Seismic Design for Buried Flexible Structures

P Waves “Push”

S Waves “Shake”

By

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President of Clark Engineers, Inc.
Presented to Foundation Performance Association
June 10, 2015

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Qualifying Statement

• No original work by this author is included herein.
• All information provided herein is from published sources.
• References are provided.
• Best effort has been made to ensure correct methods; however all methods should be verified for accuracy before using.
• PDF files of references can be provided upon written request.
• Any constructive comments from “those of skill in the art” † are greatly appreciated and may be sent to jmc@clark-engineers.com.

†Xerxes Patent US 6,397,168 B1 column 3, line 67 (20)
Purpose

1) Provide discussion and documentation on various methods used for seismic design of buried flexible structures subjected to seismic loading.

2) Provide a single source for much of the required information.

3) Expose reader/attendees to the main points that are considered.
Focus of this Presentation

0) Historical background, some seismic information, shear modulus, and seismic spectra

1) Axial stress due to P waves and S waves
2) Wang method (23) (NCHRP) (4) transverse loads on circular conduits and box culverts
3) Xerxes patent (20) (reduced shear modulus) with transverse loads on FRP UST’s
4) Sloshing
5) Liquefaction
6) Buckling of soil surrounded tubes
Historical Background

• 1980’s customer’s started requiring seismic calculations for underground storage tanks (UST’s). At this time there were no known treatises on this topic.

• Hired local consultant PhD, PE to write paper.
  - Results were based on methods used for pipe lines – axial stress due to P and S waves. Based largely on the work of Newmark (17 & 18) and Yeh (24).
  - Method was reviewed and used current Uniform Building Code (~1985). Method relied on confining pressure of the surrounding soil/backfill.
Historical Background

- In 1999 this method was updated to include derivation of equations for stresses due to P and S waves and updated for 1997 UBC and built MathCAD sheet to automate calculations.
- In ~2004 it was again updated to latest International Building Code (IBC).
- Client in New Zealand requested new update in 2015.
- Latest literature search revealed alternate methods that focused on lateral-diametrical stress not previously included.
  - Specifically Wang (23) / NCHRP (4) method and Xerxes (20) method
Current method *(axial stress)* uses soil strain to calculate stress in conduit/tank.

- Stress in buried structures is a function of *shear wave velocity* \(C_s\).
- Shear wave velocity is a function of *shear modulus* of surrounding soil – \(G_m\).
- Shear modulus is a function of *soil type* and *confining pressures*.
- Resulting stresses are calculated in axial direction e.g. for a pipeline and horizontally (perpendicular to long axis).
- A check for *slippage* is included for the axial stress condition.
- *Sloshing effects* are checked.

Historical Background

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Examples of Longitudinal Effects for P Waves and S Waves:

P Wave
“Push”

S Wave
“Shake”

Examples of Longitudinal Effects for L Waves and R Waves:

L Wave
“Love”

R Wave
“Rayleigh”

Some Earthquake Characteristics

Figure 1.1 shows classification of dynamic problems

- 10-20 repetitions of shaking with different amplitudes
- Irregular time history
- Period within each pulse 0.1 to 3.0 seconds
- Time of loading 0.02 to 1.0 seconds
- Soil strain ranges from .0001 to .001 in/in

Reference K. Ishihara, Soil Behavior in Earthquake Geotechnics, ©1996, pp. 2-4 (11)
Some Earthquake Characteristics

Figure 1.2 (ibid) shows variation of soil properties with strain.

- Some soils exhibit dilatancy - dilate or to contract during drained shear or pore water pressure changes.
- Its effect begins to appear when soil strain reaches 0.0001 to 0.001 or 0.01% to 0.1%

Progressive changes in soil properties during load repetition such as *degradation in stiffness* of saturated soils or hardening of dry or partially saturated soils can occur as a consequence of dilatancy during shear.

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Some Earthquake Characteristics

- Shear modulus decreases with strain and damping increases with strain (ibid p. 33-34).

Reference strain

\[ \gamma_r = \frac{\tau_f}{G_0} \]

where

- \( \gamma_r \) = strain at failure for linear elastic \( G_0 \)
- \( \tau_f \) = shear stress at failure
- \( G_0 \) = shear modulus of soil
Some Earthquake Characteristics

Figure 3.11 shows the relationship between shear modulus and damping ratio. (ibid)

Note that the secant shear modulus is reduced to half the initial shear modulus when the shear strain becomes equal to the reference strain [i.e. when $\frac{\gamma_a}{\gamma_r} = 1.0 \rightarrow \frac{G}{G_0} = \frac{1}{1+1} = \frac{1}{2}$]
# Shear Modulus \( G_o \)

Tables I and II provide empirical values for initial values of shear modulus.

## Table I.

<table>
<thead>
<tr>
<th>Reference</th>
<th>( A )</th>
<th>( F(e) )</th>
<th>( n )</th>
<th>Material</th>
<th>Sample Size</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prange (1981)</td>
<td>7230</td>
<td>((2.97 - e)^2 / (1 + e))</td>
<td>0.38</td>
<td>Ballast</td>
<td>Dia.: 100 cm</td>
<td>Resonant column</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{50}^{**} = 40 \text{ mm} )</td>
<td>Length: 60 cm</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( U_c^{***} = 3.0 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kokusho and Esashi (1981)</td>
<td>13000</td>
<td>((2.17 - e)^2 / (1 + e))</td>
<td>0.55</td>
<td>Crushed rock</td>
<td>Dia.: 30 cm</td>
<td>Triaxial</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{50} = 30 \text{ mm} )</td>
<td>Length: 60 cm</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( U_c = 10 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kokusho and Esashi (1981)*</td>
<td>8400</td>
<td>((2.17 - e)^2 / (1 + e))</td>
<td>0.60</td>
<td>Round gravel</td>
<td>Dia.: 30 cm</td>
<td>Triaxial</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{50} = 10 \text{ mm} )</td>
<td>Length: 60 cm</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( U_c = 20 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tanaka et al. (1987)</td>
<td>3080</td>
<td>((2.17 - e)^2 / (1 + e))</td>
<td>0.60</td>
<td>Ballast</td>
<td>Dia.: 10 cm</td>
<td>Triaxial</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{50} = 10 \text{ mm} )</td>
<td>Length: 20 cm</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( U_c = 20 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Goto et al. (1987)</td>
<td>1200</td>
<td>((2.17 - e)^2 / (1 + e))</td>
<td>0.85</td>
<td>Ballast</td>
<td>Dia.: 30 cm</td>
<td>Triaxial</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{50} = 2 \text{ mm} )</td>
<td>Length: 60 cm</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( U_c = 10 )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Undisturbed</td>
<td></td>
<td></td>
<td></td>
<td>Ballast</td>
<td>Dia.: 30 cm</td>
<td>Triaxial</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( D_{50} = 10.7 \text{ mm} )</td>
<td>Length: 60 cm</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( U_c = 13.8 )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Reference K. Ishihara, Soil Behavior in Earthquake Geotechnics, ©1996, pg. 100 (11)

*This value is used in Xerxes model – slide 98.

** \( D_{50} \) - 50% finer than value

*** \( U_c - \frac{D_{60}}{D_{10}} \)

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Shear Modulus $G_o$

Table II.

<table>
<thead>
<tr>
<th>Reference</th>
<th>A</th>
<th>$F(e)$</th>
<th>$n$</th>
<th>Soil Material</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sand</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardin-Richart (1963)</td>
<td>7000</td>
<td>$(2.17 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Round grained Ottawa sand</td>
<td>Resonant column</td>
</tr>
<tr>
<td></td>
<td>3300</td>
<td>$(2.97 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Angular grained crushed quartz</td>
<td>Resonant column</td>
</tr>
<tr>
<td>Shibata-Soelarno (1975)</td>
<td>42000</td>
<td>$(0.67 - e) / (1 + e)$</td>
<td>0.5</td>
<td>Three kinds of clean sand</td>
<td>Ultrasonic pulse</td>
</tr>
<tr>
<td>Iwasaki et al. (1978)</td>
<td>9000</td>
<td>$(2.17 - e)^2 / (1 + e)$</td>
<td>0.38</td>
<td>Eleven kinds of clean sand</td>
<td>Resonant column</td>
</tr>
<tr>
<td>Kokusho (1980)</td>
<td>8400</td>
<td>$(2.17 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Toyoura sand</td>
<td>Cyclic triaxial</td>
</tr>
<tr>
<td>Yu-Richart (1968)</td>
<td>7000</td>
<td>$(2.17 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Three kinds of clean sand</td>
<td>Resonant column</td>
</tr>
<tr>
<td><strong>Clay</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardin-Black (1968)</td>
<td>3300</td>
<td>$(2.97 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Kaolinite, etc.</td>
<td>Resonant column</td>
</tr>
<tr>
<td></td>
<td>4500</td>
<td>$(2.97 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Kaolinite, $I_p^{**} = 35$</td>
<td>Resonant column</td>
</tr>
<tr>
<td>Marcuson-Wahls (1972)</td>
<td>450</td>
<td>$(4.4 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Bentonite, $I_p = 60$</td>
<td>Resonant column</td>
</tr>
<tr>
<td>Zen-Umehara (1978)</td>
<td>2000~4000</td>
<td>$(2.97 - e)^2 / (1 + e)$</td>
<td>0.5</td>
<td>Remolded clay, $I_p = 0~50$</td>
<td>Resonant column</td>
</tr>
<tr>
<td>Kokusho et al. (1982)</td>
<td>141</td>
<td>$(7.32 - e)^2 / (1 + e)$</td>
<td>0.6</td>
<td>Undisturbed clays, $I_p = 40~85$</td>
<td>Cyclic triaxial</td>
</tr>
</tbody>
</table>

* $\sigma'_0$ : kPa, $G_o$ : kPa, **$I_p$ : Plasticity Index

Reference K. Ishihara, Soil Behavior in Earthquake Geotechnics, ©1996, pg. 89 (11)
Table III shows some representative values of initial soil shear modulus \((G_o)\) for gravels, sands and clays.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Soil Density (pcf)</th>
<th>Void Ratio</th>
<th>Type</th>
<th>Shear Modulus ((G_o)) (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Depth (ft)</td>
<td>2</td>
</tr>
<tr>
<td>Gravely</td>
<td>120</td>
<td>0.5</td>
<td>Round Gravel</td>
<td>8786</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gravel</td>
<td>3594</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Gravel</td>
<td>2578</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Crushed Rock</td>
<td>13427</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Undisturbed Gravel</td>
<td>7390</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ballast</td>
<td>10786</td>
</tr>
<tr>
<td>Sands</td>
<td>120</td>
<td>0.5</td>
<td>Round grained Ottawa</td>
<td>6399</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Ang’r gr’nd crshd qtz</td>
<td>6599</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3 kinds clean sand</td>
<td>2340</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>11 kinds clean sand</td>
<td>6138</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Toyura sand</td>
<td>7679</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3 kinds clean sand</td>
<td>6399</td>
</tr>
<tr>
<td>Clays</td>
<td>100</td>
<td>0.5</td>
<td>Kaol.te etc</td>
<td>7378</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Kaol.te PI=35</td>
<td>10061</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Bent’te PI=60</td>
<td>2508</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Remold clay</td>
<td>3248</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Remold clay</td>
<td>8943</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Undist clay PI=40~85</td>
<td>2403</td>
</tr>
</tbody>
</table>

Reference K. Ishihara, Soil Behavior in Earthquake Geotechnics, ©1996, pp. 100 and 89 (11)
Reference provides equations to calculate \(G\) based on vertical soil stress.
Response Spectra vs. ASCE 7 Design Curves

Example

1994 Northridge Earthquake

<table>
<thead>
<tr>
<th></th>
<th>Horizontal</th>
<th>Vertical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Acceleration</td>
<td>1.78 g</td>
<td>1.047 g</td>
</tr>
<tr>
<td>Peak Velocity</td>
<td>47.37 in/s</td>
<td>-28.469 in/s</td>
</tr>
<tr>
<td>Peak Displacement</td>
<td></td>
<td>6.7 in</td>
</tr>
<tr>
<td>Initial Velocity</td>
<td>0.67 in/s</td>
<td>0.53 in/s</td>
</tr>
<tr>
<td>Initial Displacement</td>
<td>1.73 in</td>
<td>1.944 in</td>
</tr>
</tbody>
</table>

ASCE 7 Mapped Acceleration 1994

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Longitude</td>
<td>34.23046 N</td>
</tr>
<tr>
<td>Latitude</td>
<td>-118.5369</td>
</tr>
<tr>
<td>$S_s$</td>
<td>1.905 g</td>
</tr>
<tr>
<td>$S_1$</td>
<td>.614 g</td>
</tr>
</tbody>
</table>

After doing code calculations

$$a_p = 18.387 \frac{ft}{s^2} = 57.1\% \ g$$

$$V_p = 25 \frac{cm}{s} = 9.8 \frac{ft}{s}$$

• Use response spectra if close to site
• Otherwise use code values

*See slide 19.*
Example of Response Spectra

• Actual data has many spikes
• Using maximum and minimum values for acceleration and displacement then maximum and minimum spectra curves can be plotted

See ref “Response Spectra as a Useful Design and Analysis Tool for Practicing Structural Engineers,” Sigmund A. Freeman, 2007 (9)

• Use average for design

“Note: The code is based on a 2475 year event. The structure is designed for 2/3 of that. It is assumed that in a major event the structure will go beyond the elastic limits and survive in the inelastic range by ductility. An important part of the structural design is to provide ductility.” – S. Freeman

Figure 10(a) from ref 9 shows a typical response spectra on a tripartite plot. This reference describes the method to create an average response spectra.
Proposed Method to Obtain Velocity from Response Spectra

1) Using $a_p$ from ASCE 7** code calculations (see slides 27-30), enter tripartite graph where average acceleration crosses the data plot.
2) Construct line perpendicular to average to intersect $a_p$.
3) Read velocity at this intersection.

