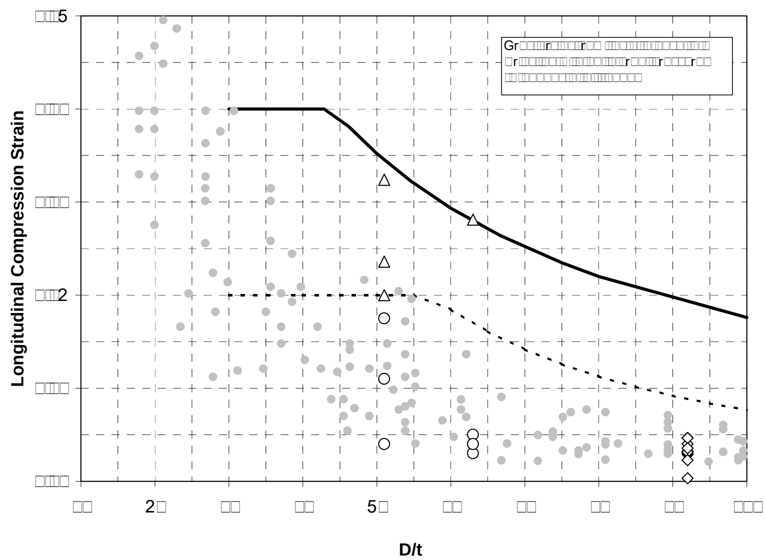
[illegible]

5. [illegible]

6. [illegible]



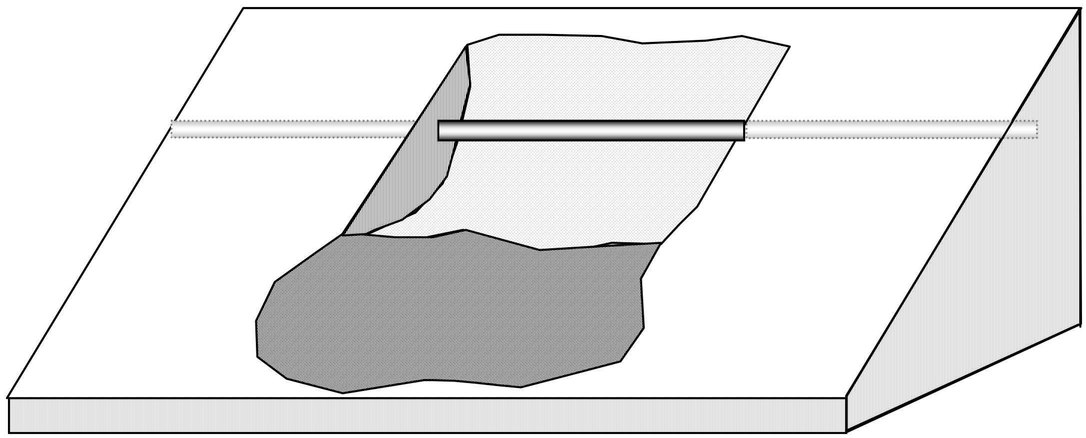
(a) Comparison of Test Data on Maximum Tested Strain Over Gauge Length



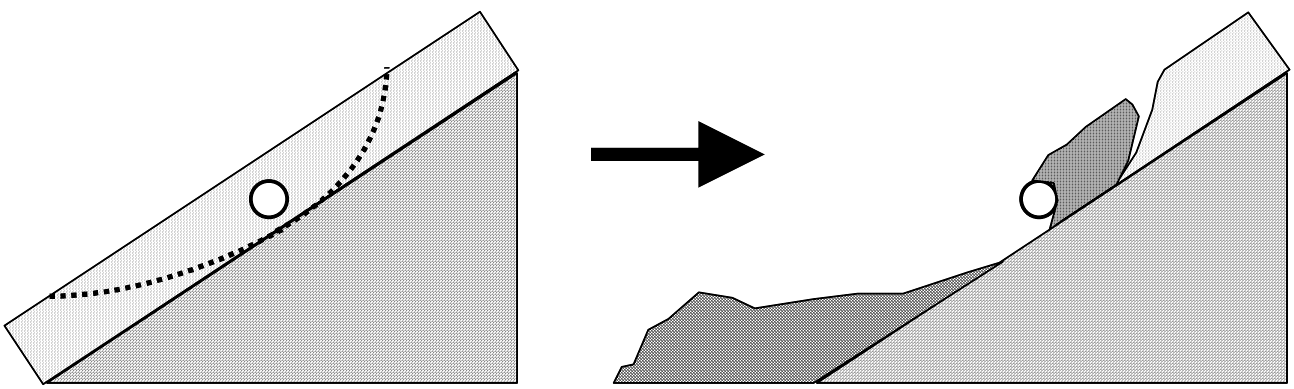
(b) Comparison of Test Data on Wrinkling and Maximum Moment

- Equation (3-1)
- Equation (3-3a) for X70, $p/p_y = 0.72$, $\Delta/t = 0.10$
- △ Mohareb et al. (1994)
- ○ Ghodsi et al. (1994)
- □ Zimmerman et al. (1995)
- ◇ Dorey et al. (2001)

Figure C3.1 Comparison of Recommended Compression Strain Limits with Test Data



(a) Pipe Span Created by Landslide or Lateral Spread



(b) Soil Load Created by Shallow Soil Slide

Figure C3.2 Examples of Load-Controlled Configurations for Buried Pipelines

C4.0 PIPELINE ANALYSIS PROCEDURES

Experience in the oil and gas industry with respect to the analytical evaluation of buried pipeline response to large permanent ground deformation dates back to the mid-1970s. This experience includes simple evaluation methodologies as well as more sophisticated finite element approaches. A brief summary of this history and the experience of pipelines undergoing large ground deformations is presented as a means to introduce the discussion of analytical methods and results in these guidelines.

Differences in the relative stiffness between the pipe and the soil lead to relative pipe-soil movements. Large relative movement is an indication that the pipeline has considerably more strength than the surrounding soil at that particular location. Therefore, high-strength pipeline material is generally more beneficial in resisting highly localized soil deformation.

The geometry of the pipeline may allow for concentration of loading and relative displacement, leading to the development of locally large strains in the pipeline. Because these load conditions are produced by an imposed displacement, relatively large strains can be accepted provided the pipeline is in good condition and the girth welds are capable of developing gross-section yielding of the pipe. To accommodate the large strains, it is beneficial to have a ductile material (such as the steels considered in this evaluation) that hardens gradually between its yield and ultimate strength capacity. This allows for a more uniform distribution of strain and lessens the likelihood of local tearing of the pipe wall.

The first approach for estimating the response of buried pipelines to large ground deformations is generally credited to Newmark and Hall (1975). Their approach provided a means to quantify the amount of strain that could be accumulated in a buried pipeline crossing a fault. The basic components of the analysis are the stress-strain properties of the pipeline steel, the ground deformation geometry, and an estimate of the variation of the axial loading placed on the pipeline by the soil. The analysis assumed a straight pipeline configuration across a fault-like discontinuity.

A modification of the Newmark and Hall approach, developed by Kennedy et al. (1977), allowed the incorporation of bending effects into the analysis. The Kennedy et al. method accounted for bending by relating the lateral soil forces on the pipeline to an assumed amount of imposed pipeline curvature. The modified approach of Kennedy et al. is best suited for instances of large ground movement for which the bending strength of the pipeline, while significant, is not a dominant feature of the pipeline response. For small amounts of deformation where bending effects are significant, Kennedy et al. outlined the process for including a local bending analysis into their methodology. However, this process is cumbersome and is rarely considered practical compared to finite element methods.

Both the Newmark and Hall and Kennedy et al. approaches are practically limited to consideration of initially straight pipelines, single components of lateral offset (either vertical or horizontal), constant soil-pipeline interaction parameters and negligible pipe bending stiffness. By the late 1970s, use of nonlinear finite element techniques were accepted as the preferred means to analyze all but the most simple of problems. Finite element approaches provide a means to rapidly investigate the effects of changes in backfill characteristics, pipeline material, wall thickness, and pipeline alignment. In addition, the assumptions used to formulate closed-form approaches like those of Newmark and Hall and Kennedy et al. have not been substantiated in comparison with finite element analyses as recommended in these guidelines.

The mechanics of implementing a finite element analysis have changed little in the last 15 to 20 years. The primary advancement in performing analyses has been in the availability of powerful desktop personal computers and compatible nonlinear analysis software that accounts for material yielding and large deformations.

The recommended analysis approach models the pipeline with beam-like elements capable of capturing nonlinear material behavior and the effect of large deformations. It is preferred that the analysis software include a pipe element in its element library with the capability to model internal pressure and provide output at various circumferential locations. These characteristics limit the amount of effort necessary to define pipe properties and the effort related to interpretation of the output from the analysis.

The recommended analysis approach can not account for localized straining that occurs once pipe wall buckling initiates. The recommended approach will underestimate local and global post-buckling strains in the region of the buckle. However, because of the displacement-controlled loading, and the fact that research into pipeline behavior has not, to date, been able to develop monotonically increasing strains of sufficient magnitude to produce rupture in a buckle (i.e., no rupture except by burst testing), this is considered acceptable. Also, refined calculations into the post-buckling region, based upon the use of nonlinear shell elements, have not yet been established to the point where they can be considered to be design tools.

C4.1 Pipe Element Definition

The one-half diameter limitation on the length of pipe elements in areas of high strain is related to the gauge length typical of full-scale pipeline tests. Shorter pipe elements promote the localization of plastic strains that have been observed in the tests.

Stress intensification and flexibility factors are based upon elastic pipeline response and are not applicable to analysis of nonlinear pipe behavior. The use of multiple straight pipe elements to represent bends and elbows introduces localization of strains that may be overly conservative considering the greater flexibility typical of elbows. However, the beneficial effect of elbow flexibility decreases with increasing D/t and internal pressure. Conservatism in the treatment of elbows is warranted because of the lack of sufficient tests to determine applicable strain limits for elbows. However, tests on Grade B elbows have demonstrated that they have the ability to withstand severe deformation while maintaining internal pressure integrity (Yoshizaki et al., 1998).

The use of sections of elbows cut to provide angle changes other than standard 90° or 45°. The wall thickness of elbow fittings is typically not uniform around the circumference will vary depending among different manufacturers. Because of these factors, sections of elbow fittings require special attention because of the potential for a significant mismatch in wall thickness between the pipe and the elbow section. This mismatch in wall thickness can lead to stress risers and reduce the strain capacity of the welded connection.

The length of the pipeline model should be sufficient to adequately capture the anchoring effects of the soil outside the zone of ground movement. Extending the pipeline model a considerable distance outside the zone of ground movement typically does not significantly lengthen the solution time for the analysis as the pipeline response is mostly elastic. In cases where a particular analysis program is impacted by an extended pipeline model, a shorter length can be used if analyses are performed to confirm that longer models do not appreciably change the maximum computed strains.

It is assumed that the pipeline analyses are performed using a multi-linear uniaxial stress-strain curve to represent the pipeline material. The procedure for defining the compressive stress-strain curve is an attempt to capture trends exhibited in full-scale pipe tests. Pipeline test results demonstrate an abrupt softening in the moment versus curvature behavior of unpressurized pipelines strained beyond the point of maximum moment capacity (Yoosef-Ghodsi et al., 1994, Mohareb et al. 1994, Zimmerman et al., 1995).

Not capturing the reduction in moment capacity that accompanies formation of a local buckle in the pipe wall is not a significant shortcoming. As discussed in Section C4.2, the representation of the soil loading does not account for the reduction in maximum soil load at large relative pipe-soil displacement. The use of a constant maximum soil load forces continued deformation of the pipeline once yielding is reached. Localization of bending strains is not a concern because the pipeline is limited to the displacement experienced by the surrounding soil.

C4.2 Pipe Stress-Strain Definition

(No additional commentary provided)

C4.3 Soil Spring Definition

Relative movement of the surrounding soil with respect to a buried structural element imparts loads on that element. The magnitude of these loads was initially investigated to understand the performance of footings, piles and soil anchors. In the early to late 1970s, several research programs developed soil loading relationships specifically for buried pipelines. A key characteristic of soil loading is that it increases only to the point at which gross failure of the soil occurs. For example, the maximum lateral load that can be imparted to a buried pipe is related to the load necessary to develop a failure plane in the soil. Once this load has been reached, further relative displacement serves primarily to move soil along the failure plane.

The limited nature of soil loading on the pipeline is commonly represented in analytical approaches by modeling the soil with bilinear springs. This approach is conservative because forces necessary to carry soil along the established failure planes are typically less than those needed to initially generate the failure plane in the soil. An approximate analogy in mechanics is the relationship between static and sliding friction.

It is possible to estimate the reduction in soil loads for large relative displacements based upon soil mechanics analyses or test data. The analysis approach using beam-like pipe elements in the analysis of pipe response is not compatible with such soil loading representations because of the inability to capture the reduction in moment capacity at large pipe strains. If a more refined estimate of soil loading is used, the modeling of the pipe should be modified to capture the localized straining that occurs in the vicinity of a buckle in the pipe wall.

There is considerable uncertainty regarding the relationships used to compute soil spring properties. Most of this uncertainty is related to estimates of the soil strength parameters. The uncertainty in estimating soil strength parameters for pipeline analyses in cohesive soils is further complicated by the fact that pipelines are typically located above the water table and within the desiccation zone of the soil. The strength of partially saturated desiccated soils is not well defined in soil mechanics practice.

The equations provided for the parameters N_{qh}, N_{ch}, N_{qv} N_{cv}, and N_{γ} are developed from empirical curves found in the literature. The equations are particularly useful when implementing calculations in a spread sheet or computer program. The curves from which the equations were developed are provided in Figures C4.1 to C4.4.

Most of the soil loading relationships in these guidelines are nearly the same as those recommended by the Gas and Liquid Fuel Lifelines Committee of the ASCE Technical Council on Lifeline Earthquake Engineering (ASCE, 1984). One exception is the relationship for estimating axial soil loads in cohesive soils. These guidelines utilize the adhesion curve developed by Honegger (1999b).

There is considerable recent research supporting the use of the approach in these guidelines for estimating soil loads. Examples include centrifuge modeling tests (Paulin et al., 1995, 1996), field tests (Cappelletto et al., 1998, Honegger, 1999b, Rizkalla et al., 1996), and finite element analysis (Altaee et al., 1996). There is some evidence from laboratory tests that the lateral soil loads determined by the equations in these guidelines may overestimate the maximum soil loads (Paulin et al., 1998). However, this evidence is from an ongoing proprietary research program, and there are a number of technical issues relating to modeling similitude and load rate that require resolution. Until the findings from this proprietary research become available for review or other data are published from tests at seismic loading rates, the expressions contained in these guidelines represent the best approaches available. As an alternative, it is always permissible to utilize test data for a particular soil condition.

C4.4 Extent of Pipeline Model

(No additional commentary provided)

C4.5 Representation of Applied Ground Movement

The use of abrupt transition in the applied ground movement at faults and boundaries of lateral spreads and landslides is required in these guidelines unless site-specific information is available to justify other patterns. This is not an unrealistic assumption based upon the observation of abrupt transition patterns in past earthquakes. The assumption of an abrupt transition is conservative in that the computed strains are likely to be substantially greater than those computed assuming a more gradual transition. This approach is consistent with the representation of soil loading and analysis methods used in these guidelines. That is, the definition of ground deformation hazard, as represented by the soil loads and deformation pattern, is conservative while the assessment of pipeline response, represented by the strain criteria and pipeline modeling approaches, is believed to be more median-centered.

An alternate approach for expressing ground displacement based upon observations from Japanese earthquakes is to assume a pattern defined by a cosine function raised to the power of n as given in equations (C4-1) and (C4-2).

For slide-type displacements:

$$\begin{aligned} y(x) &= 1-\delta\left(\cos\frac{\pi x}{W_s}\right)^n && \text{for } \frac{x}{W_s} \le 0.5 \\ &= 1- \delta\cdot\left(\cos\left[\pi\left(1-\frac{x}{W_s}\right)\right]\right)^n && \text{for } \frac{x}{W_s} > 0.5 \end{aligned} \qquad \text{(C4-1)}$$

For fault-type displacements:

$$y(x) = 1-\delta\left(\cos\frac{\pi x}{2W_f}\right)^n \qquad \text{(C4-2)}$$

where:

$y(x)$ = displacement as a function of location, x, within the zone of deformation

x = distance across the zone of deformation

H = rupture depth, km

δ = amount of displacement

W_s = width of slide zone

W_f = width of fault zone

The cosine functions given by equations (C4-1) and (C4-2) are graphed in Figure C4.5 for various powers of *n*.

A displacement pattern corresponding to an *n* of 10 is representative of slide and fault displacements. A more abrupt transition, approximated by an *n* value greater than 100 has also been observed in some earthquakes. Other displacement patterns established based upon historical observation or rational analysis may be used for characterizing ground movements.

Practically, current empirical or analytical methods for quantifying seismic ground movement hazards are also not capable of quantifying the behavior at boundaries. The most appropriate basis for estimating more gradual transitions in ground movement is historical evidence. This type of information is typically limited to areas of high seismic activity where a historical record of observed ground deformations exits. If the basis for a more gradual transition is judgment, the sensitivity analyses should include an abrupt transition in ground deformation pattern.

