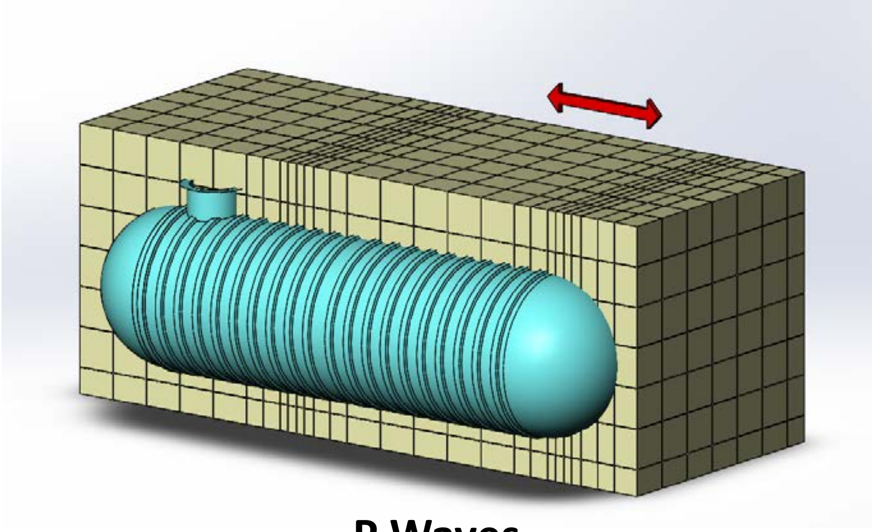
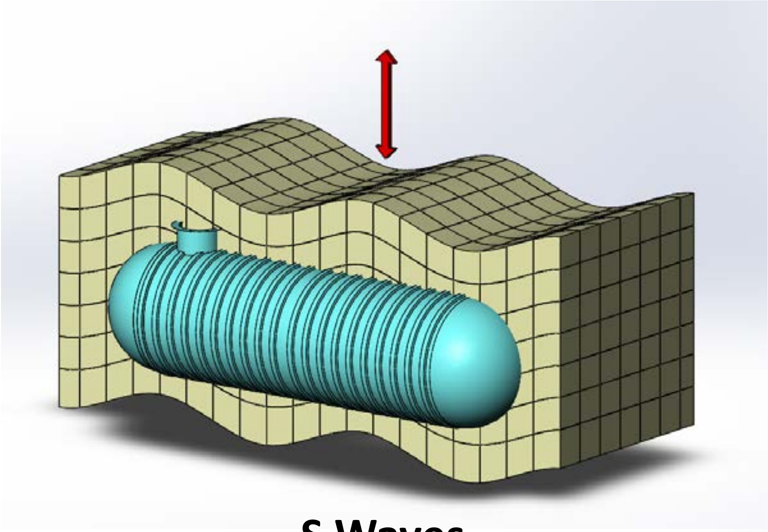
Seismic Design for Buried Flexible Structures



P Waves
"Push"



S Waves
"Shake"

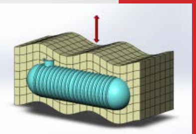
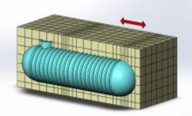
By

John M. Clark, MS,PE

President of Clark Engineers, Inc.

Presented to Foundation Performance Association

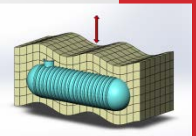
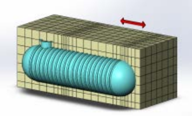
June 10, 2015

Qualifying Statement

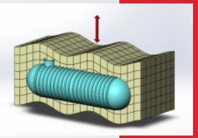
- No original work by this author is included herein.
- All information provided herein is from published sources.
- References are provided.
- Best effort has been made to ensure correct methods; however all methods should be verified for accuracy before using.
- PDF files of references can be provided 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.

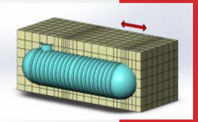
†**Xerxes Patent US 6,397,168 B1 column 3, line 67 (20)**

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.**

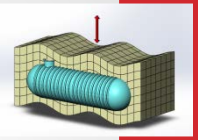


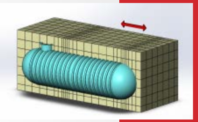


Focus of this Presentation

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

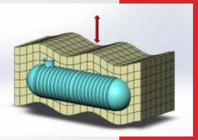
1) Axial stress due to P waves and S waves
2) Wang method (23) (NCHRP) (4) transverse loads on circular conduits and box culverts
3) Xerxes patent (20) (reduced shear modulus) with transverse loads on FRP UST's
4) Sloshing
5) Liquefaction
6) Buckling of soil surrounded tubes

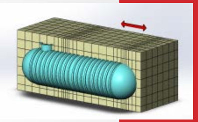




Historical Background

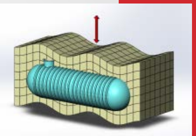
- 1980's customer's started requiring seismic calculations for underground storage tanks (UST's). At this time there were no known treatises on this topic.
- Hired local consultant PhD, PE to write paper.
  - Results were based on methods used for pipe lines – axial stress due to P and S waves. Based 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.

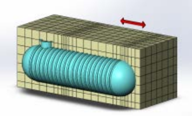




Historical Background

- 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



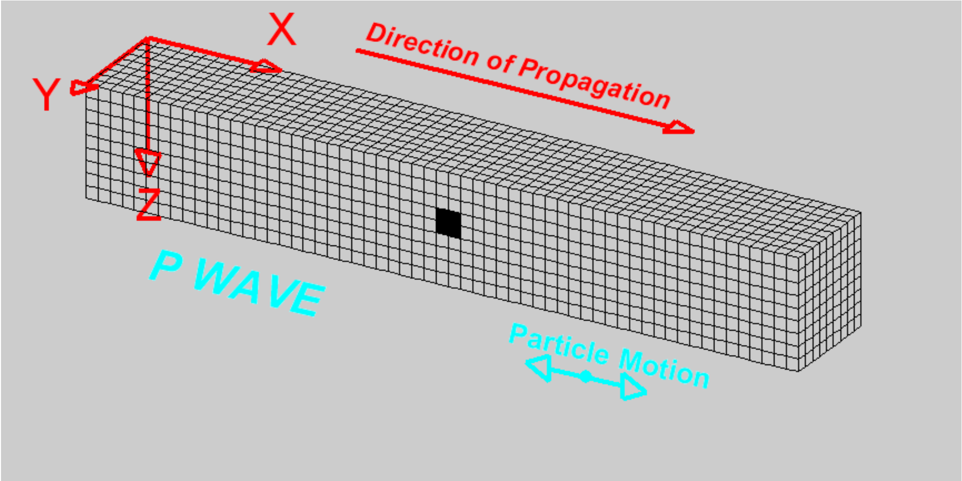


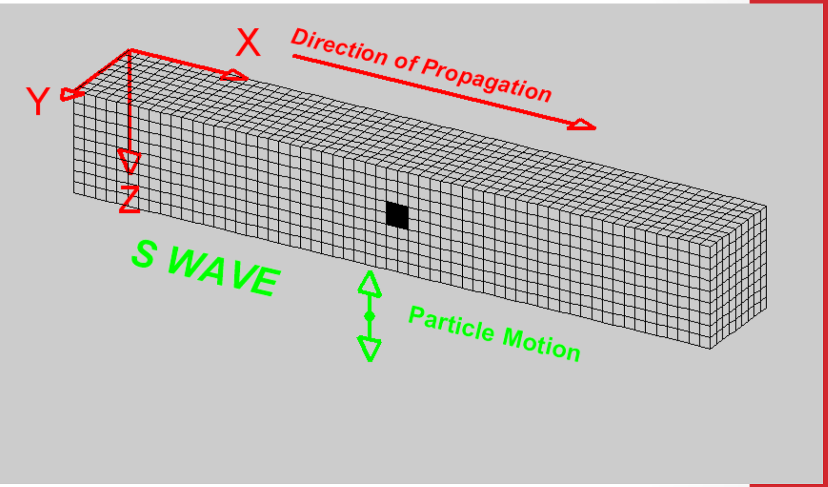
Historical Background

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.

