Hatch Mott MacDonald

# TECHNICAL MEMORANDUM

## Transpecos Pipeline from Waha to Ojinaga

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| TO: | *Isaac Derr* | DATE: | *July 1, 2015* |
| FROM: | *Don Wotring* | REFERENCE: | *Project No. 353157* |
| CC: | *Jon Davies, Scott Kibby* | | |
| SUBJECT: | *Potential Seismic Hazards along Proposed Pipeline Alignment* | | |

## 1. EXECUTIVE SUMMARY

This memorandum summarizes the potential seismic hazards along the route of the proposed Transpecos Pipeline. The seismic design parameters are provided in Section 3, which includes earthquake magnitude probability, seismic acceleration parameters, and faults. The potential hazards, design values, and recommendations are provided in Section 4. The potential hazards evaluated include: seismic wave propagation, fault rupture, liquefaction (predominately settlement), lateral spread movement (due to liquefaction), and seismic-induced slope instability.

- **Seismic Wave Propagation –** stresses and strains induced by a seismic wave are expected to be acceptable and not considered a credible seismic hazard.
- **Fault Rupture –** Based on available USGS information, it doesn't appear that any known or inferred faults cross the proposed alignment. Therefore, fault rupture is not a credible seismic hazard.
- **Liquefaction –** Liquefaction and resulting pipeline settlement is not a credible hazard for locations where the pipeline is in rock. However, liquefaction is potentially a credible hazard that should be evaluated for the following specific locations – 1) areas where groundwater is less than 15 meters below the ground surface, 2) areas of uncompated or poorly compacted cohesionless fills, and 3) areas along river channels and their historic flood plains, marshes, and estuaries, or soils less than 1,000 years old that may contain loose deposits of cohesionless soil. Recommendations for a preliminary assessment of a liquefaction hazard posed to the pipeline are provided in Section 4.4.
- **Lateral Spread Movement –** Lateral spread movement is induced by liquefaction and potentially is a credible hazard that should be evaluated at the specific locations susceptible to liquefaction. For potentially liquefiable locations, site-specific information is required. Once site-specific information is obtained, Section 4.5 provides an equation that can be used to estimate the extent of lateral spread movement.
- **Seismic-Induced Slope Stability –** Seismic-induced slope stability is a credible hazard that should be evaluated for certain slopes. Section 4.6 provides graphs that can be used to estimate the slope factor of safety as a function of average shear strength (i.e., average friction angle) and slope angle. Section 4.6 also provides an equation and graph to estimate permanent slope

displacement due to a seismic event. Site-specific information and a more detailed evaluation is required for those slopes that have a factor of safety less than or equal to 1.1 for a seismic event, and have an estimated displacement that equals or exceeds 10 cm.

Conclusions and recommendations, including recommendations for additional required site-specific information, are summarized in Section 5.

# 2. PURPOSE

## 2.1 General

This memorandum summarizes the potential seismic hazards along the route of the proposed Transpecos Pipeline. The seismic design criteria are summarized and are based on United States Geological Survey (USGS) data in accordance with design code ASCE 7 (with March 2013 errata). The potential hazards and design considerations are discussed. Conclusions and recommendations are provided at the end of this memorandum.

## 2.2 Project Background

The Transpecos pipeline proposed alignment is shown in Figure 1. The southwestern terminus is located west of the Rio Grande River northwest of Ojinaga, Mexico. The northeast terminus is located in the Waha Basin northwest of Coyanosa, Texas. The straight line distance between the two terminal locations is approximately 140 miles. Due to the length of the pipeline, seismic criteria were determined at both terminal locations. The latitude and longitude used for each terminal and the approximate middle of the alignment are shown in Table 1 and labeled in Figure 1.

**Table 1: Global Site Coordinates used for Evaluation**

| Terminal Location | Latitude | Longitude |
|---|---|---|
| SW, near Ojinaga, MX (A) | 29.671481 N | -104.537372 W |
| Near middle of pipeline (B) | 30.4659 N | -103.604689 W |
| NE, near Coyanosa, TX (C) | 31.273481 N | -103.099547 W |


**Figure 1: Transpecos Pipeline Alignment**

## 3. SEISMIC DESIGN PARAMETERS

### 3.1 General

Seismic design parameters for pipeline design were generally developed for the site coordinates of the two terminal locations provided in Table 1. For earthquake probability, parameters were developed for the site coordinates near the middle of the pipeline alignment. USGS online design mapping tools were used to develop these design parameters.

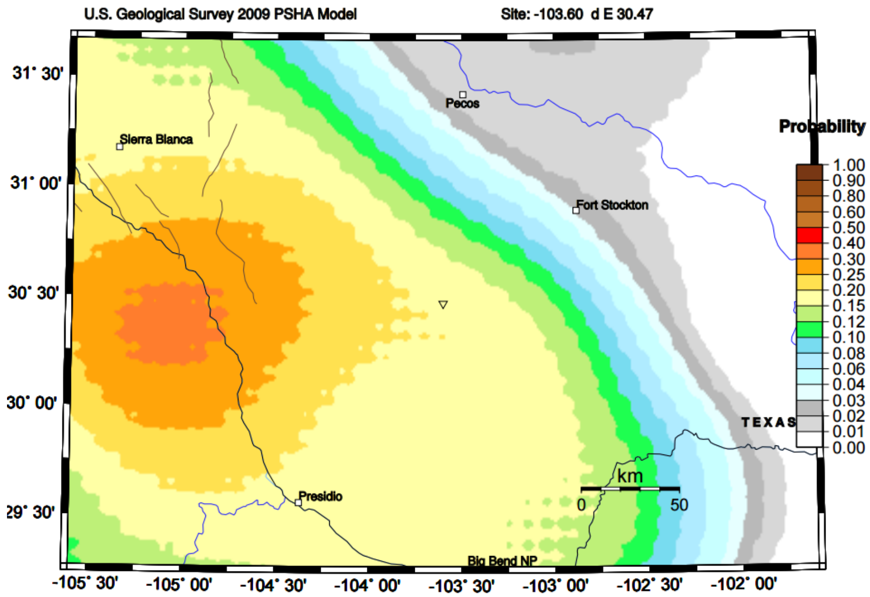
geohazards.usgs.gov/eqprob/2009 (Earthquake probability)

