

Appendix D
Northern Impoundment
Preliminary Vibration Analysis



Appendix D - Northern Impoundment Preliminary Vibration Analysis

San Jacinto River Waste Pits Site
Harris County, Texas

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11187072 | Report No 13



Table of Contents

- 1. Introduction..... 1
- 2. Drivability and Hammer Types 2
- 3. Estimation of Vibration Velocity and Acceleration..... 3
 - 3.1 Peak Particle Velocity (PPV)..... 3
 - 3.2 Peak Particle Acceleration (g force) 4
- 4. Slope Stability..... 4
 - 4.1 Analysis Approach 5
 - 4.1.1 Effects of Material Thickness and Strength (Cohesive Material) on Stability... 5
 - 4.1.2 Effects of Friction Angle (Cohesionless Material) on Stability 6
 - 4.2 Impact of Vibrations from Pile Driving on Slope Stability..... 7
 - 4.2.1 Pseudostatic Approach..... 7
 - 4.2.1.1 Cohesive Material 8
 - 4.2.1.2 Cohesionless Material 9
 - 4.2.2 Excess Pore Pressure Approach..... 10
- 5. Other Potential Vibration Impacts 11
 - 5.1 Settlement..... 11
 - 5.2 Impact to Structures..... 12
- 6. Conclusions..... 13
- 7. References 14



1. Introduction

The remedy for the Northern Impoundment at the San Jacinto River Waste Pits Site in Harris County, Texas selected by the EPA in the October 2017 Record of Decision (ROD), provides for the use of a Best Management Practice (BMP). The ROD describes the BMP as being implementable (based on information available at that time) and effective preventing or minimizing the release of waste material during removal activities to ensure compliance with Applicable or Relevant and Appropriate Requirements (ARARs). The 30% Preliminary Remedial Design (30% RD) presents a phased construction approach utilizing a cantilever retaining wall as the BMP to segregate the Northern Impoundment into five cells. The preliminary geotechnical engineering analysis performed by Ardaman & Associates, Inc. (Ardaman) indicates that a cantilever wall that consists of a tubular pipe pile with a pair of intermediary sheet pile (PAZ66/AZ38-700N) combination-wall system (combi-wall) and/or double H-Beam wall to be driven in excess of elevation -80 feet North American Vertical Datum 1988 (NAVD88) would be required to implement this approach.

The installation of the combi-wall system using a conventional diesel impact hammer was analyzed by Ardaman and the initial results based on wave equation analysis using GRLWEAP, a one-dimensional wave analysis equation program, indicates that a large diesel impact hammer is required to drive the piles to the specified depth. The GRLWEAP analysis is provided in the *Ardaman Geotechnical Engineering Report* (Appendix B). As discussed in Appendix B, further analysis performed by Ardaman, indicates that the vibration caused by pile driving could decrease the stability of the slope in the vicinity of the pile installation and result in the potential for a release of waste to the San Jacinto River.

This report expands on the vibration impact analysis performed by Ardaman. It includes a preliminary evaluation of the potential effects of vibrations during pile driving due to the following:

- The potential release of waste from the Northern Impoundment to surface water from a slope failure caused by the acceleration force from vibrations acting on the slope.
- The potential release of waste from the Northern Impoundment to surface water from a slope failure due to the development of excess pore water pressure caused by the ground motion from the vibrations.
- The potential release of waste from the Northern Impoundment from densification and settlement caused by the vibrations, from the pore water that potentially could contain waste constituents due to the shifting of the waste material from the area in which settlement occurred.
- The potential impact of vibrations on surrounding structures.

Figure 1 is a map of the Northern Impoundment showing the boring locations and the cross Section location evaluated for purposes of slope stability analysis.