Examples of Longitudinal Effects for P Waves and S Waves:

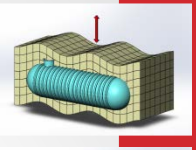


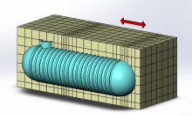


P Wave
“Push”

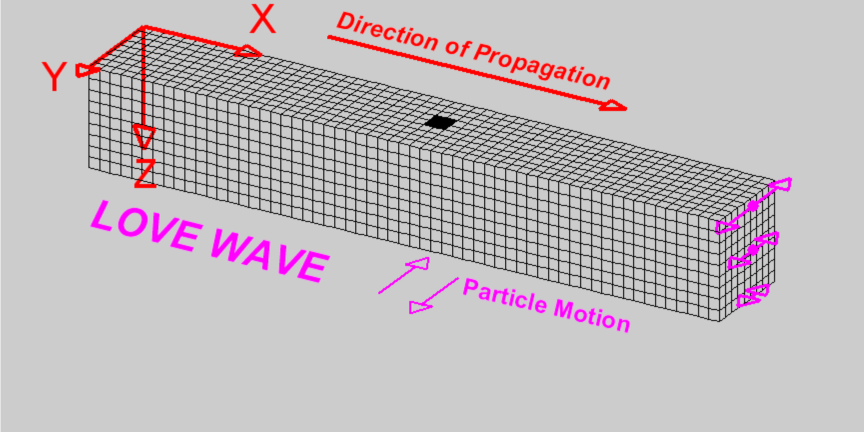
S Wave
“Shake”

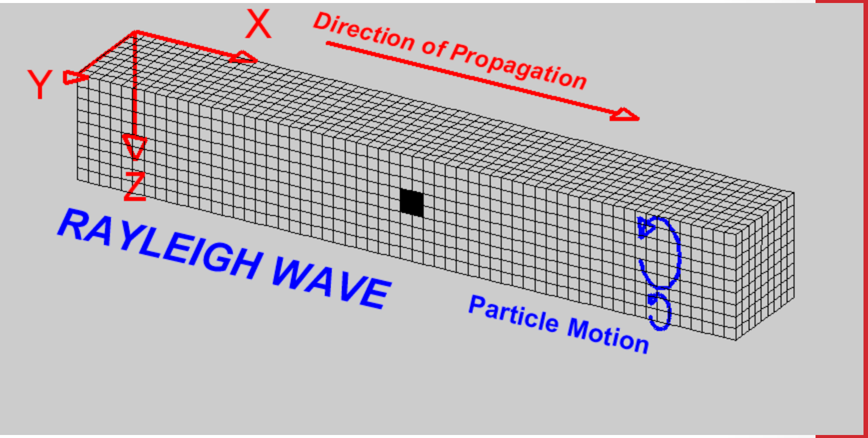
Reference http://web.ics.purdue.edu/~braile/edumod/waves/WaveDemo.htm (26)





Examples of Longitudinal Effects for L Waves and R Waves:

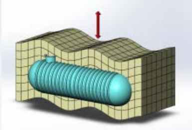


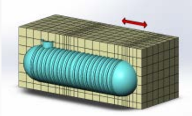


L Wave
"Love"

R Wave
"Rayleigh"

Reference http://web.ics.purdue.edu/~braile/edumod/waves/WaveDemo.htm (26)





Some Earthquake Characteristics

Figure 1.1 shows classification of dynamic problems

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

- 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

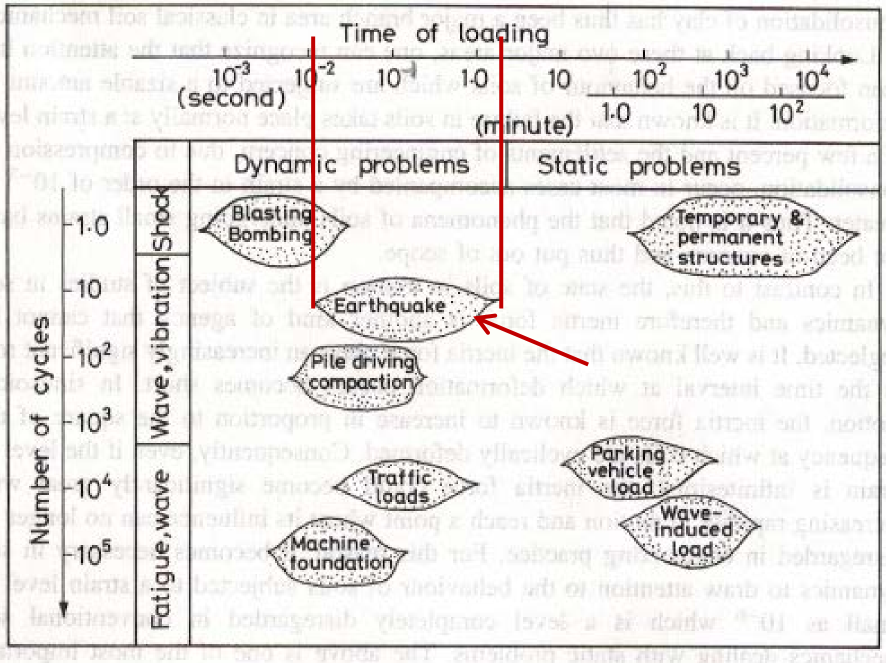
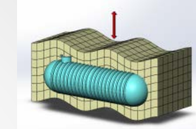
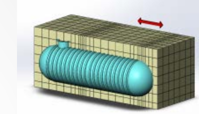


Fig. 1.1 Classification of dynamic problems.





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%

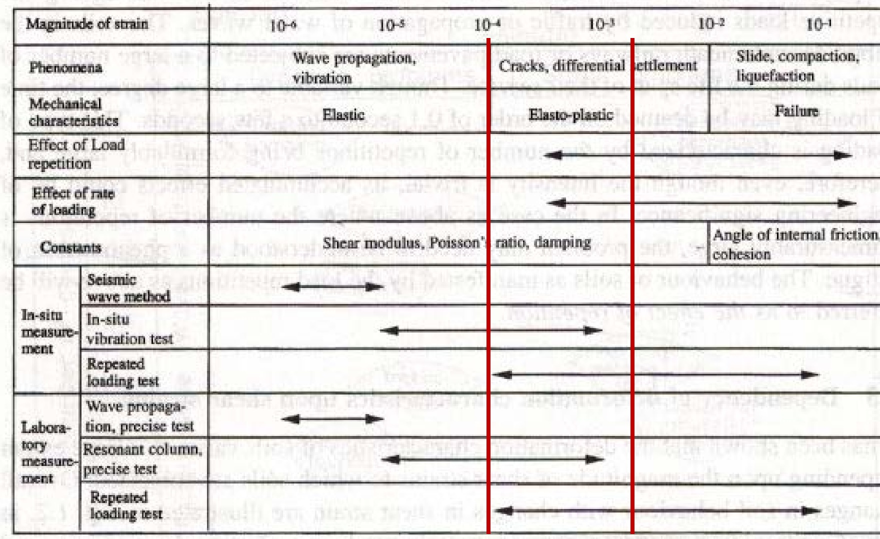
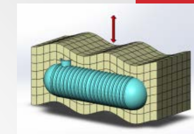
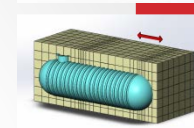


Fig. 1.2 Variation of soil properties with strain.