earthquake.usgs.gov/designmaps/us/application.php (Seismic design parameters)

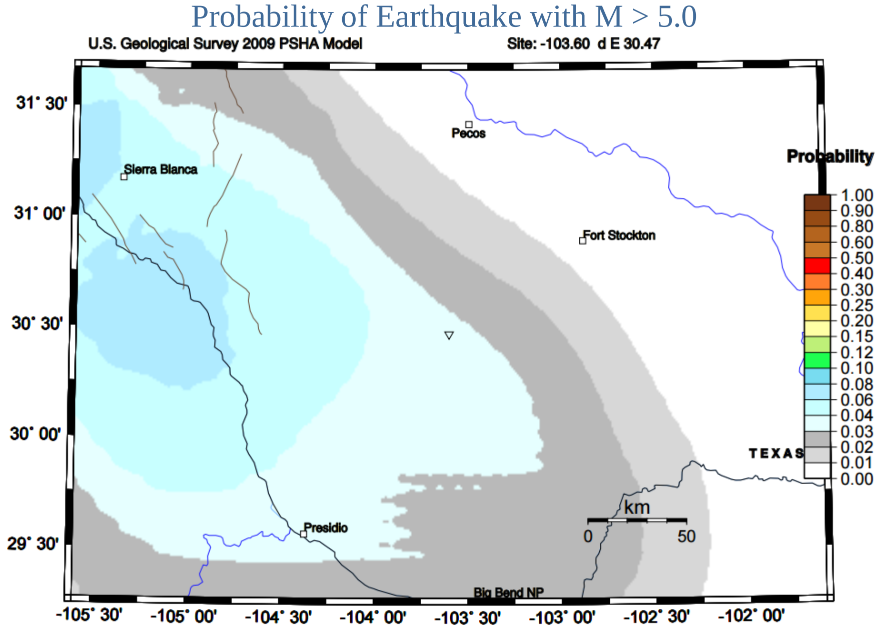
earthquake.usgs.gov/hazards/qfaults/map (Fault)

## 3.2 Moment Magnitude Probability

The probability of an earthquake with moment magnitudes equaling or exceeding 5.0 and 6.0 within 50 years are shown in Figure 2. These probability plots were developed based on the site coordinates near the center of proposed pipeline alignment, which is provided in Table 1. The horizontal distance scale represents at maximum distance of 50 km.




Probability of Earthquake with M > 5.0




Probability of Earthquake with M > 6.0

**Figure 2: Earthquake Magnitude Probability – Near Center of Pipeline Alignment**

The maximum probability is approximately 25% for an earthquake having a moment magnitude equaling or exceeding 5.0 within 50 years. Similarly, the maximum probability is approximately 4% for an earthquake having a moment magnitude equaling or exceeding 6.0 within 50 years. For both of these scenarios, a dome of higher probability is located approximately 100 km northwest of Presidio, TX.

## 3.3 Acceleration Parameters

The mapped acceleration parameters were developed based on the design code ASCE 7 (with March 2013 errata). A risk category of I, II, or III was assumed. The design parameters for two different site classifications are provided – Site Class B (Rock) and Site Class D (Stiff soil). The site classification criteria are shown in Table 2. If these site classifications are not representative of the actual conditions, the following data would need to be modified.

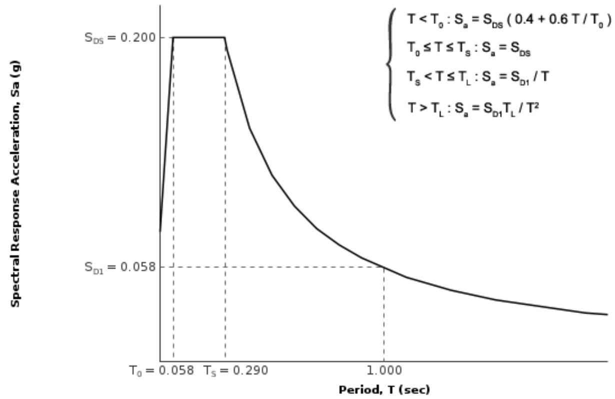
**Table 2: Design Seismic Acceleration Parameters**

| Site Class | $\bar{v}_s$ (ft/s) | $\bar{N}$ (bpf) | $s_u$ (psf) |
|---|---|---|---|
| A "Hard Rock" | > 5,000 | N/A | N/A |
| B "Rock" | 2,500 to 5,000 | N/A | N/A |
| C "Very Dense Soil and Soft Rock | 1,200 to 2,500 | > 50 | > 2,000 |
| D "Stiff Soil" | 600 to 1,200 | 15 to 50 | 1,000 to 2,000 |
| E "Soft Clay Soil" | < 600 | < 15 | < 1,000 |