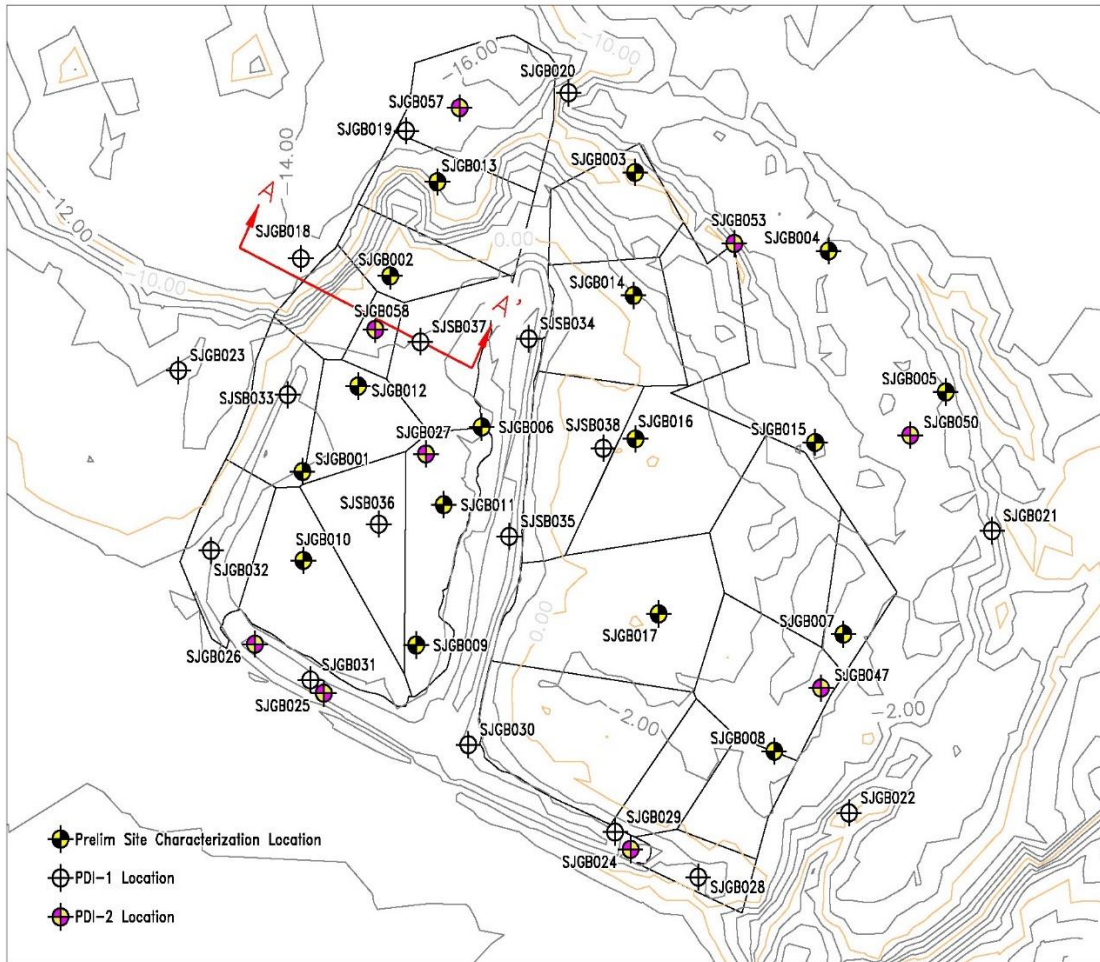


Figure 1 - Northern Impoundment Boring Locations, Topography, and Cross Section A-A' Location

2. Drivability and Hammer Types

Pile drivability and the equipment requirements are important aspects of a vibration impact analysis, as vibrations are directly related to the site conditions and the equipment necessary to drive the piles to the specified depths. The preliminary evaluation of initial inputs (such as peak particle velocity (PPV), ground acceleration, frequencies, etc.) required for the vibration analyses is largely related to the type of equipment used to advance the piles.

GHD Services Inc. (GHD) performed an additional evaluation using GRLWEAP based on data from borings SJGB053 and SJGB057 (see Figure 1 for boring locations) to predict the drivability of 66-inch and 84-inch diameter open-ended tubular steel pipe piles to a target depth of at least 100 feet. For this evaluation, Mohr-Coulomb parameters were inferred for the various soil layers interpreted from the boring logs. The GHD assessment also assumed the use of a pile toe inside friction reducer, which effectively prevents the development of significant internal friction during driving.



The GHD analysis showed that it may be possible to achieve the target depth with the 66-inch diameter pipe pile using an American Pile Driving Equipment (APE) D100-42 and D80-42 impact hammer. However, the results also indicated that a significant risk of driving refusal could occur at elevation -70 feet NAVD88 where a dense sand layer is encountered. The risk of premature refusal is greater if driving is temporarily interrupted, resulting in “setup” (i.e., significantly increased shaft resistance due to radial consolidation of cohesive soils in contact with the pile shaft). Premature driving refusal is predicted with the 84-inch diameter pipe pile for both hammers (D100-42 and D80-42) which indicates that driving the 84-inch diameter piles to a target depth of -100 feet NAVD88 is likely impracticable without a larger hammer.

The inputs for the vibration analysis presented herein (PPV, ground acceleration, frequencies, etc.) are based on the evaluations performed by Ardaman and GHD using a D100-42 hammer to advance the piles through the dense sand. These evaluations assume a maximum 66-inch diameter pipe pile. The 84-inch diameter pile would require an even larger hammer that would produce higher vibrations than are considered in this report.

The drivability analyses would require updating during future phases of the remedial design to incorporate information on the impact of specific pile driving equipment anticipated to be used.

3. Estimation of Vibration Velocity and Acceleration

During pile driving, ground vibrations are generated as a result of elastic deformation of the soil. The vibrations are then propagated through the soil as elastic waves. However, the wave motions generated by pile driving are complex due to the effect of soil damping and geometric damping. For this study, the CalTrans (2013) method was selected, as it uses the wave propagation theory and incorporates the pile driving equipment and soil type to predict the PPV, which is the best indicator of damage potential during pile driving. The PPV values are used to calculate peak particle acceleration, a value that is then used in the slope stability analyses.

3.1 Peak Particle Velocity (PPV)

CalTrans (2013) published a method for predicting vibration amplitudes, in terms of PPV, for pile driving using a propagation model. Based on the CalTrans document, the estimated PPV from an impact hammer can be represented by:

$$PPV_{Impact\ Hammer} = PPV_{Ref}(25/D)^n \times (E_{equip}/E_{Ref})^{0.5}$$

Where:

$$PPV_{Ref} = 0.65 \text{ in/sec for a reference pile driver at 25 ft}$$

$$D = \text{distance from pile hammer to the receiver in ft}$$

$$n \text{ is the vibration attenuation rate through ground and is between 1.1 and 1.5}$$

$$\text{(see table extracted from the Caltran's document below)}$$

$$E_{Ref} = 36,000 \text{ ft-lb (rated energy of reference pile driver)}$$

$$E_{equip} = \text{rated energy of impact hammer in ft-lbs}$$

The calculated PPV values based on the maximum rated energy of the APE D100-42 hammer ranges from 17.5 inches per second (in/sec) at a distance of 5 feet from the pile to 0.26 in/sec at a



distance of 100 feet from the pile for a Class I soil type (n=1.4). The soil classes from the CalTrans document are listed below.