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.





Some Earthquake Characteristics

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

Reference strain

$$\gamma_r = {}^{\tau_f}/_{G_0}$$

where γ_r = strain at failure for linear elastic G_0

τ_f = shear stress at failure

G_0 = shear modulus of soil

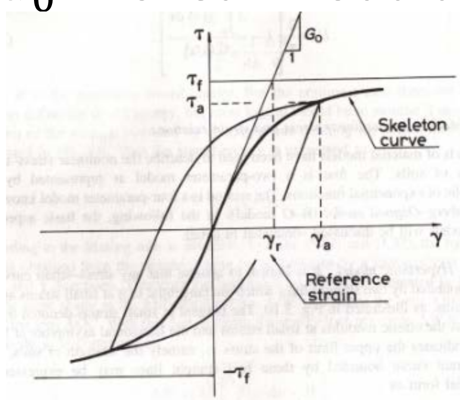
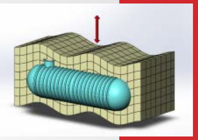
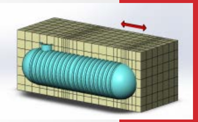


Fig. 3.10 Definition of reference strain.





Some Earthquake Characteristics

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

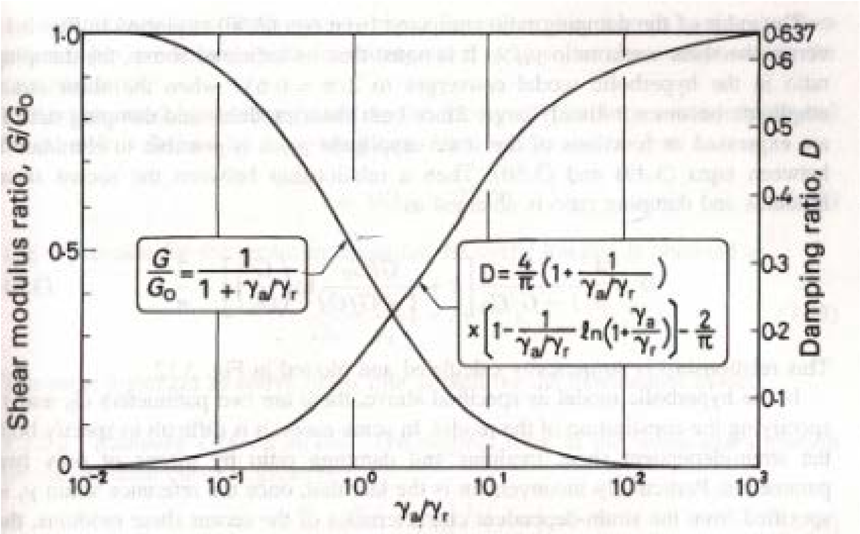
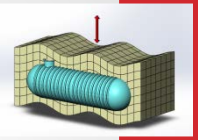
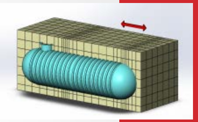


Fig. 3.11 Shear modulus ratio and damping ratio for hyperbolic model.

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 ${}^{\gamma_a}/_{\gamma_r} = 1.0 \rightarrow \frac{G}{G_0} = \frac{1}{1+1} = \frac{1}{2}$]

Shear Modulus G_o





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

Table I.

Table 6.3 Constants in proposed empirical formulae of initial shear modulus for gravels may be estimated by $\mathbf{G_o = AF(e)(\sigma'_0)^{n}}$ ($\mathbf{G_o}$ and $\mathbf{\sigma'_0}$ in kPa)

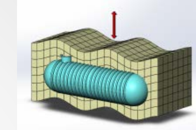
Reference	A	F(e)	n	Material	Sample Size	Test Method
Prange (1981)	7230	$(2.97-e)^2/(1+e)$	0.38	**Ballast** $D_{50}{}^{**} = 40$ mm $U_c{}^{***} = 3.0$	Dia.: 100 cm Length: 60 cm	Resonant column
Kokusho and Esashi (1981)	13000	$(2.17-e)^2/(1+e)$	0.55	**Crushed rock** $D_{50} = 30$ mm $U_c = 10$	Dia.: 30 cm Length: 60 cm	Triaxial
Kokusho and Esashi (1981)*	8400	$(2.17-e)^2/(1+e)$	0.60	**Round gravel** $D_{50} = 10$ mm $U_c = 20$	Dia.: 30 cm Length: 60 cm	Triaxial
Tanaka et al. (1987)	3080	$(2.17-e)^2/(1+e)$	0.60	**Ballast** $D_{50} = 10$ mm $U_c = 20$	Dia.: 10 cm Length: 20 cm	Triaxial
Goto et al. (1987)	1200	$(2.17-e)^2/(1+e)$	0.85	**Ballast** $D_{50} = 2$ mm $U_c = 10$	Dia.: 30 cm Length: 60 cm	Triaxial
Undisturbed						
Nishio et al. (1985)	9360	$(2.17-e)^2/(1+e)$	0.44	**Ballast** $D_{50} = 10.7$ mm $U_c = 13.8$	Dia.: 30 cm Length: 60 cm	Triaxial

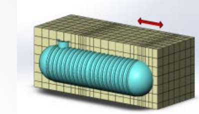
Reference K. Ishihara, Soil Behavior in Earthquake Geotechnics, ©1996, pg. 100 (11)

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

** D_{50} - 50 % finer than value

*** U_c - D_{60}/D_{10}





Shear Modulus G_o

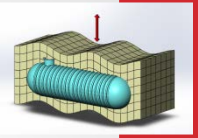
Table II.