*There are probably other methods. If period can be determined, use the corresponding value.

**ASCE 7 calculations using mapped acceleration.

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0) Historical background, some seismic information, shear modulus, and seismic spectra

1) **Axial stress due to P waves and S waves**

2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts

3) Xerxes patent (20) (reduced shear modulus) with transverse loads on FRP UST’s

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5) Liquefaction

6) Buckling of soil surrounded tubes
Four Primary Stresses for Compression Waves and Shear Waves

- Compression Wave Axial \( \sigma_{ac} \)
- Compression Wave Bending \( \sigma_{ab} \)
- Shear Wave Axial \( \sigma_{as} \)
- Shear Wave Bending \( \sigma_{bs} \)
Compression Wave Stress

**Axial Stress**

\[ \sigma_{ac} = \frac{E_A V_p F_m}{C_p} \]

where \( C_p \) = compression wave velocity

\[ V_p = \frac{a_p \times 48 \text{ in/}(\text{sec})}{g}; \]  
\( a_p \) = particle acceleration; 
\( g \) = acceleration of gravity 
\( E_A \) = Axial modulus of elasticity of pipe or tank 
\( F_m \) = percent retention of modulus

**Bending Stress**

\[ \sigma_{bc} = \frac{0.385 E_A R a_p F_m}{C_p^2} \]

where \( R \) = radius of tank or pipe

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Shear Wave Stresses

Axial Stress

\[ \sigma_{as} = \frac{E_A \cdot V_p \cdot F_m}{2 \cdot C_s} \]

where \( C_s \) = shear wave velocity

Bending Stress

\[ \sigma_{bs} = \frac{E_A R \cdot a_p F_m}{C_s^2} \]

If \( a_p \) and \( V_p \) are provided, use these values. Otherwise, use method in ASCE 7.

Note that stresses increase with decreasing shear wave velocity.
Example of IBC/ASCE 7 Determination for Particle Acceleration ($\alpha_p$)

1) Determine seismic accelerations from USGS site (same as acceleration map in ASCE 7-10)

2) Enter longitude and latitude in earthquake USGS site. [Link](http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf)

3) Determine $S_s$ and $S_1$ accelerations and velocities by IBC/ASCE 7 method (either MAP or USGS site)

† Note this site provides accelerations world wide.
Mapped seismic acceleration for Continental US is shown below

http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf

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1) Axial Stress Due to P Waves and S Waves

**Example Using USGS Site**


**Seismic Load**

<table>
<thead>
<tr>
<th>Report Title</th>
<th>Northridge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thu June 4, 2015 16:17:03 UTC</td>
</tr>
<tr>
<td>(which utilizes USGS hazard data available in 2008)</td>
<td></td>
</tr>
<tr>
<td>Site Coordinates</td>
<td>34.2131°N, 118.5369°W</td>
</tr>
<tr>
<td>Site Soil Classification</td>
<td>Site Class D – &quot;Stiff Soil&quot;</td>
</tr>
<tr>
<td>Risk Category</td>
<td>IV (e.g. essential facilities)</td>
</tr>
</tbody>
</table>


Latitude ≡ "36.23046 N"
Longitude ≡ −118.5369
Site_Class ≡ "D"
Loc ≡ "Northridge, CA"
S_S ≡ 190.5%
S_L ≡ 61.4%

USGS—Provided Output

\[ S_e = 1.905 \text{ g} \]
\[ S_{se} = 1.905 \text{ g} \]
\[ S_{se} = 1.270 \text{ g} \]
\[ S_s = 0.614 \text{ g} \]
\[ S_{se} = 0.922 \text{ g} \]
\[ S_{se} = 0.614 \text{ g} \]
# Code Calculations

<table>
<thead>
<tr>
<th>Code</th>
<th>Code = “ASCE 7-10”</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Loc = “Northridge, CA”</td>
</tr>
<tr>
<td>Short Period</td>
<td>$S_S = 190.5%$</td>
</tr>
<tr>
<td>Long Period</td>
<td>$S_1 = 61.4%$</td>
</tr>
</tbody>
</table>

Mapped accelerations are found with USGS program “Seismic Hazard Curves and Uniform Response Spectra” most recent version.

## Soil Profile Type

<table>
<thead>
<tr>
<th>Used in Analysis</th>
<th>Site_Class = “D”</th>
</tr>
</thead>
</table>

Note: Per 2012 IBC, Sect 1613.5.2, pg 340, "when the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used".

Site Classes
- A  Hard Rock
- B  Rock
- C  Very dense soil and soft rock
- D  Stiff soil
- E  Soft clay soil
- F  Soils vulnerable to potential failure or collapse (see ASCE 20.3.1, page 203)
Code Calculations

\( S_{DS} \) and \( S_{DL} \) per Section 11.4.4 (ASCE 7-10, page 65) (1)

Design spectral response acceleration parameter at short periods

\[
S_{DS} = \frac{2^*}{3} \cdot S_{MS} = \frac{2}{3} \cdot (F_a \cdot S_S) \\
S_{DL} = \frac{2^*}{3} \cdot S_{M1} = \frac{2}{3} \cdot (F_v \cdot S_1)
\]

*Recall 2/3 factor per (slide 18)

Site Coefficients \( F_a \) and \( F_v \) per Tables 11.4-1 and 11.4-2 (ASCE 7-10, page 66) for site class Site_Class = “D” (only Site_Class = “D” values shown)
Determine $F_a$

$$F_a = \begin{cases} 
F'_a a_0 & \text{if } S_S \leq S'_S 0 \\
\text{interp}(S'_S, F'_a, S_S) & \text{if } S'_S 0 < S_S < S'_S \text{last } (S'_S) \\
F' a_{\text{last }} (F'_a) & \text{if } S_S \geq S'_S \text{last } (S'_S)
\end{cases}$$

Determine $F_v$

$$F_v = \begin{cases} 
F'_v v_0 & \text{if } S_1 \leq S'_{10} \\
\text{interp}(S'_1, F'_v, S_L) & \text{if } S'_{10} < S_1 < S'_1 \text{last } (S'_1) \\
F' v_{\text{last }} (F'_v) & \text{if } S_1 \geq S'_1 \text{last } (S'_1)
\end{cases}$$

(MCAD routines for interpolation are shown)

1) Axial Stress Due to P Waves and S Waves

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For Rigid Nonbuilding Structures (ASCE 7-10 15.4-5)
Assume weight of 1 lbf

\[ V_{Tank} = 0.3 \cdot S_{DS} \cdot I_e \cdot 1\text{lbf} \]
where \( I_e = 1.5 \)

Seismic Importance Factor per
ASCE 7-10, Table 1.5-2, pg 5 (1) for
Category IV: substantial hazard to
community (worst case)

Effective Particle Acceleration
Let \( W_p = 1\text{lbf} \)

\[ M = \frac{W_p}{g}; \quad a_p = \frac{V_{Tank}}{M} \]

Effective Particle Velocity

\[ V_P = \frac{a_p \cdot 48\text{in/sec}}{g}; \]
or

\[ V_P = \frac{a_p \cdot 25\text{in/sec}}{g} \]

From Northridge Earthquake; use \( a_p \) to compute \( V_p \), then
compute longitudinal seismic stresses or use \( V_p \) provided for
specific site.

From tripartite plot.

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Combined Stress

Once all stresses are calculated, use SRSS Method for combined stress.

\[ \sigma_{comb} = \sqrt{\sigma_{ac}^2 + \sigma_{ab}^2} + \sqrt{\sigma_{as}^2 + \sigma_{ab}^2} \]

Since waves are out of phase and compression waves arrive first.
1) Axial Stress Due to P Waves and S Waves

Derivation of Axial and Bending Stresses


The particle displacement in the x directions is

\[ X = X(x - c_p \cdot t) \]

where \( x \) = displacement at time zero

\( c_p \) = compression wave velocity

\( t \) = time say one second

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Let us consider a situation in which a seismic wave travels from point 1 to point 2 in the figure above. The particle displacement of the soil in the X direction of the buried structure can be given by:

\[ X = X(x - c_p \cdot t) \]

where \( x \) = displacement at time zero
\( c_p \) = compression wave velocity
\( t \) = time say one second

The various derivatives of the displacement \( X \) with respect to \( x \) and \( t \) are given by the following relations:

- \[ \frac{\delta X}{\delta x} = f'(x - c_p \cdot t) \] \hspace{1cm} a
- \[ \frac{\delta^2 X}{\delta x^2} = f''(x - c_p \cdot t) \] \hspace{1cm} b
- \[ \frac{\delta X}{\delta t} = -c f'(x - c_p \cdot t) \] \hspace{1cm} c
- \[ \frac{\delta^2 X}{\delta t^2} = c^2 f''(x - c_p \cdot t) \] \hspace{1cm} d

Equating a and c gives:
\[ \frac{\delta X}{\delta x} = \frac{1}{c_p} \cdot \frac{\delta X}{\delta t} \]

Equating b and d gives:
\[ \frac{\delta^2 X}{\delta x^2} = \frac{1}{c_p^2} \cdot \frac{\delta^2 X}{\delta t^2} \]

The soil strain is simply \( \varepsilon = \frac{\delta X}{\delta x} \)
which can be related to the particle velocity of the soil \( \frac{dX}{dt} \) by
\[ \varepsilon = \frac{dX}{dt}/(c_p) \]
where \( dX/dt = v_p \) max

Axial stress due to P waves.

Then the stress is simply the modulus of elasticity \( (E) \) times the strain, or
\[ \sigma_a = \pm E \cdot v_p/(c_p) \]
Bending Stress Due to an Axial Shear Wave

The shear wave velocity is given as:
\[ c_s = \left(\frac{(1 - 2\nu)/2(1 - \nu)}{1 - \nu}\right)^{1/2} c_p \]

The particle displacement in the y direction can be written as
\[ Y = Y(x - c_s t) \]

From elementary beam theory, the radius of curvature of a beam is:
\[ \frac{1}{R} = \kappa = \frac{d^2 y}{dx^2} \]

"if the soil is linearly elastic and homogeneous, the displacement will satisfy the differential equation
\[ \frac{\delta^2 x}{\delta t^2} = c^2 \frac{\delta^2 x}{\delta X^2} \]

rewriting this equation as
\[ \frac{\delta^2 x}{\delta X^2} = \frac{\delta^2 x}{\delta t^2} / c^2 \]

It is clear that the curvature of the beam is a function of the particle acceleration of soil by
\[ \kappa = \frac{d^2 x}{dt^2} / c_s^2 \]

where \( a_{s0} = \frac{d^2 x}{dt^2} \) is the maximum ground acceleration due to an axial shear wave.