Soil Class	Description of Soil Material	Value of "n" measured by Woods and Jedele	Suggested Value of "n"
I	Weak or soft soils: loose soils, dry or partially saturated peat and muck, mud, loose beach sand, and dune sand, recently plowed ground, soft spongy forest or jungle floor, organic soils, top soil. (shovel penetrates easily)	Data not available	1.4
II	Competent soils: most sands, sandy clays, silty clays, gravel, silts, weathered rock. (can dig with shovel)	1.5	1.3
III	Hard soils: dense compacted sand, dry consolidated clay, consolidated glacial till, some exposed rock. (cannot dig with shovel, need pick to break up)	1.1	1.1
IV	Hard, competent rock: bedrock, freshly exposed hard rock. (difficult to break with hammer)	Data not available	1.0

This range of PPVs is used as input to estimate the peak particle acceleration (g force) as discussed in the following section.

3.2 Peak Particle Acceleration (g force)

Wiss (1967) presented a range of observed soil frequencies (f) during pile driving and a typical soil frequencies range for three soil types. For alluvial fill, the soil frequency is typically between 5 and 10 cycles per second (cps). For clay soil, the frequency is between 13 and 25 cps; and for sandy soil, the frequency is between 30 and 40 cps. The peak acceleration (A) as a function of PPV and soil frequency (CalTrans 2013) is:

$$A = 2\pi fV \quad \text{where } V = \text{PPV and } A \text{ is in the same unit as PPV}$$

$$\text{or } A_g = 0.0163fV \quad \text{where } A_g \text{ is the peak acceleration in term of } g\text{-force}$$

Using the range of PPVs calculated from the previous Section and a soil frequency of 7.5 cps for alluvial fill, which is the predominant soil type for the upper 30 feet, the peak particle acceleration (g-force) ranges from 2.1g at a distance of 5 feet from the pile to 0.03g at a distance of 100 feet from the pile. At a distance of 20 feet from the pile, the peak particle acceleration is approximately 0.3g. These peak particle acceleration values will be used to evaluate the effects of pile driving vibrations on slope stability. As depicted in the *Preliminary 30% Remedial Design - Northern Impoundment*, the pile locations are anticipated to be closer than 20 feet from the slope; therefore, the peak particle acceleration values at the slope would be even greater.

4. Slope Stability

According to Lamens (2020): "the stability of a slope may be negatively affected by pile driving in two ways: (a) through dynamic or inertia-related effects and (b) through excess pore pressure development, diminishing the effective stress and, correspondingly, the mobilizable shear strength in the soil".



4.1 Analysis Approach

In order to properly assess the influence of pile driving on the slope stability (i.e. to analyze the reduction of factor of safety due to the vibrations), it is first required to evaluate the pre-pile driving stability of the slope. Accordingly, it is critical to establish a representative soil model with appropriate geometrical and geotechnical properties. This is typically done by collecting geotechnical information at the top and at the bottom of the slope, so that the soil profile along the slope is properly captured in the analysis.

The northwestern part of the Northern Impoundment is one area where piles would be driven onto and/or near the toe of a relatively steep slope and the slope stability evaluation focuses on this area. Figure 1 shows the cross Section (A-A') location where the evaluation was performed. The surrounding boring logs (SJGB018 and 019) show very soft surficial material in areas near the toe of the slope and the geotechnical data are limited because of poor recovery of this soft material during the sampling. There is also uncertainty because there are not any borings directly on the slope and the slope soil profile had to be developed from adjacent borings located more than 100 feet apart.

Due to the uncertainties in the soil conditions on the slope and the potential for these conditions to be different than those observed in the limited number of borings, a parametric study was performed to evaluate how changes in material thicknesses and strength would affect slope stability (under static conditions). Thus, the slope stability of these two soil types was evaluated for these same conditions considering the vibration impacts.

This preliminary evaluation focused on both cohesive (clay) and cohesionless (sand) material on the slope. The 6-inch thick articulated concrete block mat (ACBM) that is currently present on the northwest slope was not included in the preliminary evaluation as a cap material. It is expected that the ACBM would behave similarly to the underlying soil type under conditions in which there are vibrations from pile driving. The weight of the ACBM will increase the horizontal force and potential for failure caused by the vibrations. The impact of vibrations on slopes on which ACBM is present would need to be further evaluated as the design progresses.

4.1.1 Effects of Material Thickness and Strength (Cohesive Material) on Stability

The following figures show how the factors of safety vary under static conditions (without vibrations) with different material thickness and strength, considering a very soft (low strength) cohesive material at the surface. The left side of Figure 2 shows the calculated factors of safety for cohesive material with undrained shear strength of 100 pounds per square foot (psf) and thickness of 5, 10 and 15 feet on the slope. The right side of Figure 2 shows the factors of safety for cohesive material with undrained shear strength of 150 psf and the same thicknesses.

