Appendix D

Task 5.1, Technical Memorandum Seismic Displacements

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29 December 2010

Mr. Don Ballantyne
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Subject: Seismic Displacements for the Proposal for Professional Service to Complete a Downtown Anchorage Seismic Risk Assessment and Land Use Regulations to Mitigate Seismic Risk for Municipality of Anchorage

Dear Mr. Ballantyne:

The characterization of the magnitude and type of potential ground movement during major earthquakes in each of the four local ground failure hazard zones and its resulting impact on buildings and lifelines is an integral part of the seismic risk assessment for the Anchorage downtown area. This memorandum describes the proposed levels of potential seismic displacement for use in the Downtown Anchorage Risk Assessment. Using a scenario consistent with the International Building Code 2006 (IBC) design ground motions and the strength of materials derived from numerous sources, the probable range of seismic displacements have been developed.

With a sound understanding of the amount and type of ground movement possible, the structural engineers of this team can provide structural detail guidelines required for foundations and structural frames to establish minimum design standards for use in the development area.

DEFINITION AND DELINEATION OF SLOPES

The study area has been divided into five categories (zones) based on seismic risk as outlined in the 1979 Harding Lawson study:

- Zone 5 - Very-high – Areas of previous seismically-induced landslides. Includes zones of tension cracks above the head wall scarp, toe bulge, and pressure ridge areas. Although portions of these previous slides may remain relatively undisturbed from future strong shaking, these slides will be the more likely site of future seismically induced sliding.

- Zone 4 - High – Fine-grained, surficial and subsurface deposits within the vicinity of steep slopes, includes area above and below the slope. Highly susceptible to all types of seismically-induced ground failure, including liquefaction, translational sliding, lurching, land spreading, cracking, and subsidence.

- Zone 3 - Moderate – Fine-grained surficial and subsurface deposits, including the Bootlegger Cove Clay and other silt, clay and peat deposits. May experience ground cracking and horizontal ground movement due to land spreading or lurching, and subsidence due to consolidation.

- Zone 2 - Moderate-Low – Mixed coarse and fine-grained glacial deposits in lowland areas, thick deposits of channel, terrace, flood plain and fan alluvium. May have very low susceptibility; may experience minor ground cracking, localized settlement due to consolidation, and perhaps liquefaction or lurching of localized saturated zones of fine-grained material.

- Zone 1 - Lowest – Includes exposed bedrock, thin alluvium and colluvium over bedrock. May experience minor ground cracking and acceleration of normal mass wasting process in unconsolidated material such as rock falls and snow avalanches.

The existing topography was provided by the Planning Department of the Municipality of Anchorage and was input into a Geographic Information System (GIS). The downtown area was then layered with the seismic zones described above. Four foot contours of the topography were used to describe slope geometries throughout the study area.

Twenty cross sections were drawn in the study area to characterize a representative range of the slopes found within each zone. An overlay of the study area with seismic zones and cross section lines is provided below in Figure 1 and in Appendix B.

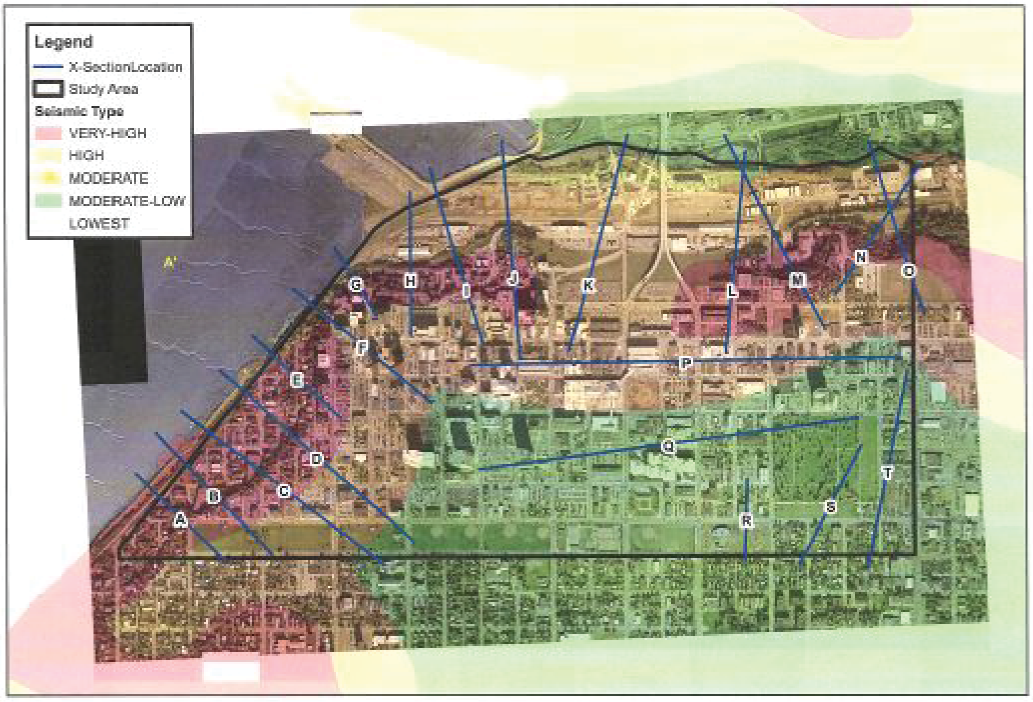

Figure 1. Overlay of Downtown Anchorage with Seismic Zones and Cross-Sectional Lines

Each of the twenty cross sections was studied and 61 slopes within these cross sections were analyzed. The number of slopes analyzed within each of the zones was:

- Zone 5: 40 slopes
- Zone 4: 15 slopes
- Zone 3: 1 slope
- Zone 2: 5 slopes

As suggested in the descriptions of the zones above, Zones 4 and 5 are expected to produce the greatest number of slopes with large seismically-induced permanent displacements. These two zones also have the highest concentration of slopes within their boundaries. Zone 3 has only small acreage within the study area and is located in a flat plain. For this reason, only one slope was analyzed. Zone 2 is relativity flat with small undulating hills.

ANTICIPATED SLOPE DISPLACEMENTS DUE TO SEISMIC SHAKING

In this study, the potential seismic displacement for each slope was estimated using the Bray and Travasarou (2007) simplified seismic slope displacement procedure. This procedure requires as input the slope geometry and soil properties to estimate the slope's characteristic dynamic resistance (i.e., its yield coefficient) and its dynamic response characteristics (i.e., the potential sliding block's fundamental period). The seismic demand was defined using the International Building Code (2006) for the likely level of earthquake shaking. The details of this analysis are described in Appendix A.

In many of the cross sections, multiple steep slopes that could produce shallow landslides existed within the mass of a potentially large translational slide. Given this complexity, all slopes, both shallow and translational, were independently evaluated to calculate anticipated displacements. Histograms of all calculated seismic slope displacements are in Appendix A.

For each cross section, if the seismic displacements for the large translational slides were greater than the anticipated seismic displacements for the shallow slides located within the larger slide mass, the displacements for the larger slide were utilized in the subsequent analysis. The smaller slopes on the slide mass were assumed to essentially "go along for the ride." Alternatively, if the large translational slide does not mobilize, it is probable that the shallower landslides will dominate the seismic displacements, and the shallower slides were thus utilized.

The slopes and their calculated seismic displacements were then placed on the Harding Lawson maps. Using engineering judgment, the plan view square footage for the potential displacements were calculated for each seismic region. The results of the seismic slope displacement analyses are presented in Figure 2 and Figure 3. The results are categorized by seismic zone in Figure 2 and by ranges of the calculated seismic displacement in Figure 3.