The bending strain from beam theory is:
\[ \varepsilon = \frac{y}{R}, \]

where \( y \) is the distance to the extreme fiber from the neutral axis of the beam. Since
\[ \kappa = \frac{1}{R} = \frac{d^2 x}{dt^2} / c_s^2 \]

we can write
\[ \varepsilon = \frac{y \cdot a_{s0}}{c_s^2} \]

Then the bending stress is simply
\[ \sigma = E \cdot \varepsilon \]

where \( E = \) modulus of elasticity of the beam section.

\[ \sigma = E \cdot \frac{y \cdot a_{s0}}{c_s^2} \]

QED
Short Section Effect (**lm**)

Slipping must be considered (similar to development length of rebar)

- If the tank length L is less than or equal to 2*lm then the seismic design stress due to wave propagation is controlled by "Slippage".

\[
lm = \frac{\varepsilon_m A}{f} \cdot E_A \cdot F_m
\]

Maximum long term slippage length


Maximum long term soil strain

\[
\varepsilon_m = \frac{\sigma_{comb}}{E_A F_m}
\]

Circular cross-sectional area of the tank shell wall

\[
A = \pi D t
\]

Frictional force per unit length between the soil-tank interface

\[
f = \pi \cdot dia \cdot P_r \cdot \mu
\]
1) Axial Stress Due to P Waves and S Waves

**Short Section Effect (\(lm\))**

\[ P_r = \left( \frac{1 + k_o}{2} \right) \gamma_{soil} \cdot H \]

Average radial soil pressure on tank

where

\( \gamma_{soil} = \) unit weight of surrounding soil

\[ \mu^+ = \frac{k_a + F_w \cdot k_o + k_p}{2 + F_w} \]

Coefficient of friction between soil and cylinder. \( \mu \) approaches \( k_o \) as \( F_w \) \( \rightarrow \) large.

\( F_w = 1 \) or greater

\[ k_a = \frac{1 - \sin \phi}{1 + \sin \phi} \]

Active press coefficient

\[ k_p = \frac{1 + \sin \phi}{1 - \sin \phi} \]

Passive press coefficient

\[ k_o = 1 - \sin \phi \]

At rest press coefficient for smooth wall

\( k_o \) = Coefficient of lateral soil pressure

Use 0.7 for ribbed shell wall (see any soils reference on soil rest pressure) or compute for smooth walls

\( \mu^+ \)

\[ \mu^+ \]

\[ \frac{k_a + F_w \cdot k_o + k_p}{2 + F_w} \]

\( F_w \) \( \rightarrow \) large.

\[ 1 - \sin \phi \]

\[ 1 + \sin \phi \]

\[ 1 - \sin \phi \]

\( 1 - \sin \phi \)

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Total axial stress is

\[ \sigma_{A'} = \frac{f \cdot L_{ss}}{2A} \]

where \( L_{ss} \) = straight shell length – excludes end caps

Control = if \( L < 2lm \), slip, no slip

Seismic Stress Due to Dynamic Soil Pressure (Hoop Stress)

\[ K_c = 4.0 \quad K_s = 5.0 \]

Dynamic stress concentration factors, reference Newmark and Rosenblueth, Fundamentals of Earthquake Engineering (18)

\[ \sigma_{\theta_c} = K_c \cdot \rho \cdot C_p \cdot V_p \]
Maximum hoop stress induced by normal stress

\[ \sigma_{\theta_s} = K_s \cdot \rho \cdot C_s \cdot V_s \]
Maximum hoop stress induced by shear stress
1) Axial Stress Due to P Waves and S Waves

Long and Short Period Regions for $T_L$
Seismic Stress Due to Dynamic Soil Pressure (Hoop Stress)

ASCE 7-10, Table 1.5-1, p. 2, Risk Category IV, Buildings or other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released an is sufficient to pose a threat to the public if released.

Per ASCE 7-10, p. 153, note d, use $T_L = 4$ s for occupancy categories I, II, III (1).
Design Example

1) Axial Stress Due to P Waves and S Waves
Design Example

1) Axial Stress Due to P Waves and S Waves

**Set Values for Some Variables:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tank inside diameter</td>
<td>120 in</td>
</tr>
<tr>
<td>Tank Capacity</td>
<td>20000</td>
</tr>
<tr>
<td>( R_T = \frac{\text{dia}}{2} )</td>
<td>60 in</td>
</tr>
<tr>
<td>( H_{o,b} )</td>
<td>5 ft</td>
</tr>
<tr>
<td>Tank burial depth (grade to top of tank)</td>
<td></td>
</tr>
<tr>
<td>( F_{A,D} )</td>
<td>0.8</td>
</tr>
<tr>
<td>Long term axial tensile modulus retention factor</td>
<td></td>
</tr>
<tr>
<td>( F_{A,S} )</td>
<td>0.8</td>
</tr>
<tr>
<td>Long term axial tensile strength retention factor</td>
<td></td>
</tr>
<tr>
<td>( F_{A,U} = 10000 \text{psi} )</td>
<td></td>
</tr>
<tr>
<td>Tank wall ultimate axial tensile strength, psi. This value is known for tank wall by test data per CSI</td>
<td></td>
</tr>
<tr>
<td>( F_{H,U} = 10000 \text{psi} )</td>
<td></td>
</tr>
<tr>
<td>Tank wall ultimate hoop tensile strength, psi. This value is known for tank wall by test data per CSI</td>
<td></td>
</tr>
<tr>
<td>( E_A = 900000 \text{psi} )</td>
<td></td>
</tr>
<tr>
<td>Axial tensile modulus, psi. This value is known for tank wall by test data per CSI</td>
<td></td>
</tr>
</tbody>
</table>

**Calculations:**

**Shear and compression wave velocities:**

\[
\gamma_{soil} = 120 \text{ psf} \quad \text{Unit weight of soil, } \text{lb/ft}^3 \quad \text{(crushed rock), worse case compared to pea gravel.}
\]

\( g = 32.17 \text{ ft/sec}^2 \) \quad \text{Gravitational constant. Use 32.2 ft/sec}^2 \text{ or 386.4 in/sec}^2. \]

\[
\rho = \frac{\gamma_{soil}}{g} \quad \rho = 0.00018 \text{ lb/ft}^2 \text{sec}^2 / \text{in}^4 \quad \text{Unit mass of soil, lb/sec}^2 / \text{in}^4.
\]

\[
G = (300000) \text{ psi} \quad \text{Soil shear modulus at 5ft bury, psi. (crushed rock), } k' := 0.3 \times \text{last}(G)
\]

\[
C_s = \sqrt{\frac{G}{\rho}} \quad C_s = (1076) \text{ ft/sec} \quad \text{Shear wave velocity, in/sec.}
\]

\[
\nu = 0.4 \quad \text{Poisson’s Ratio (Soil Mechanics by R. W. Lambe and R. V. Whitman, John Wiley & Sons, 1969)}
\]

\[
C_p = \left( 2 - \frac{1 - \nu}{1 + 2 \nu} \right)^{0.5} \quad C_p = (2636) \text{ ft/sec} \quad \text{Compression wave velocity, in/sec.}
\]

**Seismic Load**

- **Site Class:** "D"
- **Location:** Loc = "Northridge, CA"
- **Code:** Code := "ASCE 7-10"
- **Short Period:** \( S_S = 100 \text{.5 %} \)
- **Long Period:** \( S_L = 61.4 \%

**Mapped accelerations are found with USGS program “Seismic Hazard Curves and Uniform Response Spectra” most recent version**

- **Site Classes:**
  - A: Hard Rock
  - B: Rock
  - C: Very dense soil and soft rock
  - D: Stiff soil
  - E: Soft clay soil
  - F: Soils vulnerable to potential failure or collapse (see ASCE 20.3.1, page 203)

**Design spectral response acceleration parameter at short periods**

\[
S_{D,S} = \frac{2}{3} S_{M,S} = \frac{2}{3} \left( F_{A} S_{S} \right)
\]

**Design spectral response acceleration parameter at a period of 1 sec**

\[
S_{D,1} = \frac{2}{3} S_{M,1} = \frac{2}{3} \left( F_v S_1 \right)
\]
1) Axial Stress Due to P Waves and S Waves

**Design Example**

**Site Coefficients** $F_s$, and $F_v$, per Tables 11.4-1 and 11.4-2 (ASCE 7-10, page 66)

For site class $Site\ Class = "D"$

<table>
<thead>
<tr>
<th>$F_s$</th>
<th>$S_s$</th>
<th>$F_v$</th>
<th>$S_L$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.6</td>
<td>0.25</td>
<td>2.4</td>
<td>0.10</td>
</tr>
<tr>
<td>1.4</td>
<td>0.50</td>
<td>2.0</td>
<td>0.20</td>
</tr>
<tr>
<td>1.2</td>
<td>0.75</td>
<td>1.8</td>
<td>0.30</td>
</tr>
<tr>
<td>1.1</td>
<td>1.00</td>
<td>1.6</td>
<td>0.40</td>
</tr>
<tr>
<td>1.0</td>
<td>1.25</td>
<td>1.5</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Determine $F_s$

$F_s = F_{b_0} \text{ if } S_s \leq S_{b_0}$

\[ F_s = \frac{1.00}{\text{interp}(S_s, F_s, S_b) \text{ if } S_{b_0} < S_s < S_{b_{\text{min}}}(S_s)} \]

$F_{b_{\text{min}}(S_s)} \text{ if } S_s > S_{b_{\text{min}}(S_s)}$

Determine $F_v$

$F_v = F_{b_0} \text{ if } S_L \leq S_{b_0}$

\[ F_v = \frac{1.50}{\text{interp}(S_L, F_v, S_l) \text{ if } S_{l_0} < S_L < S_{l_{\text{min}}}(S_L)} \]

$F_{l_{\text{min}}(F_v)} \text{ if } S_L > S_{l_{\text{min}}(S_L)}$

$S_{b_0}$ and $S_{b_0}$ per Section 11.4.4 (ASCE 7-10, page 65)

**Seismic Stresses Due to Wave Propagation for No Slip Condition**

- $E_A = 900000\psi$
- $V_p = -27.43\text{ in/sec}$
- $V_p = \frac{10}{\text{sec}}$
- $\sigma_{b_{\text{max}}} = \frac{E_A R_T \cdot F_M}{C_s^2}$
- $\sigma_{b_{\text{max}}} = (64)\text{ psi}$
- $\sigma_{b_{\text{max}}} = (702)\text{ psi}$
- $\sigma_{b_{\text{max}}} = (860)\text{ psi}$
- $\sigma_{b_{\text{max}}} = (4.1)\text{ psi}$

Axial tensile stress due to compression waves, psi.

Axial bending stress due to shear waves, psi.

Maximum ground particle velocity due to shear waves, in/sec (see Yen, “Seismic Analysis of Slender Buried Seams”, Bulletin of the Seismological Society of America, Vol 64, No 5, pp 1551-1562)

Maximum ground particle velocity due to compression waves, in/sec

Calculate the axial stress from a seismic event using Square Root Sum Squares (SRSS) method (because the maximum values for each stress do not occur at the same time).

$\sigma_{A_{\text{max}}} := \left(\sigma_{b_{\text{max}}}^2 + \sigma_{b_{\text{max}}}^2\right)^{0.5} + \left(\sigma_{b_{\text{max}}}^2 + \sigma_{b_{\text{max}}}^2\right)^{0.5}$

Combined axial stress due to seismic event, psi. Include long term property loss.

$\sigma_{A_{\text{max}}} = (1505)\text{ psi}$

**Seismic Stresses Due to Wave Propagation (Short Section Effect) for Slip Condition**

If the tank length $L$ is less than or equal to 2’1m than the seismic design stress due to wave propagation is controlled by “Slippage”. Calculate the value of $V_p$. Use the value unless otherwise specified by customer.