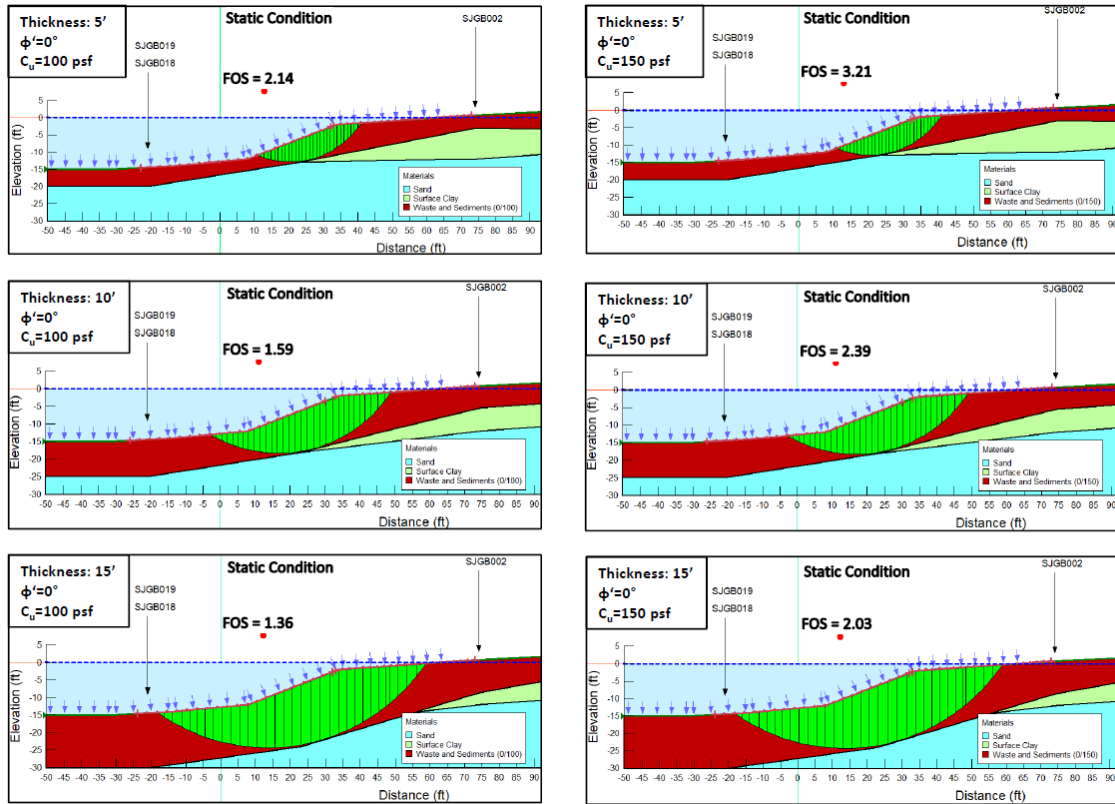


Figure 2 - Effects of Sediments Properties and Layer Thickness (Cohesive Material) on Factors of Safety in a Slope Stability Analysis

These figures show that both the thickness and the strength affect the static slope stability. Based on the boring logs (SJGB018, 019, and 002), there is a potential for any of the conditions depicted on the above figures to exist somewhere along the slope.

4.1.2 Effects of Friction Angle (Cohesionless Material) on Stability

A cohesionless material on a slope similar to the slope at the northwest corner of the Northern Impoundment has the potential for a shallow slip surface failure. This type of failure is dependent on the slope geometry and the friction angle of the material. The material thickness does not influence the result. The following figures show how the factors of safety vary with the friction angles under static conditions for the geometry on northwestern slope.

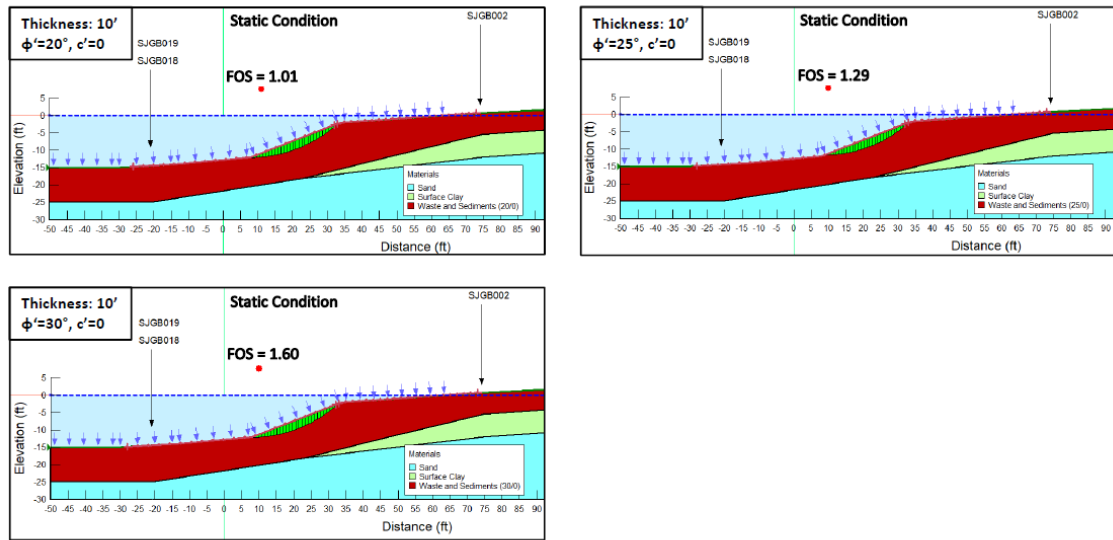


Figure 3 - Effects of Sediments Properties (Cohesionless Material) on Factors of Safety in a Slope Stability Analysis

These figures show that if a low friction angle cohesionless material is on the slope in the northwest portion of the Northern Impoundment (at any thickness), the slope would be marginally stable for a shallow slip surface failure under static conditions. The higher friction angle material is more stable and likely more representative of the Northern Impoundment soils, but may still be susceptible to failure with vibrations as discussed below.