C4.6 Sensitivity Analyses

Sensitivity analyses are recommended to provide some indication of the sensitivity of computed pipeline strains to various input parameters. The primary reason for performing sensitivity analyses is to assess the impact of uncertainty regarding definition of the earthquake hazard and soil loading parameters.

The recommended variations in these guidelines are based upon the premise that the soil spring definitions in these guidelines tend to overestimate soil loads. Some determination of the expected variability in modeling parameters should be included in the scope of any assessment. While higher soil loads generally produce more severe pipeline distortion, there are certain exceptions. A simple example illustrates this point.

Consider a pipeline alignment shown in Figure C4.6 subjected to right-lateral strike-slip faulting. The pipeline has an induction bend west of the fault and a 90° 3R elbow on the east side of the fault approximately 350 m from the induction bend. Given the shallow crossing angle, the response of the pipeline is governed by axial loads between the two bend locations. With this configuration, the two bends have the potential to behave as anchors. Overestimating the axial soil loads on the pipeline will tend to focus high strains at the induction bend. As the pipeline strains axially, the soil loads gradually reduce and may be insignificant at the 3R elbow if the total axial strain is sufficient to allow no relative displacement at the elbow. If the soil loads are underestimated, the soil will tend to slide along a much greater length of pipeline because the strains induced by the low soil loads are not sufficient to match the ground displacement. In this case, the strains at the induction bend may be reduced but one leg of the 3R elbow will be forced by high lateral soil loads to match the difference between the ground displacement and the axial pipe elongation. In effect, soil reaction forces developed at the 3R elbow will be pulling the pipe between the bends to accommodate the displacement mismatch. This condition can shift the most critical location of pipeline straining from the induction bend to the 3R elbow.

This example demonstrates that possible variations in soil spring definition can not only define the variability in computed strains but may also indicate a potential for alternate critically loaded portions of the pipeline alignment. When selecting variations of soil springs for sensitivity analyses, the potential for such behavior should be considered. This may involve a more extreme variation in soil spring definition and a separate assessment to determine the likelihood of experiencing such an extreme variation.

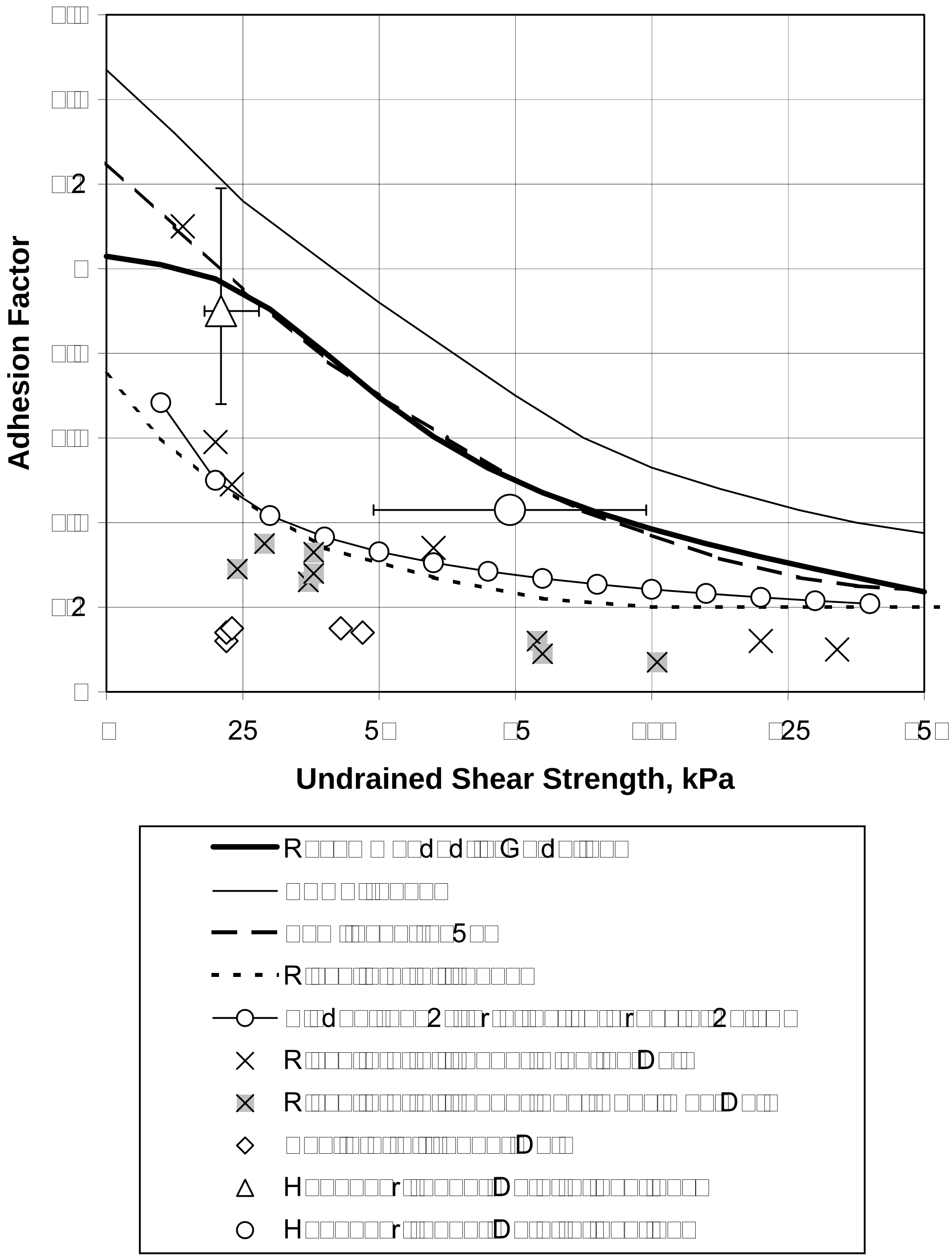


Figure C4.1 Plotted Values for the Adhesion Factor, α

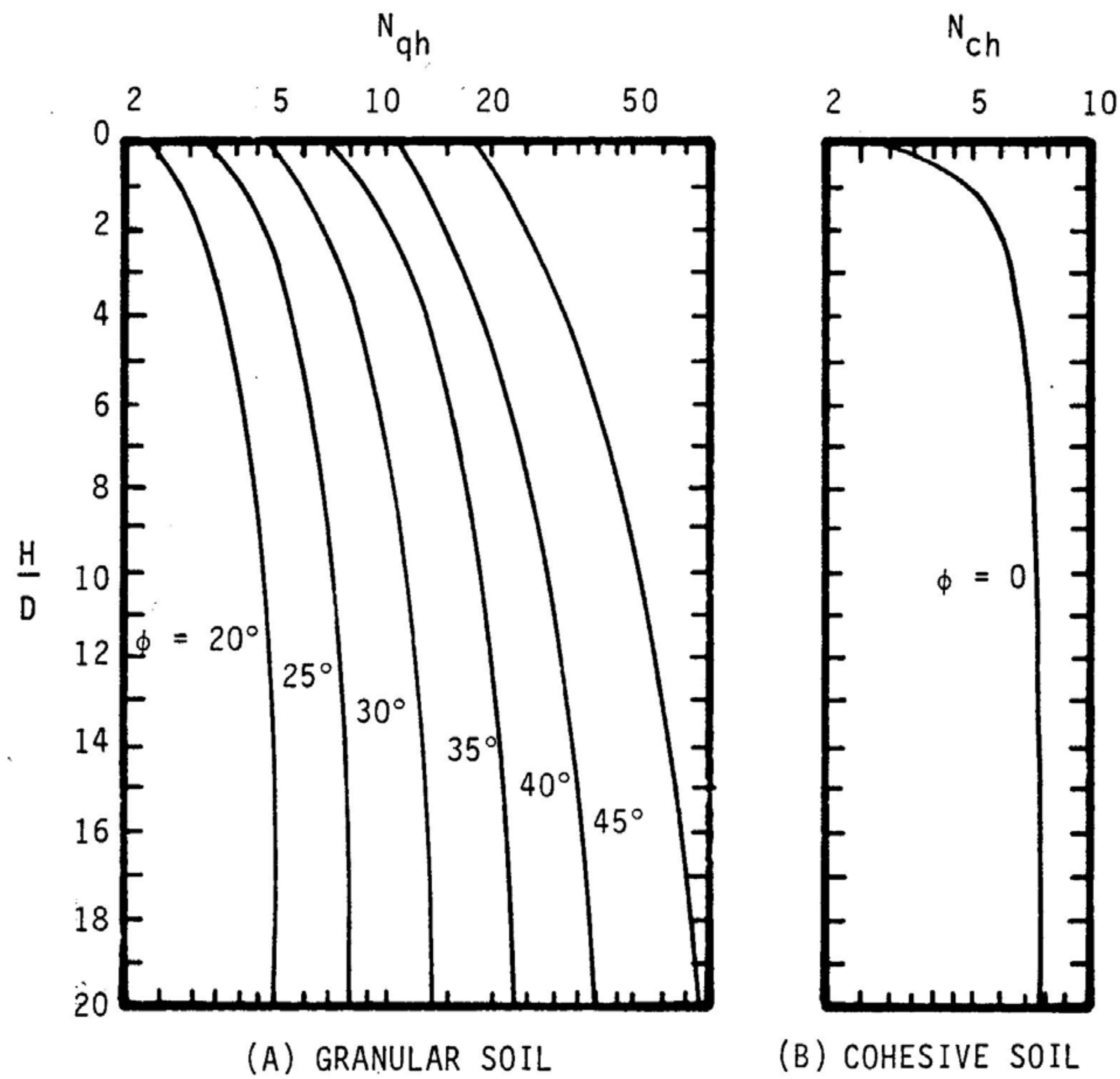
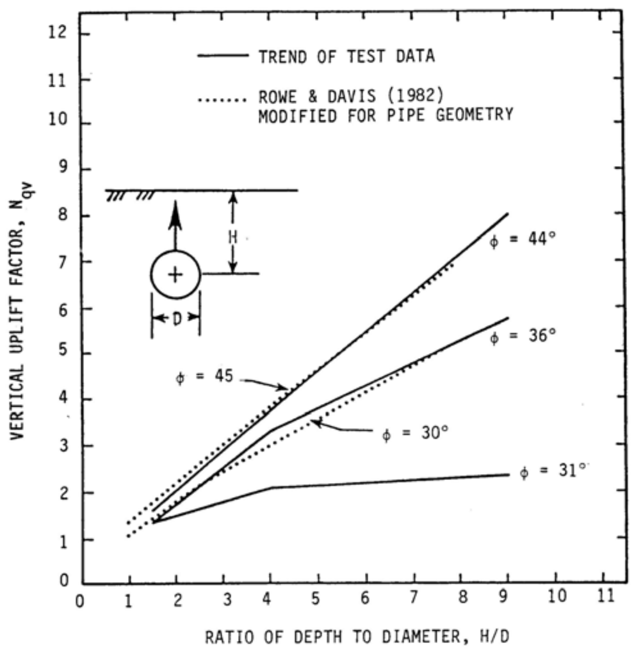


Figure C4.2 Values of N_{qh} and N_{ch} (from Hansen, 1961)



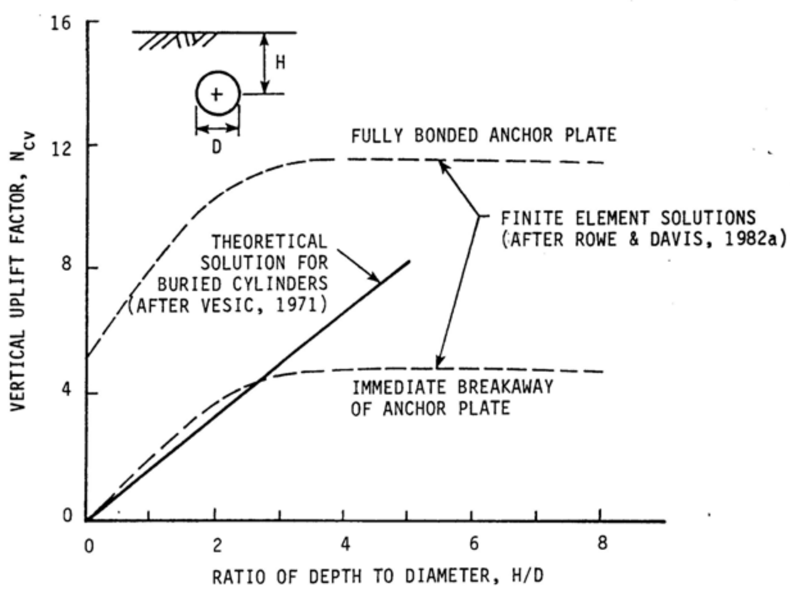


Figure C4.3 Ranges for Values of N_{qv} and N_{cv} (from Trautman and O'Rourke, 1983)

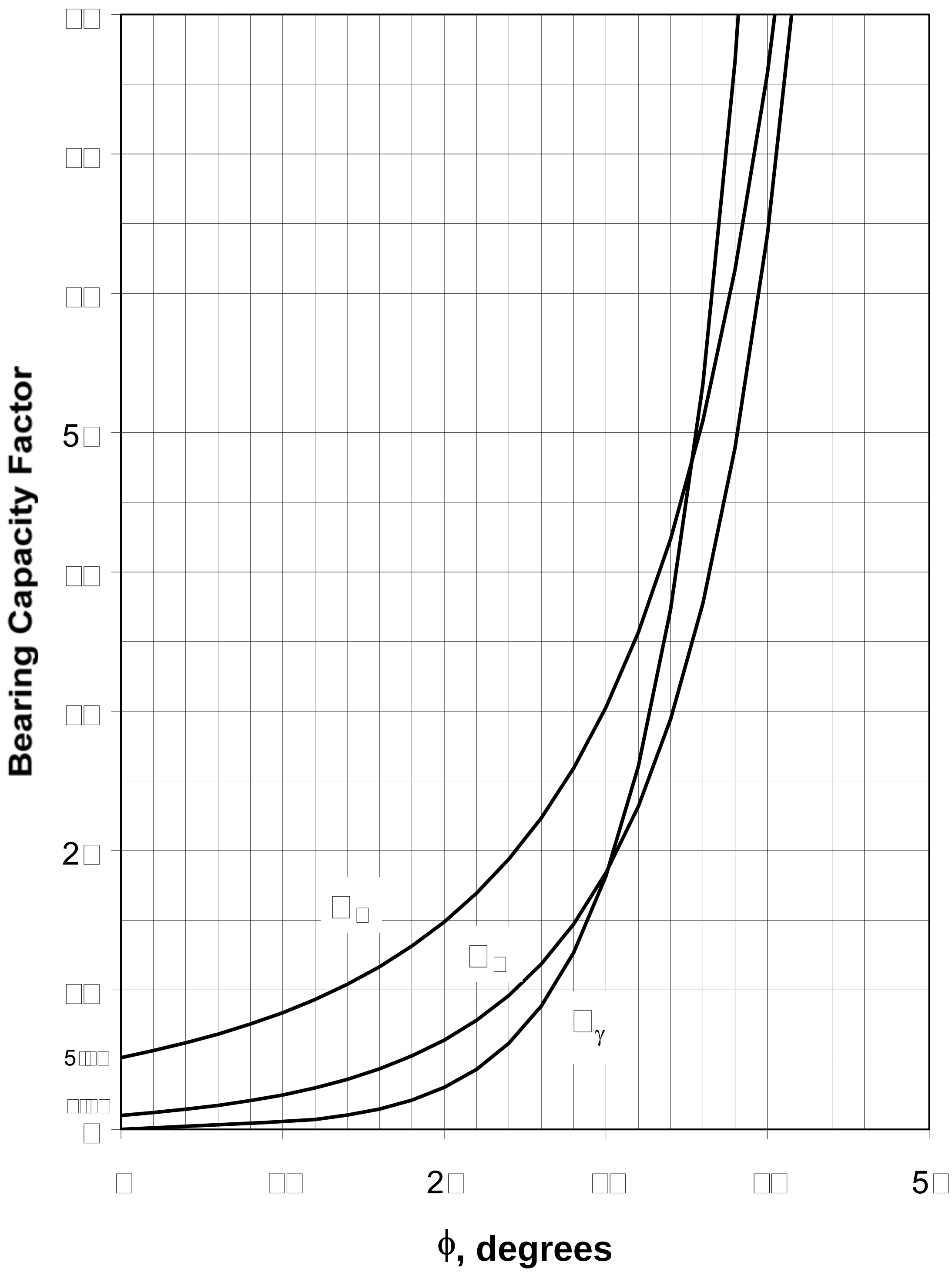
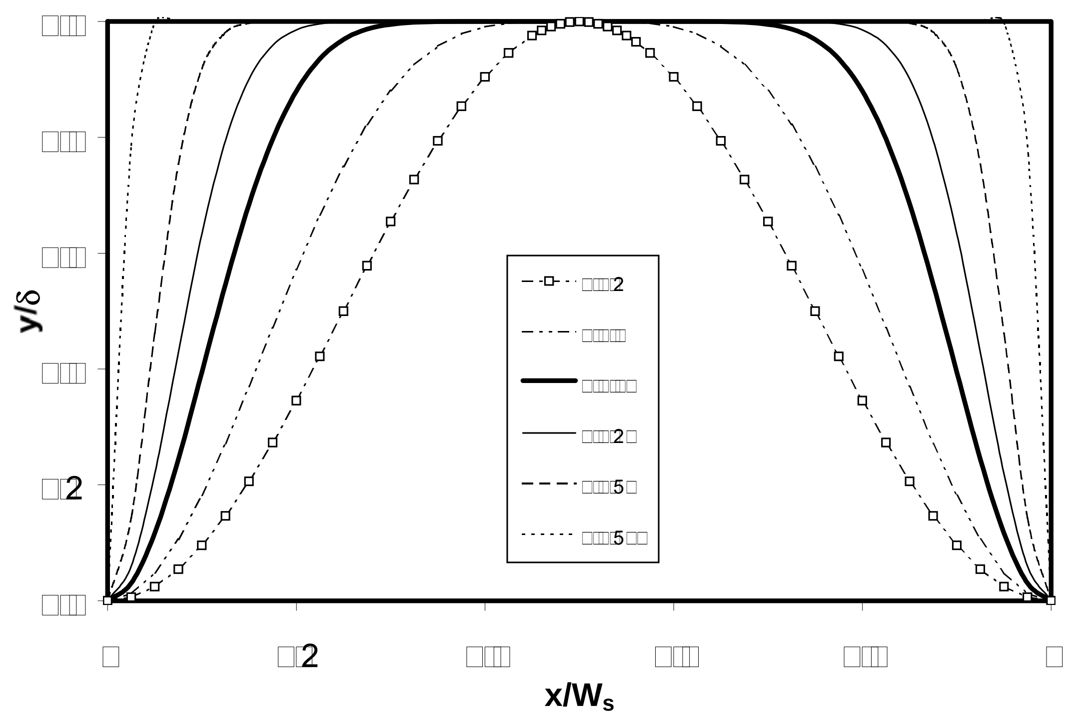
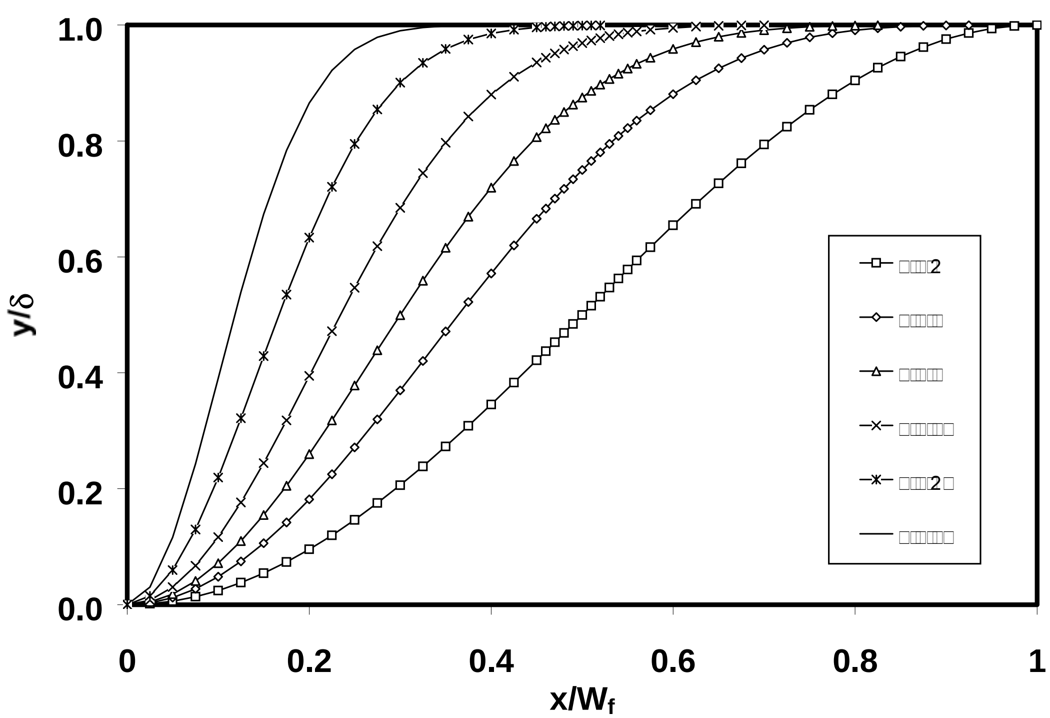