- $D_{\text{tank}} = 120\text{ in}$
- $W_p = 1\text{ lbf}$
- $M := \frac{W_p}{g}$
- $\mu = 0.49$
- $k_0 = 0.7$
- $H = 10\text{ ft}$

Distance from grade to tank centerline. Worst case (max $\sigma_{A_{\text{max}}}$) is at deepest bury of 7 ft from grade to tank top.
1) Axial Stress Due to P Waves and S Waves

Design Example

\[ P_1 := \left( \frac{1 + k_0}{2} \right) \tan H \]
\[ P_2 := 7.1 \text{ psi} \quad \text{Average radial soil pressure on tank} \]
\[ f := \pi \cdot \text{diam} \cdot P_1 \cdot \mu \]
\[ f = 1307 \text{ lbf/in} \quad \text{Frictional force per unit length between the soil-tank interface.} \]
\[ A := \pi \cdot \text{D}_{\text{in}} \cdot \text{L}_{\text{in}} \]
\[ A = 98.2 \text{ in}^2 \quad \text{Cross-sectional area of the tank shell wall} \]
\[ P_1 := 7.1 \text{ psi} \quad \text{Average radial soil pressure on tank} \]
\[ f = 1307 \text{ lbf/in} \quad \text{Frictional force per unit length between the soil-tank interface.} \]
\[ \epsilon_m := \frac{\sigma_A}{E_A F_m} \quad \epsilon_m = (0.001932) \text{ in/in} \quad \text{Maximum long term soil strain} \]
\[ \text{ln} := \frac{\epsilon_m A}{f} \quad \text{ln} = (118) \text{ in} \quad \text{Maximum long term slippage length} \]
\[ H_{\text{buy}} = 5 \text{ ft} \]

If the tank length \( L \) is less than 2\( L_m \) then the seismic design stress due to wave propagation would be:
\[ \sigma_A := (1565) \text{ psi} \]
\[ \sigma_A := \frac{f \cdot L_m}{2 \cdot A} \quad \sigma_A = 2185 \text{ psi} \]

Seismic Stresses Due to Wave Propagation for Slip or No Slip Condition

Combining the stress equations gives:
\[ \sigma_{A_{\text{in}}} := \text{if}(L_{\text{in}} < 2 \cdot L_{\text{m}}, \sigma_A, \sigma_A) \]
\[ \sigma_{A_{\text{in}}} = (1565) \text{ psi} \]

This "IF" equation will give the correct axial stress value for any tank length.

Note: Under a "No Slippage" condition, the \( \sigma_A \) stress does not vary by tank length.

\[ L_{\text{in}} = 328.5 \text{ in} \quad 2 \cdot \text{ln} = (235) \text{ in} \quad \sigma_A = (1565) \text{ psi} \]

Seismic Stress due to Dynamic Soil Pressure

\[ \text{K}_C := 4.0 \quad \text{K}_S := 5.0 \quad \text{Dynamic stress concentration factors, see Newmark & Rosenblueth, Fundamentals of Earthquake Engineering.} \]

\[ \sigma_{0c} := K_c \cdot \rho \cdot C_P \cdot V_p \quad \sigma_{0c} = (624) \text{ psi} \quad \text{Maximum hoop stress induced by normal stress} \]

\[ \sigma_{0s} := K_s \cdot \rho \cdot C_s \cdot V_s \quad \sigma_{0s} = (319) \text{ psi} \quad \text{Maximum hoop stress induced by shear stress} \]
1) Axial Stress Due to P Waves and S Waves

Summary of Axial Stresses vs. Shear Modulus ($G_m$)

<table>
<thead>
<tr>
<th>$G$ (psi)</th>
<th>1625*</th>
<th>5000</th>
<th>30000</th>
<th>1625**</th>
<th>5000**</th>
<th>30000**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burial Depth</td>
<td>5 ft.</td>
<td>5 ft.</td>
<td>5 ft.</td>
<td>5 ft.</td>
<td>5 ft.</td>
<td>5 ft.</td>
</tr>
<tr>
<td>$a_p$</td>
<td>57.1% g</td>
<td>57.1% g</td>
<td>57.1% g</td>
<td>57.1% g</td>
<td>57.1% g</td>
<td>57.1% g</td>
</tr>
<tr>
<td>$V_p$</td>
<td>48 in/s</td>
<td>48 in/s</td>
<td>48 in/s</td>
<td>9.8 in/s</td>
<td>9.8 in/s</td>
<td>9.8 in/s</td>
</tr>
<tr>
<td>$L_{ss}$</td>
<td>27.37 ft.</td>
<td>27.37 ft.</td>
<td>27.37 ft.</td>
<td>27.37 ft.</td>
<td>27.37 ft.</td>
<td>27.37 ft.</td>
</tr>
<tr>
<td>$2lm$</td>
<td>86.5 ft.</td>
<td>48.4 ft.</td>
<td>19.6 ft.</td>
<td>25.5 ft.</td>
<td>11.7 ft.</td>
<td>4.2 ft.</td>
</tr>
<tr>
<td>$\sigma_{ac}$</td>
<td>3018</td>
<td>1721</td>
<td>702</td>
<td>619</td>
<td>353</td>
<td>144</td>
</tr>
<tr>
<td>$\sigma_{bc}$</td>
<td>76</td>
<td>25</td>
<td>46</td>
<td>76</td>
<td>25</td>
<td>4</td>
</tr>
<tr>
<td>$\sigma_{as}$</td>
<td>3696</td>
<td>2107</td>
<td>860</td>
<td>758</td>
<td>432</td>
<td>176</td>
</tr>
<tr>
<td>$\sigma_{bs}$</td>
<td>1187</td>
<td>386</td>
<td>64</td>
<td>1187</td>
<td>386</td>
<td>64</td>
</tr>
<tr>
<td>$\sigma_A'$</td>
<td>6901</td>
<td>3863</td>
<td>1565</td>
<td>2032</td>
<td>933</td>
<td>332</td>
</tr>
<tr>
<td>$\sigma_{A \text{ noslip}}$</td>
<td>2185</td>
<td>2185</td>
<td>2185</td>
<td>2185</td>
<td>2185</td>
<td>2185</td>
</tr>
<tr>
<td>Condition</td>
<td>Slip</td>
<td>Slip</td>
<td>No Slip</td>
<td>No Slip</td>
<td>No Slip</td>
<td>No Slip</td>
</tr>
<tr>
<td>$\sigma_{A \text{ control}}$</td>
<td>2185</td>
<td>2185</td>
<td>1565</td>
<td>2032</td>
<td>933</td>
<td>332</td>
</tr>
<tr>
<td>$\sigma_{\theta c}$</td>
<td>.145</td>
<td>255</td>
<td>624</td>
<td>30</td>
<td>52</td>
<td>124</td>
</tr>
<tr>
<td>$\sigma_{\theta s}$</td>
<td>74</td>
<td>130</td>
<td>319</td>
<td>74</td>
<td>130</td>
<td>319</td>
</tr>
<tr>
<td>$\sigma_{\theta \text{comb}}$</td>
<td>114</td>
<td>200</td>
<td>491</td>
<td>80</td>
<td>140</td>
<td>343</td>
</tr>
</tbody>
</table>

- None of the references for this method discussed shear modulus reduction. This effect is shown.
- Axial stresses are higher with lower shear modulus.
- Hoop stresses are lower with lower shear modulus.

*For comparison to Xerxes method (slide 110)
**Tripartite plot for Northridge Earthquake used for velocity

www.clark-engineers.com • 936.273.6200
0) Historical background, some seismic information, shear modulus, and seismic spectra
1) Axial stress due to P waves and S waves
2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts
3) Xerxes (20) patent (reduced shear modulus) with transverse loads on FRP UST’s
4) Sloshing
5) Liquefaction
6) Buckling of soil surrounded tubes
2) Wang/NCHRP Method

Ovaling/Racking Method

1993  Jaw-Nan Wang, with Parsons Brinckerhoff Quade & Douglas, Inc. Published (23)

“Seismic Design of Tunnels” Monograph 7

www.clark-engineers.com  •  936.273.6200
Ovaling/Racking Method

2008 This method was updated and published in the NCHRP Report 611 (4)

“Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments” – Chapter 9 Buried Structures

Can be downloaded at http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_611.pdf
Discussion for:

- Circular Conduits and Tanks (Ovaling)
- Box culverts (Racking)
Chapter 9 of Report 611 documents FEA and finite difference studies done on a wide range of soil/structure stiffness ratios and provides “closed form” solutions based on these studies with comparisons to computer results.

Wang found that transverse stress were most important for softer soils with caveat that longitudinal stresses can occur with “stiff backfill” e.g. pea gravel or crushed stone as is used in FRP UST’s. It is now recognized that confining pressure can decrease with increasing dynamic strain. This is addressed farther on herein.

The following tables show the range of studies done by Wang.
Table 9-1. Parameters used in the parametric analysis.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Descriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure Types</td>
<td><strong>FLEXIBLE CULVERTS:</strong></td>
</tr>
<tr>
<td></td>
<td>- Corrugated Aluminum Pipe</td>
</tr>
<tr>
<td></td>
<td>- Corrugated Steel Pipe</td>
</tr>
<tr>
<td></td>
<td>- Corrugated HDPE Pipe</td>
</tr>
<tr>
<td></td>
<td><strong>RIGID CULVERTS:</strong></td>
</tr>
<tr>
<td></td>
<td>- Reinforced Concrete Pipe</td>
</tr>
<tr>
<td></td>
<td>- Reinforced Concrete Box Type</td>
</tr>
<tr>
<td>Burial Depths</td>
<td>5d, 3d, 2d, 1d, 0.5d, (<em>d</em> represents the diameter of a circular pipe or the height of a box concrete culvert)</td>
</tr>
<tr>
<td>Cross Section Geometry Types</td>
<td>- Circular</td>
</tr>
<tr>
<td></td>
<td>- Square Box</td>
</tr>
<tr>
<td></td>
<td>- Rectangular Box</td>
</tr>
<tr>
<td></td>
<td>- Square 3-sided</td>
</tr>
<tr>
<td></td>
<td>- Rectangular 3-sided</td>
</tr>
<tr>
<td>Diameters of Circular Culverts</td>
<td>- 5 feet (Medium Diameter)</td>
</tr>
<tr>
<td></td>
<td>- 10 feet (Large Diameter)</td>
</tr>
<tr>
<td>Wall Stiffness of Circular Culverts</td>
<td><strong>FLEXIBLE CULVERTS:</strong></td>
</tr>
<tr>
<td></td>
<td>- I=0.00007256 ft^4/ft, E= 2.9E+07 psi (Steel)</td>
</tr>
<tr>
<td></td>
<td>- I=0.00001168 ft^4/ft, E= 1.0E+07 psi (Aluminum)</td>
</tr>
<tr>
<td></td>
<td>- I=0.0005787 ft^4/ft, E= 1.1E+05 psi (HDPE)</td>
</tr>
<tr>
<td>Size Dimensions of Box Culverts</td>
<td><strong>RIGID CULVERTS:</strong></td>
</tr>
<tr>
<td></td>
<td>- 10 feet x 10 feet: Square Box and Square 3-sided</td>
</tr>
<tr>
<td></td>
<td>- 10 feet x 20 feet: Rectangular Box and Rectangular 3-sided</td>
</tr>
<tr>
<td>Wall Stiffness of Box Culverts</td>
<td><strong>RIGID CULVERTS:</strong></td>
</tr>
<tr>
<td></td>
<td>- I=0.025 ft^4/ft, t=0.67 ft, E= 4.0E+06 psi (Concrete)</td>
</tr>
<tr>
<td></td>
<td>- I=0.2 ft^4/ft, t=1.33 ft, E= 4.0E+06 psi (Concrete)</td>
</tr>
<tr>
<td>Properties of Surrounding Ground*</td>
<td><strong>FLEXIBLE CULVERTS:</strong></td>
</tr>
<tr>
<td></td>
<td>- E=3,000 psi (Firm Ground)</td>
</tr>
<tr>
<td></td>
<td>- E=7,500 psi (Very Stiff Ground)</td>
</tr>
<tr>
<td></td>
<td>- Total Unit Weight = 120 psf</td>
</tr>
</tbody>
</table>

*Note: The Young's Modulus values used in this study are for parametric analysis purposes only.*
2) Wang/NCHRP Method

Table 9-2. Parametric Analysis Set 1—culvert lining properties (Reference Set).

<table>
<thead>
<tr>
<th>Culvert Properties</th>
<th>Rigid Culvert (Concrete Pipe)</th>
<th>Flexible Culvert (Corrugated Steel Pipe)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert Diameter, ft</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Young’s Modulus, $E/(1-v^2)$, used in 2-D Plane Strain Condition, psi</td>
<td>4.0E+06</td>
<td>2.9E+07</td>
</tr>
<tr>
<td>Moment of Inertia I, $ft^4/ft$</td>
<td>0.025 $ft^4/ft$</td>
<td>0.00007256 $ft^4/ft$ ($=1.505 \text{ in}^4/\text{ft}$)</td>
</tr>
<tr>
<td>Sectional Area $A$, $ft^2$ per ft</td>
<td>0.67</td>
<td>0.02</td>
</tr>
<tr>
<td>$EI$ (lb-$ft^2$ per ft)</td>
<td>1.44E+07</td>
<td>3.03E+05</td>
</tr>
<tr>
<td>$AE$ (lb per ft)</td>
<td>3.86E+08</td>
<td>8.35E+07</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.3</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Note: Ground condition (firm ground with $E_m = 3000$ psi, $\nu_m = 0.3$).