4.2 Impact of Vibrations from Pile Driving on Slope Stability

Two methods were considered for the initial and preliminary evaluation of vibrations due to pile driving on slope stability. The first method is a pseudostatic or seismic approach that is commonly used to evaluate seismic impacts on slope stability. The second method is the evaluation of failure due to the development of excess pore pressures (EPP) from vibrations during pile driving.

4.2.1 Pseudostatic Approach

In this method, the effect of the vibrations is represented by simulated accelerations that produce inertial forces that act on the centroid of the soil mass. This effect is incorporated in the limit equilibrium method model (Slope/W) through the use of a horizontal pseudostatic coefficient (k_h), which is often considered to be equal to $k_h = k \cdot A_{max} / g$, where A_{max} is the peak horizontal acceleration and k is a magnification factor. The k factor is usually determined from response analysis depending on the mode of vibration and soil damping. Based on published research on one-dimensional response during blasting and seismic events on a slope, the k factor ranges from 0.1 to 0.8 (Wong, et. al., 2000). Since the mechanisms involved during blasting and seismic events and those related to vibrations from pile driving are different, the use of the pseudostatic method is considered a preliminary assessment of influence of pile driving generated inertial forces on slope stability. For this study, the k factor is assumed to be 0.5 resulting in $k_h = 0.5 \cdot A_{max} / g$. The acceleration values evaluated in Section 3.2 were considered initially as A_{max} to estimate the



range of k_h . These coefficient values ranging from 0.02 to 0.15 were considered in the analysis based on the vibration from the impact hammer as described in Section 3.1. This is considered to represent a range of A_{max} that could occur from approximately 20 feet to 100 feet of the pile driving location. It is important to note that the use of the k_h in this study is based on the similar use of response peak ground acceleration (PGA) coefficient by Wong, et. al. (2000) in the analysis of slope stability due to vibrations from blasting. To our knowledge, no publication exists that provides direct correlation between the k_h and peak particle acceleration; thus, the results presented below should be considered qualitative. These results, however, can be used to compare with Ardaman's slope stability analysis results using different soil conditions and similar k_h values to highlight the influence of vibration on differing soil profiles and properties.

4.2.1.1 Cohesive Material

Results of the analyses showing the estimated impact of vibrations using different seismic coefficients for cohesive soils are provided below. The results are shown for a 10-foot thick material with undrained shear strength of 100 psf and 150 psf.

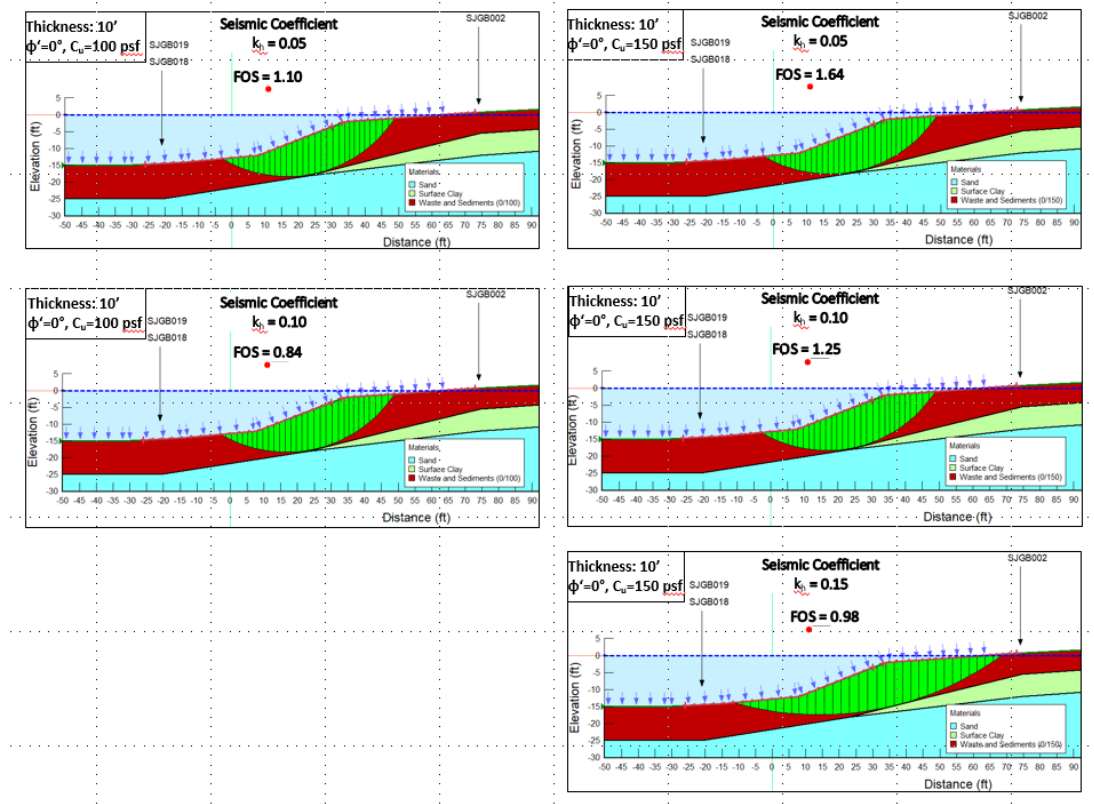


Figure 4 - Slope Stability Analysis with Varying k_h for Cohesive Material

For the material with undrained shear strength of 100 psf, using the lower end seismic coefficient of 0.05 results in a slope that is marginally stable at a factor of safety of 1.10. The slope is unstable with a factor of safety of 0.84 when a seismic coefficient of 0.10 is applied. For the material with undrained shear strength of 150 psf, the slope is more stable, but the factor of safety is below 1.0 when a seismic coefficient of 0.15 is considered. The results shown in Figure 4 are for 10 feet of the

cohesive material on top of the slope. The slopes would be less stable if the material is thicker than 10 feet and more stable if the material is below 10 feet in thickness.