Table 6.1 Constants in proposed empirical equations on small strain modulus: $^{*}G_o = AF(e)(\sigma_0')^{\,n}$ (Kokusho, 1987)

	Reference	*A*	*F(e)*	*n*	Soil Material	Test Method
Sand	Hardin-Richart (1963)	7000	$(2.17 - e)^2/(1+e)$	0.5	Round grained Ottawa sand	Resonant column
		3300	$(2.97 - e)^2/(1+e)$	0.5	Angular grained crushed quartz	Resonant column
	Shibata-Soelarno (1975)	42000	$(0.67 - e)/(1+e)$	0.5	Three kinds of clean sand	Ultrasonic pulse
	Iwasaki et al. (1978)	9000	$(2.17 - e)^2/(1+e)$	0.38	Eleven kinds of clean sand	Resonant column
	Kokusho (1980)	8400	$(2.17 - e)^2/(1+e)$	0.5	Toyoura sand	Cyclic triaxial
	Yu-Richart (1968	7000	$(2.17 - e)^2/(1+e)$	0.5	Three kinds of clean sand	Resonant column
Clay	Hardin-Black (1968)	3300	$(2.97 - e)^2/(1+e)$	0.5	Kaolinite, etc.	Resonant column
		4500	$(2.97 - e)^2/(1+e)$	0.5	Kaolinite, $I_p^{**} = 35$	Resonant column
	Marcuson-Wahls (1972)	450	$(4.4 - e)^2/(1+e)$	0.5	Bentonite, $I_p = 60$	Resonant column
	Zen-Umehara (1978)	2000~4000	$(2.97 - e)^2/(1+e)$	0.5	Remolded clay, $I_p = 0\sim50$	Resonant column
	Kokusho et al. (1982)	141	$(7.32 - e)^2/(1+e)$	0.6	Undisturbed clays, $I_p = 40\sim85$	Cyclic triaxial

$^{*}\sigma_0'$: kPa, G_o : kPa, $^{**}I_p$: Plasticity Index

Reference K. Ishihara, Soil Behavior in Earthquake Geotechnics, ©1996, pg. 89 **(11)**



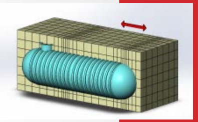
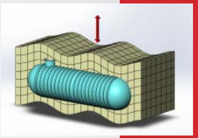


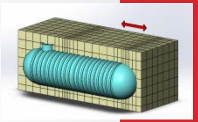
Table III shows some representative values of initial soil shear modulus (G_0) for gravels, sands and clays

Table III.

Soil Type	Soil Density (pcf)	Void Ratio	Type	Shear Modulus (G_0) (psi)				
				Depth (ft)				
				2	4	6	8	15
Gravely	120	0.5	Round Gravel	8786	13318	16986	20186	29434
			Gravel	3594	5448	6948	8257	12040
			Gravel	2578	4647	6559	8377	14293
			Crushed Rock	13427	19658	24569	28781	40668
			Undisturbed Gravel	7390	10026	11984	13601	17934
			Ballast	10786	14036	16374	18266	23194
Sands	120	0.5	Round grained Ottawa	6399	9049	11083	12798	17524
			Ang'lr gr'nd crshd qtz	6599	9332	11430	13198	18072
			3 kinds clean sand	2340	3310	4054	4681	6409
			11 kinds clean sand	6138	7987	9318	10394	13199
			Toyura sand	7679	10859	13300	15357	21029
			3 kinds clean sand	6399	9049	11083	12798	17524
Clays	100	0.5	Kaol.te etc	7378	10434	12779	14756	20205
			Kaol.te PI=35	10061	14228	17426	20122	27553
			Bent'te PI=60	2508	3547	4344	5016	6869
			Remold clay	3248	4226	4930	5500	6984
			Remold clay	8943	12647	15490	17886	24491
			Undist clay PI=40~85	2403	3399	4163	4807	6582

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

	Horizontal	Vertical
Peak Acceleration	1.78 g	1.047 g
Peak Velocity	47.37 in/s	-28.469 in/s
Peak Displacement		6.7 in
Initial Velocity	0.67 in/s	0.53 in/s
Initial Displacement	1.73 in	1.944 in

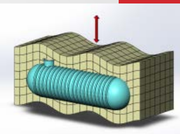
ASCE 7 Mapped Acceleration 1994	
Longitude	34.23046 N
Latitude	-118.5369
S_s	1.905 g
S_1	.614 g

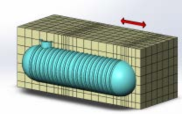
After doing code calculations

$a_p = 18.387 \ {}^{ft}/_{s^2} = 57.1\% \ g$ $V_p = 25 \ {}^{cm}/_{s} = 9.8 \ {}^{ft}/_{s}*$

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

*See slide 19.





Example of Response Spectra

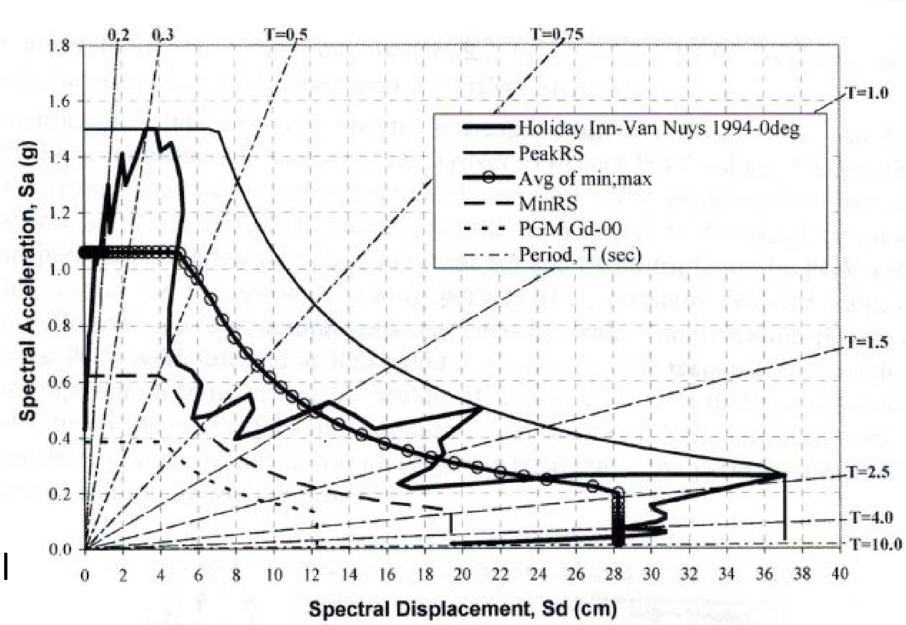
- Actual data has many spikes
- Using maximum and minimum values for acceleration and displacement then maximum and minimum spectra curves can be plotted

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

- Use average for design

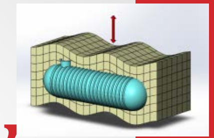
“Note: The code is based on a **2475 year event**. The structure is designed for 2/3 of that. It is assumed that in a major event the structure will go beyond the elastic limits and survive in the inelastic range by ductility. An important part of the structural design is to provide ductility.” – S.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

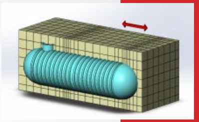
Figure 10(a) from ref 9 shows a typical response spectra on a **tripartite** plot. This reference describes the method to create an average response spectra.