Figure C4.4 Plotted Values of Bearing Capacity Factors (N_q, N_c, and N_γ)



a) Cosine Function Representation of Slide Displacement



b) Cosine Function Representation of Fault Displacement

Figure C4.5 Cosine Function for Representing Variation in Ground Displacement Patterns

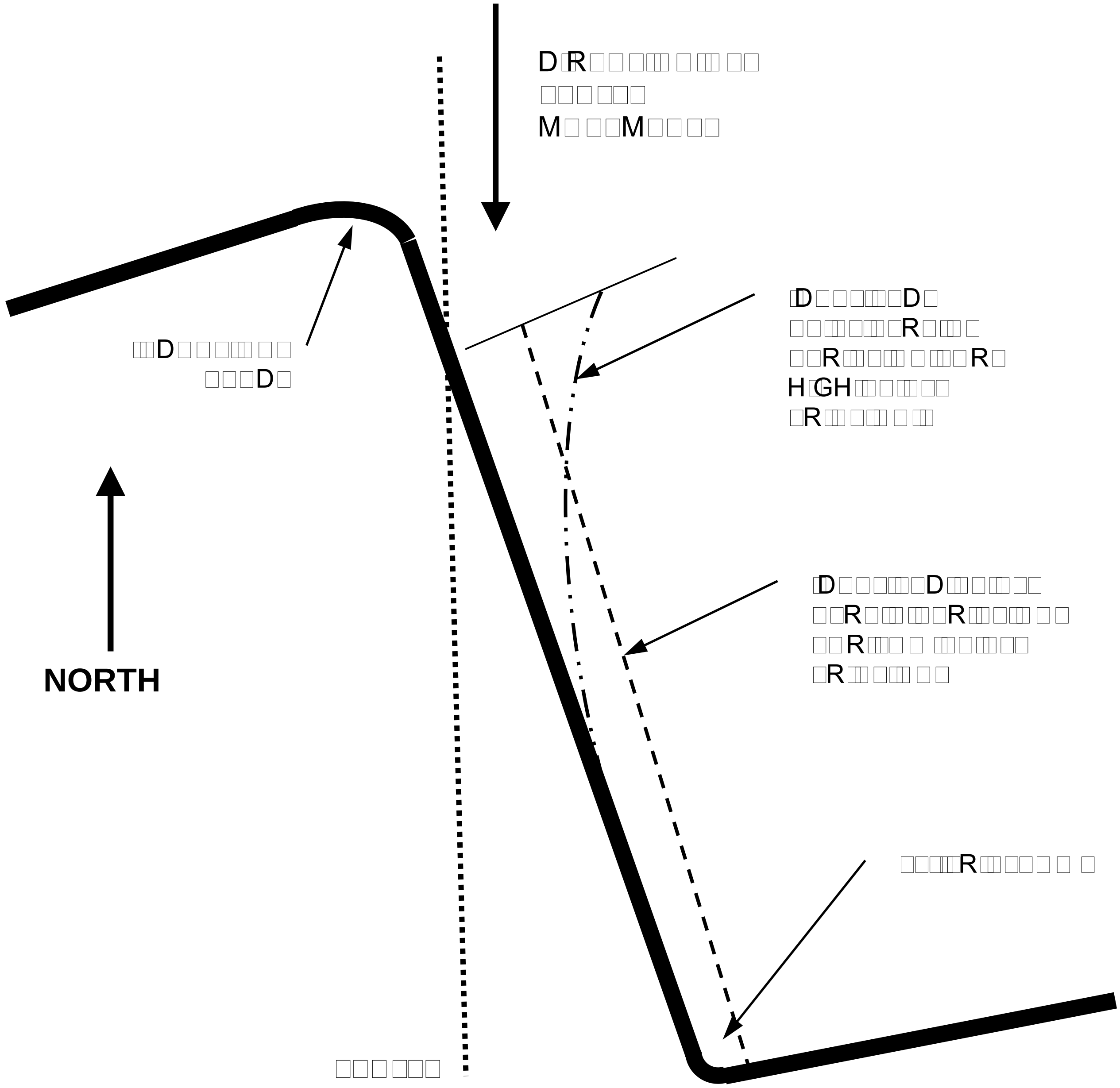


Figure C4.6 Example of Pipeline Configuration Illustrating Consequences of Variability in Soil Spring Forces

C5.0 MITIGATION OPTIONS

As with pipeline design and assessment guidelines, the benefits of implementing mitigation measures have generally not been demonstrated by experience in actual earthquakes or in controlled tests. A notable exception being the limited field testing performed to validate the concept of dual layers of geotextile wrapping. The presentation of mitigation options is limited to a general discussion of various techniques that have been implemented or considered in the past. For this reason, no additional discussion of potential mitigation options is provided in this commentary.

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CONSIDERATIONS FOR ESTABLISHING PERFORMANCE GOALS

Existing pipeline standards emphasize minimum requirements for normal design, construction, and operation. Many pipeline companies exceed the requirements stipulated by the standards for safety and reliability. With regard to seismic hazard mitigation, the appropriate level of seismic design is typically determined on a project-by-project basis using site-specific information to define the seismic hazard exposure and the current state-of-practice. In practice, this often results in the definition of seismic performance goals for the pipeline being based largely upon criteria adopted from other non-pipeline projects, with similar consequences to the public.

A fundamental question to be answered in performing a seismic assessment for a new or existing pipeline is what level of performance is necessary. These guidelines assume a definition in terms of an annual probability of exceeding an unacceptable condition such as excessive pipe deformation or loss of pressure boundary integrity. This probability is then equated to the probability of occurrence of the earthquake.

A.1 Objectives

The main objective of seismic design for natural gas transmission and liquid hydrocarbon pipelines is to provide life/safety protection from the effects of stipulated design earthquakes distributed along the pipeline route. A secondary, but important, objective is to minimize capital loss and disruptions to operations.

The goal for seismic design of natural gas transmission and liquid hydrocarbon pipelines is to achieve a balanced design to withstand the effects of earthquakes and other loads which is both safe and economically feasible. The design should take into account the nature and importance of the project, cost implications, and risk assessment centering around such items as public safety, loss of product or service, and damage to property and the environment. The treatment of the seismic hazards should be consistent with the treatment of other natural and manmade hazards.

The concept of a balanced design may be difficult to achieve because many of the parameters necessary for making rational decisions are either unknown or not well defined. The goal of a balanced design is for all major components of a pipeline system to have failure risks and consequences that are consistent with system performance objectives. Pipelines and support facilities should be designed to withstand the effects of earthquake ground shaking and permanent ground movements that have a reasonable probability of occurrence along the route during their operational life.

A.2 Dual-Level versus Single-Level Earthquake Hazard Definition

The use of dual-level earthquake design criteria is an outgrowth of practices developed for the nuclear power industry and other projects (e.g., offshore structures) where seismic damage has the potential for severe safety or environmental consequences. The origin of the two-level design earthquake was primarily intended for the design of buildings and surface structures, with their contained electrical and mechanical systems and aboveground piping systems. The first major pipeline project to adopt dual-level earthquake criteria was the Trans-Alaska pipeline, but design implementation was restricted primarily to aboveground pipeline segments.

Current practice for the seismic design of major pipeline facilities (new construction) often follows the precedent set for offshore oil production platforms, Department of Energy critical facilities, and nuclear power plants in that two levels of earthquake hazard are selected for design. In this approach, the low level event generally has a return period on the order of 200 to 500 years. The higher level event is generally one that has a return period on the order of 1,000 years or more, depending upon the nature of the facility.

The lower level event is often viewed as a threshold for continued operation of the pipeline within accepted safety margins and little if any permanent deformation of the pipeline is permitted. The pipeline response to higher level event is typically required to prevent a threat to safety, although significant structural damage could occur. Specific criteria for the response of the pipeline to these events, including the amount of permissible damage, varies according to the type of structure or component and its function. The selection of earthquake return periods and performance objectives should be established through a project-specific risk assessment.

There is no standard nomenclature for referring to the two earthquake levels. The lower level event is often referred to as the "probable design earthquake" (PDE), the "design basis earthquake" (DBE), or the "strength level earthquake" (SLE). The higher level event is commonly referred to as the "contingency design earthquake" (CDE), "maximum credible earthquake" (MCE), "safe shutdown earthquake" (SSE), or "ductility level earthquake" (DLE).

For the design or assessment of buried pipelines it is not clear whether a two-level design is necessary or practical in many cases. The fundamental difference in the condition expected of buried pipelines and typical structures and equipment following the low-level earthquake should be considered if a dual-level earthquake design is specified. Establishing a lower level earthquake for structures and equipment generally assures that the post-earthquake condition is essentially the same as the pre-earthquake condition for the less severe earthquake. This is expected because their primary earthquake loading condition is a transitory inertial excitation. However, the primary earthquake load on buried pipelines is permanent ground displacement. The pipeline's total strain capacity to resist ground displacement in the future is reduced because of the strains produced by the low-level earthquake displacements. Also, it is possible that landslide and lateral spread hazards are not direct functions of the level of ground shaking necessary to trigger large displacements. These factors should be considered with respect to the purpose of specifying a dual-level earthquake to provide an increased likelihood of normal operation for moderate events and prevent loss of pressure integrity for major events.

The decision on whether or not to specify a dual-level earthquake design for a buried pipeline may also be influenced by considering measures that may be required following an earthquake that produces permanent ground displacement. Following the earthquake, a decision will be necessary regarding the suitability of the pipeline for continued short-term and long-term service. Considerations impacting this decision include the consequences if the pipeline performance is compromised by future ground displacement, the likelihood for future earthquake generated ground displacement during the life of the pipeline, the remaining capacity of the pipeline to withstand future ground displacement, and the consequences to customers if pipeline operations are interrupted.

Given that this decision process generally occurs regardless of the level of ground displacement experienced by the pipeline, it is not clear what benefits are to be gained by specifying two levels of earthquake hazard. This is especially true for cases where analysis provides information on pipe strains as a function of ground displacement such that the pipeline response for lower level events is understood.

Another consideration relates to the assessment of permanent ground displacement patterns after an earthquake when the ground displacement was insufficient to lead to loss of pipe contents. With the exception of well-defined abrupt surface expressions of displacement, it is often necessary to compare the positions of surface features following the earthquake that were known before the earthquake in order to evaluate the extent of ground deformation. Examples of such surface features include roads, sidewalks, fence lines, and survey monuments. In the case of pipelines within a liquefiable soil experiencing lateral spread movement, the problem is compounded by the fact that the ground movements within the liquefiable layer may not be well represented by surface ground displacement. In most earthquakes, detailed surveys are necessary to determine the amount of ground displacement. In remote locations, it is not unusual to have no pre-existing features from which to ascertain ground displacement. In such cases, assessment of pipeline condition is often qualitative and relies on limited excavation of the pipeline to observe any signs indicative of large strains (e.g., wrinkles, damaged coating, gaps between the soil and the pipe, ovaling of the pipe). It may also be decided to use an instrumented pigging device to identify any abnormalities in the pipe wall or alignment. In any case, assessing pipeline response to ground displacement following an earthquake may take several days or weeks.

These problems can be reduced somewhat by performing surveys to locate surface features or to install survey monuments prior to the earthquake in the vicinity of areas expected to experience ground displacement. Post-earthquake surveys using GPS equipment can then provide a relatively rapid assessment of the amount of ground displacement that has occurred in the vicinity of the pipeline. This entails some long-term costs for verifying the survey and maintaining survey monuments.

Whether a pipeline is to be constructed onshore or offshore may dictate the need to account for differences in the consequences of seismic damage when establishing performance goals. Seismic damage to an offshore pipeline would result in a more difficult repair situation than for an onshore pipeline, particularly in an extreme climate or deep water, or in cases where the pipeline is deeply buried to protect against anchor dragging, ice gouging, etc. The increased difficulty is due mainly to mobilizing offshore equipment and gaining access to the pipeline. Considering the differences in damage consequences for offshore and onshore pipelines and production platforms, less frequent seismic hazards (larger earthquake hazards) should be considered for offshore pipelines than for onshore pipelines. This can be achieved by selecting annual probabilities of earthquake hazards for offshore pipelines that are less than what would be selected for onshore pipelines (i.e., longer average return periods for seismic events).

If life safety is the primary criteria, the annual probability for experiencing a design earthquake hazard for an offshore pipeline may not need to be as low as for an offshore production platform, because pipeline damage does not generally lead to direct life-safety consequences. Pipeline failures tend to be localized (at one or more locations) and repairs might be implemented in a relatively short time frame if the water depth is not great. Such disruptions would have limited adverse consequences with respect to life safety. A very important exception is offshore oil pipelines and pipelines located in very deep water for which the environmental consequences and costs associated with pipeline repairs may be very severe. In these cases, the design criteria may be more stringent that what is necessary to meet established life-safety performance objectives.

A.3 Perspective on Seismic Risk

The principal consequence of the rupture of a natural gas transmission pipeline is the potential for release and subsequent ignition of gas in areas where there is a potential for injury or significant property damage. Other consequences include service interruption and the cost of repair and cleanup. Rupture of liquid hydrocarbon pipelines may have severe environmental consequences that are of equal or greater concern to the pipeline owner and govern the conservatism in defining the earthquake hazard. One of the most important and difficult issues associated with earthquake risk mitigation for natural gas and liquid hydrocarbon pipelines is the determination of the appropriate level of investment in seismic hazard mitigation versus the benefit that would be derived. Costs and benefits are difficult to assess due to the limited knowledge of earthquake recurrence, the uncertainty involved in projecting pipeline damage, and the quantification of the impact of pipeline service interruptions.