4.2.1.2 Cohesionless Material

Results of the analyses showing the estimated impact of vibrations using different seismic coefficients for shallow slip surface failures of cohesionless soils are below. The results are shown for both the 25-degree friction angle and the 30-degree friction angle. The 20-degree friction angle presented in Figure 3 was marginally stable under static conditions. Therefore, it was assumed the slope would not be stable under conditions in which there are vibrations and further evaluation was not conducted.

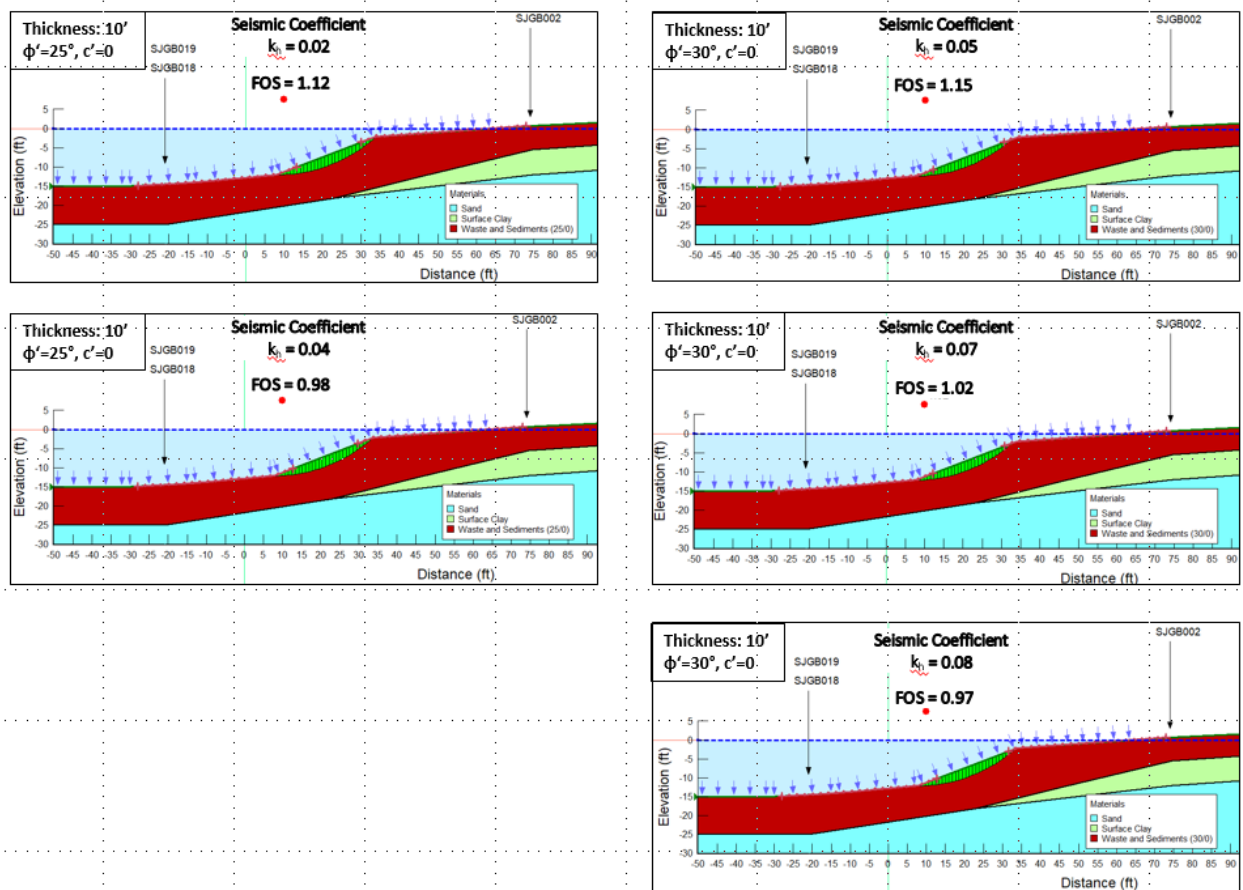


Figure 5 - Slope Stability Analysis with Varying k_h for Cohesionless Material

For the 25-degree friction angle material, a low seismic coefficient (of 0.02) results in a slope that is marginally stable, and a seismic coefficient of only 0.04 produces a factor of safety below 1.0. The higher friction angle material (30 degrees) is more stable, but when a 0.08 seismic coefficient is applied, the factor of safety on the slope falls below 1.0. The stability against a shallow slip surface failure for the cohesionless material is not affected by the thickness of the material on the slope. The risk of a shallow slip surface failure during construction under this condition could result in a release of waste material from the Northern Impoundment to the San Jacinto River.