(a)

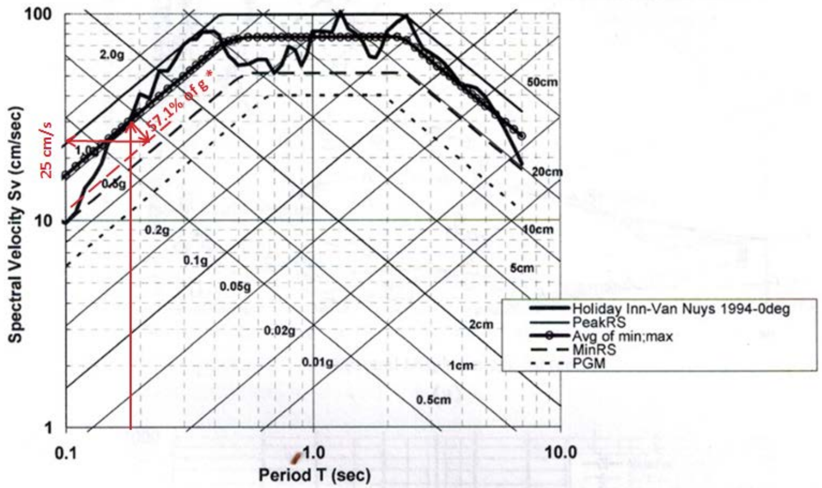
10



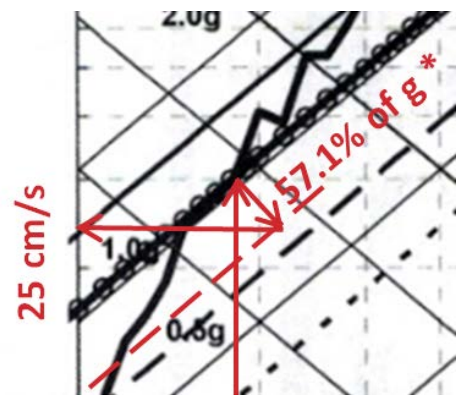


Proposed* Method to Obtain Velocity from Response Spectra

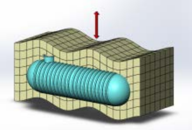
1) Using a_p from ASCE 7** code calculations (see slides 27-30), enter **tripartite** graph where average acceleration crosses the data plot.
2) Construct line perpendicular to average to intersect a_p.
3) Read velocity at this intersection.

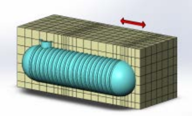


(b)



*There are probably other methods. If period can be determined, use the corresponding value.
**ASCE 7 calculations using mapped acceleration.

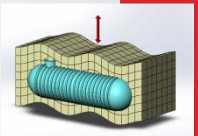


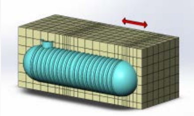


0) Historical background, some seismic information, shear modulus, and seismic spectra

1) Axial stress due to P waves and S waves

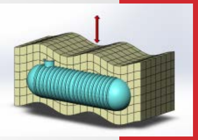
2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts
3) Xerxes patent (20) (reduced shear modulus) with transverse loads on FRP UST's
4) Sloshing
5) Liquefaction
6) Buckling of soil surrounded tubes

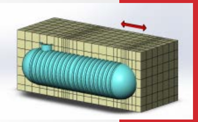




Four Primary Stresses for Compression Waves and Shear Waves

- Compression Wave Axial σ_{ac}
- Compression Wave Bending σ_{ab}
- Shear Wave Axial σ_{as}
- Shear Wave Bending σ_{bs}





Compression Wave Stress

Axial stress

$$\sigma_{ac} = \frac{E_A \cdot V_p \cdot F_m}{C_p}$$

where C_p = compression wave velocity

$$V_p = \frac{a_p \times 48^{in}/_{(sec)}}{g};$$

Shear wave velocity from 1994 Northridge EQ
Or use alternate method from response spectra from slide 19, etc.

a_p = particle acceleration; *Specified or determined from code method*

g = acceleration of gravity

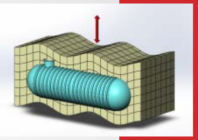
E_A = Axial modulus of elasticity of pipe or tank

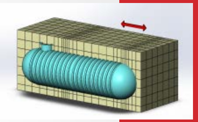
F_m = percent retention of modulus

Bending Stress

$$\sigma_{bc} = \frac{0.385 \cdot E_A \cdot R \cdot a_p \cdot F_m}{{C_p}^2}$$

where R = radius of tank or pipe





Shear Wave Stresses

Axial Stress

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

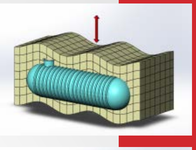
where C_s = shear wave velocity

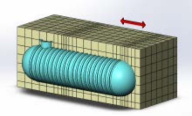
Bending Stress

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

If $\mathrm{a_p}$ and $\mathrm{V_p}$ are provided, use these values. Otherwise, use method in ASCE 7.

Note that stresses increase with decreasing shear wave velocity.





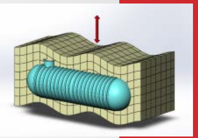
Example of IBC/ASCE 7 Determination for Particle Acceleration (a_p)

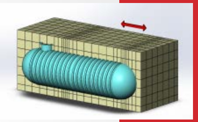
1) Determine seismic accelerations from USGS site (same as acceleration map in ASCE 7-10)
2) Enter longitude and latitude in earthquake USGS site.
   http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf (28)†
3) Determine S_s and S_1 accelerations and velocities by IBC/ASCE 7 method (either MAP or USGS site)

† **Note this site provides accelerations world wide.**

1) Axial Stress Due to P Waves and S Waves

Mapped seismic acceleration for Continental US is shown below





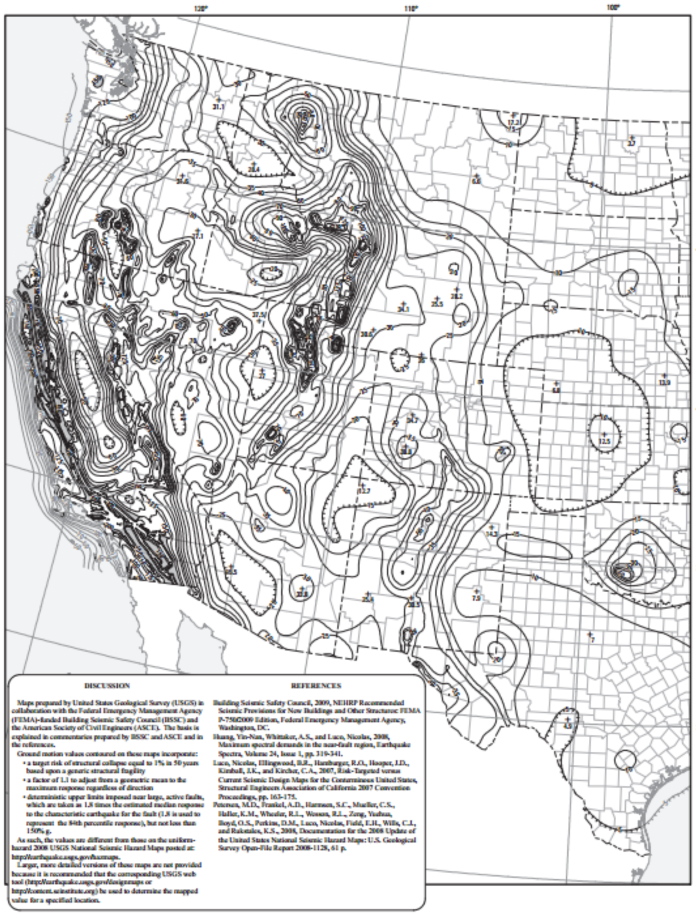


Figure 22-1 *S*, Risk-Targeted Maximum Considered Earthquake (MCE$_R$) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