Table 9-3. Analyses performed for variable embedment depths.

<table>
<thead>
<tr>
<th>Cases Analyzed</th>
<th>Soil Cover $H$ (feet)</th>
<th>Culvert Diameter $d$ (feet)</th>
<th>Embedment Depth Ratio, $H/d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>50</td>
<td>10</td>
<td>5</td>
</tr>
<tr>
<td>Case 2</td>
<td>30</td>
<td>10</td>
<td>3</td>
</tr>
<tr>
<td>Case 3</td>
<td>20</td>
<td>10</td>
<td>2</td>
</tr>
<tr>
<td>Case 4</td>
<td>10</td>
<td>10</td>
<td>1</td>
</tr>
<tr>
<td>Case 5</td>
<td>5</td>
<td>10</td>
<td>0.5</td>
</tr>
<tr>
<td>Case 6</td>
<td>2</td>
<td>10</td>
<td>0.2</td>
</tr>
</tbody>
</table>
2) Wang/NCHRP Method

Table 9-4. Culvert lining compressibility and flexibility used in analysis.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Rigid Culvert (Concrete Pipe)</th>
<th>Flexible Culvert (Corrugated Steel Pipe)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressibility Ratio, $C$</td>
<td>0.011</td>
<td>0.05</td>
</tr>
<tr>
<td>Flexibility Ratio, $F$</td>
<td>0.482</td>
<td>22.6</td>
</tr>
</tbody>
</table>

Table 9-5. Free-field ground strain and diameter change.

<table>
<thead>
<tr>
<th>Case No. (Embedment Ratio)</th>
<th>Free-Field Maximum Ground Shear Strain (from FLAC Analysis) $\gamma_{max}$</th>
<th>Closed-Form Free-Field Ground Diameter Change Using Eq. 9-3 $\Delta D = 0.5 D^* \gamma_{max}$ (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1 (H/d=5)</td>
<td>0.0129</td>
<td>0.065</td>
</tr>
<tr>
<td>Case 2 (H/d=3)</td>
<td>0.0085</td>
<td>0.043</td>
</tr>
<tr>
<td>Case 3 (H/d=2)</td>
<td>0.0064</td>
<td>0.032</td>
</tr>
<tr>
<td>Case 4 (H/d=1)</td>
<td>0.004</td>
<td>0.02</td>
</tr>
<tr>
<td>Case 5 (H/d=0.5)</td>
<td>0.003</td>
<td>0.015</td>
</tr>
<tr>
<td>Case 6 (H/d=0.2)</td>
<td>0.0022</td>
<td>0.011</td>
</tr>
</tbody>
</table>

Note: The maximum free-field ground shearing strain is the maximum shearing strain that could occur within the full depth of the culvert (that is, from the crown to the invert). In the pseudo-static FLAC analysis, the maximum ground shearing strains occur at the invert in all cases.

[CLARK ENGINEERS, Inc]
Table 9-6. Culvert diameter change—effect of interface slippage condition.

<table>
<thead>
<tr>
<th>Case No. (Embedment Ratio)</th>
<th>Culvert Diameter Change (ft) for Full-Slip Interface Using Eq. 9-7</th>
<th>Culvert Diameter Change (ft) for No-Slip Interface Using FLAC Analysis</th>
<th>Diameter Change Ratio for No-Slip to Full-Slip</th>
</tr>
</thead>
<tbody>
<tr>
<td>For Flexible Culvert</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 1 (H/d=5)</td>
<td>0.169</td>
<td>0.129</td>
<td>0.77</td>
</tr>
<tr>
<td>Case 2 (H/d=3)</td>
<td>0.111</td>
<td>0.082</td>
<td>0.74</td>
</tr>
<tr>
<td>Case 3 (H/d=2)</td>
<td>0.084</td>
<td>0.059</td>
<td>0.70</td>
</tr>
<tr>
<td>Case 4 (H/d=1)</td>
<td>0.052</td>
<td>0.036</td>
<td>0.68</td>
</tr>
<tr>
<td>Case 5 (H/d=0.5)</td>
<td>0.039</td>
<td>0.024</td>
<td>0.62</td>
</tr>
<tr>
<td>Case 6 (H/d=0.2)</td>
<td>0.029</td>
<td>0.018</td>
<td>0.62</td>
</tr>
<tr>
<td>For Rigid Culvert</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case 1 (H/d=5)</td>
<td>0.042</td>
<td>0.034</td>
<td>0.80</td>
</tr>
<tr>
<td>Case 2 (H/d=3)</td>
<td>0.028</td>
<td>0.021</td>
<td>0.77</td>
</tr>
<tr>
<td>Case 3 (H/d=2)</td>
<td>0.021</td>
<td>0.015</td>
<td>0.72</td>
</tr>
<tr>
<td>Case 4 (H/d=1)</td>
<td>0.013</td>
<td>0.009</td>
<td>0.67</td>
</tr>
<tr>
<td>Case 5 (H/d=0.5)</td>
<td>0.010</td>
<td>0.006</td>
<td>0.57</td>
</tr>
<tr>
<td>Case 6 (H/d=0.2)</td>
<td>0.007</td>
<td>0.004</td>
<td>0.51</td>
</tr>
</tbody>
</table>
2) Wang/NCHRP Method

Table 9-8. Parametric analysis set 3—culvert lining properties.

<table>
<thead>
<tr>
<th>Culvert Properties</th>
<th>Rigid Culvert (Concrete Pipe)</th>
<th>Flexible Culvert Aluminum CMP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert Diameter, ft</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Young's Modulus, E/(1-ν²), psi</td>
<td>4.0E+06</td>
<td>1.0E+07</td>
</tr>
<tr>
<td>Moment of Inertia, ft⁴/ft</td>
<td>0.2</td>
<td>0.00001168</td>
</tr>
<tr>
<td>Sectional Area, ft² per ft</td>
<td>1.333</td>
<td>0.01125</td>
</tr>
<tr>
<td>EI (lb·ft² per ft)</td>
<td>1.152E+08</td>
<td>1.682E+04</td>
</tr>
<tr>
<td>AE (lb, per ft)</td>
<td>7.678E+08</td>
<td>1.62E+07</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Compressibility, C</td>
<td>0.005</td>
<td>0.256</td>
</tr>
<tr>
<td>Flexibility Ratio, F</td>
<td>0.060</td>
<td>411.7</td>
</tr>
</tbody>
</table>

Note: Ground condition (firm ground with Eₘ = 3,000 psi, νₘ = 0.3).

Table 9-7. Parametric analysis set 2—culvert lining properties.

<table>
<thead>
<tr>
<th>Culvert Properties</th>
<th>Rigid Culvert (Concrete Pipe)</th>
<th>Flexible Culvert (Corrugated Steel Pipe)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert Diameter, ft</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Young's Modulus, E/(1-ν²), psi</td>
<td>4.0E+06</td>
<td>2.9E+07</td>
</tr>
<tr>
<td>Moment of Inertia, ft⁴/ft</td>
<td>0.025</td>
<td>0.00007256</td>
</tr>
<tr>
<td>Sectional Area, ft² per ft</td>
<td>0.67</td>
<td>0.02</td>
</tr>
<tr>
<td>EI (lb·ft² per ft)</td>
<td>1.44E+07</td>
<td>3.03E+05</td>
</tr>
<tr>
<td>AE (lb, per ft)</td>
<td>3.86E+08</td>
<td>8.35E+07</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Compressibility, C</td>
<td>0.005</td>
<td>0.025</td>
</tr>
<tr>
<td>Flexibility Ratio, F</td>
<td>0.061</td>
<td>2.856</td>
</tr>
</tbody>
</table>

Note: Ground condition (firm ground with Eₘ = 3000 psi, νₘ = 0.3).
Table 9-9. Parametric analysis set 4—culvert lining properties.

<table>
<thead>
<tr>
<th>Culvert Properties</th>
<th>Flexible Culvert (Corrugated HDPE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert Diameter, ft</td>
<td>5</td>
</tr>
<tr>
<td>Young’s Modulus, E/(1-ν²), psi</td>
<td>1.1E+05</td>
</tr>
<tr>
<td>Moment of Inertia, ft² per ft</td>
<td>0.0005787</td>
</tr>
<tr>
<td>Sectional Area, ft² per ft</td>
<td>0.0448</td>
</tr>
<tr>
<td>EI (lb-ft² per ft)</td>
<td>9.17E+03</td>
</tr>
<tr>
<td>AE (lb, per ft)</td>
<td>7.10E+05</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.45</td>
</tr>
<tr>
<td>Compressibility, C</td>
<td>2.927</td>
</tr>
<tr>
<td>Flexibility Ratio, F</td>
<td>94.424</td>
</tr>
</tbody>
</table>

Note: Ground condition (firm ground with \(E_m = 3,000\) psi, \(\nu_m = 0.3\)).

Table 9-10. Parametric analysis set 5—very stiff ground condition.

<table>
<thead>
<tr>
<th>Culvert Properties</th>
<th>Rigid Culvert (Concrete Pipe)</th>
<th>Flexible Culvert (Corrugated Steel Pipe)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Culvert Diameter, ft</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Young’s Modulus, E/(1-ν²), psi</td>
<td>4.0E+06</td>
<td>2.9E+07</td>
</tr>
<tr>
<td>Moment of Inertia, ft²/ft</td>
<td>0.025</td>
<td>0.00007256</td>
</tr>
<tr>
<td>Sectional Area, ft² per ft</td>
<td>0.67</td>
<td>0.02</td>
</tr>
<tr>
<td>EI (lb-ft² per ft)</td>
<td>1.44E+07</td>
<td>3.03E+05</td>
</tr>
<tr>
<td>AE (lb, per ft)</td>
<td>3.86E+08</td>
<td>8.35E+07</td>
</tr>
<tr>
<td>Poisson’s Ratio</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Compressibility, C</td>
<td>0.027</td>
<td>0.127</td>
</tr>
<tr>
<td>Flexibility Ratio, F</td>
<td>1.217</td>
<td>57.122</td>
</tr>
</tbody>
</table>

Note: ground condition (very stiff ground with \(E_m = 7,500\) psi, \(\nu_m = 0.3\)).
Table 9-11. Soil and structure parameters used in the analysis.

<table>
<thead>
<tr>
<th>Structural Configurations and Soil Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
</tr>
<tr>
<td>Case 2</td>
</tr>
<tr>
<td>Case 3</td>
</tr>
<tr>
<td>Case 4</td>
</tr>
<tr>
<td>Case 5</td>
</tr>
</tbody>
</table>

Note: For each case, the effects of culvert embedment depth (of 50 feet, 30 feet, 20 feet, 10 feet, and 5 feet, measured from ground surface to top of the culvert roof) were studied.

Table 9-12. Racking stiffness of culverts and flexibility ratios.

<table>
<thead>
<tr>
<th></th>
<th>Structural Racking Stiffness $K_s$ (kips/ft)</th>
<th>Flexibility Ratio $F_{REC}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>172</td>
<td>0.97</td>
</tr>
<tr>
<td>Case 2</td>
<td>172</td>
<td>2.4</td>
</tr>
<tr>
<td>Case 3</td>
<td>115</td>
<td>2.9</td>
</tr>
<tr>
<td>Case 4</td>
<td>57</td>
<td>7.3</td>
</tr>
<tr>
<td>Case 5</td>
<td>43</td>
<td>19.3</td>
</tr>
</tbody>
</table>
Embedment Depth Ratio (H/d)

H/d is used in studies by Wang.