Pipelines are subject to risk of damage due to a number of causes totally unrelated to earthquakes. Among these are corrosion, outside force, material failure, and construction defects. An analysis of reportable incidents for natural gas transmission and gathering lines in the U.S. (Jones et al., 1986) indicates that approximately 685 failures, classified as propagating ruptures, punctures, blowouts or tears, occurred over a 14.5-year period. The total length of natural gas transmission pipelines during this period was about 500,000 km. This leads to an average of approximately 1×10^{-4} failures per kilometer of pipeline per year. For a postulated 50-year service life, this failure rate equates to approximately 5×10^{-3} failures per kilometer or one failure per 200 km (Nyman and Hall, 1991).

If one considers the hypothetical case of an earthquake with an estimated recurrence interval of 500 years causing damage over a 100-km segment of the pipeline route, five such pipeline failures would be comparable to the statistics for normal operation. This failure rate probably represents an upper bound for most welded steel pipelines. Admittedly, this analogy is rough and approximate, but it does illustrate the order of magnitude of the relative threat posed by strong earthquakes compared to normal operation.

In light of the foregoing example, it seems appropriate that improvement of seismic resistance through capital expenditures should be tempered by recognition of the hazards associated with day-to-day operations. While mitigation of the seismic threat through design is usually the preferred alternative, it may be more reasonable in certain cases to focus attention on minimizing the consequences of destructive earthquakes. In particular, the safety impact of extensive pipeline damage can be reduced through improved line break isolation, earthquake contingency plans, repair plans, and seismic hardening of control systems and communications.

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SEISMIC DESIGN PROVISIONS IN EXISTING STANDARDS

Requirements for liquid hydrocarbon pipelines in the United States are set forth in Title 49 of the Code of Federal Regulations (CFR), Parts 192 and 195 (U.S. Department of Transportation, 1998a, 1998b), which regulate the design, construction, operation, and maintenance of natural gas and crude oil pipelines, respectively. ASME B31.4 and ASME B31.8 are consensus standards developed in accordance with the requirements of the American National Standards Institute (ANSI) which have been developed and maintained by technical committees of the American Society of Mechanical Engineers (ASME) (1998, 2000). The ANSI standards are supplemented by industry standards such as API Recommended Practice 1111 for the design construction, operation, and maintenance of offshore hydrocarbon pipelines (American Petroleum Institute, 1993).

In Canada, liquid hydrocarbon and natural gas pipelines are designed in accordance with Canadian Standards Association (CSA) Z662, Design of Oil and Gas Pipeline Systems. CSA Z662 covers both onshore and offshore pipelines.

In Europe, two standards are commonly used to control the design and operation of offshore pipelines: British Standard BS 8010, Code of Practice for Pipelines, Part 3 (British Standards Institute, 1993), and Rules for Submarine Pipeline Systems published by Det Norske Veritas (DNV) (2000).

None of the aforementioned standards provide specific requirements for seismic design of crude oil and natural gas pipeline systems. Typically, these standards acknowledge that consideration should be given to the various types of seismic hazards, but they do not prescribe specific criteria or methodology. Reference is often made to “giving consideration to,” “providing reasonable protection,” or “taking reasonable precautions,” but the level of design treatment is undefined. The responsibility for assuring that adequate provisions have been made to address seismic hazards is delegated to the pipeline designer and the appropriate regulatory body.

The references made by the above-listed standards to seismic design is summarized below.

B.1 ANSI B31.4, Pipeline Transportation Systems for Liquid Hydrocarbons and Other Liquids

ANSI B31.4 is the U.S. standard for the design, construction, operation, and maintenance of liquid hydrocarbons pipelines, both onshore and offshore. ANSI B31.4 makes only limited reference to seismic design by acknowledging the need to consider certain situations involving earthquake-induced loads. No specific criteria, requirements or guidelines are offered. The major objective is to create a general awareness of the need to evaluate seismic hazards.

1. Paragraph 401.5.3, *Earthquake* (relating to dynamic effects). “Consideration in the design shall be given to piping systems located in regions where earthquakes are

known to occur." This paragraph acknowledges the need for seismic design, but offers no guidance or specific requirements.

2. Paragraph 402.1, *General* (relating to design criteria). "… the design engineer shall provide reasonable protection to prevent damage to the pipeline from unusual external conditions which may be encountered in river crossings, offshore and inland coastal water areas, bridges, areas of heavy traffic, long self-supported spans, unstable ground, vibration, weight of special attachments, or forces resulting from abnormal thermal conditions." This provision goes on to state that protective measures, such as increasing the wall thickness, may be used. This provision simply acknowledges the possibility of extreme events for which some special protection is warranted. No specific requirements are set forth for any of these load conditions, nor is there any delineation of what constitutes "reasonable protection."

3. Paragraphs 402.3.3, *Limits of Calculated Stresses Due to Occasional Loads*, and 419.6.4, *Stress Values.* "The sum of the longitudinal stresses produced by pressure, live and dead loads, and those produced by occasional loads, such as wind or earthquake, shall not exceed 80% of the specified minimum yield strength of the pipe…" This limit is intended for aboveground piping or elevated pipeline spans and is not applicable to extreme situations involving large ground movements due to faulting or liquefaction.

B.2 ANSI B31.8, Gas Transmission and Distribution Piping Systems

ANSI B31.8 is the U.S. standard for the design, construction, and operation of onshore and offshore gas transmission and distribution systems. In a manner similar to the ANSI standard for liquids lines (B31.4), the ANSI B31.8 standard for gas transmission and distribution lines makes very limited reference to seismic design of onshore pipelines, but has more extensive treatment of seismic hazards (without specifics) for offshore pipelines (B31.8, Chapter 8).

The B31.8 provisions that mention seismic design are as follows:

1. Paragraph 833.4(c), relating to the calculation of longitudinal stress. External loads such as weight and wind that contribute to longitudinal bending stress for aboveground piping are mentioned. This section applies to primary seismic inertial loads but not to secondary loads on buried piping.

2. Paragraph 841.13, *Protection of Pipelines and Mains from Hazards*, Item (a). "When pipelines and mains must be installed where they will be subject to natural hazards, such as washouts, floods, unstable soil, landslides, earthquake related events (such as surface faulting, soil liquefaction, soil and slope instability characteristics), or other conditions which may cause serious movement of, or abnormal loads on, the pipeline, reasonable precautions shall be taken to protect the pipeline, such as increasing the wall thickness, constructing revetments, preventing erosion, and installing anchors." This paragraph effectively identifies the potential for seismic hazards but does not

prescribe any specific criteria or requirements for design or operational precautions or what constitutes “reasonable precautions.”

3. Paragraph A835, *Anchorage for Buried Piping*. “When a submerged pipeline is to be laid across a known fault zone, or in an earthquake-prone area where new faults are a possibility, consideration shall be given to the need for flexibility in the pipeline system and its components to minimize the possibility of damage due to seismic activity.”

4. Paragraph A841.1, *Design Conditions*, Item (f). Seismic activity is identified as a prospective offshore design condition.

5. Paragraph A841.33, *Design Environmental Conditions*, Item (d). Seismic events are identified as a prospective offshore environmental load condition.

6. Paragraph A842.23, *Alternate Design for Strain.* This paragraph permits design to strain criteria in excess of yield strain for situations where the pipeline experiences a predictable, noncyclic displacement of its support and provided the consequences of yielding are not detrimental to the integrity of the pipeline. The paragraph goes on to say that permissible strain criteria should be determined on the basis of available ductility and buckling behavior. This provision of B31.8 is quite important in that it offers confirmation of the appropriateness of using strain limits as the design criteria for pipelines subjected to seismic ground movements, albeit such limits are undefined.

7. Paragraph A843, *On-Bottom Stability.* “Pipeline design for lateral and vertical stability is governed by seafloor bathymetry, soil characteristics, and by hydrodynamic, seismic, and soil behavior events having a significant probability of occurrence during the life of the system. … The pipeline system shall be designed to prevent horizontal and vertical movements, or shall be designed so that any movements will be limited to values not causing design strength to be exceeded. …Typical factors to be considered in the stability design include: …(c) liquefaction, (d) slope failure.” This paragraph states that offshore pipelines should be designed to withstand seismic effects without exceeding design strength, but there is no link back to the provision in Paragraph A842.23 for design on the basis of strain limits.

8. Paragraph A843.5, *Soil Liquefaction.* “Design for the effects of liquefaction shall be performed for areas of known or expected occurrence. …. Seismic design conditions used to predict the occurrence of bottom liquefaction or slope failure shall have the same recurrence interval as used for the operating design strength calculations for the pipeline.” This paragraph calls for the design of offshore pipelines to mitigate the effects of liquefaction and slope failure, but no specific criteria or methodology are defined.

B.3 API Recommended Practice 1111, Design, Construction, Operation, and Maintenance of Offshore Hydrocarbon Pipelines

API Recommended Practice (RP) 1111 sets out criteria for the design of offshore pipelines. It is intended to be used in conjunction with ANSI B31.4 and B31.8 standards for liquids and gas transmission pipelines. The procedures presented in API RP 1111 are essentially the same as that prescribed by the ANSI standards, and the allowable stress provisions from B31.4 and B31.8 are incorporated by reference. The mention of seismic requirements are quite limited as summarized below:

1. Section 2.1.4, *Dynamic Forces*. A general statement is made that the design should consider dynamic forces imposed on the pipeline and the resulting stresses, and that these stresses may be caused by a number of possible loading conditions, one of which is seismic activity. The provision serves only as an alert to consider seismic events as a source of dynamic loads.

2. Section 2.4.2.1, *On-Bottom Stability*. The discussion of considerations for on-bottom stability includes the effects of natural phenomena. The potential for earthquakes to cause the liquefaction of sea-bottom sediments and associated settlement or floatation of a pipeline is specifically mentioned as a potential situation requiring consideration.

B.4 BS 8010, Code of Practice for Pipelines, Part 3, Pipelines Subsea: Design, Construction and Installation

British Standard BS 8010, Part 3, provides recommendations for the design, construction, installation, testing, and commissioning of subsea pipelines. BS 8010 contains numerous references to seismic activity as an environmental load condition that must be considered. The provisions of BS 8010 that relate to seismic design are listed below:

1. Section 4.2.2.5, *Functional and Environmental Loads.* Earthquakes are identified as an environmental load condition requiring consideration, and cross-reference is made to Appendix B of BS 8010 for further discussion.

2. Section 4.2.5.4, *Equivalent Stress*. This section specifies that equivalent stress (computed in accordance with the von Mises' stress criterion) shall be less than 0.96 times the specified minimum yield stress for environmental load conditions.

3. Section 4.2.6, *Alternative Design for Strain*. This section states that the limit on equivalent stress set forth in Section 4.2.5.4 may be replaced by an allowable strain limit of 0.1%.

4. Section 4.5.4.2, *Soil Instability,* Item (a). This section states that consideration should be given to stabilization enhancement methods in locations where the ground along the pipeline route might become unstable due to a number of reasons, including seismic activity.

5. Section 4.12.1, *Possible Causes of Damage*, Item (I). Seismic activity is named as a possible cause of damage that requires consideration.

6. Section B.1.10, *Seismic Action*. This section calls for consideration of the effects of seismic ground motions along the pipeline route. Specific mention is given to the possibility of soil liquefaction. No guidance is offered on the appropriate level of treatment.

7. Section B.2.7.5, *Liquefaction*, Item (b). This section calls for consideration of soil liquefaction, and seismic action is identified as a potential cause of liquefaction.

8. Section C.1.3, *Axial Compression*, and Section C.1.4, *Bending*. These two subsections under Appendix C, Buckling, provide stress limits for local buckling.

B.5 CSA Z662-96, Oil and Gas Pipeline Systems

The Canadian standard for oil and gas pipeline systems provides stress design requirements for operating pressure, thermal expansion ranges, temperature differential, and sustained force and wind loadings. Other load conditions are not specifically addressed. The Canadian standard provides for the design of pipelines using limit state criteria defined in its non-mandatory Appendix C, although the prescribed load conditions for which the limit state criteria are to be applied are the same as in the mandatory sections of the standard.

B.6 DNV Rules for Submarine Pipeline Systems

The DNV Rules for submarine pipeline systems specify requirements for the design, construction, installation, operation, and maintenance of subsea pipelines. The DNV Rules contain several references to seismic activity as a load condition that must be considered. These provisions are summarized below.

1. Section 2.1.1, *Environmental Phenomena*. This section requires consideration of all environmental phenomena that impair pipeline system function or reduce system reliability. Seismic activity is listed as an environmental phenomenon that should be considered.

2. Section 3.3.1.3, relating to environmental loads. This section states that the return period to be taken for environmental loads shall be greater than 100 years.

3. Section B.1, *Local Buckling*. This section of Appendix B provides methodology for determining critical buckling stress based upon a Ramberg-Osgood representation of the stress-strain curve for steel.

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PROBABILISTIC SEISMIC HAZARD

Nearly all modern earthquake design codes rely on probabilistic seismic hazard analyses to define the level of ground shaking. The basic procedures for estimating the ground shaking hazard in a probabilistic hazard analysis are similar to those used in a deterministic approach where a specific earthquake magnitude and location are specified. The main difference in a probabilistic approach is that multiple earthquakes are considered to contribute to the level of ground shaking likely to be experienced at a particular location.

The general process for performing a probabilistic seismic hazard analysis includes the following steps, illustrated graphically in Figure C.1:

1. Characterize the type and location of earthquake source zones.

2. Define the frequency of occurrence of earthquakes of varying magnitude within the source zones.

3. For a particular location, use attenuation relationships to compute the level of ground shaking from each earthquake from each source zone and weight the resulting ground shaking by the likelihood of earthquake occurrence.

Earthquake source zones are typically characterized as point, line, or area sources (Figure C.2). A point source represents an earthquake at a single location and is typically incorporated into line-source or area-source models. A line source, often used to represent earthquakes occurring along faults, can have an earthquake point source centered at any portion of the line source considered to rupture in an earthquake. An area source is often used to model regions with no known active faults and point sources are assumed to occur at random locations within the defined area.

The rate of occurrence of earthquakes is estimated from regional earthquake records, regional strain rate information, or specific fault investigations that provide estimates of average fault slip rates. A particular concern in establishing earthquake occurrence rates is to maintain consistency between regional deformation associated with the model and deformation related to regional strain measurements.

Two common methods used to estimate recurrence rate for earthquakes of varying magnitude include the use of the Gutenberg-Richter relationship and a characteristic model of earthquake recurrence. The Guttenberg-Richter relationship assumes a continuous distribution of earthquake magnitudes between zero and the maximum magnitude for the particular fault. The form of this relationship is as follows:

$$\log N(M) = a - bM \qquad \text{(C-1)}$$

where:

$N(M)$ = earthquake moment magnitude [$6.0 < M < 8.0$]

a, b = constants determined from regional earthquake catalogs

The *a* parameter in (C-1) corresponds to a magnitude between –0.05 and +0.05. In probabilistic ground motion hazard analyses, it is customary to truncate the relationship at an earthquake magnitude between 5.00 and 6.50. This truncation is necessary to avoid overestimating regional deformation produced by a large number of very small earthquakes. For areas with known active faults, the maximum earthquake magnitude given by the Gutenberg-Richter equation is generally limited to the magnitude computed based upon empirical relationships (e.g., Wells and Coppersmith, 1994) between magnitude and fault length or fault area assuming the entire fault ruptures.

The concept of a characteristic earthquake is generally accepted as an appropriate means to address hazards from individual faults. The characteristic earthquake approach is based upon the hypothesis that earthquakes generated by a particular fault tend to have similar magnitudes instead of the continuous magnitude distribution assumed by the Gutenberg-Richter model. The characteristic earthquake is typically related to an earthquake that ruptures an entire fault segment.

For area sources, earthquake occurrence rates are nearly always based upon a Gutenberg-Richter recurrence model with the maximum earthquake magnitude estimated using available information and considerable judgment.

In addition to weighting the ground motions at a site produced by each earthquake source, the uncertainty in selection of attenuation relationship can be incorporated into the probabilistic hazard. This may be accomplished by selecting multiple attenuation relationships, with a weighting factor based upon judgment or by accounting for the variability of a particular attenuation relationship directly.

Probabilistic hazard studies produced for areas where there are no well-defined active faults (i.e., modeled entirely with area sources) have a common characteristic: ground motion estimates increase with decreasing probability of occurrence and these motions are associated with smaller magnitude earthquakes occurring closer to the site of interest.

For estimating wave propagation strains or assessing landslide hazard, the ground shaking estimates from a probabilistic hazard analysis can be used directly. For the assessment of liquefaction or lateral spread displacement, it is necessary to define an earthquake magnitude and distance. While this information is generally available as a by-product of the probabilistic hazard analysis, it is often not reported. Deaggregation studies extract information from probabilistic hazard analyses on the contribution of individual point sources within line and area source models. There is little guidance on how to select an appropriate combination of magnitude and distance from a deaggregation study. Past project experience indicates that selecting a combination that captures 50% to 84% of the total seismic hazard may be appropriate. This is illustrated in the Commentary.