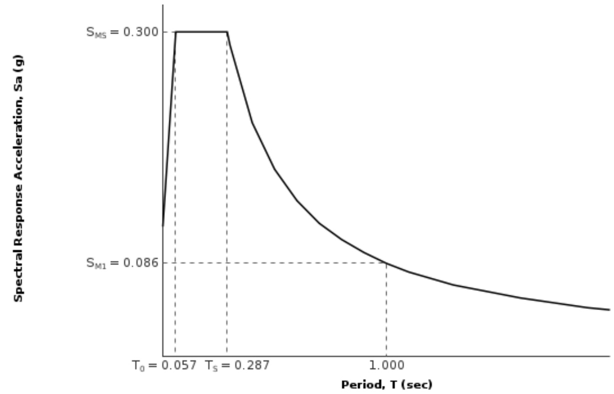
The resulting design seismic acceleration parameters are summarized in Table 3. The response spectrums (design and risk-targeted maximum considered earthquake) are provided in Figures 3 and 4.

**Table 3: Design Seismic Acceleration Parameters**

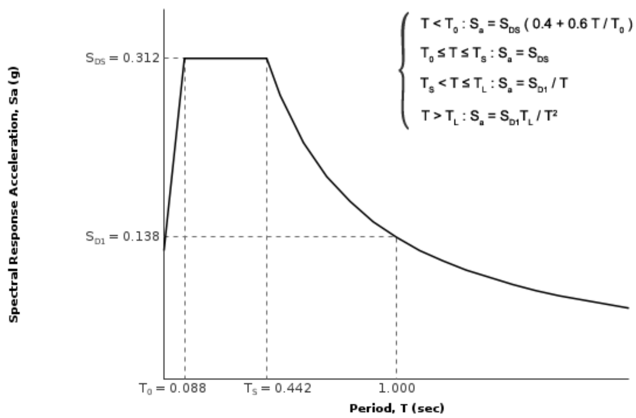
| Site Class | Ss | $S_1$ | $S_{DS}$ | $S_{MS}$ | $PGA_M$ | Risk Cat. |
|---|---|---|---|---|---|---|
| SW, near Ojinaga, MX | | | | | | |
| B "Rock" | 0.300 g | 0.086 g | 0.200 g | 0.300 g | 0.126 g | B |
| D "Stiff Soil" | | | 0.312 g | 0.468 g | 0.195 g | C |
| NE, near Coyanosa, TX | | | | | | |
| B "Rock" | 0.147 g | 0.045 g | 0.098 g | 0.147 g | 0.07 g | A |
| D "Stiff Soil" | | | 0.157 g | 0.236 g | 0.112 g | B |



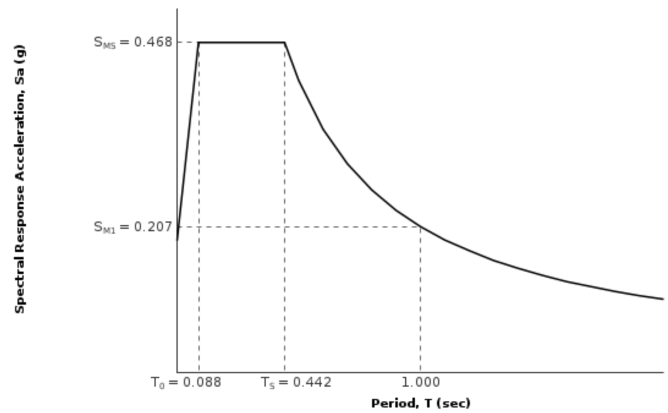

Design Response – Site Class B "Rock"




MCE$_R$ Response – Site Class B "Rock"

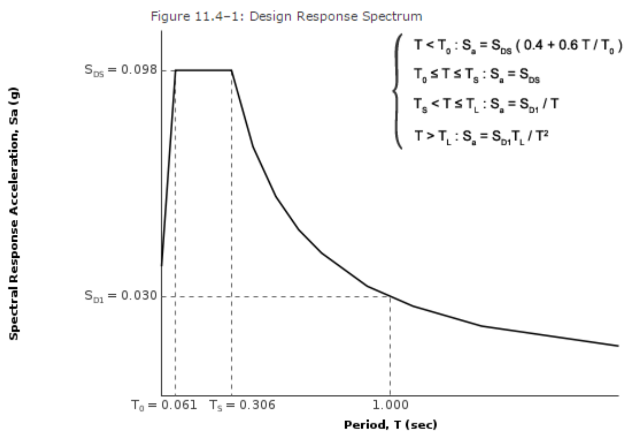



Design Response – Site Class D "Stiff Soil"

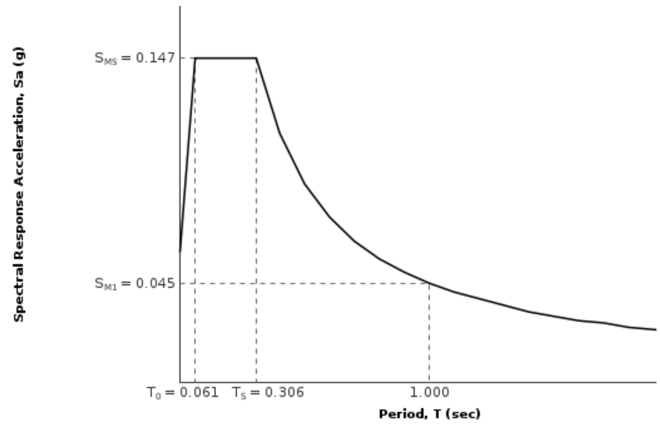



MCE$_R$ Response – Site Class D "Stiff Soil"