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Most FRP and Steel UST’s for gasoline storage have H/d ratio from $0.26 \leq H/d \leq 1.75$

<table>
<thead>
<tr>
<th>Dia* (ft)</th>
<th>H min (ft) (UL Listing)**</th>
<th>H max (ft) (UL Listing)</th>
<th>H/d (min)</th>
<th>H/d (max)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>3</td>
<td>7</td>
<td>0.75</td>
<td>1.75</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>7</td>
<td>0.50</td>
<td>1.16</td>
</tr>
<tr>
<td>8 (92in)</td>
<td>3</td>
<td>7</td>
<td>0.38</td>
<td>1.13</td>
</tr>
<tr>
<td>10</td>
<td>3</td>
<td>7</td>
<td>0.30</td>
<td>0.70</td>
</tr>
<tr>
<td>12 (138in)</td>
<td>3</td>
<td>7</td>
<td>0.26</td>
<td>0.58</td>
</tr>
</tbody>
</table>

*Typical FRP diameters

**All buried tanks used for storage of gasoline and fuel oil must be listed by Underwriters’ Laboratories (UL) or other recognized third party testing laboratory e.g. Factory Mutual (FM).
Wang shows that the results are almost unchanged for $H/d > 1.0$ and for $H/d < 1.0$ results change gradually.

New method accounts for hoop thrust stress and diametrical bending stress for circular tanks and for racking for rectangular culverts.
Flexible Culverts (9.2.1)*

- Rely on firm soil support
- Depend on large strain capacity to hold shape
  - Flexible pipe is simply a liner for a hole in the soil – ie pipe goes along for the ride

*Numbers in parenthesis refer to appropriate section in Report 611

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Flexible Tanks and Pipes (9.2.1)

- Design considerations
  - Buckling
  - Flexibility limit
    - Moment capacity (generally not an issue)
    - Use pipe stiffness ($PS$) to compute flexibility

$$PS = \frac{P}{\Delta w} = \frac{EI}{0.149R^3w} \text{ (psi/in)}$$
where $w =$ width of section tested
$R =$ radius

- $PS$ normalizes stiffness for radius

Typical range of pipe stiffness is ~10 to 20 psi ± for flexible FRP tanks
Two Main Factors

1) Bending moment and hoop thrust evaluation
   • Bending demand can be high

2) Soil support is critical for flexible pipe
   • Can be lost due to *liquefaction* or other permanent ground failure mechanisms (see discussion starting on slide 121)
General Effects of Earthquakes and Potential “New” Failure Models

Ground Shaking (9.3.1) (See slides 8 and 9 for videos of wave types)

- Two different types of waves with two sub types
- Body Waves – within Earth’s crust
  - Longitudinal compressional: (P) Push waves
  - Transverse and shear: (S) Shake waves
- Travel in any direction
General Effects of Earthquakes and Potential “New” Failure Models

- Surface Waves – along Earth’s surface
  - Rayleigh waves cause the ground to shake in an elliptical motion, with no transverse motion. Ref. earthquake.usgs.gov
  - Love waves have a horizontal motion that is transverse to the direction the wave is traveling. Ref. earthquake.usgs.gov

- Unified evaluation procedure is developed for seismic evaluations and realistic design for buried culvert and pipe structures.

- Wang/NCHRP does not include effect of reduced modulus based on shear strain (it is included in studies).
Rigid Culverts and Pipes

- Strain capacity much lower
- Not as dependent on soil support as flexible culverts
- Must apply soil pressure, active pressure, surcharge pressure, etc. to obtain total stress condition
Principle Types of Transient Ground Deformations (TDA):

- Axial
- Curvature
- Ovaling or Racking

Some Terminology

- PDA: Peak Ground Deformation
- TDA: Transient Ground Deformation

[Diagrams showing ovaling and racking]

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Focus of Chapter 9 (NCHRP 611)

- Transverse deformations (9.3.1)
- Limited length structures generally do not develop significant axial curvature (beam bending) unless embedded in **stiff, strong soil** as is the case for flexible UST’s but with effect of increasing strain **now** recognized.
- Ovaling or racking develops when waves propagate perpendicular to the longitudinal axis

![Pipe Diagram](image-url)
Vertically propagating shear wave is predominant form of earthquake loading governing ovaling/racking

1) Horizontal component is most severe except for very near source

2) Vertical ground strains are generally much smaller than shear strain because shear modulus is lower than constrained modulus

3) Amplification of vertically propagating shear wave is much higher in soft weak soil

Evaluated using under two-dimensional plain strain condition per K. Ishihara
Ground Failure Modes (Ground Instability)

- Faulting
- Landslides
- Liquefaction (*more on slide 121*)
  - Induced lateral spread
  - Settlement
  - Floatation, etc.
- Tectonic uplift and subsidence
- Can cause permanent deformations
Permanent Deformation

• Can be catastrophic to a culvert or pipeline
• Usually localized
• Typically requires ground improvement

Therefore:
Avoid possible ground failure situations or provide an easy means for repair if unavoidable.
General Methodology

Recommended procedures for ovaling and racking analysis and design.

Ovaling

- Change in diameter \( \Delta_D = \frac{\Delta}{D} \)
- Buckling is key failure mode for flexible conduits
- For rigid conduits, thrust and moment are important
Determine Seismic Demands

**Step 0:** Determine seismic demands from *actual site data* or from appropriate method eg. IBC, ASCE-7 (same as before, use actual site data or determine from code)

Determine mapped acceleration $S_S$ and $S_L$ from USGS seismic hazard curves and ASCE 7 (see slides 24-30)

\[ V_{Tank} = 0.3 \cdot S_{DS} \cdot I_e \cdot 1. lbf \]  
(ASCE 7-10† 15.4-5)

Ref 11.4.4 ASCE 7-10 p. 65 (1)

Importance factor table 1.5-2 p.

5 Category IV: Substantial Hazard to Community

†A reference for buried structures was not found in ASCE 7-10.
Determine Seismic Demands

b) Seismic acceleration per unit force

\[ a_P = \frac{V_{Tank}}{M} \]

where \( M = \) mass for 1lbf

\[ M = \frac{1\text{ lbf}}{g} \]

c) \( V_P = a_P \cdot \frac{48 \text{ in/sec}}{g} \)  
   (Ref Earthquake Spectra: Northridge Earthquake of January 17, 1994
   Reconnaissance report-Vol 1 (15) or use site specific values if available
   or use response spectra)

OR

\[ V_P = a_P \cdot \frac{25 \text{ cm/s}}{g} \]  
   From Northridge Response Spectra (see slides 18 and 19)
Determine Seismic Demands \((9.4)\)

**Step 1:** Maximum Free Field Strain

\[
\gamma_{\text{max}} = \frac{V_s}{C_{SE}}
\]

where \(C_{SE}\) = effective shear wave velocity

Shear Wave Velocity

\[
V_s = \frac{V_p}{\sqrt{\frac{1 - 2\nu_m}{2(1 - \nu_m)}}}
\]

where \(V_p\) = velocity from compression wave

\(\nu_m\) = Poisson’s ratio for surrounding soil
Determine Seismic Demands

Step 1 continued:

Alternate

\[ \gamma_{max} = \frac{\tau_{max}}{G_m} \]

\[ \tau_{max} = \left( \frac{PGA}{g} \right) \sigma_v R_d \]

\[ \sigma_v = \gamma_t \cdot (H_{TOP} + d) \]

\[ R_d = 1.0 - 0.00233 \cdot z \quad z < 30 \text{ ft.} \]

\[ G_m = \text{effective shear modulus} \]

or use “SHAKE” program analysis

\[ G_m = G_o \] for these calculations

(Reduce to include strain reduction per Ishihara (11))
Determine Seismic Demands

Step 2: Maximum free field diameter change

\[ \Delta_{D_{EQ-FF}} = 0.5 \gamma_{max} D \]

If hole cavity in soil is considered (yes for this case)

\[ \Delta_{D_{EQ}} = \pm 2 \gamma_{max} (1 - \nu_m) D \]

\[ \nu_m = \text{Poisson’s ratio for surrounding soil} \]

\[ D = \text{diameter} \]

Good for flexible conduits in competent ground.
Alternate $\gamma_{max}$ based on FEA studies by Wang (1993 and NCHRP Report 611 2008)

Basis: define relative stiffness between circular lining and surrounding ground.

Compressibility Ratio $C$

$$C = \frac{E_m(1-\nu_m^2)R_T}{E_A \cdot t_{eff} (1+\nu)(1-2\nu)} = \left(\frac{R_T}{t_{eff}}\right) \left(\frac{E_m}{E_A}\right) \left(\frac{1-\nu_m^2}{(1+\nu)(1-2\nu)}\right)$$

Geometry Stiffness Ratio for Tank/Soil

(Definitions on next slide)
Alternate $\gamma_{max}$ based on FEA studies by Wang (1993 and NCHRP Report 611 2008)

Flexibility Ratio $F$

$$F = \frac{E_m(1-\nu_m^2)R_T^3}{6EI (1+\nu)} = \left(\frac{2R_T^3}{t_{eff}^3}\right) \left(\frac{E_m}{E}\right) \left(\frac{1-\nu_m^2}{(1+\nu)}\right)$$

where $EI =$ flexural rigidity of pipe/tank

$\nu =$ Poisson’s ratio of pipe/tank

$E_m:$ strain compatible elastic modulus of surrounding soil (soil report or estimate from available literature)

e.g. K. Ishihara, etc. (11)

$\nu_m =$ Poisson’s ratio of surrounding soil

$R_T =$ tank or pipe radius

Rigid Ring $F < 1$

Flexible Ring $F > 1$

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**Full Slip Condition** (Occurs only in soft soils or very high seismic loading - 9.5.1 p. 111)

Change in Diameter

\[ \Delta D_{EQ} = \pm \frac{1}{3} (k_1 F_{\gamma max} D) \]

where \( k_1 = 12 (1 - v_m) \cdot (2F + 5 - 6v_m) \)

Max hoop thrust

\[ T_{max} = \frac{1}{6} k_1 \left[ \frac{E_m}{(1 - v_m)} \right] R \cdot \gamma_{max} \]

Max Hoop Bending

\[ M_{max} = \frac{1}{6} k_1 \left[ \frac{E_m}{(1 - v_m)} \right] R \cdot \gamma_{max} \]

\[ = R \cdot T_{max} \]
No Slip Condition

\[ T_{max} = k_2 \left[ \frac{E_m}{2 \cdot (1-2\nu_m)} \right] R\gamma_{max} \]

where:

\[ k_2 = 1 + \frac{F_T[(1-2\nu_m)-(1-2\nu_m)C_T]-\frac{1}{2}(1-2\nu_m)^2+2}{F_T[(3-2\nu_m)+(1-2\nu)C_T]} + \ldots \]

\[ C_T[ -8\nu_m + 6\nu_m^2 + 6 - 8\nu_m ] \]
“In most cases the condition at the interface is between slip and no slip,” p. 111. According to Fahimifar and Vakilzadeh in *Numerical and Analytical Solutions for Ovaling Deformation in Circular Tunnels Under Seismic Loading* (8),

“Note that no solution is developed for calculating diametric strain and maximum moment under no-slip condition. It is recommended that the solutions for full slip condition be used for no-slip condition. The more conservative estimates of the full-slip condition is considered to offset the potential underestimation due to pseudo-static representation of the dynamic problem [1].”

Therefore use full slip method.
## Summary for Horizontal Seismic Stresses per Wang/NCHRP Method for Hoop Load

<table>
<thead>
<tr>
<th></th>
<th>$G$ 1625**</th>
<th>5000</th>
<th>10000</th>
<th>30000</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$</td>
<td>4549</td>
<td>14000</td>
<td>28000</td>
<td>84000*</td>
</tr>
<tr>
<td>$\gamma%$</td>
<td>.913%</td>
<td>.52 in/m</td>
<td>0.368%</td>
<td>.212%</td>
</tr>
<tr>
<td>$\Delta_D$</td>
<td>$\sim 1\frac{5}{16}$ in</td>
<td>$\sim \frac{3}{4}$ in</td>
<td>$\sim \frac{1}{2}$ in (+)</td>
<td>$\sim \frac{5}{16}$ in</td>
</tr>
<tr>
<td>$\sigma_T$</td>
<td>4</td>
<td>2</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>$\sigma_m$</td>
<td>623</td>
<td>356</td>
<td>252</td>
<td>145</td>
</tr>
<tr>
<td>$\sigma_{Total}$</td>
<td>627</td>
<td>358</td>
<td>254</td>
<td>146</td>
</tr>
</tbody>
</table>

All stresses and moduli values are in psi.

*E = 2(1+ν)∙G. Max value ~29000 per Bowles (6)

ν = 0.4

**Xerxes Patent value corresponds with E = 4549 psi
Racking for Rectangular Conduits (9.5.2)

Racking: differential sideways movements between top and bottom.