The results of probabilistic hazard studies are of no value for estimating fault displacement. However, if probabilistic hazard studies are available for areas with known fault activity, there will be information on specific faults used in the hazard study (e.g., fault length, fault width, slip rate, dip angle, type of faulting).

It is recommended that assistance from individuals with special expertise in defining earthquake surface fault hazards be used to fault motions for assessing pipeline performance. However, a very approximate estimate of the likelihood of an earthquake of the maximum magnitude on a fault can be made based upon the reported fault slip rate and fault dimensions. The key assumption in this approach is that all geologic slip is associated with the estimate of the maximum earthquake that can occur on a fault. This assumption overestimates the occurrence rate since it does not account for slippage from smaller, more frequent earthquakes. Nonetheless, the approach can be used to compare relative risk from numerous faults and is often adequate for performing preliminary engineering assessments.

The slip rate can be used to estimate the rate of accumulation of seismic moment using the definition of seismic moment rate, M_r:

$$M_r = \mu \cdot RLD \cdot FW \cdot sr \tag{C-2}$$

where:

a, b	=	constants determined from regional earthquake catalogs
M_r	=	moment rate
μ	=	shear modulus of earth's crust (typically taken as $3\text{x}10^{10}$ N/m^2)
RLD	=	fault rupture length at depth
FW	=	fault width
sr	=	fault slip rate

The moment released, M_e by an earthquake of magnitude M is computed as

$$M_e = 10^{1.5(M)+9} \tag{C-3}$$

The earthquake magnitude, M, can be estimated using empirical relationships from Wells and Coppersmith (1994), one of which is provided below:

$$M = 4.38 + 1.49 \cdot \log(RLD) \qquad (\sigma = 0.26 \text{ units of magnitude}) \tag{C-4}$$

The mean recurrence period, RP, for an earthquake of magnitude M can be estimated as M_e/M_r which is given by the expression below.

$$RP = \frac{10^{1.5(M)+9}}{\mu \cdot RLD \cdot FW \cdot sr} \tag{C-5}$$

By assuming that an earthquake of a selected magnitude is responsible for all of the fault slip, the simplified approach always overestimates the probability of occurrence of a particular earthquake. Comparisons with published recurrence intervals for many faults in the United States indicate that the simplified approach typically overestimates earthquake probability by less than a factor of two. Since the simple approach always overestimates the likelihood of an earthquake of a specific magnitude, it will generally significantly underestimate the earthquake magnitude for a defined return period. This is the reason the simplified approach is limited to consideration of the maximum earthquake magnitude for a particular fault.

If the maximum magnitude earthquake on a particular fault is not judged to be appropriate or a more refined estimate of return period is needed, assistance from individuals with expertise in seismology is required.

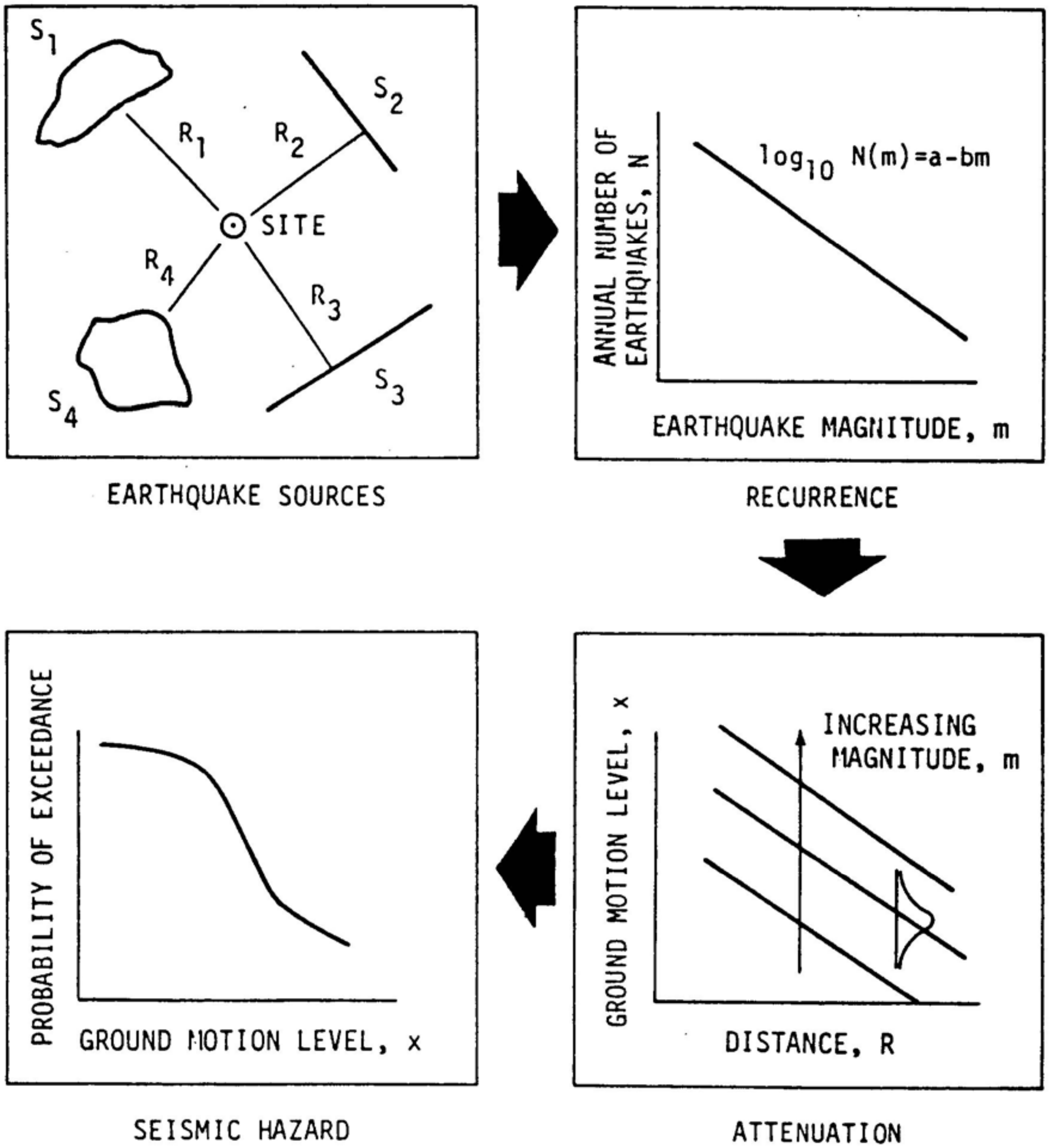

Figure C.1 Probabilistic Seismic Hazard Process (from ASCE, 1984)

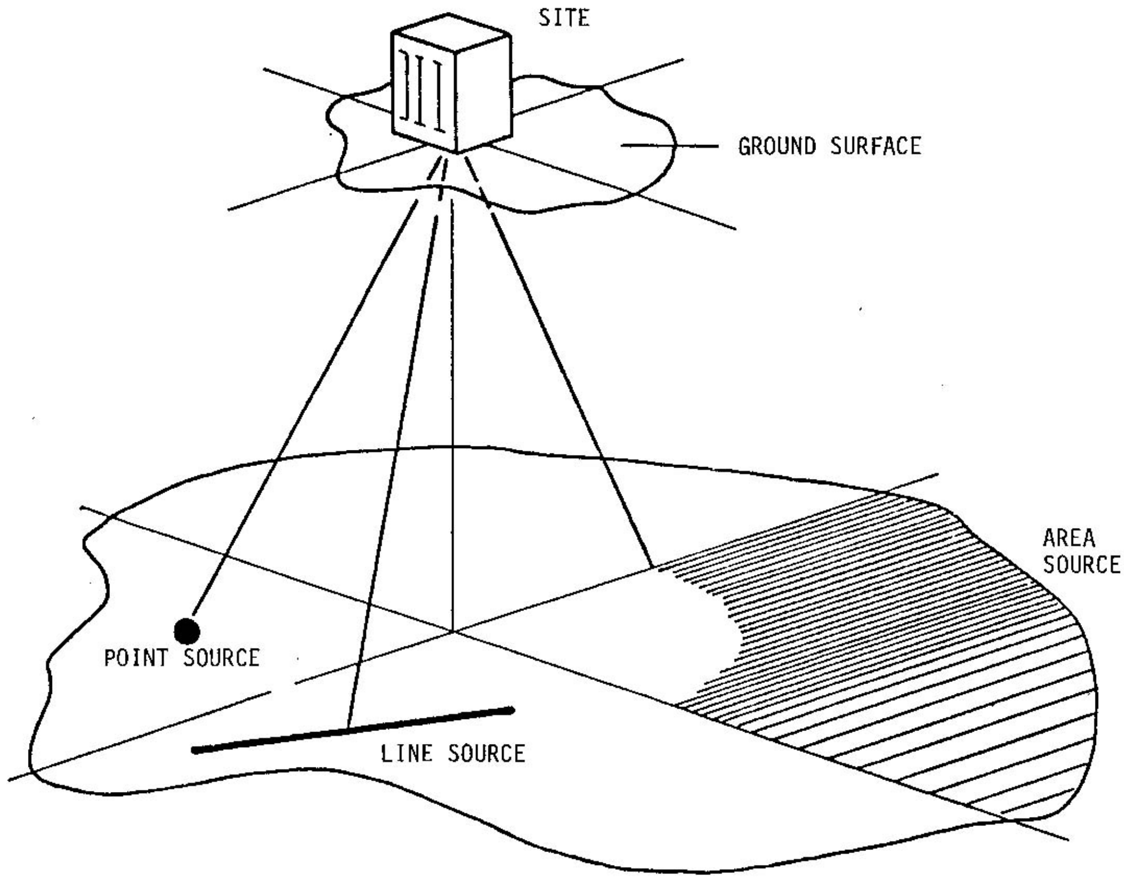

Figure C.2 Common Types of Earthquake Source Models (from ASCE, 1984)

EXAMPLE CALCULATIONS

This example calculation is provided to serve several purposes. First, the example illustrates the application of various equations provided in the guidelines. Second, the example provides a basis for understanding the magnitude of numerical values (e.g., ground displacements, soil spring forces) associated with estimating parameters for a seismic assessment of a buried pipeline. Finally, the example illustrates the processes involved in performing a seismic assessment of a buried pipeline. To assist in presenting the calculations, the example has been framed in the context of a hypothetical pipeline alignment. While parts of the example are extracted from actual past project experience, the results are for illustrative purposes only.

Problem Description:

The site in question is the portion of an NPS 20 pipeline alignment at the south bank of the Flowing River in the western United States. The NPS 20 pipeline crosses the river in a straight alignment. The site is being evaluated for potential liquefaction hazards. The plan and profile of the south side of the river crossing is shown in Figure D.1. The ground is typically flat south of the crossing (less than 0.5% slope) until the pipeline reaches a hill approximately 650 ft (200 m) south of the riverbank. Surface geology maps indicate the hill is an outcropping of weathered shale. The pipeline in the vicinity of the river crossing is classified as a Class 2 location in accordance with ASME B31.8.

The performance requirement established for the NPS 20 pipeline is defined as a mean annual probability of pipeline rupture from earthquake causes less than 0.2%. This is equivalent to a 500-year average return period. A review of available geologic maps and regional hazard information available from the government indicate two potential earthquake sources. One source is the Shakem fault, located 19.3 miles (31 km) from the river crossing. The other source is the Enditall fault, located more than 30 miles (48 km) from the site. Both faults are characterized by strike-slip fault movement with surface fault expression. Seismic parameters available for the two faults in question are summarized in Table 1. The variability in fault length and slip rate is for illustration purposes only. In actual practice, there is considerable variability in all fault descriptive parameters that are usually related to the particular interpretation of subsurface data by different geologists and seismologists.

Table 1: Summary of Fault Descriptive Parameters

Fault	**Length, km (range)**	**Depth km**	**Slip mm/yr**	**Sediment Depth km**
Shakem	80 (70 - 120)	11	5±2	1
Enditall	175 (100 - 250)	13	1±0.5	1

The pipeline, installed in 1960, is fabricated from API 5L Grade X42 seamless pipe with a coal tar coating. The pipe wall thickness is typically 0.280 inches (7.1 mm) except at the river crossing where the thickness is 0.375 inches (9.5 mm). The river crossing portion of the pipeline also has 3 inches (76 mm) of concrete coating for buoyancy control. The maximum allowable operating pressure for the pipeline is 825 psi (5,700 kPa) although the typical operating pressure is below 500 psi. No internal inspection of the pipeline has been performed to confirm the condition.

Liquefaction is considered possible based upon regional seismic hazard maps and knowledge of past subsurface investigations for other construction projects near the pipeline crossing. Given this information, subsurface investigations to collect SPT (one location) and CPT data (three locations) were performed at the sites indicated in Figure D.1. SPT and CPT data were collected at a common location to provide a direct correlation between the SPT and CPT data. In addition to the SPT data, SPT drilling provided an opportunity to collect soil samples. Samples were collected from each SPT using sample liners and later evaluated to determine density and grain size distribution. Some results from the subsurface investigation are provided in Table 2 and Figures D.2 through D.4.

Table 2: Selected Data from Subsurface Investigations

SPT 1		**CPT 3**	
Sample Depth	20 ft	Test Depth	20 ft
SPT Value	8	q_c (bars)	50
% Fines	19	f_s (bars)	0.2
Average Density (pcf)	110		

Step 1: Define Earthquake for Performing the Seismic Assessment

In most cases, the earthquake hazard will be defined by geologists or seismologists familiar with the tectonic setting of the site being assessed. The following approach, based upon methods described in Section 2 and Appendix C is provided to illustrate the process for obtaining an approximate quantification of the seismic hazard with limited data.

The information on fault length, depth, slip rate, and direction of slip are used to estimate the risk of an earthquake and the ground shaking levels in the event of an earthquake. The calculations necessary to perform this estimate for the Shakem fault are summarized below.

Earthquake Magnitude:

Use the Wells and Coppersmith (1994) relationship for unknown or unspecified type of faulting

$$M = 4.38 + 1.49\left[\log(RLD)\right] = 4.38 + 1.49\left[\log(80)\right] = 7.2$$

Mean Recurrence Interval:

μ = $3(10)^{10}$ N/m^2

M_e = $10^{1.5(M)+9} = 10^{1.5(7.2)+9} = 6.31(10)^{19}$ N-m

M_r = $\mu \cdot RLD \cdot FW \cdot sr = 3(10)^{10}(80,000)(11,000)(.005) = 1.32(10)^{17}$ N-m/yr

Mean recurrence interval estimated to be no more frequent than $\frac{M_e}{M_r} = 478$ years or approximately 500 years.

Amount of Fault Movement:

Although the pipeline does not cross a fault, the design fault dispalcements are computed for the purpose of the example problem. The pipeline is in a Class 2 location so the design fault displacement will be based upon the mean average fault displacement. For the Shakem fault:

$$\log(AD) = 0.88SRL - 1.43 = 0.88\log(80) - 1.43 = 0.24$$

AD = 1.8 m

The potential variability in the above estimates of fault movement and return period can be estimated by examining the results for possible combinations of fault rupture length and slip rate. The results are summarized in the following tables.

Table 3a: Earthquake Magnitude from Fault Rupture Length

Fault Rupture Length, km		
70	**80**	**120**
7.1	7.2	7.5

Table 3b: Mean Fault Displacement from Fault Rupture Length

Fault Rupture Length, km		
70	**80**	**120**
1.6 m	1.8 m	2.5 m

Table 3c: Earthquake Recurrence Interval (years)

Slip Rate mm/yr	Fault Rupture Length, km		
	70	**80**	**120**
3	710	840	1,390
5	430	500	830
7	306	360	600

Similar values for the Enditall fault are provided below:

Magnitude: 7.4 - 8.0

Fault Movement: 2.1 - 4.8 m

Recurrence Interval: 1,120 - 14,430 years

For the best estimate fault parameters, the Enditall fault average return period is estimated to be 5,610 years compared to the approximately 500 years for the Shakem fault. If fault crossing evaluations were to be performed for the Enditall fault, the design fault displacements would be obtained by multiplying the mean average fault displacement by 0.18, the performance goal (500 years) divided by half of the average return period of the fault (2,805 years). This would result in design fault displacements of 0.38 m to 0.86 m. Since both faults are strike-slip, there is no need for a correction to derive fault displacement components in three orthogonal displacements.

It is important to note the considerable variability in the estimates of the earthquake hazard. The level of uncertainty indicated by the results in Table 3 is typical of actual experience in estimating the earthquake hazard. It is possible, but not certain, that additional geologic investigations including trenching across the fault, deep borings, or seismic refraction studies could better define the basic fault descriptors.