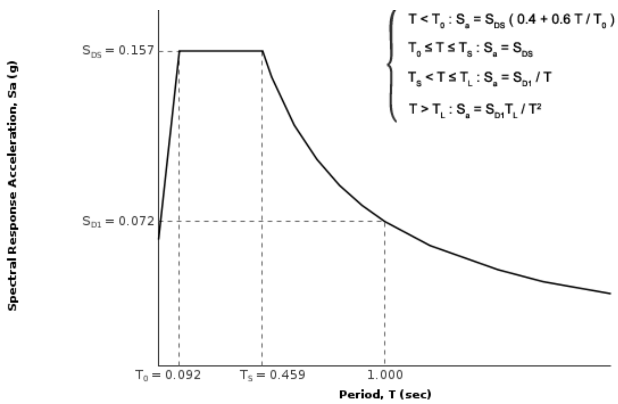
**Figure 3: Response Spectrums – SW Terminus near Ojinaga, MX**



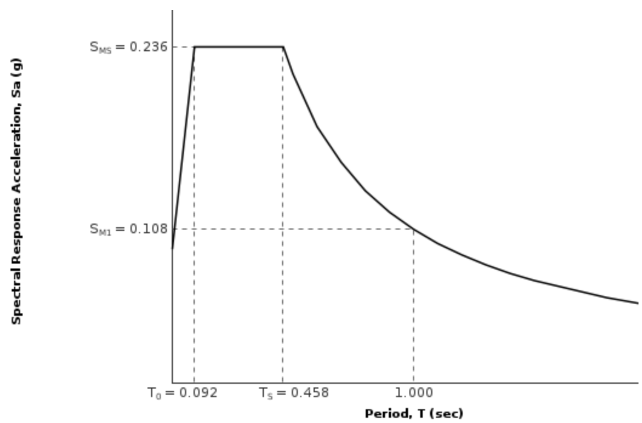

Design Response – Site Class B "Rock"




MCE$_R$ Response – Site Class B "Rock"




Design Response – Site Class D "Stiff Soil"




MCE$_R$ Response – Site Class D "Stiff Soil"

**Figure 4: Response Spectrums – NE Terminus near Coyanosa, TX**

As is expected from the earthquake magnitude contour plots shown in Figure 2, the SW terminus northwest of Ojinaga, MX has larger design seismic acceleration parameters than the NE terminus. At the SW terminus, the risk category for site classification B "Rock" is B and for D "Stiff Soil" is C.

## 3.4 Faults

Figure 5 shows the known or inferred Quaternary fault locations in the vicinity of the pipeline alignment. Along the Rio Grande River, there are a number of faults that are approximately parallel to the river at this location. The USGS anticipated age (i.e., time of most recent deformation) for these faults range between <130,000 years (green) and < 1.6 million years (black). The anticipated slip rates are generally estimated as less than 2 mm per year.