Results in differential inertial strain.

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Step 1: Estimate $\gamma_{max}$ - free field ground strain

$$\gamma_{max} = \frac{V_s}{C_{SE}}$$

where $C_{SE} =$ effective shear wave velocity

$$V_s = \frac{V_P}{\sqrt{(1-2\nu_m) / 2(1-\nu_m)}}$$
shear wave velocity
Method for Rectangular Structure

OR

\[ \gamma_{max} = \frac{T_{max}}{G_m} \]

\[ \gamma_{max} = \left( \frac{PGA}{g} \right) \sigma_v R_d \]

PGA = peak ground acceleration (\(a_p\) - defined previously by code – slide 18)

where \(\sigma_v = \gamma_m \cdot (H_{TOP} + d)\)

\[ R_d = 1.0 - 0.00233 \cdot z \quad z<30 \text{ ft.}^* \]

\(G_m = \) effective shear modulus

Determine differential free-field relative displacements \(\Delta_{free-field}\) at top and bottom elevations.

\[ \Delta_{Free-field} = H \cdot \gamma_{max} \]

\[ H = H_{Top} \quad \text{and} \quad H_{Bottom} \]

*For design > 30 ft. refer to Report 611 (4)

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Method for Rectangular Structure

**Step 2:** Determine racking stiffness from simple frame analysis by applying a unit load at the top and determine unit moment thrust and shear at each point of design interest.

\[ k_{rec} = \frac{1 \text{kip}}{\Delta} \]

Force applied at top left corner.
Method for Rectangular Structure

Step 3: Flexibility Ratio

\[
F_{REC} = \left( \frac{G_m}{k_{REC}} \right) \cdot \frac{L}{H}
\]

\( L = \) width of structure

\( k_{REC} = \) racking stiffness \( = \frac{P_{\text{1kip}}}{\Delta_{TOP}} \left( \frac{\text{kip}}{\text{in}} \right) \)

Step 4: Racking Ratio

\[
R_{REC} = \frac{2F_{REC}}{1 + F_{REC}}
\]

Ratio of actual racking to free-field racking.
Method for Rectangular Structure

\[ F = 1 \]  Ground and structure have same distortion

\[ F \to 0 \]  Perfectly rigid structure-no racking regardless of ground free-field distortion

\[ F > 1 \]  Flexible structure and distortion is magnified compared to free-field distortion

Figure 9-9. Racking ratio between structure and free-field.
Method for Rectangular Structure

**Step 5:** Racking deformation

$$\Delta R = R_{REC} \cdot \Delta_{Free-field}$$

**Step 6:** Compute seismic demands in terms of internal forces

- $M = \text{moment}$
- $T = \text{thrust}$
- $V = \text{shear}$

By imposing $\Delta R$ on the structure.

$$M = M_{\text{unit load}} \cdot \frac{\Delta R}{\Delta_{\text{unit load}}}$$
$$T = T_{\text{unit load}} \cdot \frac{\Delta R}{\Delta_{\text{unit load}}}$$
$$V = V_{\text{unit load}} \cdot \frac{\Delta R}{\Delta_{\text{unit load}}}$$
Seismic effects must be added to other load cases to obtain total stress normal load effects

Note that Wang/NCHRP method does not discuss sloshing. This must be included in the analysis.
FEA Models
Some of the FEA models used by Wang are provided herein.

2) Wang/NCHRP Method

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2) Wang/NCHRP Method

FEA Model

Figure 9-13. Case 3 finite difference mesh (soil cover = 20 feet).

Figure 9-14. Case 4 finite difference mesh (soil cover = 10 feet).

Figure 9-15. Case 5 finite difference mesh (soil cover = 5 feet).

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2) Wang/NCHRP Method

FEA Model

Figure 9-17. Culvert beam element number.

Figure 9-18. Soil deformations subjected to pseudo lateral acceleration of 0.3g.

Figure 9-19. Culvert lining thrust force distribution (for flexible culvert in Set 1, Case 1 geometry).

Figure 9-20. Culvert lining bending moment distribution (for flexible culvert in Set 1, Case 1 geometry).

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2) Wang/NCHRP Method

FEA Model

Figure 9-37. Various concrete box culvert sectional shapes and sizes used in the parametric analysis—Set 6.

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Design Example (Tank/Pipe)

Design Example Wang / RCHRP Method

Maximum Ovaling

\[ \Delta D = \frac{1}{3} K_{1T} F T_{\gamma \max} \delta_{\text{Ovaling}} \]

\[ \Delta D = (0.75) \text{ in} \]

Modulus Range (E_m)

Ref Bowles 4th ed pg 125

<table>
<thead>
<tr>
<th>Material</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose Gravel</td>
<td>7250 to 21700 psi</td>
</tr>
<tr>
<td>Dense Gravel</td>
<td>14500 to 29000 psi</td>
</tr>
</tbody>
</table>

Axial Stress

\[ T_{\text{max}} = \frac{T_{\text{tmax}}}{(1 + v)} \]

\[ T_{\text{max}} = 12.25 \text{ lbf} \]

Maximum Thrust under Full Slip

\[ \sigma_{\text{thrust}} = \frac{T_{\text{max}}}{A_{p,\text{tot}}} \]

\[ \sigma_{\text{thrust}} = 2.19 \text{ psi} \]

Note: No solution is developed for calculating diametric strain and moment under no-slip condition. It is recommended that the solutions for full-slip condition be used.

Maximum Bending Moment in shell / rib wall

\[ M_{\text{max}} = \frac{1}{6} K_{1T} E_m F T_{\gamma \max} \rho_{\text{rib}} \]

\[ M_{\text{max}} = 735 \text{ lbf in} \]

Maximum Bending Stress in shell / rib wall

\[ \sigma_{T_{\text{rib}}} = \frac{M_{\text{max}}}{S_{jj}} \]

\[ \sigma_{T_{\text{rib}}} = \frac{256}{153} \text{ psi} \]

\( S = \frac{2.07}{4.82} \text{ in}^3 \)
0) Historical background, some seismic information, shear modulus, and seismic spectra
1) Axial stress due to P waves and S waves
2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts

3) **Xerxes (20) patent (reduced shear modulus) with transverse loads on FRP UST’s**

4) Sloshing
5) Liquefaction
6) Buckling of soil surrounded tubes
Xerxes Method

Granted Patent
Patent No. US 6,397,168 B1
Date of Patent May 28, 2002


A brief summary of some pertinent points follow.
Based on FEA

3) Xerxes Method

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3) Xerxes Method

FEA Soil Mesh

FIG. 6

FIG. 7
3) Xerxes Method

**FEA Ring Mesh**

See slide 107 for rib cross section.

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Xerxes Method

• Modeled 1994 Northridge Earthquake (see slides 108 and 109)
• From column 3 of patent
  • S waves come from epicenter of earthquake
  • Eventually intersect Earth’s surfaces
  • Split into two types and travel along Earth’s surface
    • Rayleigh waves
    • Love waves
• Patent argues that backfill loses its active confining pressure under vertical seismic acceleration of 1g so shear modulus is very low, perhaps zero – ref to discussion B K. Ishihara (11)
• Accounted for by using very low dynamic shear modulus
Xerxes Method

• Rayleigh waves with wavelength of 20 times diameter or the tank

• Model includes
  • Backfill
  • Tank shell
  • Reinforcing ribs

• Soil shear modulus decreases with increasing level of cyclic shear strain. Damping increases with increasing shear strain (see slide 13).

• G assumed to decrease with increasing strain over time history for each model per figures 8A and 8B.
Figure 8A and 8B show that shear modulus decreases with increasing strain and damping ratio increases.
Xerxes Method

• Initial shear modulus by Kokusho and Esashi gives, (from Table I):

\[ G_o = \frac{8400(2.17-e)^2(\sigma_0)^{0.60}}{(1+e)} \] (kPa) (1)  

(2) Xerxes Method

• Apply factor \( \frac{G}{G_o} \) from shear modulus curve using \( \frac{G}{G_o} = 0.15 \) gives

\[ G_o = \frac{1260(2.17-e)^2(\sigma_0)^{0.60}}{(1+e)} \] (kPa) (2)

\[ G_o = \frac{582(2.17-e)^2(\sigma_0)^{0.60}}{(1+e)} \] (psi) (3)

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Xerxes Method

- Confining pressure is taken as $\frac{3}{8} \gamma H$
- Void ratio assumed to be $e = 0.4$ and $\gamma_{soil} = 120 \text{pcf}$
- Results in shear modulus varied from top to bottom of 360 psi to 2257 psi (constant over bottom half)*
- Young’s modulus varied similarly from 1008 psi to 6320 psi.*
- Tank properties are $E = 900,000 \text{ psi}, \nu = 0.3, \gamma_{FRP} = 0.061 \text{pci}$

Metric Evaluation

*These values confirmed by independent check see Appendix A, slide 136.
Section Properties (for Half Rib)

Rib space = 16 $\frac{1}{2}$ in

Reported in patent

$A = 3.25in^2$

$I = 4.0in^4$* (use 1.75in$^4$)

$S = 2.35in^3$

$EI = 1,575,000 lbs - in^2$

$EI/in = 190,909 \frac{lbs-in^2}{in}$

Calculated

$A = \frac{5.6in^2}{2} = 2.8in^2$

$I = \frac{3.4in^4}{2} = 1.7in^4$

$S = 2.07in^3$

$EI = \frac{3,059,400}{2} = 1,529,700 lbs - in^2$

$EI/in = 185,424 \frac{lbs-in^2}{in}$

* Found to be incorrect. This is probably for full rib.

[Diagram of half rib section properties]
Xerxes Method

• Seismic accelerations applied horizontally and vertically in separate analyses.
• Time history of 15 seconds from Northridge Earthquake used

**Horizontal Analysis**

Peak Acceleration = 1.78 g @ 8.36 s
Peak Velocity = 47.37 in/s @ 7.92 s
Initial Velocity = 0.67 in/s
Initial displacement = 1.73 in
Peak values occurred @ ~8 s
Xerxes Method

Vertical Analysis

Peak Acceleration = 1.047 g @ 8.58 s
Peak Velocity = -28.469 in/s @ 8.52 s
Peak Displacement = 6.7 in @ 7.94 s
Initial Velocity = 0.53 in/s
Initial displacement = 1.944 in

• Results may be added algebraically but this may result in overly conservative results
• [For combined results the SRSS method is recommended]
• Results reported in Xerxes Patent are summarized in Tables IV and V.
3) Xerxes Method

**Xerxes Method**

Table IV.

<table>
<thead>
<tr>
<th>Column</th>
<th>Model</th>
<th>Figure</th>
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<th>Δ (in)</th>
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* Compare to previous methods

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* With HORIZONTAL soil displacements

Min: -595, Max: 1555

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Min: -1146, Max: 868 for range above

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Comparison of Three Methods Using Lowest Shear Modulus

Note that Xerxes method does not include sloshing. This must be included.

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All units in psi or inches.

*The Wang method has same order of magnitude for hoop stress as Xerxes method.