PGA at Site:

Use Campbell attenuation relationship for a soil site:

$F = 0 \qquad S_{SR} = 0 \qquad S_{HR} = 0 \qquad R = 31$

$H_T = 0 \qquad H_S = 1 \qquad H_B = 12 \qquad \alpha = 90^\circ$

$$W = 10^{-1.01+0.32M} = 10^{-1.01+0.32(7.2)} = 16$$

$$d_S = \frac{1}{2}\left[H_B + H_T - W\sin(\alpha)\right] \geq H_S = \frac{1}{2}\left[12 + 0 - 16\right] = -3.85$$

$$d_S = H_S = 1$$

$$R_s = \sqrt{d_S^2 + R^2} = \sqrt{1^2 + 33^2} = 31 \text{ km}$$

$$\begin{aligned}\ln(PGA) = &-3.512 + 0.904M - 1.328\ln\sqrt{R_S^2 + \left[0.149e^{0.647M}\right]^2} \\ &+ \left[1.125 - 0.112\ln(R_S) - 0.0957M\right]F + \left[0.440 - 0.171\ln(R_S)\right]S_{SR} \\ &+ \left[0.405 - 0.222\ln(R_S)\right]S_{HR} + f_A(D) + E\Phi^{-1}(x) \\ = &-3.512 + 0.904(7.2) - 1.328\ln\sqrt{31^2 + \left[0.149e^{0.647(7.2)}\right]^2}\end{aligned}$$

PGA = 0.18 g

The range of PGA, adopting the variability in earthquake magnitude from Table 3 and not including variability in the Campbell attenuation relationship, is 0.17 - 0.22 g. Similar PGA values for the Enditall fault are 0.13 g to 0.20 g.

Although the levels of ground shaking are comparable for the Shakem and Enditall faults, the Shakem fault is determined to be the controlling earthquake hazard because the recurrence interval is more compatible with the stated performance goal.

For the remainder of this example, the central fault description values (length = 80 km, slip rate = 5 mm/yr) are used.

Step 2: Assess Liquefaction Potential for Subsurface Soils

Calculations to assess the potential for liquefaction are performed for the same depth at location 1 where SPT and CPT data were collected. For the SPT calculations, hammer energy was assumed to be 60% and the tests were performed with a standard diameter sampler. A gradation curve for a sample retrieved at the depth being assessed for location 1 is provided in Figure D.4.

Depth = 20 ft = 6.1 m

Depth to water table = 6 ft (1.8 m) at time of test

Dry density = 110 pcf

Total overburden stress = $\sigma_{vo} = 20(110) = 2{,}200$ psf = 1.1 tsf = 105 kPa

Effective stress = $\sigma'_{vo} = \sigma_{vo} - 62.4(20-6) = 1{,}326$ psf =0.663 tsf = 63.5 kPa

$P_a = 14.5 \text{ psi} = 100 \text{ kPa}$ = 2,088 psf

z = D/m = 6.1

$$r_d = \frac{1 - 0.4113\sqrt{z} + 0.04052z + 0.001753z^{1.5}}{1 - 0.4177\sqrt{z} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2}$$

$$r_d = \frac{1 - 0.4113\sqrt{6.1} + 0.04052(6.1) + 0.001753(6.1)^{1.5}}{1 - 0.4177\sqrt{6.1} + 0.05729(6.1) - 0.006205(6.1)^{1.5} + 0.001210(6.1)^2} = 0.96$$

$$CSR = 0.65PGA\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d = 0.65(0.18)\left(\frac{2,200}{1,326}\right)0.96 = 0.19$$

SPT Assessment:

$N = 8$

$$C_N = \sqrt{\frac{P_a}{\sigma'_{vo}}} = \sqrt{\frac{2,088}{2,200}} = 1.25$$

$$C_R = 0.75 + 0.25\frac{(z-4)}{6} = 0.75 + 0.25\frac{(6.1-4)}{6} = 0.84$$

$FC = 19\%$

$$\alpha = e^{1.76 - \frac{190}{FC^2}} = e^{1.76 - \frac{190}{19^2}} = 3.23$$

$$\beta = 0.99 + \frac{FC^{1.5}}{1000} = 0.99 + \frac{19^{1.5}}{1000} = 1.07$$

$$(N_1)_{60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S = 8 \cdot 1.26 \cdot 1 \cdot 1 \cdot 0.84 \cdot 1 = 8.4$$

$$(N_1)_{60FC} = \alpha + \beta(N_1)_{60} = 3.23 + 1.07(8.4) = 12.2$$

$$MSF = \frac{10^{3.81}}{M^{4.42}} = \frac{10^{3.81}}{7.2^{4.42}} = 1.05$$

K_σ is 1.0 since the effective stress is less than the reference pressure, P_a

$$\begin{aligned} CRR &= \left(\frac{4.8(10)^{-2} - 4.721(10)^{-3}x + 6.136(10)^{-4}x^2 - 1.673(10)^{-5}x^3}{1 - 1.248(10)^{-1}x + 9.578(10)^{-3}x^2 - 3.285(10)^{-4}x^3 + 3.714(10)^{-6}x^4}\right)MSF \cdot K_\sigma \\ &= \left(\frac{4.8(10)^{-2} - 4.721(10)^{-3}12.2 + 6.136(10)^{-4}12.2^2 - 1.673(10)^{-5}12.2^3}{1 - 1.248(10)^{-1}12.2 + 9.578(10)^{-3}12.2^2 - 3.285(10)^{-4}12.2^3 + 3.714(10)^{-6}12.2^4}\right)1.05 \cdot 1.0 \\ &= 0.14 \end{aligned}$$

Since the CRR of 0.14 is less than the CSR of 0.19, liquefaction is expected for 0.18 *g PGA*.

CPT Assessment:

q_c = 50 bar = 5,000 kPa

f_s = 0.2 bar = 20 kPa

$$F = \left[\frac{f_s}{q_c - P_a}\right]100\% = \left[\frac{20}{5000-100}\right]100\% = 0.41\%$$

$$Q_1 = \frac{q_c - \sigma_{vo}}{P_a}\left(\frac{P_a}{\sigma'_{vo}}\right) = \frac{5000-105}{100}\left(\frac{100}{63.5}\right) = 77.1$$

$$I_1 = \sqrt{(3.47 - \log Q_1)^2 + (1.22 + \log F)^2} = \sqrt{(3.47 - \log 77.1)^2 + (1.22 + \log 0.41)^2} = 1.79$$

$$Q_{0.5} = \frac{q_c - \sigma_{vo}}{100\text{ kPa}}\left(\frac{100\text{ kPa}}{\sigma'_{vo}}\right)^{0.5} = \frac{5000-105}{100}\left(\frac{100}{63.5}\right)^{0.5} = 61.4$$

$$I_{0.5} = \sqrt{(3.47 - \log Q_{0.5})^2 + (1.22 + \log F)^2} = \sqrt{(3.47 - \log 61.4)^2 + (1.22 + \log 0.41)^2} = 1.88$$

n = 0.5

$$C_Q = \left(\frac{P_a}{\sigma'_{vo}}\right)^n = \left(\frac{100}{63.5}\right)^{0.5} = 1.26$$

$$\begin{aligned} K_c &= -17.88 + 33.75 I_n - 21.63 I_n^2 + 5.581 I_n^3 - 0.403 I_n^4 \\ &= -17.88 + 33.75(1.88) - 21.63(1.88)^2 + 5.581(1.88)^3 - 0.403(1.88)^4 \\ &= 1.16 \end{aligned}$$

$$(q_{c1N})_{FC} = K_c \cdot C_Q\left(\frac{q_c}{P_a}\right) = 1.26 \cdot 1.16\left(\frac{5,000}{100}\right) = 73$$

$$CRR = \left[93\left(\frac{(q_{c1N})_{FC}}{1000}\right)^3 + 0.08\right]MSF = \left[93\left(\frac{73}{1000}\right)^3 + 0.08\right]1.05 = 0.12$$

Since the CRR of 0.12 is less than the CSR of 0.19, liquefaction is expected for 0.18 *g PGA*.

From similar calculations for the other boring locations, the extent of liquefiable material for the site is estimated. The cross-section of the site is provided in Figure D.1 that indicates the extent of potentially liquefiable material at the riverbank.

Step 3: Assess Potential for Lateral Spread Movement

The site is representative of a free face condition. For this example, thickness of saturated sand deposits with $(N_1)_{60}$ less than 15 is taken as 75% of the average liquefiable thickness of 15.3 m or 11.5 m, the average fines content of the material with $(N_1)_{60}$ less than 15 is 5%, and $D50_{15}$ is 0.10 mm. In practice, this determination is made based upon a review of all SPT and CPT data.

Lateral spread displacement for the onshore portion of the crossing can be estimated as a function of the distance from the crest of the bank using a free-face condition.

M = 7.2

R = 31 km

$R_o = 10^{0.89M-5.64} = 10^{0.89(7.2)-5.64} = 5.86$

$R^* = R + R_o = 36.86$

T_{15} = 11.5

F_{15} = 5

$D50_{15}$ = 0.10

H = 20

For sloping ground conditions:

$$\begin{aligned}Log(LSD) = &-16.213 + 1.532M - 1.406 LogR^* \\ &-0.012R + 0.338 LogS + 0.540 LogT_{15} \\ &+3.413 Log(100 - F_{15}) - 0.795 Log(D50_{15} + 0.1)\end{aligned}$$

For free-face conditions:

$$\begin{aligned}Log(LSD) = &-16.713 + 1.532M - 1.406 LogR^* \\ &-0.012R + 0.592 LogW + 0.540 LogT_{15} \\ &+3.413 Log(100 - F_{15}) - 0.795 Log(D50_{15} + 0.1)\end{aligned}$$

Lateral spread displacement is computed for various distances from the toe of the riverbank as summarized in Table 4. The maximum distance of interest coincides with a transition between river deposits and the weathered shale hillsides bounding the river. Lateral spread displacements on the slope of the bank are estimated using the ground slope equation even though the slope exceeds the 0% to 6% recommended range in slope for applicability of the equation. This is likely a conservative assumption considering the much larger displacements estimated by the ground slope equation compared to the free-face equation. The surface ground displacement profile assumes free-face conditions beyond the crest of the riverbank since the ground profile in this area is relatively flat. This assumption is consistent with the formulation of data used in the Youd, Hansen, and Bartlett equations. The resulting surface ground displacement profile is illustrated in Figure D.5.

Table 4: Computed Lateral Spread Displacements

Station (m)	**Distance from Toe (m)**	**W & S (%)**	**Slope LSD (m)**	**Free-Face LSD (m)**
2355	0	N/A	N/A	N/A
2596	241	8.3	2.7	N/A
2605	250	7.8	2.6	1.4
2655	300	6.7	2.5	1.3
2705	350	5.7	2.4	1.2
2739	384	5.2	2.3	1.1

Step 4: Estimate Relative Lateral Spread Displacement Affecting the Pipeline

Typically, the ground displacement resulting in loads on the pipeline is less than that indicated in the plot of surface displacements for portions of the pipeline within the liquefiable material. The ground displacements at the pipeline depth are estimated as a function of the ratio of the liquefiable layer depth below the pipe to the total depth of the liquefiable layer. Alternatively, a more conservative approach is to assume that the pipe is subjected to the same horizontal ground displacements as the surface. This assumption is consistent with block-like movement of the liquefiable layer over the deeper non-liquefiable soil.

Step 5: Compute Vertical Settlement from Pore Water Dissipation

The amount of vertical settlement related to pore water dissipation is estimated using the average computed factor of safety against liquefaction and Figure 2.4. For the location previously assessed, the factor of safety against liquefaction is CRR/CSR = 0.13/0.19 = 0.7. Figure 2.4 was developed for clean sands. The use of $(N_1)_{60FC}$ = 12.2 instead of $(N_1)_{60}$ is assumed to account for the presence of fine-grained material. From Figure 2.4, the post-liquefaction volumetric strain at the location where liquefaction is estimated as approximately 3%. The vertical settlement that the pipeline will experience is estimated as 3% of the depth of liquefiable soil beneath the pipeline and is plotted in Figure D.5. The maximum differential vertical displacements that the pipeline is exposed to is less than 15 cm over 35 m of pipe. This is sufficiently small to be neglected in the assessment of pipeline performance.

Step 6: Compute the Soil Spring Properties Along the Pipeline

Soil spring properties vary with soil material and pipeline burial depth. Within the same soil material, soil spring values are assumed to vary linearly between two different burial depths. The following example computes soil springs near stations 2453 and 2749.

Station 2453:

This location is within the zone of soil expected to liquefy. The soil strength is treated as if the soil were a cohesive material ($\phi = 0$) with an undrained shear strength equal to 20% of the effective overburden stress.

Soil cover = 8.4 m = 27.6 ft

Water table elevation = -2.75 m = -9 ft

Ground surface elevation = -13.8 m = -45.3 ft

Diameter = D+2(3inches) = 26 inches = 2.167 feet = 0.660 m

Material = liquefied soil

$H = C + D/2$ = 8.73 m = 28.6 ft

H/D = 13.2

Effective stress = 28.6(110-62.4) = 1,361 psf = 65 kPa

c = 0.2(1,361) = 272 psf = 0.272 ksf = 13 kPa

The primary direction of soil movement is in the axial direction. Therefore, assume all lateral (horizontal and vertical) movement is confined to the soils immediately surrounding the pipeline.

Axial Soil Spring:

$$\alpha = 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1}$$
$$= 0.608 - 0.123(0.272) - \frac{0.274}{(0.272)^2+1} + \frac{0.695}{(0.272)^3+1} = 1.0$$

$$T_u = \pi D \alpha c + \pi D H \bar{\gamma}\left(\frac{1+K_o}{2}\right)\tan(\delta) = \pi(2.167)(1.0)(272) = 1{,}851\frac{\text{lb}}{\text{ft}} = 27\frac{\text{kN}}{\text{m}}$$

Horizontal Soil Spring:

$$N_{ch} = 6.752 + 0.065\left(\frac{H}{D}\right) - \frac{11.063}{\left(\frac{H}{D}+1\right)^2} + \frac{7.119}{\left(\frac{H}{D}+1\right)^3}$$

$$= 6.752 + 0.065(13.2) - \frac{11.063}{(13.2+1)^2} + \frac{7.119}{(13.2+1)^3}$$

$$= 7.6$$

$$P_u = N_{ch}cD + N_{qh}\bar{\gamma}HD = 7.6(272)(2.167) = 4{,}480\,\frac{\text{lb}}{\text{ft}} = 65.4\,\frac{\text{kN}}{\text{m}}$$

$$\Delta_p = 0.04\left(H + \frac{D}{2}\right) = 0.04\left(12 \cdot 28.6 + \frac{22.167}{2}\right) = 14 \text{ inches } = 36 \text{ cm}$$

Vertical Bearing Soil Spring:

$$N_c = \cot(\phi + 0.001)\left[e^{\pi\tan(\phi+0.001)}\tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right]$$

$$= \cot(0.001)\left[e^{\pi\tan(0.001)}\tan^2\left(45 + \frac{0.001}{2}\right) - 1\right] = 5.14$$

$$N_q = e^{\pi\tan(\phi+0.001)}\tan^2\left(45 + \frac{\phi + 0.001}{2}\right) = e^{\pi\tan(0.001)}\tan^2\left(45 + \frac{0.001}{2}\right) = 1$$

$$N_\gamma = e^{0.18\varphi - 2.5} = e^{-2.5} \approx 0$$

$$Q_d = N_c cD + N_q\bar{\gamma}HD + N_\gamma \gamma\frac{D^2}{2}$$

$$= 5.14(272)(2.167) + (110 - 62.4)(2.167)(28.7) = 5{,}990\,\frac{\text{lb}}{\text{ft}} = 87.4\,\frac{\text{kN}}{\text{m}}$$

$$\Delta_{qd} = 0.125D = 0.125(26) = 3.3 \text{ inches } = 84 \text{ cm}$$

Vertical Uplift Soil Spring:

$$N_{cv} = 2\left(\frac{H}{D}\right) \le 8$$

$$= 2(9.5) = 19 > 8$$

$$N_{cv} = 8$$

$$Q_u = N_{cv}S_u D + N_{qv}\bar{\gamma}HD = 8(272)(2.167) = 4{,}715\,\frac{\text{lb}}{\text{ft}} = 68.8\,\frac{\text{kN}}{\text{m}}$$

$$\Delta_{qu} = \frac{0.12H(N_{qv}\bar{\gamma}HD) + 0.1H(N_{cv}cD)}{N_{cv}cD + N_{qv}\bar{\gamma}HD} = 0.1H = 0.1(28.7\text{ft})\left(12\frac{\text{in}}{\text{ft}}\right) = 34 \text{ inches } = 86 \text{ cm}$$

Station 2749:

Soil Cover = 4.9 ft (1.50 m)

Diameter = D = 20 inches = 1.667 ft = 0.508 m

Material = granular (c = 0)

$H = C + D/2$ = 4.9(12)+.5(20) = 68.8 inches = 5.733 ft = 1.75 m

H/D = 68.8/20 = 3.4

The internal friction angle is taken to be 35°.

The pipe is coated with fusion bonded epoxy. Use $\delta = 0.6\phi = 0.6(35) = 21°$

The primary direction of soil movement is in the axial direction. Therefore, assume all lateral (horizontal and vertical) movement is confined to the soils immediately surrounding the pipeline.