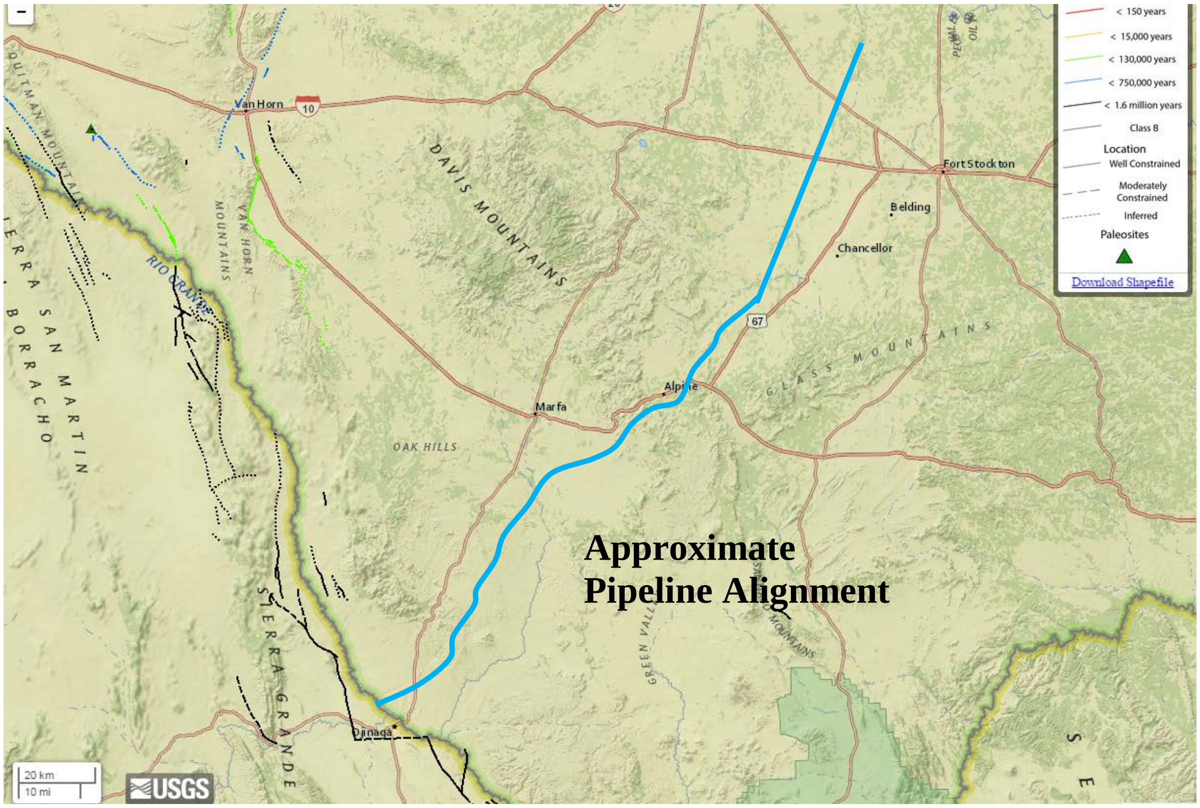



Figure 5: Quaternary Faults near Pipeline

## 4. EARTHQUAKE GENERATED HAZARDS RELATIVE TO TRANSMISSION PIPELINES

### 4.1 General

For the design of steel pressure pipelines used for transmission of natural gas and liquid hydrocarbons credible seismic hazards are limited to those that produce permanent deformation along the pipeline alignment. Seismic wave propagation (ground shaking) can only pose a significant threat to buried steel pipelines under very limited conditions. Credible seismic hazards primarily include surface fault rupture, liquefaction, liquefaction-induced lateral spread movement, and seismic-induced slope instability.

### 4.2 Seismic Wave Propagation

A steel pipeline buried in soil that is subject to seismic 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 strains and shearing strains in buried pipelines, but these strains are even smaller. The allowable tensile strains for pipes recommended by multiple sources are summarized in Table 4. In the vicinity of the proposed pipeline alignment, seismic wave propagation ground strains are anticipated to be much less than the thresholds presented in Table 4. Therefore, seismic wave propagation is not anticipated to cause noticeable damage to the pipeline.

**Table 4: Recommended Max. Strain – Steel Pipelines with Good Quality Butt Welds**

| Newmark & Hall (1975) | 1984 ASCE Guideline | 2001 ALA Steel Pipe Guidelines | 2004 PRCE Guideline | Wijewickreme et al. (2005) |
|---|---|---|---|---|
| 4% | 3 to 5% | 4% (pressure integrity goal), 2% (normal operability goal) | 2 to 4% (pressure integrity goal), 1 to 2% (normal operability goal) | 3% (10% probability of tensile rupture) 10% (90% probability of tensile rupture) |

## 4.3 Fault Rupture

If a surface fault that crosses a pipeline ruptures, the pipeline will deform longitudinally and bend to accommodate the displacement. For active fault crossings, the fault location must be delineated and measures taken to accommodate the potential deformation caused by fault displacement.

Based on the known and inferred faults shown in Figure 5, which all have very long return periods, it doesn't appear that there are known faults that cross the proposed alignment.

## 4.4 Liquefaction

Liquefaction occurs when saturated cohesionless soil is rapidly and cyclically loaded under undrained conditions. Excess pore water pressure is developed and soil particle effective stress is reduced to near zero, which causes a significant reduction in shear strength.

Not all soils are susceptible to liquefaction. If the soils and groundwater along the alignment are not susceptible to liquefaction, there is no liquefaction hazard. There are several criterial by which liquefaction susceptibility can be judged:

- Geologic process that sort soils into uniform grain size distributions and deposit them in loose states produce soils with high liquefaction susceptibility, including fluvial, colluvial, and aeolian deposits and, less frequently, in alluvial-fan, alluvial-plain, beach, terrace, playa, and estuary deposits.
- Soils of Holocene aged (approximately past 10,000 years) are most susceptible and susceptibility decreases with age.
- Liquefaction occurs only in saturated soils, so the depth to groundwater influences susceptibility. Liquefaction susceptibility is highest where groundwater is within a few meters of the ground surface.
- Soil compositional characteristics that allow for high volume change during undrained loading are most susceptible. Generally, uniform, rounded gravels, sands, and silts are most susceptible to liquefaction. Fine-grained soils may also be susceptible to liquefaction if the following conditions are met:
    - Grain size fraction finer than 0.005 mm $\leq$ 15%

- Liquid limit, LL $\leq$ 35%
- Natural water content, w $\geq$ 0.9 LL
- Liquidity index $\leq$ 0.75

Identifying whether a liquefaction hazard exists requires site-specific information on the subsurface soil properties and peak ground acceleration. However, there are some general guidelines for identifying whether or not a significant liquefaction hazard exists. Youd (1998) proposed a set of purposely conservative screening guidelines for estimating liquefaction susceptibility, which are shown in Table 5. Liquefaction potential is very low for earthquake magnitudes less than 5.0.

**Table 5: Minimum PGAs that are Capable of Liquefying Very Susceptible Natural Soils**

| Earthquake Magnitude | Peak Ground Acceleration for Very Low Liquefaction Hazard | |
|---|---|---|
| | Stiff Sites[1] | Soft Sites[2] |
| Less than 5.2 | < 0.4 g | < 0.1 g |
| 5.2 to 6.4 | < 0.1 g | < 0.05 g |
| 6.4 to 7.6 | < 0.05 g | < 0.025 g |
| Greater than 7.6 | < 0.025 g | < 0.025 g |

1 1992 AASHTO Type I and II soil
2 1992 AASHTO Type II and IV soil

Based on Figure 2, the probability of an earthquake having a magnitude equal to or greater than 5.0 over a period of 50 years ranges between approximately 1% (NE terminus) and 25% (SW terminus). Since the potential for liquefaction cannot be completely ruled out based on the screening criteria from Table 5, other screening criterial can be used.