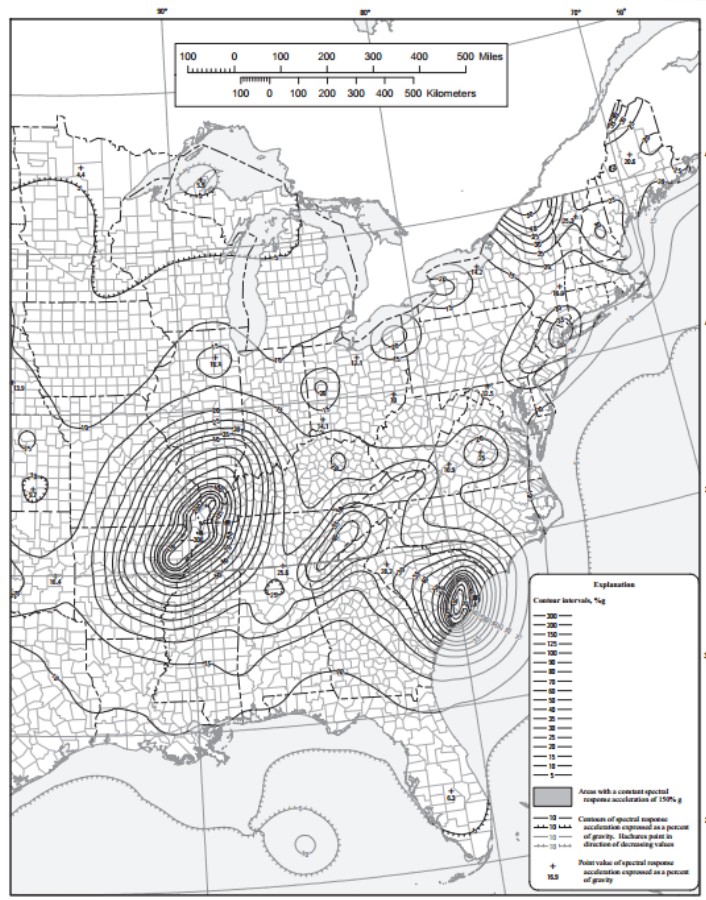
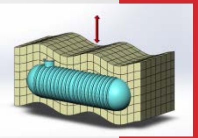


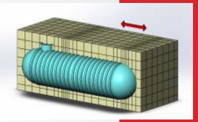
Figure 22-1 *(continued)* *S*, Risk-Targeted Maximum Considered Earthquake (MCE$_R$) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.

http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf **(28)**

1) Axial Stress Due to P Waves and S Waves

Example Using USGS Site

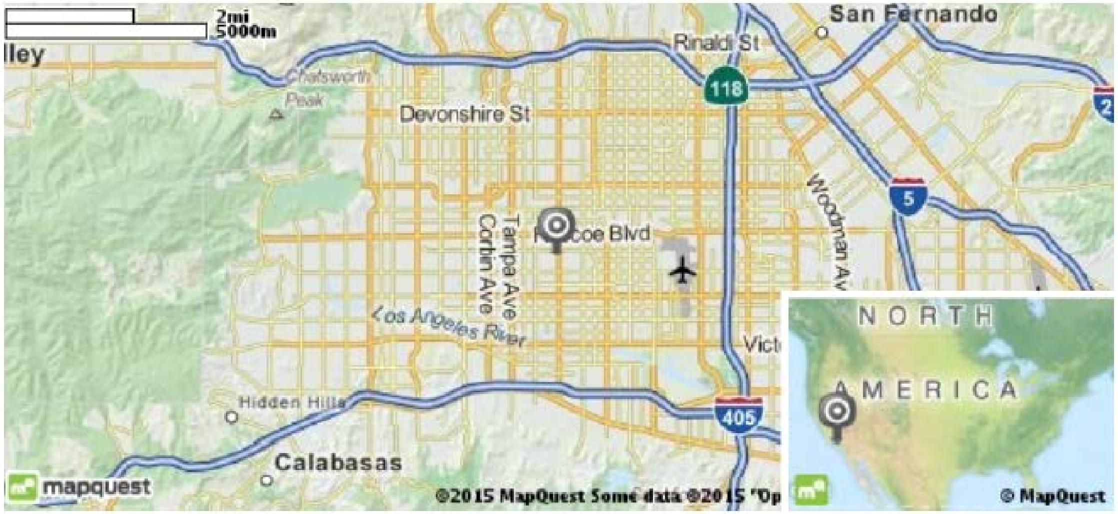




http://earthquake.usgs.gov/hazards/designmaps/usdesign.php **(27)**

Seismic Load

Report Title	Northridge Thu June 4, 2015 16:17:03 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.2131°N, 118.5369°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	IV (e.g. essential facilities)



USGS–Provided Output

S_S =	1.905 g	S_{MS} =	1.905 g	S_{DS} =	1.270 g
S_1 =	0.614 g	S_{M1} =	0.922 g	S_{D1} =	0.614 g

http://earthquake.usgs.gops/us/application.php

Latitude ≡ "36.23046 N"

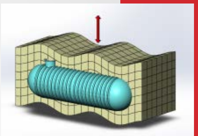
Longitude ≡ −118.5369

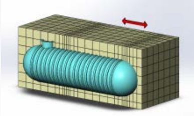
Site_Class ≡ "D"

Loc ≡ "Northridge, CA"

S_S ≡ 190.5%

S_L ≡ 61.4%





Code Calculations

Code Code = "ASCE 7-10"

Location Loc = "Northridge, CA"

Short Period $S_S = 190.5\%$

Long Period $S_1 = 61.4\%$

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

Soil Profile Type

Used in Analysis Site_Class = "D"

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

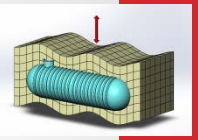
B Rock

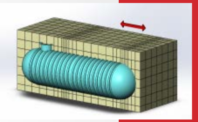
C Very dense soil and soft rock

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

$$S_{DS} = \frac{2^*}{3} \cdot S_{MS} = \frac{2}{3} \cdot (F_a \cdot S_S)$$

Design spectral response acceleration parameter at a period of 1 sec

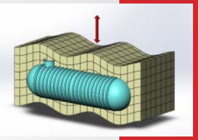
$$S_{DL} = \frac{2^*}{3} \cdot S_{M1} = \frac{2}{3} \cdot (F_v \cdot S_1)$$

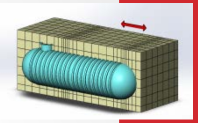
*Recall 2/3 factor per (slide 18)

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)

$$F'_a = \begin{pmatrix} 1.6 \\ 1.4 \\ 1.2 \\ 1.1 \\ 1.0 \end{pmatrix} \quad S'_S = \begin{pmatrix} 0.25 \\ 0.50 \\ 0.75 \\ 1.00 \\ 1.25 \end{pmatrix} \quad F'_v = \begin{pmatrix} 2.4 \\ 2.0 \\ 1.8 \\ 1.6 \\ 1.5 \end{pmatrix} \quad S'_L = \begin{pmatrix} 0.10 \\ 0.20 \\ 0.30 \\ 0.40 \\ 0.50 \end{pmatrix}$$

1) Axial Stress Due to P Waves and S Waves





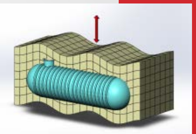
Determine F_a

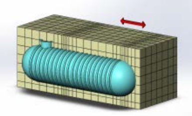
$$F_a = \begin{cases} {F'}_{a_0} \text{ if } S_S \le {S'}_{S_0} \\ \text{linterp}({S'}_S, {F'}_a, S_S) \text{ if } {S'}_{S_0} < S_S < {S'}_{S_{last\,({S'}_S)}} \\ {F'}_{a_{last\,({F'}_a)}} \text{ if } S_S \ge {S'}_{S_{last\,({S'}_S)}} \end{cases}$$