4.2.2 Excess Pore Pressure Approach

During both impact and vibratory pile driving, the formation of various stress waves will create ground motion, which can develop EPPs. These EPPs can affect the stability of the slope. It is of particular concern for cohesionless soils with the potential for liquefaction.

The correlation between ground motion and the generation of EPP is a function of site conditions and equipment used and typically is determined during a test pile program. Without test pile data, the preliminary evaluation presented below was performed based on available literature values for the EPP as a function of distance from the pile driving. The EPP values were estimated based on a study by Lamens (2020) correlating the measured EPP versus radial distance from a test pile program during the installation of tubular and steel sheet piles on a submerged sandy slope. Lamens (2020) compared the data from the test pile program with data from the literature and plotted the maximum EPP normalized with effective stress (referred to as relative excess pore water, $r_{u,max}$) versus the normalized horizontal distance (x/D) where x is the radial distance from the pile and D is the diameter of the pile and curve fit all the data with an exponential function. The exponential function proposed by Lamens is:

$$r_{u,max} = 2.6e^{-0.22(x/D)}$$

Comparison of the calculated values from this function to measured values from Lamens' test pile program indicates that the function typically overestimates the EPP. The measured $r_{u,max}$ values from the Lamens' test piles are in the order of 40% of the predicted values. Therefore, two EPP profiles within 40% of the predicted values from the exponential function were developed for the parametric slope stability evaluation. The first EPP profile assumed the EPP distribution is 0.33 times of $r_{u,max}$ and the second EPP profile assumed the EPP distribution is 0.38 times of $r_{u,max}$. These EPP profiles are then incorporated in the Slope/W model to evaluate the slope stability due to pile driving.

Results of the slope stability analysis of the two simulated EPP profiles from pile driving for 10 feet of cohesionless material on the slope with an angle of internal friction of 30 degrees are presented in Figure 6 below.

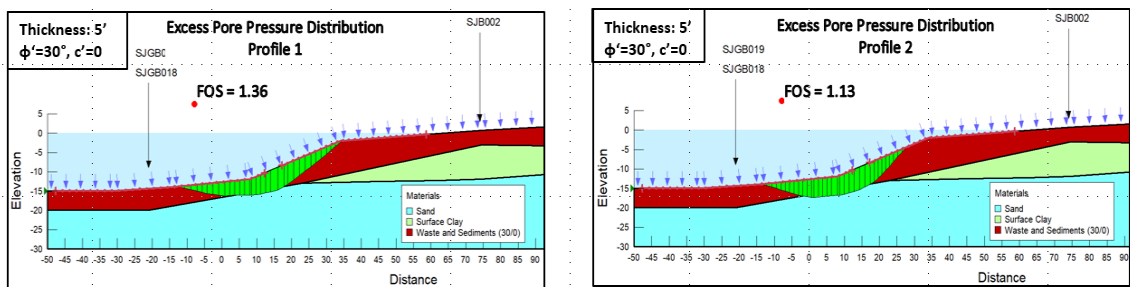


Figure 6 - Slope Stability Analysis for Two Normalized Excess Pore Pressure Profiles

The factors of safety for the two profiles are 1.36 and 1.13. This is compared to the static factor of safety shown on Figure 3 of 1.6. The result of this parametric study indicates the generation of EPP due to pile driving on a sandy slope reduces the margin of safety of the slope. Accordingly, there is a potential that the generation of EPP could cause slope failure.



5. Other Potential Vibration Impacts

The potential for settlement to occur from the vibrations and the potential impact of the vibrations to surrounding structures are evaluated in the following section.

5.1 Settlement

Settlement from vibrations due to pile driving may become problematic in loose, granular soils, especially close to the pile. For the Northern Impoundment, the concern is whether settlement could cause the release of pore water from locations beneath existing armored cap installed during the Time Critical Removal Action (TCRA), or whether shifting of the soils from settlement could cause the release of waste material into the surface water.

Several simplified methods for assessing densification or ground settlement during vibratory pile installation have been proposed by researchers during the last two decades. Models range from using acceleration amplitude and cone penetration test (CPT) tip resistance, PPV and shear wave velocity, soil compression factor, to a seven factors model that includes PPV and depth of soil layer. For this analysis, the seven factors empirical model proposed by Kim, et. al (1994) and the method proposed by Massarsch (2004) were used to provide settlement estimation for near field and far field conditions.

The Kim, et. al. (1994) method can be represented as:

$$\ln Y = 2.27 + 1.19x_1 - 0.71x_1^2 + 0.49x_2 - 0.68x_2^2 - 0.80x_3 + 1.09x_3^2 - 0.46x_4 + 0.06x_4^2 + 0.45x_5 - 0.38x_5^2 - 0.19x_6 - 0.1x_7$$

Where:

Factor Name	Range	Factor
velocity amplitude (v)	0.1 - 0.7 in/s	$x_1 = -1+(v-0.1)/0.3$
deviatoric stress (q)	2 - 15 psi	$x_2 = -1+(q-2)/6.5$
confining pressure (p)	10 - 30 psi	$x_3 = -1+(p-10)/10$
sand mixture	coarse, medium, fine	x_4 is resp. -1, 0, 1
number of vibrations	$N = 60 - 500,000$	$x_5 = -1+(N-60)/269970$
moisture content	dry, saturated	x_6 is resp. -1 and 2
initial relative density	loose, medium dense	x_7 is resp. -1 and 2