We understand that the pipeline will be blasted and trenched into rock along some portions of the alignment. Liquefaction will not occur where the pipeline is within rock.

Due to the rocky terrain and arid climate along the majority of the pipeline, we anticipate that liquefaction potential will be restricted to a few select locations. For the portions of the proposed pipeline alignment that are not in rock, areas of potential liquefaction can be determined using the relevant portions of the guidelines developed for California (SCEC, 1999)summarized below.

- Areas where the groundwater level is less than 15 meters below the ground surface
- Areas of uncompacted or poorly compacted cohesionless fills that are or may become fully saturated
- Areas along river channels and their historic flood plains, marshes, and estuaries or soils less than 1,000 years old that may contain loose deposits of cohesionless soils

The impact of liquefaction on buried pipelines is limited to relatively modest amounts of floatation, sinking, and general ground settlement from dissipation of excess pore water pressure. Pipeline floatation or sinking occurs when the pipe is located below the groundwater table within a zone of liquefiable soil. Whether a pipe will tend to float or sink depends on the buoyancy of the pipeline and residual strength of the liquefied soil. Floatation or sinking within the liquefied soil is associated with low pipeline stresses because the buoyance forces are very low compared to the bending stiffness of the pipeline.

The most significant threat to a buried pipeline from liquefaction alone is ground settlement. Ground settlement occurs when the rearrangement of soil particles leads to densification and a reduction in soil volume. If the pipeline is located in the layer of competent soil near the surface, it will be subjected to displacement associated with ground subsidence. These displacements will rarely exceed a few tens of centimeters, and only for very large earthquakes. This level of displacement is not sufficient to pose a threat of pressure loss to a modern steel pipeline based on empirical evidence. This should not be surprising considering the level of deformation from liquefaction settlement is generally much less than pipeline deformation that occurs when the pipeline is roped into a pipe trench. An exception to this general conclusion is where pipelines are connected to other structures that can result in differential settlement and much greater loading.

In lieu of a detailed liquefaction assessment, the following assumptions are suggested for a preliminary assessment of the liquefaction settlement hazard posed to the pipeline:

- Assume total settlement is 4% of the total thickness of the potentially liquefiable soil (saturated cohesionless material)
- Assume the settlement occurs at an abrupt offset adjacent to structures or foundations supported on deep foundations
- For open field situations, assume 100 m of pipeline is exposed to abrupt vertical downward movement equal to half the total settlement.

## 4.5 Lateral Spread Movement

Lateral spread involves horizontal movement of competent surficial soils due to liquefaction of an underling deposit. A simplified approach (Youd et al., 2002), which is based upon multiple linear regression of data from past earthquake observations, represents the best available method for estimating lateral spread movement, outside of numerical analyses. Youd et al. (2002) proposed two relationships for estimating lateral spread displacement, one for the failure of gently sloping ground, and the other for failure of a bank along a water body. The bank failure equation is shown below and represents a more critical case for estimating lateral spread displacement.

$$\begin{aligned} \mathit{log}(\mathrm{LSD}) = \; & -16.713 + 1.532(\mathrm{M}) - 1.406\,\mathit{log}(\mathrm{R}^*) \\ & - 0.012(\mathrm{R}) + 0.592\,\mathit{log}(\mathrm{W}) + 0.540\,\mathit{log}(\mathrm{T}_{15}) \\ & + 3.413\,\mathit{log}(100-\mathrm{F}_{15}) - 0.795\,\mathit{log}(\mathrm{D50}_{15} + 0.1) \end{aligned} \qquad \{1\}$$

where:

| | |
|---|---|
| LSD | horizontal lateral spread movement (m) |
| M | earthquake moment magnitude [6.0 < M < 8.0] |
| R | epicentral distance (km) |
| R* | $R + R_0$ |
| $R_0$ | $10^{0.89M - 5.64}$ |
| 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 atmospheric pressure and 60% driving efficiency |
| $F_{15}$ | average fines content (%) in $T_{15}$ [$0 < F_{15} < 50$] |
| $D50_{15}$ | average median particle size (mm) in T15 [$0 < D50_{15} < 50$] |

Site specific information is required to compute lateral displacement from Equation 1. However, lateral spread displacement is not anticipated for earthquakes with magnitudes less than 6.0. Based on Figure 2, , the probability for a magnitude 6.0 earthquake over a period of 50 years along the pipeline ranges between approximately 0% (NE terminus) to 4% (SE terminus). Therefore, for practical purposes, only those locations that are deemed susceptible to liquefaction and where the probability of exceeding a magnitude 6.0 exceeds 2% in 50 years need be considered a credible hazard for lateral spreading.

## 4.6 Seismic-Induced Slope Instability

Seismic-induced slope movement may be triggered in a variety of geologic and topographic settings. There is insufficient information available at this time to perform a site-specific slope stability evaluation. However, simplistic models can be used to estimate the possibility of seismic-induced slope instability. Using the Newmark sliding block analysis method (Newmark, 1965), the adjacent hillsides are treated like solid block mass as shown in Figure 6.