Determine F_v

$$F_v = \begin{cases} {F'}_{v_0} \text{ if } S_1 \le {S'}_{1_0} \\ \text{linterp}({S'}_1, {F'}_v, S_L) \text{ if } {S'}_{1_0} < S_1 < {S'}_{1_{last\,({S'}_1)}} \\ {F'}_{v_{last\,({F'}_v)}} \text{ if } S_1 \ge {S'}_{1_{last\,({S'}_1)}} \end{cases}$$

(MCAD routines for interpolation are shown)





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

$$V_{Tank} = 0.3 \cdot S_{DS} \cdot I_e \cdot 1lbf$$

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 $W_p = 1lbf$

$$M = \frac{W_p}{g}; a_p = \frac{V_{Tank}}{M}$$

Effective Particle Velocity

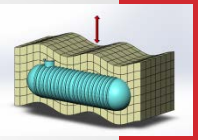
$$V_P = \frac{a_P \cdot 48^{\text{in}}/_{\text{sec}}}{\mathbf{g}};$$

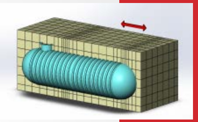
or

$$V_P = \frac{a_P \cdot 25^{\text{in}}/_{\text{sec}}}{\mathbf{g}}$$

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.



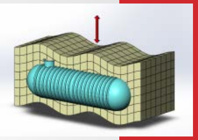


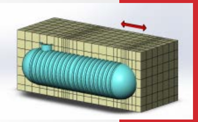
Combined Stress

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

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

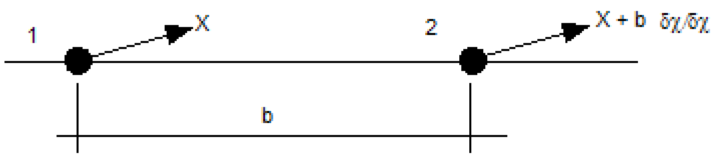
Since waves are out of phase and compression waves arrive first.





Derivation of Axial and Bending Stresses

Based on work by Dr. Nathan Newmark. Ref Newmark, N.M., "Earthquake Response Analysis of Reactor Structures," 1971 from the First International Conference on Structural Mechanics in Reactor Technology (17) and Yeh, C., "Seismic Analysis of Slender Buried Beams," 1964 from the Bulletin of the Seismological Society of America (24).



The particle displacement in the x directions is

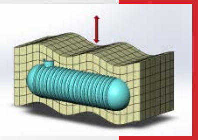
$$X = X(x - c_p \cdot t)$$

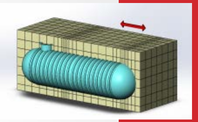
where x = displacement at time zero

c_p = compression wave velocity

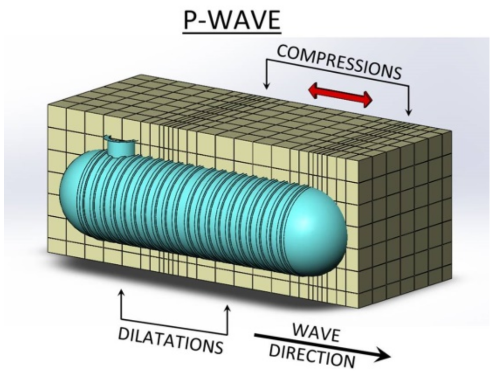
t = time say one second

1) Axial Stress Due to P Waves and S Waves





Derivation of Seismic Stresses Due to Seismic Wave Propagation



Equating a and c gives:
$\delta X/\delta x = 1/c_p \cdot \delta X/\delta t$
Equating b and d gives:
$\delta^2 X/\delta x^2 = 1/c_p{}^2 \cdot \delta^2 X/\delta t^2$
The soil strain is simply $\varepsilon = \delta X/\delta x$
which can be related to the particle velocity of the soil dX/dt by
$\varepsilon = dX/dt/(c_p)$
where dX/dt = vp max
Axial stress due to P waves.
Then the stress is simply the modulus of elasticity (E) times the strain, or
$\boldsymbol{\sigma_a = \pm E \cdot v_p/(c_p)}$

Let us consider a situation in which a seismic wave travel 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:

$\delta X/\delta x = f'(x - c_p \cdot t)$ a

$\delta^2 X/\delta x^2 = f''(x - c_p \cdot t)$ b

$\delta X/\delta t = -cf'(x - c_p \cdot t)$ c

$\delta^2 X/\delta t^2 = c^2 f''(x - c_p \cdot t)$ d

Equating a and c gives:

$$\delta X/\delta x = 1/c_p \cdot \delta X/\delta t$$

Equating b and d gives:

$$\delta^2 X/\delta x^2 = 1/c_p{}^2 \cdot \delta^2 X/\delta t^2$$

The soil strain is simply $\varepsilon = \delta X/\delta x$

which can be related to the particle velocity of the soil dX/dt by

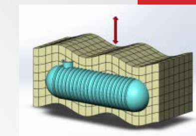
$$\varepsilon = dX/dt/(c_p)$$

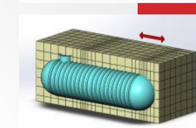
where dX/dt = vp max

Axial stress due to P waves.

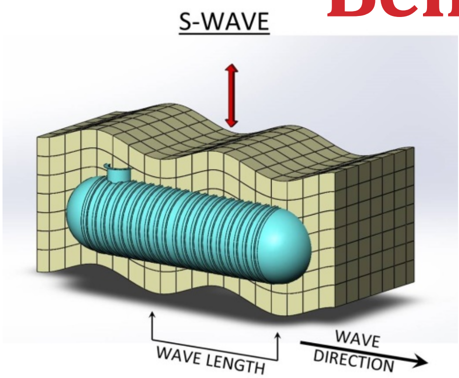
Then the stress is simply the modulus of elasticity (E) times the strain, or

$$\boldsymbol{\sigma_a = \pm E \cdot v_p/(c_p)}$$





Bending Stress Due to an Axial Shear Wave



The shear wave velocity is given as:

$$c_s = [(1-2\nu)/2(1-\nu)]^{1/2} c_p$$

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:

$$\frac{1}{R} = \kappa = d^2y/dx^2$$

"if the soil is linearly elastic and homogeneous, the displacement will satisfy the differential equation

$$\frac{\delta^2 x}{\delta t^2} = c^2 \delta^2 x/\delta X^2$$

rewriting this equation as

$$\frac{\delta^2 x}{\delta X^2} = \delta^2 x/\delta t^2 /c^2$$

It is clear that the curvature of the beam is a function of the particle acceleration of soil by

$$\kappa = d^2x/dt^2/{c_s}^2$$

where $a_{s0} = d^2x/dt^2$ is the maximum ground acceleration due to an axial shear wave.

The bending strain from ***beam theory*** is:

$$\varepsilon = y/R,$$

where y is the distance to the extreme fiber from the neutral axis of the beam. Since

$$\kappa = \frac{1}{R} = d^2x/dt^2/{c_s}^2$$

we can write

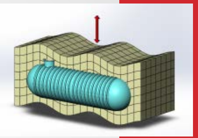
$$\varepsilon = y \cdot a_{s0}/{c_s}^2$$

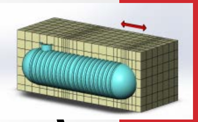
Then the bending stress is simply

$\sigma = E \cdot \varepsilon$ where E = modulus of elasticity of the beam section.

$$\boldsymbol{\sigma = E \cdot y \cdot a_{s0}/{c_s}^2}$$

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".