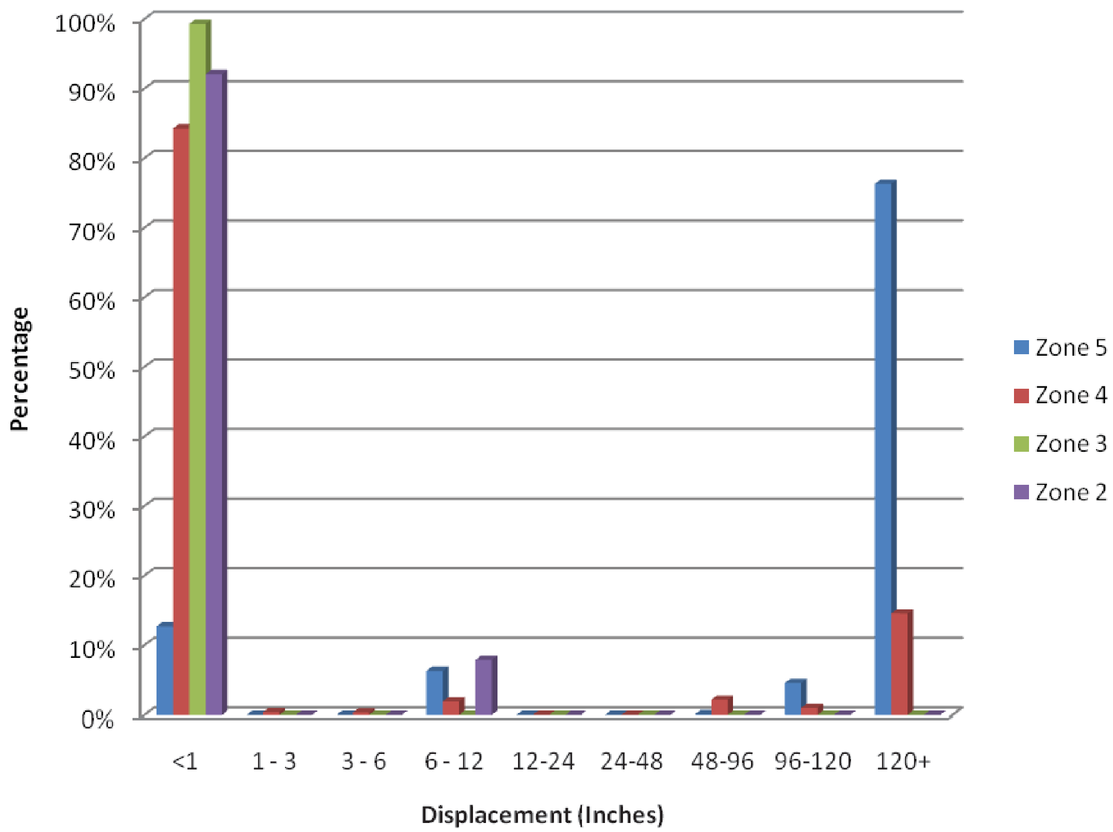

Figure 2. Displacements by Percentage of Area by Zone

Zone 5 is the region for which the largest potential seismic displacements are predicted. Over 80% of the area of Zone 5 would likely experience more than eight feet of seismic slope displacement during the design level of earthquake shaking in Anchorage. These large, translational slides are typically through the Bootlegger Cove Clay. Shallower slopes, mainly comprised of 30 to 50 feet of sands and gravels, are expected to "go along for the ride" within the larger sliding mass. However, there are areas within Zone 5 which may have 6 to 12 inches or less displacement, or even negligible estimated seismic displacement. This suggests that some refinement of the generalized seismic zonation of the 1979 Harding Lawson maps may be possible.

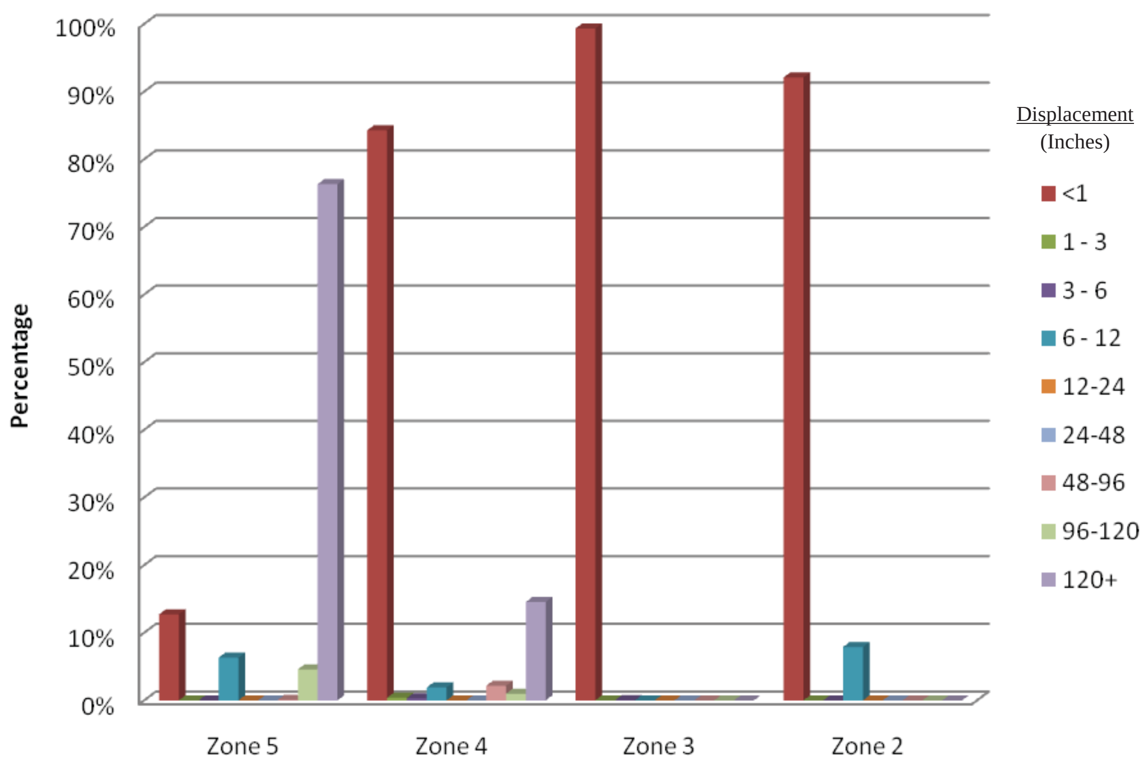

Figure 3. Displacements by Percentage of Area within each Zone

Zone 4 has the largest square footage in comparison to the other zones. Results for Zone 4 show that calculated seismic displacements can be large with almost 20% of the region possibly displacing more than four feet for the design earthquake scenario. These slides are large, translational slides within the Bootlegger Cove Clay and are without a toe berm or pressure ridge. Shallower potential slope failures comprised of the sands and gravels within the larger sliding mass are expected to "go along for the ride" within the larger sliding mass.. Approximately 3% of Zone 4 will likely displace between 6 and 12 inches for the design event. These areas are generally directly behind Zone 5. A movement of about 6 inches is commensurate with observations of movement behind the scarps from the 1964 earthquake (Long, 1973; Hansen, 1965). However, a considerable percentage of the Zone 4 is more stable within areas significantly behind Zone 5 or within the areas of the toe berm/pressure ridge.

Zone 3 has only one gentle slope within the study area. It is anticipated that this slope will displace less than 1 inch during the design event. Additionally, this shallow slide is less than 1% of the plan area of Zone 3 within the study area. Thus, Zone 3 does not present a significant seismic slope displacement hazard within the study area.

Zone 2 is characterized by large square footage and does not contain significant open faces or steep slopes. The relatively few slopes in this area are moderate and may move on the order of 6 to 12 inches during the design event. These slopes represent less than 10% of the land mass of Zone 2 within the study area.

The orientations of the calculated seismic displacements are aligned roughly with the slope topography. Gentle slopes that displace on a horizontal base sliding plane will displace largely horizontally. Steep slopes on inclined base sliding planes may displace vertically as much as they displace horizontally. Intermediate cases will include both horizontal and vertical components of displacement.