**The original method does not include hoop bending so results are low.
0) Historical background, some seismic information, shear modulus, and seismic spectra
1) Axial stress due to P waves and S waves
2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts
3) Xerxes (20) patent (reduced shear modulus) with transverse loads on FRP UST’s

4) **Sloshing**
5) Liquefaction
6) Buckling of soil surrounded tubes
Sloshing

- Convective (sloshing) component computed according to ASCE 7-10, Section 15.7.6.1 page 152. Sloshing mass is computed per ACI 350.3-06, Eqn 9-16, page 48.
  - Ref ASCE 7-10, Section 15.7.6.1.1 “Distribution of Hydrodynamic and Intertia Forces”, page 153 (1)
    “...the method given in ACI 350.3 is permitted to be used to determine the vertical and horizontal distribution of hydrodynamics and Inertia forces on the walls of circular and rectangular tanks.”
  - “Analysis of Pressurized Horizontal Vessels Under Seismic Excitation”, by Carluccio, Fabbrocino, Salzano and Manfredi (7), states that, for circular horizontal tanks with (fluid depth)/(tank radius) between 0.5 and 1.6,
    “approximate values for hydrodynamic pressures...can be obtained from solutions for the rectangular of equal dimension...”
First Mode Period (convective) sloshing

\[ \lambda = \sqrt{3.16 g \cdot \tanh \left( 3.16 \cdot \frac{d_{fl}}{L'} \right)} \]

where

\[ d_{fl} = \text{depth of fluid} \]

\[ T_c = \left( \frac{2\pi}{\lambda} \cdot \sqrt{L'} \right) \]

\[ L' = \text{effective tank length} \]

Sloshing mass is computed per ACI 350.3, Eqn 9-2, pg. 44

\[ W_c = 0.264 \left( \frac{L}{d_{fl}} \right) \cdot \tanh \left[ 3.16 \left( \frac{d_{fl}}{L'} \right) \right] \cdot W_{tf} \]
Sloshing

Fundamental period \( T_S = \frac{S_{D1}}{S_{DS}} \) ASCE 7-10 pg. 152 (1)

Seismic Response Coefficient

\[
C_{ck} = \text{if} \left[ \frac{T_{ck}}{s} \leq \frac{1.6s}{T_S}, \frac{1.5S_{D1}}{T_{ck}}, \frac{2.4S_{DS}}{(T_{ck} \div s)^2} \right]
\]

Convective Component \( R_c = 1.0 \) ACI 350.3, Section 9.4.2, page 40 (3)

\[
V_c = \left( C_c I_e \cdot \frac{W_c}{R_c} \right)
\]
Method for Sloshing Calculations

1) Iterate on fluid depth in the tank
2) Determine the first mode period per ASCE 7-10 Eqn. 15.7-12
3) Determine the spectral acceleration per ASCE 7-10 Eqn. 15.7-10 & 11
4) Determine the sloshing wave height per ASCE 7-10 Eqn. 15.7-13
5) Compute the sloshing mass per ACI 350.3 Eqn. 9-16
6) Compute the convective component of the seismic base shear per ASCE 7-10 Eqn. 15.7-6
7) Plot to find maximum value (slide 119)
Example

Example text.
4) Sloshing

Example

Sloshing force = f (fluid height)

Convective Component Vs. Fluid depth (by straight shell length)

Fluid Depth

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3) Xerxes patent (20) (reduced shear modulus) with transverse loads on FRP UST’s

4) Sloshing

5) **Liquefaction**

6) Buckling of soil surrounded tubes
Liquefaction – Flotation

- FRP tanks and manholes are vulnerable to liquefaction induced flotation
- Tanks and manholes must be within liquefiable layer
- Nearly complete liquefaction must occur
- Flotation is dependent on buoyancy
  - Larger diameter tanks more buoyant than small diameter

Reference Donald Ballantyne, Sewers Float and Other Aspects of Sewer Performance in Earthquakes, 2010, p. 18 (5)
Liquefaction

Reference Lambe and Whitman, Soil Mechanics p. 445 (13)

- Liquefaction susceptibility is greatest in fine uniform sand or silt
- Fine sands precise size range from 0.06mm to 0.2mm
- Uniformity coefficient

\[ U_c = \frac{D_{60}}{D_{10}} > 2 \]  
Fine sand

\( D_{60} \) is particle diameter at which 60% of weight is finer than \( D_{10} \) where \( D_{10} \) is 10% of weight is finer. ibid p. 32
Liquefaction

- Liquefaction results from shear stress reversal for a saturated soil.
  - During each cycle excess pore pressure accumulates
  - As pore pressure increases, shear strength decreases
  - Catastrophic failure can occur when initial shear strength equals zero
- Highly dependent on initial void ratio (e > 0.8)
Four conditions must be present for liquefaction to occur

1) The soil must be a fine-grained silt [or sand] with a uniformity coefficient ($U_c$) less than 2 for the full depth of the tank installation and below the tank.

2) The void ratio of the fine silt [or sand] must be greater than or equal to 0.8.

3) The ground water must be high, such as at grade.

4) There must be a seismic event of sufficient magnitude to cause a number of strain reversals and a high-accumulated soil strain.

Probability of these four occurring simultaneously is very low.

One study based on USGS data for p 6.7 EQ resulted in $P_{LIQF} = 0.001995\%$ or one installation 1 in 50125 tanks.

Flotation has occurred in New Zealand during Christchurch earthquake (2011)

Reference M. Power and T. Holzer, Liquefaction Maps, ©1996, pg. 1 (21)
Can be found at http://www.atcouncil.org/pdfs/atc-35.pdf
Mitigation of Potential for Liquefaction

- Avoid areas with possible liquefaction if at all possible
- If must install in location with fine sand with potential for liquefactions
  - Consider soil improvement
  - Ballast tanks to be neutral if soil liquefies
  - Install stone columns
  - Use compaction grouting
  - Directionally drill below layer for pressure lines and siphons
  - Install anchors below susceptible layers - helical piles
  - Install fins on pipe to activate more backfill – or use deadman anchors for UST’s
  - Design backfill to release pressure
  - Design so that repairs may be easily made if possible

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6) Buckling of soil surrounded tubes
Buckling of Soil Surrounded Pipes and Tanks

- Initial work done by Ulrich Luscher in 1966 – Buckling of Soil Surrounded Tubes. J. Soil Mechanics and Foundation. ASCE 92 (6) (14)
  - Valid for long pipes without end loading i.e. for hoop load only.
  - Not valid for short cylindrical tanks with axial load.
- Excellent review by Ian D. Moore, Elastic Buckling of Buried Flexible Tubes – A Review of Theory and Experiment. J. Geotechnical Engineering (115)3 - 1989. (16)
  - It includes Luscher’s data and other data sets.
  - Still only valid for long pipe lines with hoop load only.
- Proposed equation

\[
N_{\theta-CR} = (n^2 - 1) \frac{EI}{R^2} + 2G_m R \left[ \frac{2n(1 - \nu_m) - (1 - 2\nu_m)}{n^2(3 - 4\nu_s)} \right]
\]

(a)
Buckling of Soil Surrounded Pipes and Tanks

Where

\( i \) = buckling mode (use 2 for flexible pipe)

\( EI \) = flexural rigidity of tank or pipe

\( R \) = radius

\( G_m \) = shear modulus of soil

\( \nu_m \) = Poisson’s ratio of soil

For slip condition

\[
N_{\theta-CR} = (n^2 - 1) \frac{EI}{R^2} + 2G_mR \left[ \frac{1}{2n(1-\nu_m)-(1-2\nu_m)} \right] \tag{b}
\]

\[
G_m = \frac{E_m}{2(1+\nu_m)}
\]

- A third method is provided in AWWA M45 “Fiberglass Pipe Design 2nd ed. pp. 65-66. (29)

This method is too long to include in this presentation.
To account for applied stress on ends of a short section with end caps use an appropriate axial buckling equation and compute FS with an interaction equation.

\[ F_{STank} = \frac{1}{\frac{1}{F_{S\theta}} + \frac{1}{F_{SAx}}} \]

Generally requires testing to validate.
Two possible axial buckling equations are:

\[ \sigma' = \frac{1}{\sqrt{3}} \cdot \frac{E_{\text{pipe}}}{(1-v^2)} \cdot \frac{t_{\text{eff}}}{R} \]

where

- \( E_{\text{pipe}} \) is modulus of pipe material
- \( v \) is Poisson’s ratio of pipe material
- \( t_{\text{eff}} \) is effective thickness for ribbed pipe or tank

(Note that axial buckling is taken as independent of soil support)
Two possible axial buckling equations are:

- Structural Plastics Design Manual Chapters 5-10 Simpson Gumpertz & Heger (inc.), p. 9-87 (2)

\[
\sigma_{CR-AX} = \frac{2\sqrt{3}C\sqrt{D_x A \theta}}{R} 
\]

\[
D_x = \frac{E_x t^3}{12(1-\nu^2)} = \frac{(E/I/\text{in})}{(1-\nu^2)} 
\]

\[
A_\theta = \frac{E_x t}{(1-\nu^2)} 
\]

\[
C = k_o \cdot k_n \cdot k_s 
\]

\[
k_o = \frac{1}{3\sqrt{1-\nu^2}} \approx 0.6 \text{ for } \nu = 0.3 
\]

\[
k_n = \text{knockdown factor for imperfections} 
\]

\[
\sim 1.53 - 0.477 \log \frac{R}{t} \leq 0.21 
\]

\[
k_s = \text{shear reduction factor (see Fig 9-26)} 
\]
When to Use which Method for Computing Buckling FS

For no ground water
- Check hoop bucking only
- Add stress for vertical loads

For high ground water
- Check hoop bucking and axial buckling (equations a or b)
- Add stress for vertical loads and hydrostatic loads
- Use buoyant weight for soil
- Use interaction equation

For liquefaction
- Check hoop and axial buckling (equations a or b) in a heavy fluid
- Use interaction equation
Summary

A. Studies show that soil shear modulus $G_m$ is reduced with increasing strain during and earthquake. This was not considered in previous method of analysis for axial stress.

B. Three methods were discussed for computing stresses due to seismic waves in buried structures.
   1) Historical method (1960-1980’s) developed for pipelines by Newmark/Yeh and others focusing on axial stress with lateral stress also reported – sloshing addressed later on. Shear modulus reduction not addressed
   2) Wang/NCHRP method (2008) focused on lateral stress perpendicular to the long axis. This method may also be used for rectangular structures such as concrete culverts, etc. Method is based on numerous FEA studies – sloshing not addressed. Shear modulus reduction not addressed
   3) Xerxes method uses reduced shear modulus and dynamic FEA. Sloshing not addressed.
C. Comparison of methods

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<td>~1 in</td>
</tr>
<tr>
<td><strong>Ref</strong></td>
<td>Slide 45</td>
<td>Slide 83</td>
<td>Slide 110</td>
</tr>
</tbody>
</table>

Similar order of magnitude values are obtained between Xerxes Patent method and Wang/NCHRP method.

The historical axial method is not considered by later authors, thus use as an upper bound check and design using the Wang/NCHRP method.

*The Wang method has same order of magnitude for hoop stress as Xerxes method.

**The original method does not include hoop bending so results are low.
Summary

D. Sloshing should be considered and added to any method used.

   Available literature indicates that methods developed for rectangular tanks are appropriate for cylindrical horizontal tanks.

E. Potential for liquefaction should be considered. Develop means to mitigate the effect or provide for quick repairs if other options not possible.

F. For hoop loaded tanks from seismic, tanks should be checked for buckling. A method is provided for combining with axial loads.
Appendix A

Verify Values for G and E Used in Xerxes Patent

Calculate using metric units for from Grade to Springline

Depth below grade (ft)  Confining Pressure  Shear Modulus  Factor by 15% per Fig 8 in patent

<table>
<thead>
<tr>
<th>Depth below grade (ft)</th>
<th>Confining Pressure (kPa)</th>
<th>Shear Modulus (MPa)</th>
<th>Factor by 15% per Fig 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0</td>
<td>19</td>
<td>282</td>
</tr>
<tr>
<td>0.375</td>
<td>0</td>
<td>16.5</td>
<td>2481</td>
</tr>
<tr>
<td>1</td>
<td>0.8</td>
<td>29.8</td>
<td>4469</td>
</tr>
<tr>
<td>3</td>
<td>6.5</td>
<td>57.6</td>
<td>8639</td>
</tr>
<tr>
<td>8</td>
<td>17.2</td>
<td>103.7</td>
<td>15562</td>
</tr>
</tbody>
</table>

Convert to psi

<table>
<thead>
<tr>
<th>Depth below grade (ft)</th>
<th>Confining Pressure (psig)</th>
<th>Shear Modulus (psi)</th>
<th>Factor by 15% per Fig 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>41</td>
<td>115</td>
<td>282</td>
</tr>
<tr>
<td>0.375</td>
<td>360</td>
<td>1008</td>
<td>2481</td>
</tr>
<tr>
<td>1</td>
<td>648</td>
<td>1815</td>
<td>4469</td>
</tr>
<tr>
<td>3</td>
<td>1258</td>
<td>3509</td>
<td>8639</td>
</tr>
<tr>
<td>8</td>
<td>2257</td>
<td>6320</td>
<td>15562</td>
</tr>
</tbody>
</table>

Compute Modulus

\[ E = \frac{G}{(1 - v)} \]

Springline (8ft)

BACKFILL MATERIAL
PEA-GRavel
120in. DIAMETER HORIZONTAL FRP TANK

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### References


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References


28. N.d., “ASCE 7-10, Figure 22-1,” USGS from http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf


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