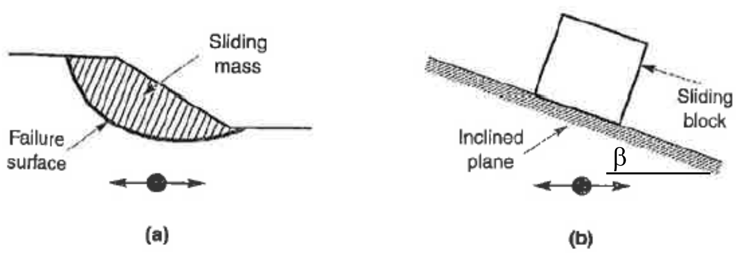



**Figure 6: Sliding Block Slope Stability Analogy**

During a seismic event, an inertial force is transmitted to the sliding block. When the inertial force acts in the downslope direction, the slope factor of safety can be computed from the following equation.

$$FS = \frac{[\cos\beta - k_h \sin\beta]\tan\phi}{\sin\beta + k_h \cos\beta} \qquad \{2\}$$

where:

| | |
|---|---|
| $\beta$ | Slope angle |
| $\phi$ | Earth material shear resistance friction angle |
| $k_h$ | pseudostatic acceleration coefficient (taken as 0.5*PGA/g) |

This relationship between the factor of safety and pseudostatic acceleration coefficient is plotted for various slope angles and shear resistance friction angles in Figure 7.

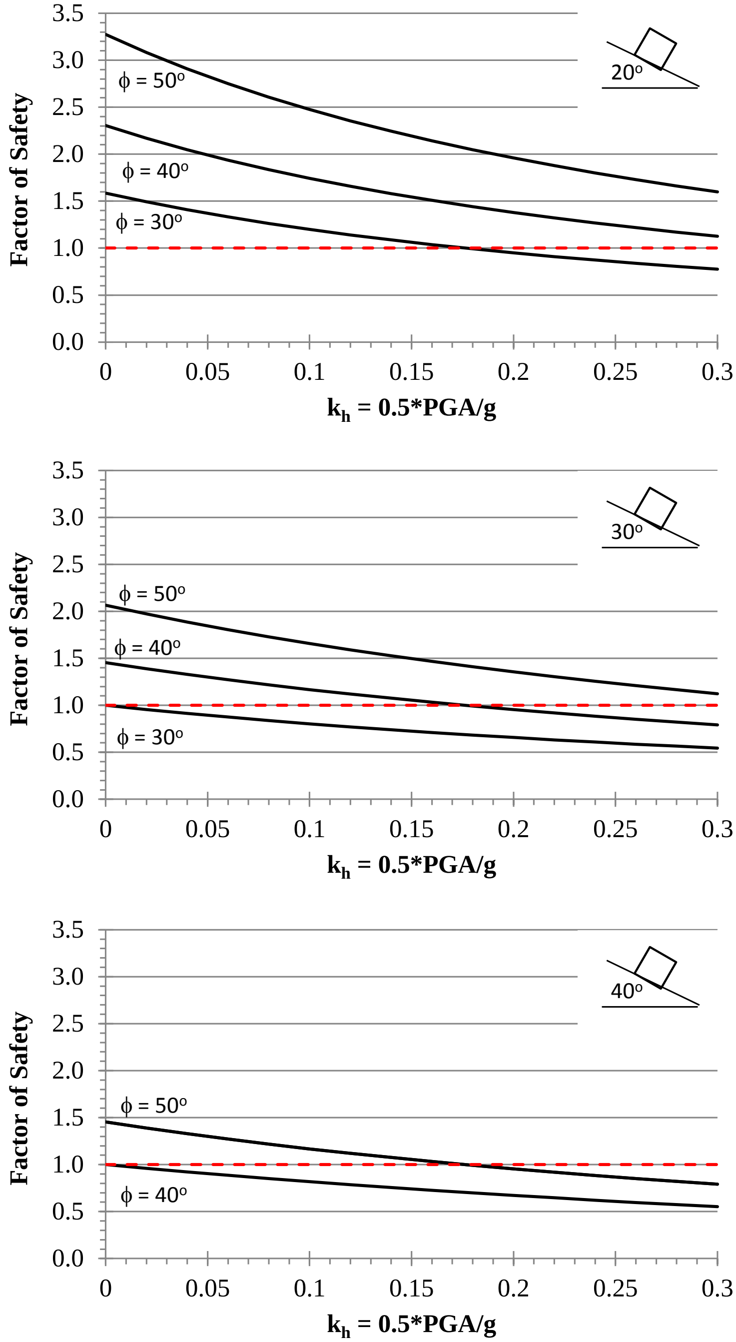



**Figure 7: Variation in Pseudostatic FS for Newmark Sliding Block Analogy**

For a PGA = 0.2 g (see worst case from Table 3), the pseudostatic coefficient, $k_h$ is 0.1. Using $k_h = 0.1$, a pseudostatic factor of safety can be estimated for seismic-induced slope instability using Figure 7.