In addition to using slope topography to estimate the likely orientation of the calculated seismic displacement vector, observations of slope movements in the study area during the 1964 Alaska earthquake were considered. For example, the L Street Slide moved 14 feet horizontally and 10 feet vertically (about a 1.5H:1V ratio). The 4th Avenue slide moved between 19 feet to 11 feet horizontally and 10 feet vertically, or approximately a 2H:1V to 1H:1V ratio of displacements. Based on a comparison with existing topography and these observations of past performance, these relationships may provide some insight into potential relative amounts of horizontal and vertical movements within each zone:

- Zone 5 with an open face: 1.5H : 1V
- Zone 4 with an open face: 1.5H : 1V
- Zone 4 without an open face: 3H : 1V
- Zone 3: primarily horizontal movement with minor vertical movement
- Zone 2: primarily horizontal movement with minor vertical movement

CLOSURE

This study was performed to provide a generalized index of the potential seismic displacements of the various slopes in the study region under the design seismic event. The resulting displacement estimates are provided for the development of general design guidelines and common structural details for foundations and building frames so that a consistent response to these hazards can be developed. Results indicate there is a significant potential for reoccurrence of large ground movements in Downtown Anchorage as a result of another major earthquake.

This work is based on generalized soil profiles and simplified analysis methods. Furthermore, this study does not include displacements along the fringe of the study area, such as the area near the coastline, since the underwater topography is unknown. The estimated displacements should not be considered a replacement for site specific evaluations for any projects within the study area.

This report was prepared in accordance with general standards of engineering practice. Geosyntec is not responsible for the use of the data or conclusions presented in this report for any purposes other than those specifically expressed herein. If you have any questions regarding this report or require any additional information, please do not hesitate to contact the undersigned at (510) 836-3034.

Sincerely,

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Project Engineer

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BIBLIOGRAPHY

Boulanger, R. W., and Idriss, I.M. (2004), "Evaluating the Potential for Liquefaction of Cyclic Failure of Silts and Clays", Center for Geotechnical Modeling, Dept. of Civil & Environmental Engineering, University of California Davis.

Bray, J. and Travasarou, T. (2007). “Simplified Procedure for Estimating Earthquake Induced Deviatoric Slope Displacements” ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133(4), p. 381-392.

Harding-Lawson Associates (1979) “Seismically-Induced Ground Failure Susceptibility”, Geotechnical Report for Municipality of Anchorage, Alaska.

Hansen, W. R. (1965). "Effects of the Earthquake of March 27, 1964 at Anchorage, Alaska", The Alaska Earthquake, March 27, 1964, Effects on Communities, USGS Professional Paper.

Idriss, I.M. (1985). "Evaluating seismic risk in engineering practice," Proc., 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Balkema, Rotterdam, p. 265-320.

International Building Code (2006).

Jibson, R., and Michael, J. (2008), “Maps Showing Seismic landslide Hazards in Anchorage, Alaska” USGS Report.

Long, E. L. (1973), "Earth Slides and Related Phenomena," The Great Alaska Earthquake of 1964, National Academy of Science, 644-773.

Mitchell, J. K., Houston, W. N., Yamane, G (1973), "Sensitivity and Geotechnical Properties of Bootlegger Cove Clay", The Great Alaska Earthquake of 1964, Committee on the Alaska Earthquake of the Division of earth Sciences, National Research Council.

Moriwaki, Y., Vicente, E. E., Lai, S. S., Moses, T. L. (1985), "A Re-evaluation of the 1964 "L" Street Slide, State of Alaska, Department of Transportation and Public Facilities.

Newmark, N. (1965), “Effects of earthquakes on dams and embankments,” Geotechnique, Vol. 15, No. 2, p. 139-160.

Shannon & Wilson (1964), "Anchorage Area Soil Studies," U.S. Army Engineer District, Alaska, Corps of Engineers

Shannon & Wilson (1965), "Remolding of Bootlegger Cove Clay with Explosives," U.S. Army Engineer District, Alaska, Corps of Engineers

Shannon and Wilson (1995), Final Geotechnical Report, Northwest Turnagain Landslide, Stability Evaluation, Anchorage Alaska.

Shannon & Wilson (1989) "Geotechnical Report, Turnagain N.E. Landslide Re-evaluation, Anchorage, Alaska" Municipality of Anchorage.

Shewbridge, S.E. (1996), "Yield Acceleration of Lined Landfills", ASCE Journal of Geotechnical Engineering, Vol. 122 (2), p. 156-158.

Stark, T. D., and Contreras, I.A. (1998), "Fourth Avenue Landslide During 1964 Earthquake", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, p. 99-109.

Woodward-Clyde (1982), Anchorage Office Complex, Geotechnical Investigation, Anchorage, Alaska.

Woodward-Clyde (1987), Geotechnical Investigation, Anchorage Courthouse addition, Anchorage, Alaska.

Yasuhara, K., Murakami, S., and Hyde, A.F.L. (2004) "Post-cyclic degradation of saturated plasticity silts," Cyclic behaviour of soils and liquefaction phenomena: Proceedings of the International Conference on Cyclic Behaviour of Soils and Liquefaction Phenomena, 31 March-02 April 2004, Bochum, Germany

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APPENDIX A

SUPPORTING DOCUMENTATION OF CALCULATIONS

A.1 ESTIMATION OF UNDRAINED SHEAR STRENGTH (S_U)

The seismic performance of buildings and lifelines in downtown Anchorage depends greatly on local ground conditions. Damage in Anchorage during the 1964 Alaskan Earthquake was largely dictated by ground failure resulting from seismic slope instability, such as from the 4th Avenue slide and L Street slide. This is recognized in Recommendation 8 of Chapter 4 of the Anchorage Downtown Comprehensive Plan (i.e., “Address Seismic Hazards”). The potential for significant seismic displacements are largely dependent on the undrained shear strength of the soil, the slope’s topography, and the intensity and duration of earthquake shaking. To estimate the shear strength of the soil, a comprehensive literature review was completed and the resulting soil characterization was reviewed by a local experienced geotechnical engineer.

A.1.A Literature review of undrained strength

Since the 1964 earthquake, the undrained shear strength of the Bootlegger Cove Clay has been carefully examined and reported on in numerous studies and journal articles. This information was assimilated, and a weighting system based on judgment was developed to assign an appropriate undrained shear strength for each of the potential slides identified in Appendix B. The studies considered primarily in this effort are summarized in Table 1.

Table 1. Considered Values for S_u

Author (s)	Proposed Values for S_u
Shannon & Wilson[1]	0.25 - 0.35 tsf
Idriss[2, 3]	$Su/\sigma'_v = 0.19\ (OCR)^{0.78}$, where OCR ~ 1.2-1.5
Mitchell, et. al. [4]	0.20 - 0.65 tsf from studies
	0.4 - 1.0 tsf from previous studies on sensitive clay
Moriwaki, et. al. [5]	$Su/\sigma'_v = 0.185\ (OCR)^{0.78}$, where OCR ~ 1.2-1.6
	0.65 - 1.6 ksf
Stark & Contreras [6]	$Su/\sigma'_v = 0.28\text{-}0.31$ (natural soils)
	$Su/\sigma'_v = 0.17\text{-}0.23$ (inside sliding mass)
Woodward-Clyde [7]	$Su/\sigma'_v = 0.19\ (OCR)^{0.78}$, where OCR ~ 1.8

Notes:

1) Shannon and Wilson (1995) Final Geotechnical Report, Northwest Turnagain Landslide, Stability Evaluation, Anchorage Alaska
2) Idriss, I.M. (1985). "Evaluating seismic risk in engineering practice," Proc., 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Balkema, Rotterdam, 265-320.
3) Boulanger, R. W., and Idriss, I.M. (2004), "Evaluating the Potential for Liquefaction of Cyclic Failure of Silts and Clays", Center for Geotechnical Modeling, Dept. of Civil & Environmental Engineering, University of California Davis
4) Mitchell, J. K., Houston, W. N., Yamane, G (1973), "Sensitivity and Geotechnical Properties of Bootlegger Cove Clay", The Great Alaska Earthquake of 1964, Committee on the Alaska Earthquake of the Division of earth Sciences, National Research Council
5) Moriwaki, Y., Vicente, E. E., Lai, S. S., Moses, T. L. (1985), "A Re-evaluation of the 1964 "L" Street Slide, State of Alaska, Department of Transportation and Public Facilities
6) Stark, T. D., and Contreras, I.A. (1998), "Fourth Avenue Landslide During 1964 Earthquake, "ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 124, 99-109
7) Woodward-Clyde (1987), Geotechnical Investigation, Anchorage Courthouse addition, Anchorage, Alaska

A.1.B Definition of Areas 1 and 2

For the purpose of this investigation, the study area was divided into two areas: Area 1 and Area 2. Area 1 is comprised of regions which failed during the 1964 earthquake and are expected to have lower strength material. Although it is expected that the strength of the materials has increased since 1964, the remolded material is not of the same strength as that of Area 2. This area is considered to have an OCR (overconsolidation ratio) of between 1.2 to 1.4. An OCR of 1.3 was selected as being representative of these areas.