This is an empirical prediction model for estimating vibration induced settlement for small to intermediate vibration levels, which is recorded at an estimated distance of 50 feet and beyond the pile for the APE D100-42 hammer type. The seven factors include vibration amplitude, deviatoric stress, confining pressure, soil gradation, duration of vibration, relative density, and moisture content of the soil. This model may not simulate the exact loading condition of driving piles, but is a reasonable method to estimate the settlement within the range of expected vibrations from the pile



driving. A parametric study was performed using the soil profile from boring SJSB027-G (see Figure 1) and varying the thickness of loose and fine sand layer at a depth of 15 feet below ground surface. The estimated settlement ranges from 0 for a 5-foot thick layer to 0.8 inch for a 25-foot thick layer at a horizontal distance of 50 feet from the wall,

The Massarsch (2004) method provides settlement estimation for a sand deposit adjacent to the pile within a zone of three times the pile diameter. The settlement is modeled as a function of a compression factor and the thickness of the compressible layer. The compression factor (α) is an empirical constant based on the driving energy and the density state of the sand deposit. The model is represented by the following equation:

$$S_{avg} = \frac{\alpha (L + 6D)}{3} \text{ where } L \text{ is the thickness of layer and } D \text{ is pile diameter}$$

An estimated 15-foot thick sand deposit using the Massarsch model yielded a range of settlement estimation of 0.6 inches for a very dense sand deposit to more than 3 inches for a very loose sand deposit.

Based on these equations, settlement likely would only be a potential issue from pile driving at the Northern Impoundment within close proximity to the pile where up to more than 3 inches of settlement was estimated within a zone that is within a distance of three times the pile diameter. This volumetric change could potentially release pore water from the material within this localized area to the San Jacinto River.

5.2 Impact to Structures

Ground vibrations due to pile driving could potentially impact surrounding structures and underground utilities. Vibration amplitudes are closely related to the type of hammer used to drive the piles. Vibration amplitudes estimated in this Section should be considered preliminary estimates.

The closest structures to the Northern Impoundment are the piles that support the I-10 Bridge. There are approximately 80 feet between the bridge piles and the nearest BMP Section where the potential combi-wall would be installed. The maximum calculated PPV based on the APE D100-42 hammer at this distance is estimated at 0.5 in/sec for a Class III soil type and the minimum PPV is approximately 0.24 in/sec for a Class I soil type using the CalTrans' equation discussed in Section 3. These calculated values are within the threshold values provided in the CalTrans' guideline. However, as discussed in Section 3 above, the soil type and the hammer type in the vibratory hammer equation presents an uncertainty in the estimation of the PPV. A larger hammer may be required due to the presence of the dense clayey silty sand layer. Therefore, before completing the RD an evaluation would have to be conducted after the hammer type and pile depths are known to determine the potential impact on structures.

The nearest known underground utility is the Exxon Pipeline with varying depths of approximately 100 to 150 feet below ground surface and vibrations from the pile driving are not expected to affect this line. However, the owner of the Exxon Pipeline will need to be contacted to confirm the location of the pipeline, and an evaluation of the potential effect from vibrations should be conducted after the hammer type and pile depths are determined.



6. Conclusions

A preliminary evaluation was performed to assess the potential effects of vibrations due to construction of the BMP proposed for the Northern Impoundment. This will continue to be evaluated as the design progresses to account for changes to the BMP and to evaluate impacts on areas containing ACBM.

The results indicated that the impact hammer required to drive the piles that are included in the 30% RD has the potential to produce vibrations and an acceleration force that could adversely impact the stability of the slope of the Northern Impoundment. A failure of this slope could cause a release of waste material to the San Jacinto River. A major uncertainty in this evaluation is the condition of the soil material on the slope. Area boring logs indicate that the near surface material is very heterogeneous and therefore a preliminary parametric study was performed to show the factor of safety on slope stability for a range of conditions that could be present at the Northern Impoundment. The parametric study shows that under certain conditions (material strength and thickness that may be present on the slope), the slope could potentially fail from the vibrations caused by pile driving. Any slope failure (even a shallow slip surface failure) has the potential to cause a release of waste material to the San Jacinto River. It is important to note that the impact hammer used in the evaluation may not be sufficient to drive some of the pile types that are in the 30% RD. If a larger hammer is required, the vibrations and potential for failure would be greater.

Another mechanism that was considered is the development of EPP caused by the ground motion from the vibrations. Again, a parametric analysis was performed under different geotechnical conditions, and the results indicate that the generated EPP from pile driving can reduce the effective stress, which may cause the slope to become unstable with a potential for failure.

Both of these mechanisms indicate that there is a risk of slope failure from vibrations caused by pile driving, and the potential for failure is greatly dependent on the geotechnical properties on the slope. Due to this uncertainty, whether piles of the size and type required can be installed without causing a slope failure due to vibrations cannot be determined until an investigation is conducted along areas of the slope where the geometry indicates that under certain conditions of material type and thickness, there is a potential for instability. A more detailed slope stability evaluation is required.

There appears to be a lower risk of a potential release due to densification and settlement. The preliminary evaluation indicated that any appreciable settlement would be localized within close proximity to the pile.

The preliminary evaluation of the potential impacts to surrounding structures indicates that the PPV from the vibrations should be within the threshold values considered acceptable, but may not be if a large hammer needs to be used. A further evaluation will need to be conducted as part of the RD after the hammer type and pile depths are known to determine whether vibrations are within threshold values considered acceptable and to determine whether monitoring of vibrations and the potential impact on structures are required.



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