Axial Soil Spring:

$$T_u = \pi D\alpha c + \pi DH\bar{\gamma}\left(\frac{1+K_o}{2}\right)\tan(\delta)$$
$$= \pi(1.667)(5.733)(110)\left(\frac{1+0.5}{2}\right)\tan(21^\circ) = 950\frac{\text{lb}}{\text{ft}} = 13.9\frac{\text{kN}}{\text{m}}$$

Horizontal Soil Spring:

$$N_{qh} = 6.816 + 2.019\frac{H}{D} - 0.146\left(\frac{H}{D}\right)^2 + 0.00765\left(\frac{H}{D}\right)^3 - 1.683(10)^{-4}\left(\frac{H}{D}\right)^4$$
$$= 6.816 + 2.019(3.4) - 0.146(3.4)^2 + 0.007651(3.4)^3 - 1.683(10)^{-4}(3.4)^4$$
$$= 12.3$$

$$P_u = N_{ch}cD + N_{qh}\bar{\gamma}HD = 12.3(110)(5.733)(1.667) = 12{,}930\frac{\text{lb}}{\text{ft}} = 189\frac{\text{kN}}{\text{m}}$$

$$\Delta_p = 0.04\left(H + \frac{D}{2}\right) = 0.04\left(12\cdot 5.733 + \frac{20}{2}\right) = 3.2 \text{ inches} = 8 \text{ cm}$$

Vertical Bearing Soil Spring:

$$N_q = e^{\pi \tan(\phi)} \tan^2\left(45 + \frac{\phi}{2}\right) = e^{\pi \tan(35)} \tan^2\left(45 + \frac{35}{2}\right) = 33$$

$$N_c = \cot(\phi + 0.001)\left[e^{\pi \tan(\phi+0.001)} \tan^2\left(45 + \frac{\phi + 0.001}{2}\right) - 1\right] = \cot(35)\left[N_q - 1\right] = 46$$

$$\begin{aligned} N_\gamma &= e^{(0.18\phi - 2.5)} \\ &= e^{(0.18(35)-2.5)} = 45 \end{aligned}$$

$$\begin{aligned} Q_d &= N_c cD + N_q \bar{\gamma} HD + N_\gamma \gamma \frac{D^2}{2} \\ &= 33(110)(5.733)(1.667) + (45)(110)\frac{(1.667)^2}{2} \\ &= 41,570 \frac{\text{lb}}{\text{ft}} = 607 \frac{\text{kN}}{\text{m}} \end{aligned}$$

$$\Delta_{qd} = 0.125D = 0.125(20) = 2.5 \text{ inches} = 5 \text{ cm}$$

Vertical Uplift Soil Spring:

$$N_{qv} = \tan(\phi)\left(\frac{\phi}{44}\right)\left(\frac{H}{D}\right) = \tan(35)\left(\frac{35}{44}\right)3.4 = 1.9$$

$$Q_u = N_{cv} S_u D + N_{qv} \bar{\gamma} HD = 1.62(110)(4.833)(1.667) = 1,431 \approx 1,430 \frac{\text{lb}}{\text{ft}}$$

$$\Delta_{qu} = \frac{0.12H(N_{qv}\bar{\gamma}HD) + 0.1H(N_{cv}cD)}{N_{cv}cD + N_{qv}\bar{\gamma}HD} = 0.12H = 0.12(58) = 7.0 \text{ inches}$$

Step 7: Perform Analysis

The analytical model for assessing pipeline response is provided in Figure D.6. In plan, the pipeline is straight with the pipe anchored against movement at the center of the river crossing and at a point approximately 1,000 ft south of the ground deformation boundary. Anchoring the pipeline at the center of the river assumes that that north bank may experience a similar ground failure and the center of the river is a point of symmetry. If no instability of the north bank occurs, this assumption will lead to an overestimate of the pipe strains at the center of the river and an underestimate of the pipe strains at the onshore ground failure boundary. The southerly anchorage point was determined based upon the recommendation in Section 4.4.

$$L_{anchor} = \frac{\pi D t \sigma_y}{T_u} = \frac{\pi(26)(0.375)52,000}{2,100} = 758 \text{ ft} = 231 \text{ m}$$

The length of the pipe elements is one pipe diameter within 100 feet of the ground displacement boundaries. Between the zones with element length of 1 pipe diameter, the pipe elements are allowed to be up to 10 feet long.

No horizontal or vertical soil springs are specified more than 100 feet from the ground displacement boundaries as the pipeline is only exposed to axial loading. Lateral movement (horizontal and vertical) is prevented for this portion of the analytical model. The ground displacement pattern is applied in 120 equal increments to a maximum displacement that is about 50% greater (4 m) than the estimates from Step 4 (2.9 m). This allows the behavior of the pipeline to be evaluated for alternate assumptions that might increase the amount of applied ground displacement.

The amount of axial ground movement relative to the pipe is provided in Figure D.7a. As can be seen in Figure D.7a, the pipeline response is nearly separated between the head and toe regions of the lateral spread. The onshore portion of the pipeline acts to restrain the pipe from ground displacements creating tension in the pipe at the head of the lateral spread. Near the toe of the lateral spread, the ground displacement produces compression in the pipe that is resisted by the pipe outside of the zone of ground displacement. Figure D.7b illustrates the variation of maximum strain magnitude (tension or compression) along the pipeline at a maximum ground displacement of 4 m. The location of maximum tensile and compressive longitudinal strains are also indicated in Figure D.7b.

The variation in maximum longitudinal strain as a function of maximum ground displacement magnitude is shown in Figure D.8. For a specific ground displacement value, the strains plotted in Figure D.8 represent the largest total strain at any of 8 equidistant circumferential locations around the pipe cross-section at any location in the pipe model. For the ground displacement estimated in Step 4, the maximum strain is approximately 0.8% tension and 0.45% compression.

Step 8: Assess Nominal Pipeline Performance for Computed Strain Levels

An assessment will first be performed assuming the pipeline construction meets the requirements of Section 3.1. For a performance requirement that pressure integrity be maintained, the allowable longitudinal tensile strain limit is assumed to be 4%. The corresponding longitudinal compressive strain limit is computed as follows:

For the concrete-coated pipe:

$$\varepsilon_{cp} = 1.76\frac{t}{D} = 1.76\frac{.375}{20} = 3.3\%$$

For the onshore portion of the pipeline (t = 0.280 inches)

$$\varepsilon_{cp} = 1.76\frac{t}{D} = 1.76\frac{.28}{20} = 2.5\%$$

The pipe strain computed for 2.9 m of maximum ground displacement is far less than the above strain limits. On this basis, it is clear that the pipeline is not at risk for the ground displacements modeled in the analysis.

Additional analyses that should be performed, but are not included in this example problem, include the following modifications:

1. Variation in the soil springs to account for uncertainties in the estimated values.

2. Variation in the ground displacement pattern that might include assessing the pipeline assuming the surface ground displacements occur at the pipeline depth.

3. Extension of the pipeline model to represent a condition at the center of the river in which the pipeline is not anchored.

Step 9: Assess Potential for Landslide Hazard South of River Crossing

In addition to the lateral spread displacement, the slope south of the river is examined to determine if a more detailed geotechnical investigation of potential slope instability is necessary. The pipeline is located within the near-surface granular soils (Group B in Figure 2.8) that overly the weathered shale (Group C in Figure 2.8) of the hill. Two potential slide mechanisms are considered:

1. A shallow disruptive slide consisting of failure at the interface between surface soils and the underlying weathered shale.

2. A deep coherent slide within the weathered shale.

The weathered shale is assumed to provide a low-permeability boundary. Therefore, the interface between the surface soil and the weathered shale is assumed to be "wet" while the weathered shale is considered to be "dry." Noting that the slope of the hillside is 10%, the critical acceleration for sliding is estimated from Figure 2.8 to be 0.24 *g* for the disruptive slide and 0.20 *g* for the coherent slide.

The ground motions at the hillside are better represented by using soft rock conditions in the attenuation relationship (S_{sr} = 1). In the Campbell attenuation relationship, this results in the following additional factor:

$$0.44 - 0.17\ln R = 0.44 - 0.17\ln(40) = -0.187$$

The following correction factor is applied to the *PGA*:

$$PGA \cdot e^{-0.187} = 0.18\ g \cdot 0.83 = 0.15\ g$$

Since the *PGA* is less than the critical acceleration, there are no concerns for potential landslide movements.