Area 1 is comprised of Profiles A through G and L through O and is representative of the soil within the L Street Slide and 4th Avenue Slide areas.

Area 2 is comprised of regions within the study area which did not fail during the 1964 earthquake. It is reasonable to estimate that the strength and OCR of the material within these regions are relatively higher than those in Area 1. Area 2 is considered to have an OCR between 1.5 to 1.8, and for this study an OCR of 1.65 was used in this area.

Area 2 is comprised of Profiles H through K and R through T and is representative of the soils near the Courthouse site and within the remainder of the study area.

A.1.C Methodology for Finding S_u

Following discussions with a local experienced geotechnical engineer, Mr. K. Mobley, the estimated value of the critical peak undrained shear strength of the Bootlegger Cove Clay was based upon the studies summarized in Table 1 using the weighting scheme shown in Table 2. Method A is largely based on the work of Woodward Clyde Consultants as described in References 2, 3, 5, and 7. Method B is based on the more recent re-examination of the Bootlegger Cove Clay by Stark and Contreras (1998; i.e., Reference 6). Method C is largely based on the work of Mitchell et al. (1973) and Shannon and Wilson (1995), which are References 4 and 1, respectively. The undrained shear strength was calculated using each method and the undrained shear strength was then assigned for a given slope.

Table 2. Values for Su and their Weighted Average

Method	Values for Su	Weighting
A	$Su/\sigma'_v = 0.19\ (OCR)^{0.78}$, where OCR ~ 1.3[1] & 1.65[2]	50%
B	$Su/\sigma'_v = 0.2$[1], $Su/\sigma'_v = 0.29$[2]	35%
C[3]	0.3 - 0.5 tsf	15%

Notes:

1. Area 1: Comprised of the L Street and 4th Avenue slides.
2. Area 2: The Courthouse site and 4th Street through 1st Street zone.
3. For Method C:
   a. If OCR = 1.3 and σ'_v < 2200 psf, Su = 0.3 tsf
   b. If OCR = 1.3 and σ'_v > 2200 psf and < 4500 psf, Su = 0.4 tsf
   c. If OCR = 1.3 and σ'_v > 4500 psf, Su = 0.5 tsf
   d. If OCR = 1.65 and σ'_v < 2500 psf, Su = 0.35 tsf
   e. If OCR = 1.65 and σ'_v > 2500 psf and < 3500 psf, Su = 0.4 tsf
   f. If OCR = 1.65 and σ'_v > 3500 psf, Su = 0.5 tsf

A.1.D Cyclic Degradation

Previous research and observations have shown in large magnitude events with long durations of shaking, in fine-grained, saturated soils, there is a significant decrease in strength and stiffness leading to large displacements (Yasuhara, et. al., 2004). The 1964 earthquake had a very long duration with between 3 to 5 minutes of strong shaking. Before a sensitive clay reaches residual strength, it is customary for the peak static strength to be reduced by each successive cyclic of loading to a peak dynamic undrained shear strength. This cyclic degradation may be between 15% to 30% of the initial static peak undrained shear strength.

For this analysis, a peak dynamic undrained shear strength of 0.75 times the peak static undrained shear strength was utilized. Back analysis of the 4th Street and L Street slides that formed during the 1964 Alaskan earthquake were valuable in selecting this value.

A.1.E Residual Undrained Shear Strength

The undrained shear strength discussed to this point represents the Bootlegger Cove's peak dynamic strength. When a sensitive soil undergoes large strain, its shear strength reduces significantly. Eventually, the soil will reach its "residual" undrained shear strength. Large shear strains may be induced by multiple cycles of intense earthquake loading, which can lead to large seismic displacements as the soil is remolded as it is deformed.

Previous research by Woodward and Clyde (1982) and Idriss (1985) indicate that the Bootlegger Cove Clay's residual undrained shear strength may be between 20 percent and 30 percent of the initial undrained shear strength when subjected to large shear strains. For this study, the clay's residual undrained shear strength was selected to be 20% of its initial undrained peak static shear strength. This value is based in part on the results of our back analysis of the L Street and 4th Street slides during the 1964 Alaskan earthquake with our model.

A typical dynamic strength versus displacement relationship for a sensitive clay is shown in Figure 4. It is likely that after displacing 6 to 12 inches (i.e., about 9 inches), the clay's strength will start reducing significantly and eventually reach its residual strength at 3 to 5 feet (i.e., about 4 feet) of displacement. This response was model with the idealized step function shown in Figure 4. For seismic displacements calculated to be greater than 12 inches, it was assumed that the clay immediately dropped to its residual strength. The calculations of Bray and Travasarou (2007) were performed again using the residual strength. Using this approach, seismic displacement was not likely to be calculated within the range of 12 to 48 inches. This implies that slopes were likely to move only a small amount (i.e., less than about a foot) or they would likely move several feet. These results mirror observations after the 1964 earthquake of displacements, wherein slopes either moved a few inches or several feet (Shannon and Wilson, 1964).

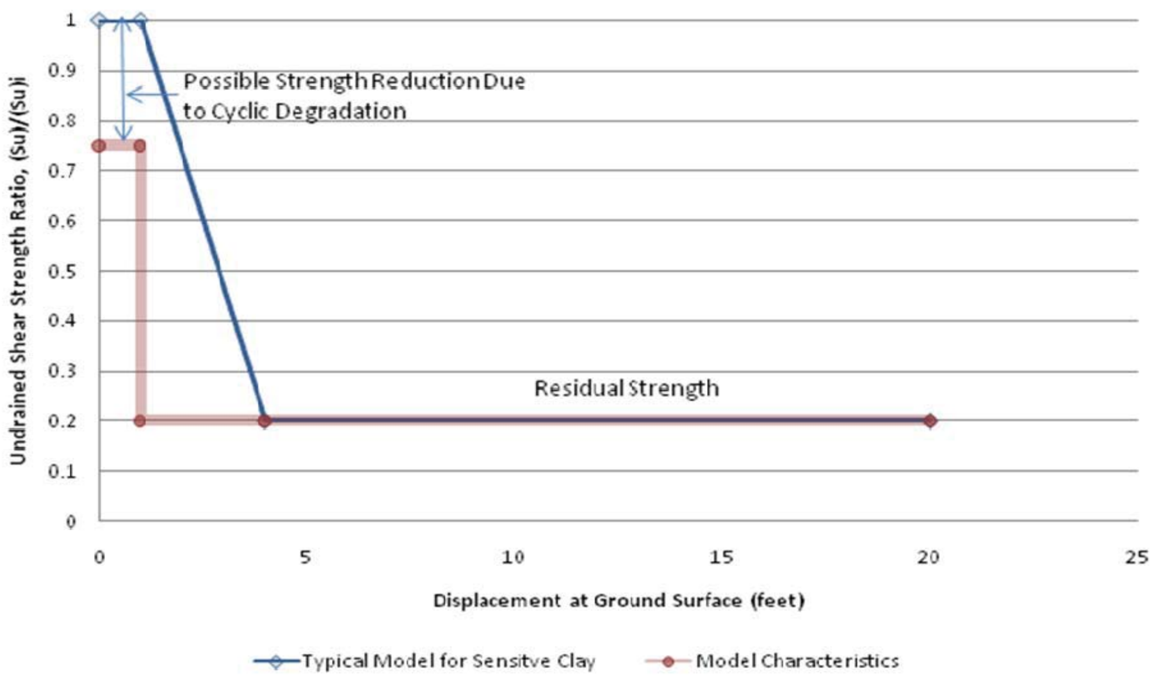


Figure 4. Idealized Dynamic Shear Strength versus Displacement for Bootlegger Clay in this Study

A.2 CALCULATION OF YIELD COEFFICIENT (k_y)

The yield coefficient (k_y) was calculated using an average of the simplified methods of block sliding (Shewbridge, 1996) ($k_{y(block)}$) and infinite slope ($k_{y(inf\ slope)}$). For each of the slopes identified, an independent yield coefficient was calculated using the dynamic undrained shear strength of the Bootlegger Cove Clay, unit weight (γ), height of the slope (H), length of the face (L_{face}) and the length of sliding mass beyond the slope (L). Because the undrained shear strength (S_u) was utilized in place of the cohesion (c), the internal friction angle (φ) was set to zero. For the infinite slope analysis, because the failure planes are nearly horizontal the angle of the failure plan to horizontal, angle β, was also set to zero.