Permanent slope displacements depend on the relationship between the yield acceleration and the maximum acceleration experienced during the earthquake. If the yield acceleration of a slope is greater than the maximum acceleration of a particular earthquake, no displacement occurs. Block yield acceleration (i.e., minimum acceleration required to produce instability of the block) can be computed using the following equation for the downslope direction.

$$k_y = a_y/g = \tan(\phi - \beta) \qquad \{3\}$$

A common equation to estimate permanent displacement of a sliding block was developed by Ambraseys and Menu (1988) and is shown below.

$$\log(u) = 0.9 + \log\left[\left(1 - \frac{a_y}{PGA}\right)^{2.53}\left(\frac{a_y}{PGA}\right)^{-1.09}\right] \qquad \{4\}$$

For a PGA = 0.2 g (see worst case from Table 3), the results of the above equation are shown in Figure 8.

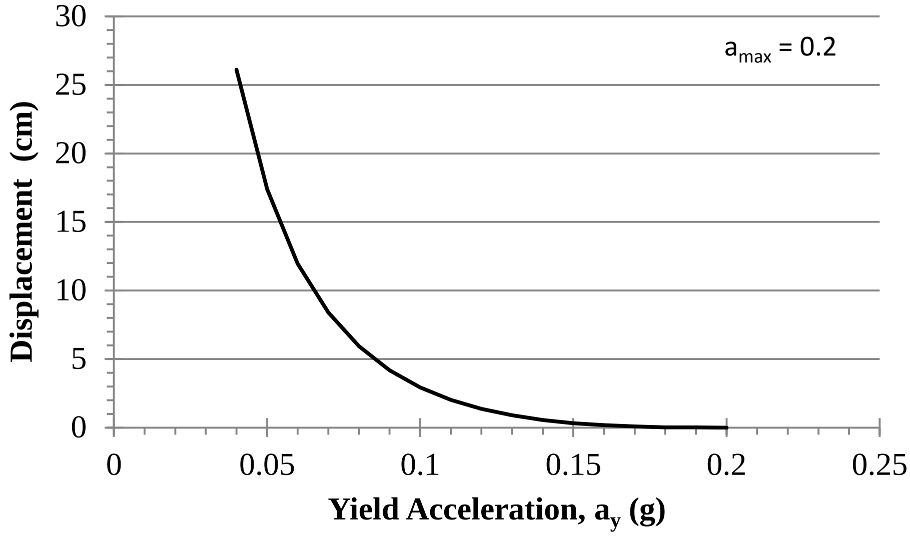



**Figure 8: Seismic-Induced Permanent Slope Displacement**

Figure 8 and equation 3 can be used together to estimate possible slope deformation for various slope angles and shear resistance friction angles for a PGA equal to 0.2.

## 5. CONCLUSIONS

Based on the information currently available and this evaluation, the credible earthquake hazards to the proposed Transpecos pipeline are limited to: liquefaction, lateral spread movement, and seismic-induced slope stability.

Liquefaction

We understand that much of the alignment will be located in rock. For these areas, liquefaction will not occur. For the portions of the proposed pipeline alignment that are not in rock, possible areas susceptible to liquefaction include:

- Areas where the groundwater level is less than 15 meters below the ground surface
- Areas of uncompacted or poorly compacted cohesionless fills that are or may become fully saturated
- Areas along river channels and their historic flood plains, marshes, and estuaries or soils less than 1,000 years old that may contain loose deposits of cohesionless soils

If, with further evaluation, it is assessed that liquefaction is possible in certain areas, the following assumptions can be used for a preliminary assessment of the liquefaction settlement hazard posed to the pipeline:

- Assume total settlement is 4% of the total thickness of the potentially liquefiable soil (saturated cohesionless material)
- Assume the settlement occurs at an abrupt offset adjacent to structures or foundations supported on deep foundations
- For open field situations, assume 100 m of pipeline is exposed to abrupt vertical downward movement equal to half the total settlement.

If there are areas that could potentially liquefy, additional site-specific information would be required for final design. This information would include:

- Soil borings in the liquefiable soil layers, including standard penetration testing
- Soil laboratory testing to determine the grain size distribution, including silt and clay content
- Groundwater observations

Lateral Spreading

In addition to settlement caused by liquefaction, lateral spreading displacement can occur in liquefied soil deposits. If the soil is not susceptible to liquefaction, lateral spreading will not occur. For those areas that are susceptible to liquefaction, additional site-specific information would be required as described above. Additionally, topographic information of the potentially liquefiable zone would be required (i.e., extents, depth, geometry of river bank, etc.). An equation is provided to estimate lateral spreading displacement with site-specific information. For practical purposes, only those locations where the probability of exceeding a magnitude 6.0 earthquake exceeds 2% in 50 years need be considered.

Seismic-Induced Slope Instability

Seismic-induced slope movement may be triggered in a variety of geologic and topographic settings and should be considered a seismic hazard if failure of the slope could impact the pipeline. There is insufficient information available at this time to perform a site-specific slope stability evaluation. However, simplistic models are presented to estimate the potential for unstable slopes during a seismic event. The pseudostatic factors of safety are plotted for various slope angles and shear resistance friction angles. Figure 7 can be evaluated assuming $k_h = 0.1$, which corresponds to a PGA = 0.2. Similarly, Figure 8 can be evaluated together with Equation 3 (slope geometry and strength) to estimate possible slope displacement due to a seismic event with a PGA = 0.2.

Site-specific information will be required for hillsides that appear unstable or marginally stable. For the purposes of this memorandum an unstable or marginally stable hillside will have a factor of safety less than or equal to 1.1 and would have displacements greater than or equal to 10 cm. For these hillsides, site-specific information similar to what was outlined for liquefaction will be required.

## 6. REFERENCES

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