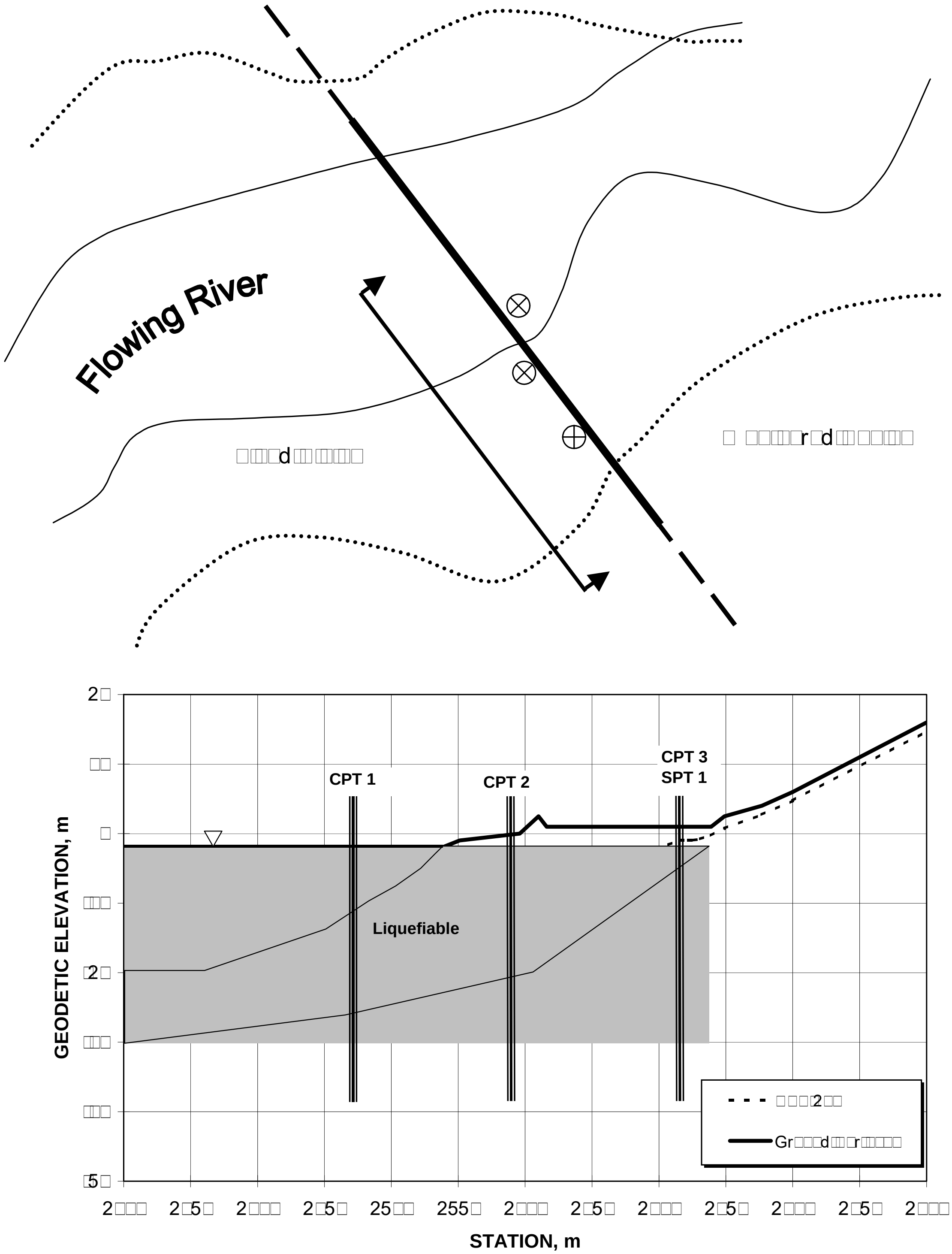

Figure D.1 General Plan and Profile of NPS 20 Pipeline River Crossing

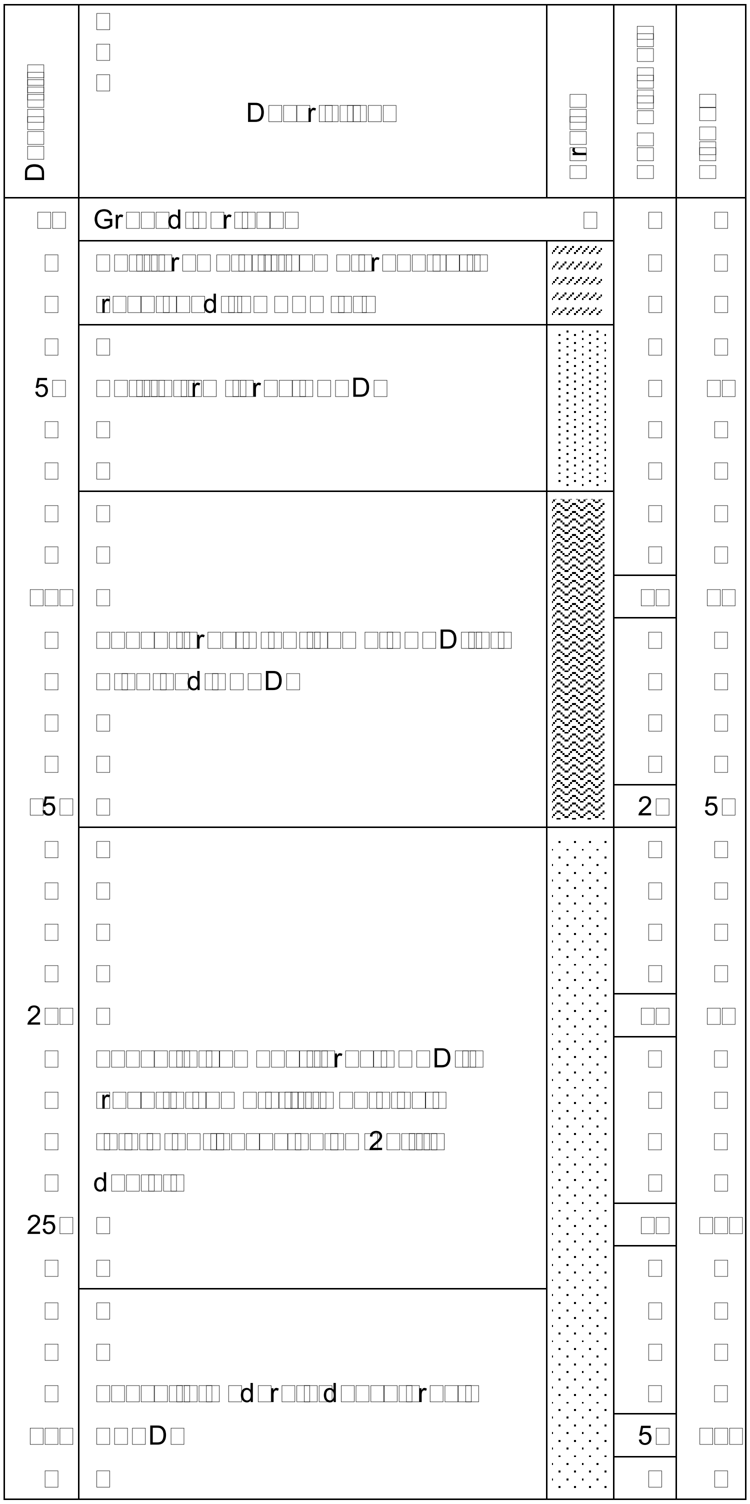

Figure D.2 SPT Boring Results

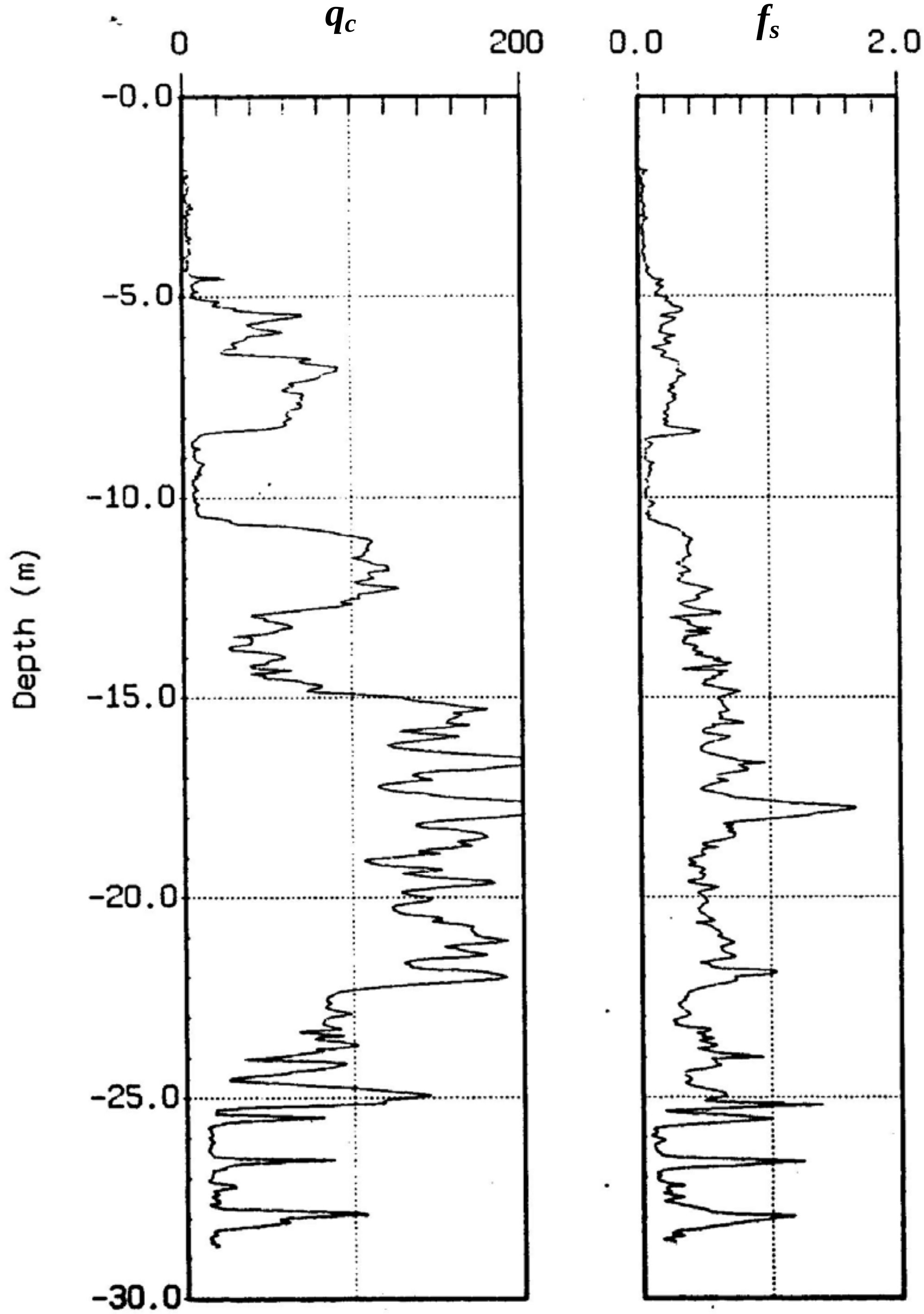

Figure D.3 Log from CPT 3

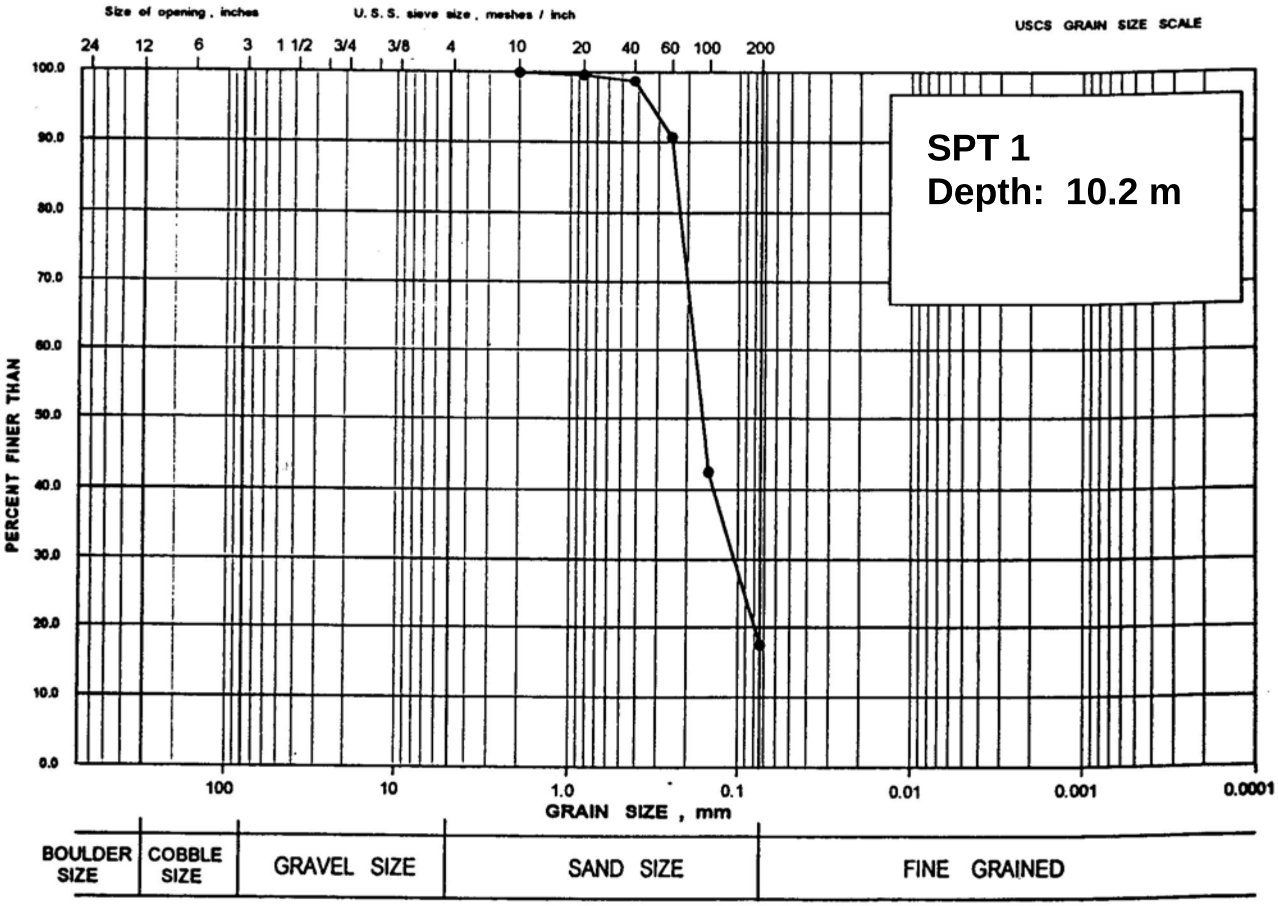

Figure D.4 Gradation Curve from SPT Sample

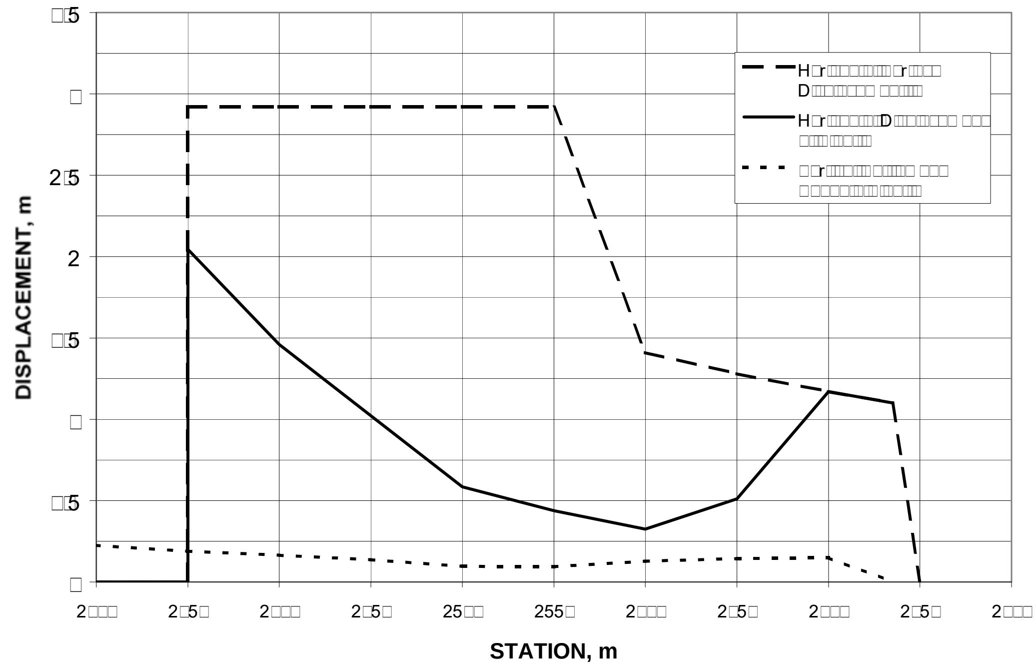

Figure D.5: Estimated Liquefaction-Induced Ground Displacement

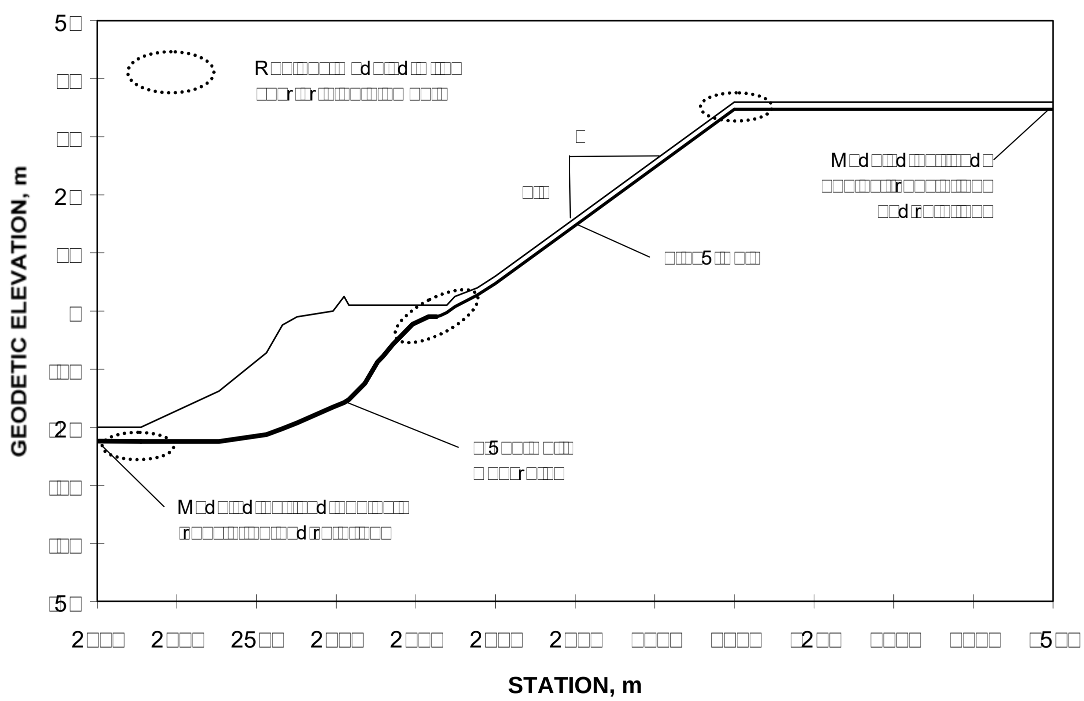

Figure D.6 Schematic Representation of Model Used to Analyze Pipeline Response

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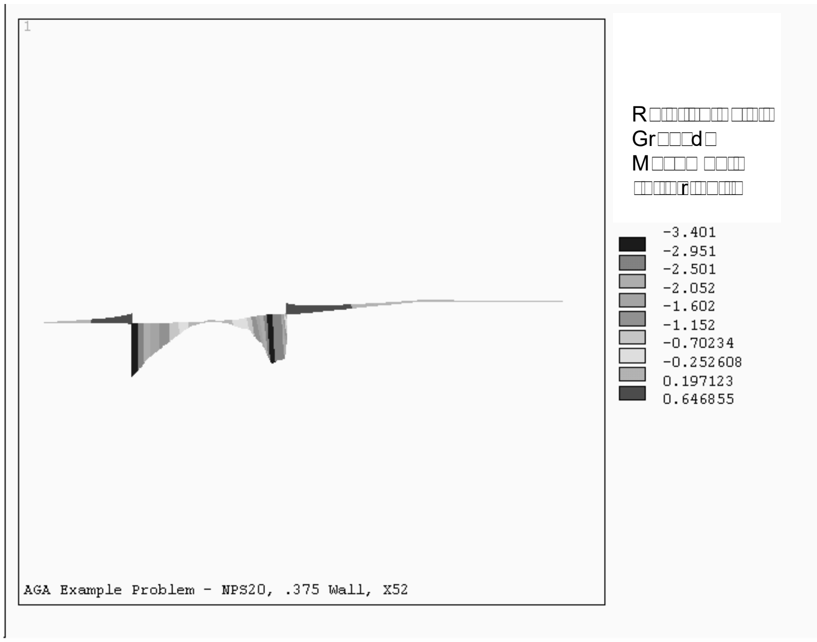

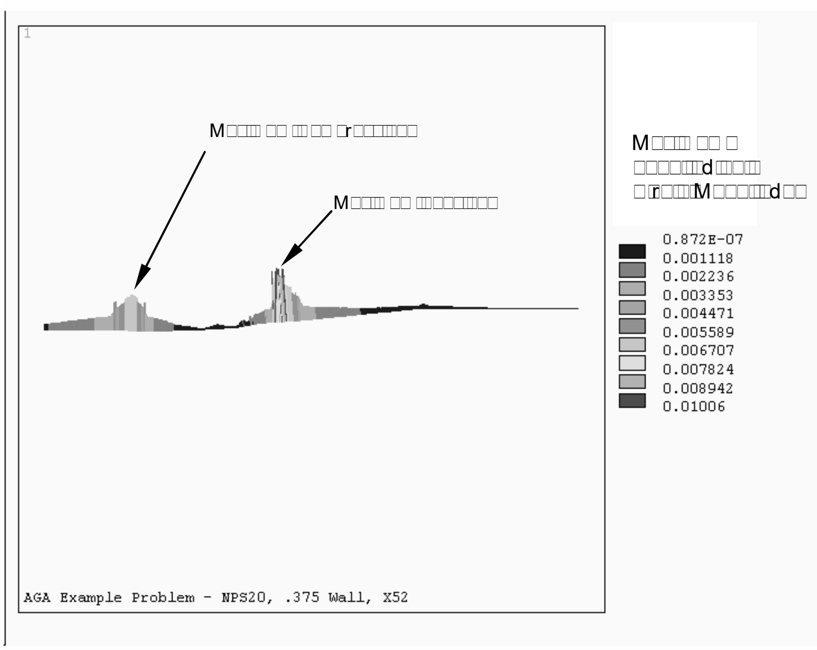

Figure D.7 Maximum Total Compression Strain at 4-m Maximum Ground Displacement

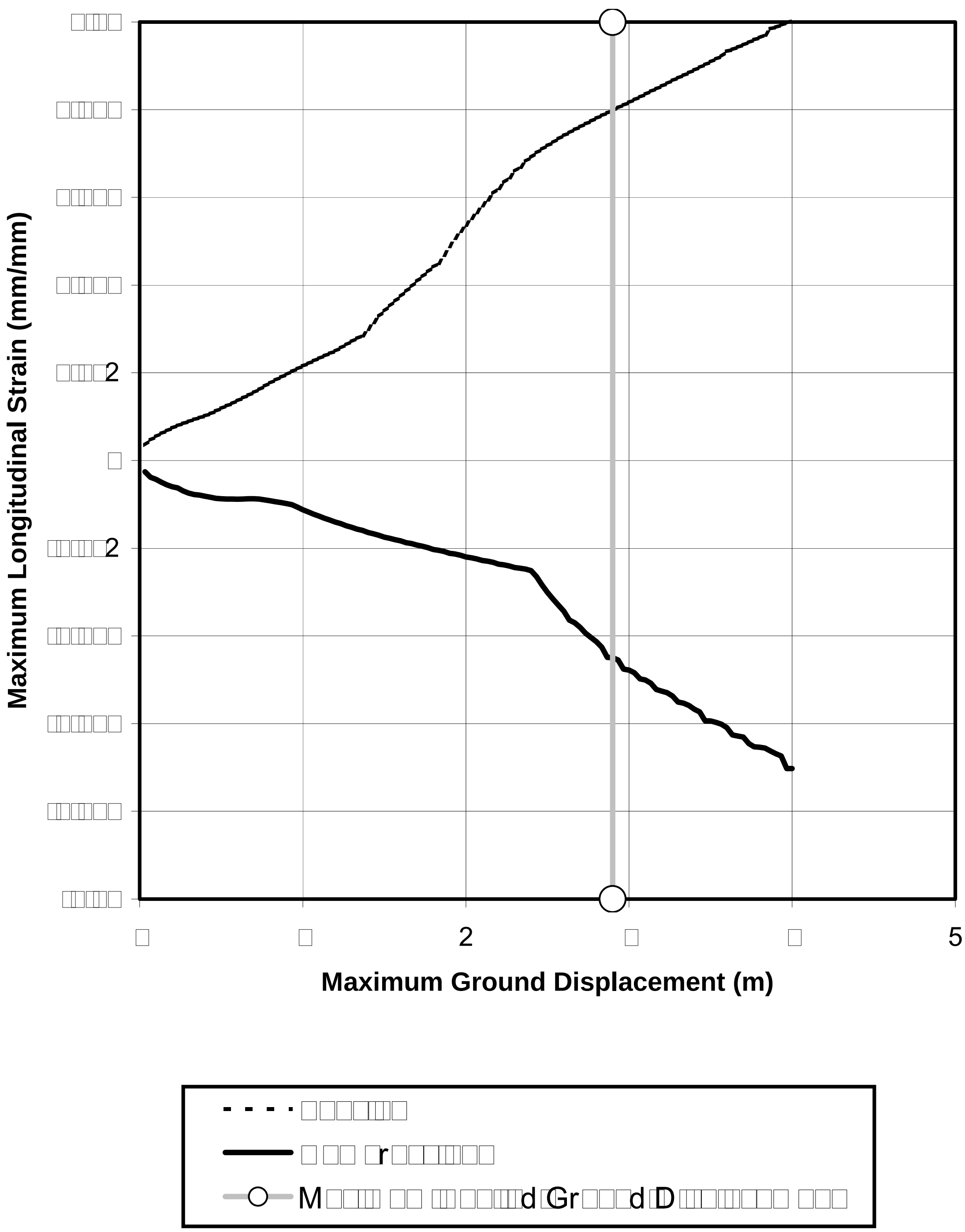

Figure D.8 Variation of Strain with Maximum Ground Displacement

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