The slopes identified for this study generally have geometries that are intermediate between the simplified block sliding mechanism and the simplified infinite slope mechanism. Thus, as an approximation, each slope's k_y value was calculated using a procedure that captured each of these mechanisms. Then the k_y values calculated by each procedure were averaged to provide a best estimate of the actual k_y value of the slope. This simplified k_Y calculation procedure was checked by performing 2-D SLOPEW analyses of six slopes and its results were found to compare reasonably well.

A.3 ESTIMATION OF SPECTRAL ACCELERATION (Sa)

As stated in the Memorandum "Downtown Anchorage Seismic Risk Assessment – Task 1, Proposed Earthquake Scenario" dated January 23, 2009, the scenario earthquake was selected to be consistent with the International building Code (IBC) design ground motions. The IBC requires buildings to be designed to two-thirds of the probabilistic ground motion that has a 2 percent chance of occurring in 50 years.

The geology underlying downtown Anchorage suggests that the site could be classified as a Site Class D (Stiff soil profile) or Site Class E (Soft Soil Profile) according to the 2006 International Building Code. As will be explained in Section A.4, Calculation of Seismic Displacements, the spectral acceleration (Sa) at the degraded period of the sliding mass ($1.5T_s$), is ultimately used to evaluate the displacements. Most of the degraded periods used for the stability analysis are less than 0.6 seconds. Between 0.1 seconds and 0.6 seconds, a Site Class D has higher spectral accelerations than a Site Class E. Therefore, because this presents a more critical case, Site Class D was utilized.

Using the IBC 2006 and the USGS "Seismic Hazard Curves and Uniform Hazard Response Spectra" application, the 5% damped elastic response spectrum for this event was generated. Using the IBC 2006 Figures 1613.5 (11) and (12) and zip codes 99501, 99510, and 99513, the values of S_S and S_1 were estimated to be:

- S_S: 1.49
- S_1: 0.55

These values for S_S and S_1 were also compared to data generated for Latitude: 61.2165N and Longitude: 149.8996 W.

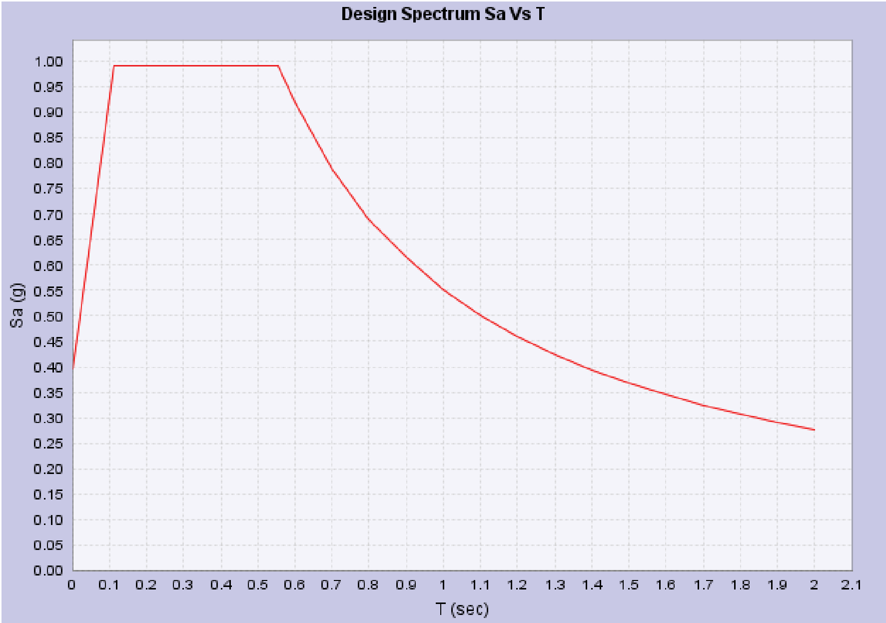


Figure 5. Design Acceleration Response Spectrum for Study Area (5% damping)

A.4 CALCULATION OF SEISMIC DISPLACEMENTS (D)

The seismic performance of a clay slope is typically evaluated in terms of seismically induced permanent displacement through a Newmark (1965)-type sliding block analysis. The Bray and Travasarou (2007) simplified procedure was utilized in this study. This procedure provides probabilistically based estimates of seismic displacement resulting from slope instability. It has been calibrated with several case histories and agrees with other widely accepted methods. This method takes advantage of nearly 700 recorded time histories from over 40 earthquakes, employs a calibrated, realistic soil model, and correctly captures key sources of uncertainty. This procedure has been widely used in the United States and has been adopted by the British Columbia, Canada Government's Building Policy Advisory Committee for implementing "The Geotechnical Slope Stability (Seismic) Regulation M-268."

The Bray and Travasarou (2007) procedure utilizes the moment magnitude (M_w), fundamental period of the potential sliding mass (T_s), the spectral acceleration at the degraded period of the sliding mass ($S_a(1.5T_s)$), and the yield coefficient (k_y) to estimate seismic slope displacement. Each slope was characterized by these parameters and the median seismic displacement for each slope was calculated.

The calculated median seismic displacement should be interpreted as an index of the seismic performance of the slopes in Anchorage. There are considerable uncertainties involved in this estimation, and there is inherent variability in seismic displacements for slopes with similar conditions. There will be a distribution of seismic displacements about each median calculated value. Thus, the calculated median seismic displacement provides an index of the likely seismic performance of each slope analyzed in this study.

For each zone, the calculated seismic displacements were categorized into these bins:

- < 1”
- 1” – 3”
- 3” – 6”
- 6” – 12”
- 12” – 24”
- 24” – 48”
- 48” – 96”
- 96” – 120”
- 120” and above.

The results in comparing the displacement for each slope are presented in Figure 6 and Figure 7 below.

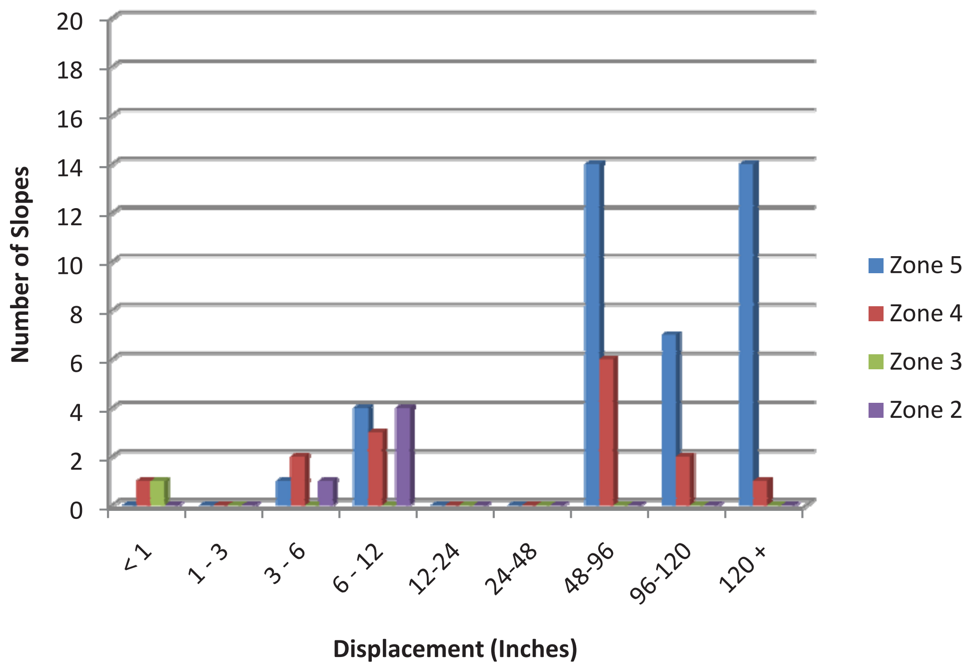


Figure 6. Histogram of Displacements by Slope per Zone

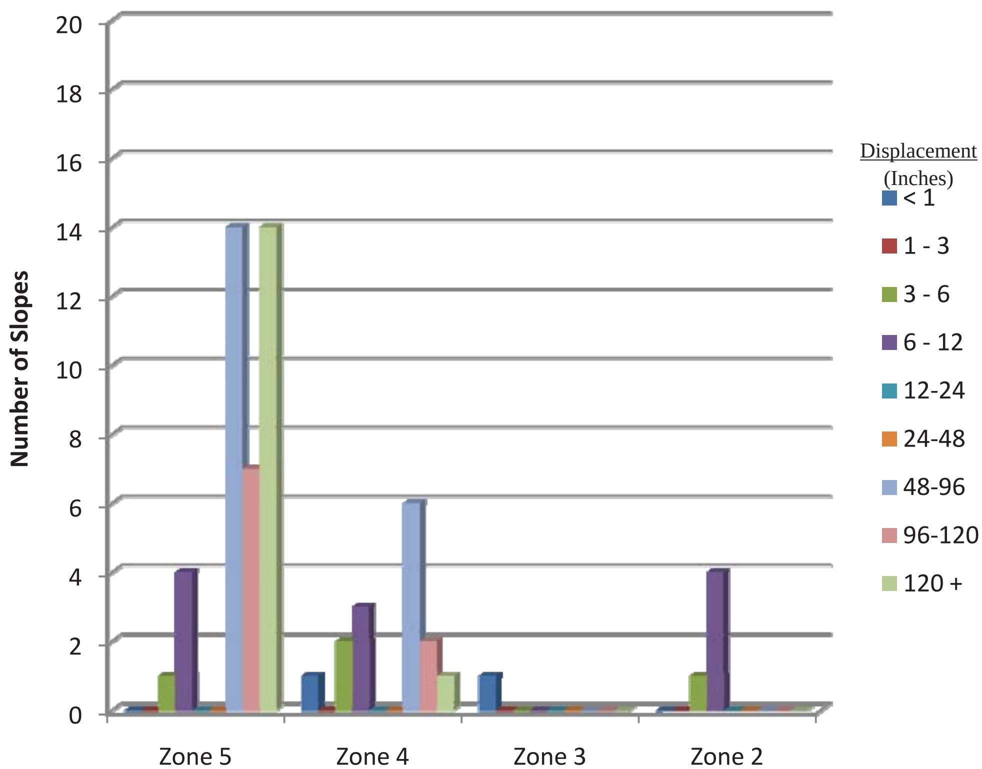
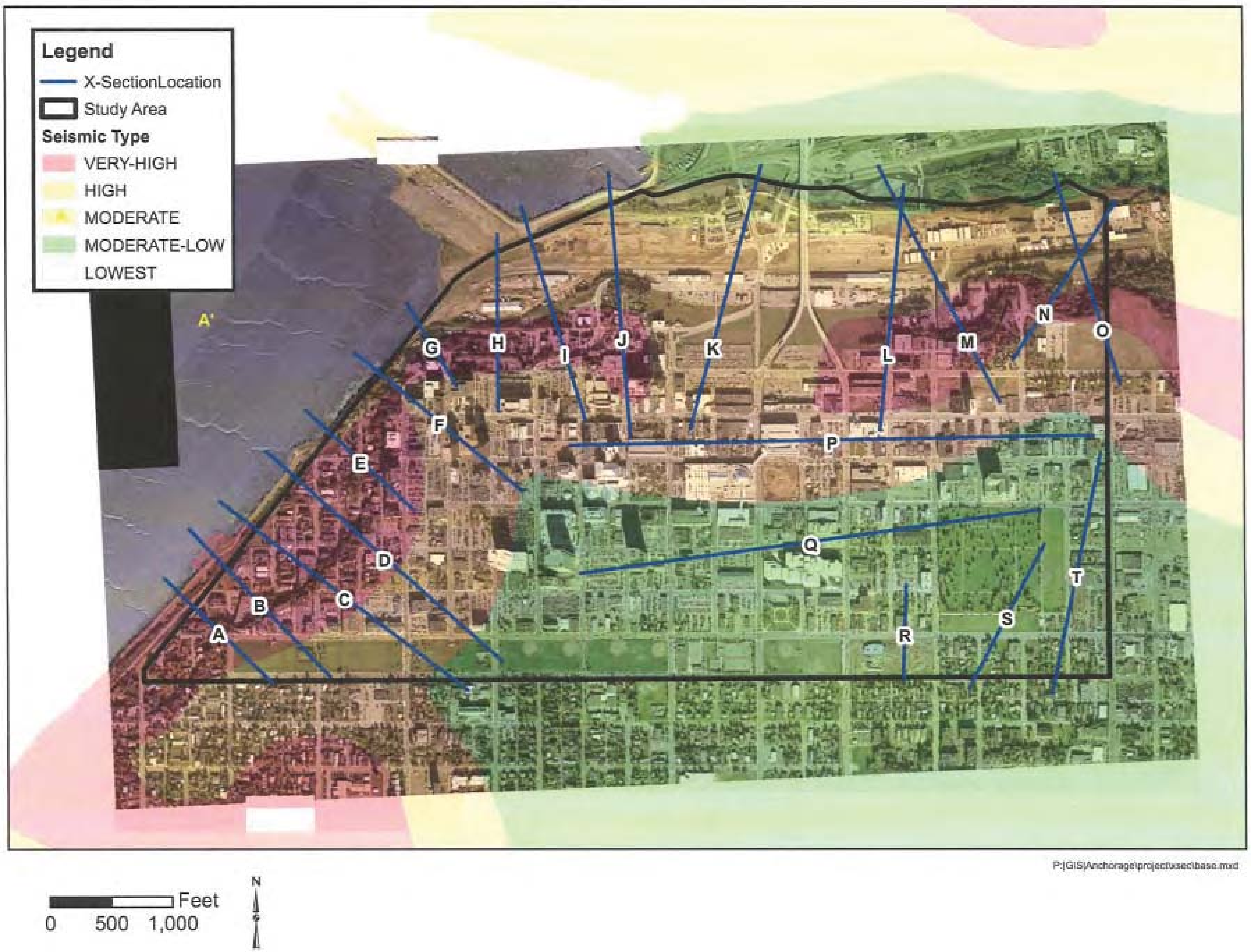


Figure 7. Histogram of Slope Displacements per Zone

APPENDIX B

Cross Section Locations and Profiles


Legend
X-SectionLocation
Study Area
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VERY-HIGH
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MODERATE
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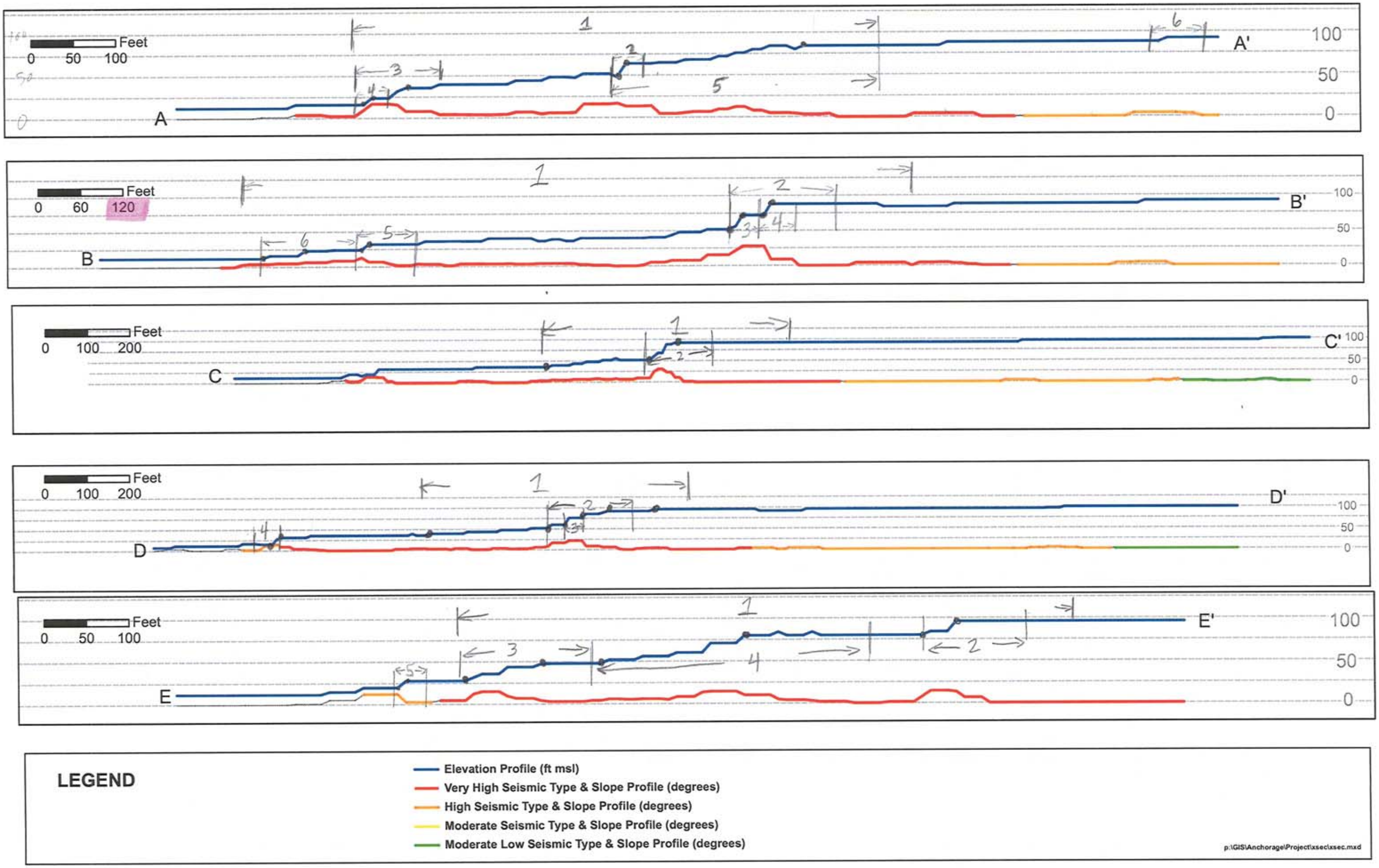
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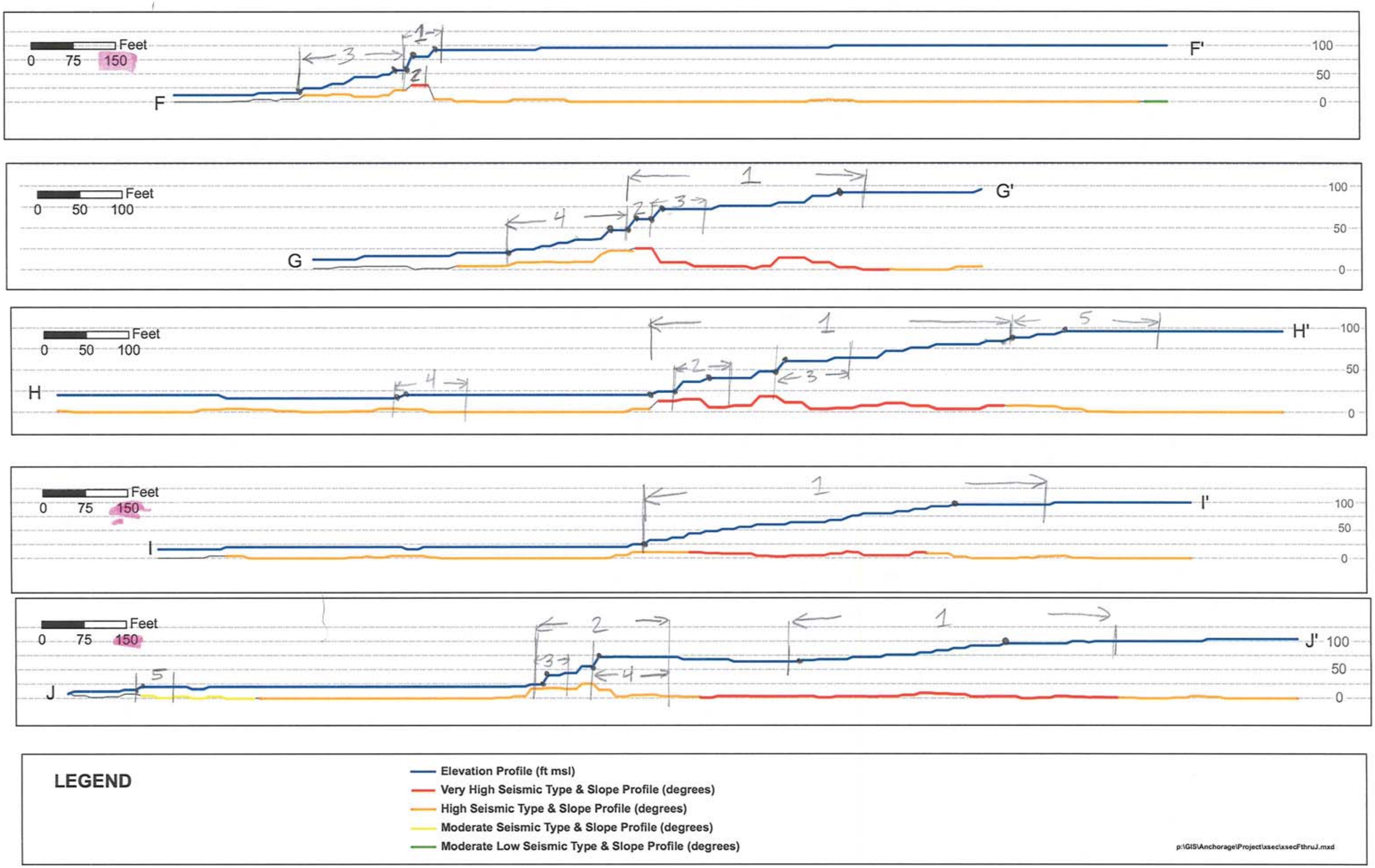
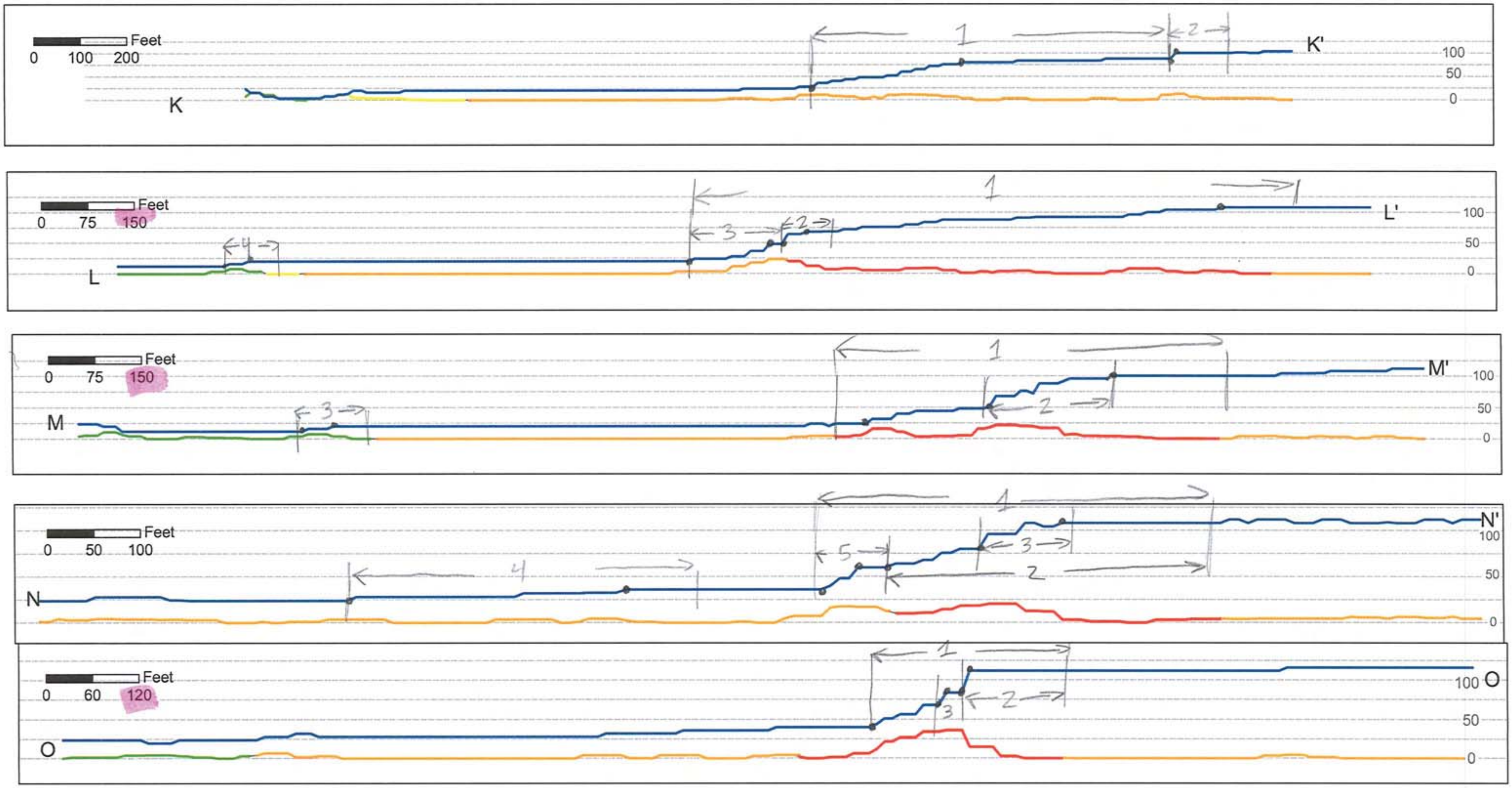
A'
100
50
0
Feet
0 60 120
B
B'
100
50
0
Feet
0 100 200
C
C'
100
50
0
Feet
0 100 200
D
D'
100
50
0
Feet
0 50 100
E
E'
100
50
0
LEGEND
Elevation Profile (ft msl)
Very High Seismic Type & Slope Profile (degrees)
High Seismic Type & Slope Profile (degrees)
Moderate Seismic Type & Slope Profile (degrees)
Moderate Low Seismic Type & Slope Profile (degrees)
p:\GIS\Anchorage\Project\xsec\xsec.mxd


Feet
0 75 150
F
F'
100
50
0
Feet
0 50 100
G
G'
Feet
0 50 100
H
H'
Feet
0 75 150
I
I'
Feet
0 75 150
J
J'
LEGEND
Elevation Profile (ft msl)
Very High Seismic Type & Slope Profile (degrees)
High Seismic Type & Slope Profile (degrees)
Moderate Seismic Type & Slope Profile (degrees)
Moderate Low Seismic Type & Slope Profile (degrees)
p:\GIS\Anchorage\Project\xsec\xsecFthruJ.mxd



LEGEND

- Elevation Profile (ft msl)
- Very High Seismic Type & Slope Profile (degrees)
- High Seismic Type & Slope Profile (degrees)
- Moderate Seismic Type & Slope Profile (degrees)
- Moderate Low Seismic Type & Slope Profile (degrees)

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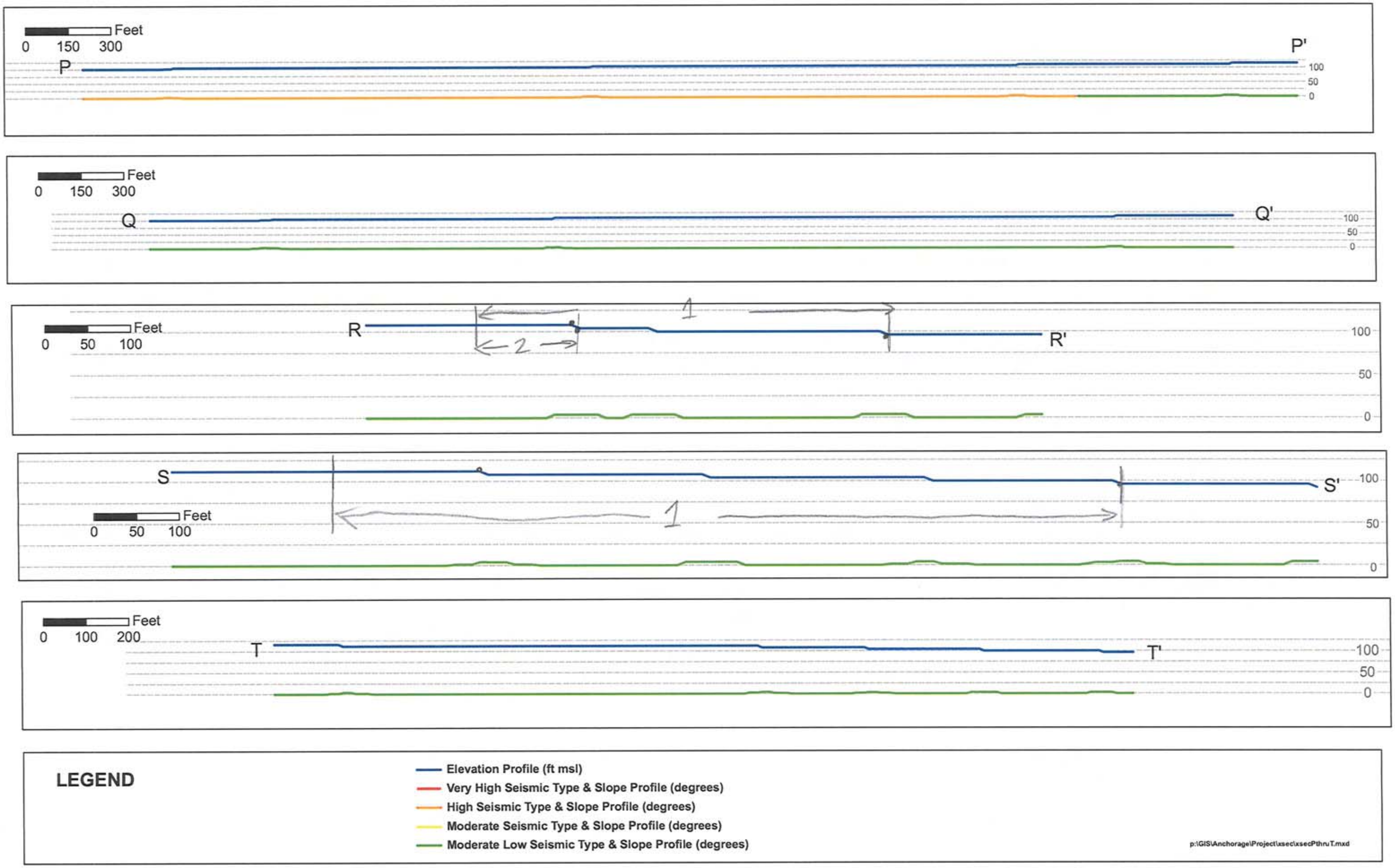
Feet
0 150 300
P
P'
100
50
0
Feet
0 150 300
Q
Q'
100
50
0
Feet
0 50 100
R
R'
1
2
100
50
0
S
S'
Feet
0 50 100
1
100
50
0
Feet
0 100 200
T
T'
100
50
0
LEGEND
Elevation Profile (ft msl)
Very High Seismic Type & Slope Profile (degrees)
High Seismic Type & Slope Profile (degrees)
Moderate Seismic Type & Slope Profile (degrees)
Moderate Low Seismic Type & Slope Profile (degrees)
p:\GIS\Anchorage\Project\xsec\xsecPthruT.mxd