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

Maximum long term slippage length

Ref Shah, Chu, "Structural Analysis of Underground Structural Elements," ASCE Journal of the Power Division, July 1974, pp. 53-61. (22)

where

$$\varepsilon_m = \frac{\sigma_{comb}}{E_A \cdot F_m}$$

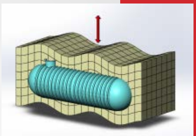
Maximum long term soil strain

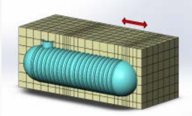
$$A = \pi D t$$

Circular cross-sectional area of the tank shell wall

$$f = \pi \cdot dia \cdot P_r \cdot \mu$$

Frictional force per unit length between the soil-tank interface





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

$\mu^{\dagger} = \frac{k_a+F_w \cdot k_o+k_p}{2+F_w}$ Coefficient of friction between soil and cylinder. μ 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

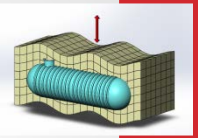
$k_p = \frac{1+\sin\phi}{1-\sin\phi}$ Passive press coefficient

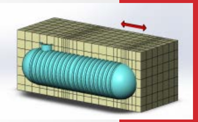
$k_o = 1 - \sin\phi$ At rest press coefficient for smooth wall

$k_o =$ 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

†Ref Bowles "Foundation Analysis and Design" 5th ed. p. 899 (6)





Short Section Effect (*lm*)

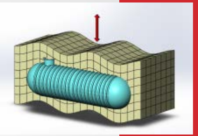
Total axial stress is

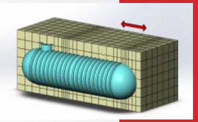
$$\sigma_{A'} = \frac{f \cdot L_{ss}}{2A}$$

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

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

Ref Iqbal, Goodling, "Seismic Design of Buried Piping," Second ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, Dec 1975, p. 153. (10)





Seismic Stress Due to Dynamic Soil Pressure (Hoop Stress)

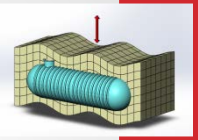
$K_c = 4.0$ $\quad$ $K_s = 5.0$ $\quad$ Dynamic stress concentration factors, reference Newmark and Rosenblueth, Fundamentals of Earthquake Engineering (18)

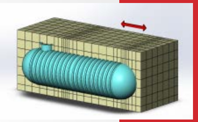
$$\sigma_{\theta_c} = K_c \cdot \rho \cdot C_p \cdot V_p$$

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





Long and Short Period Regions for T_L

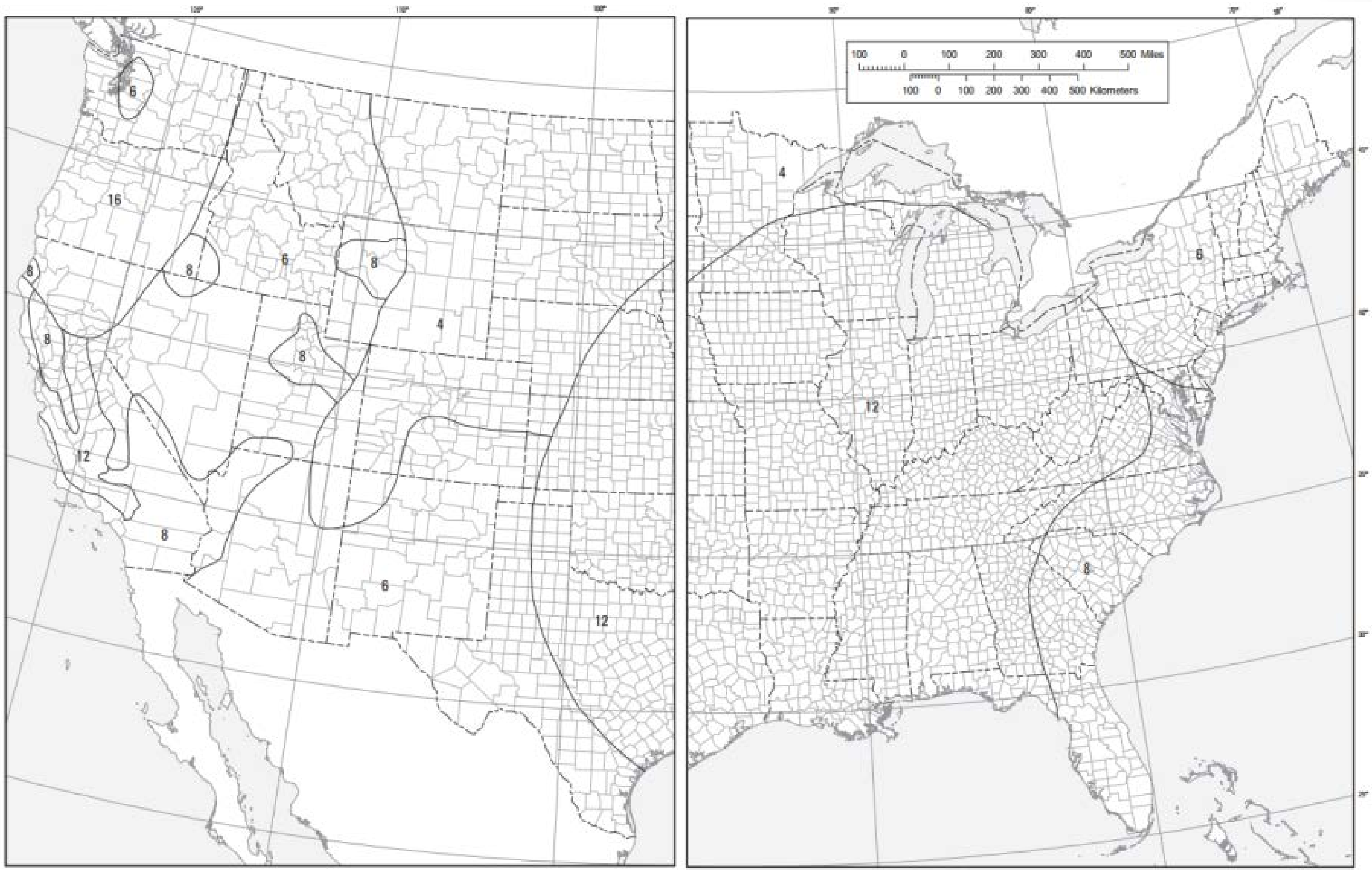
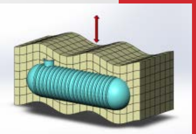
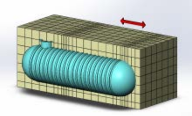


Figure 22-12 Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States.

Figure 22-12 (*continued*) Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States.

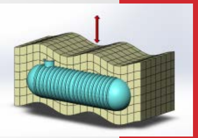


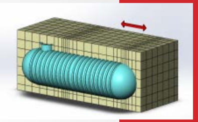


Seismic Stress Due to Dynamic Soil Pressure (Hoop Stress)

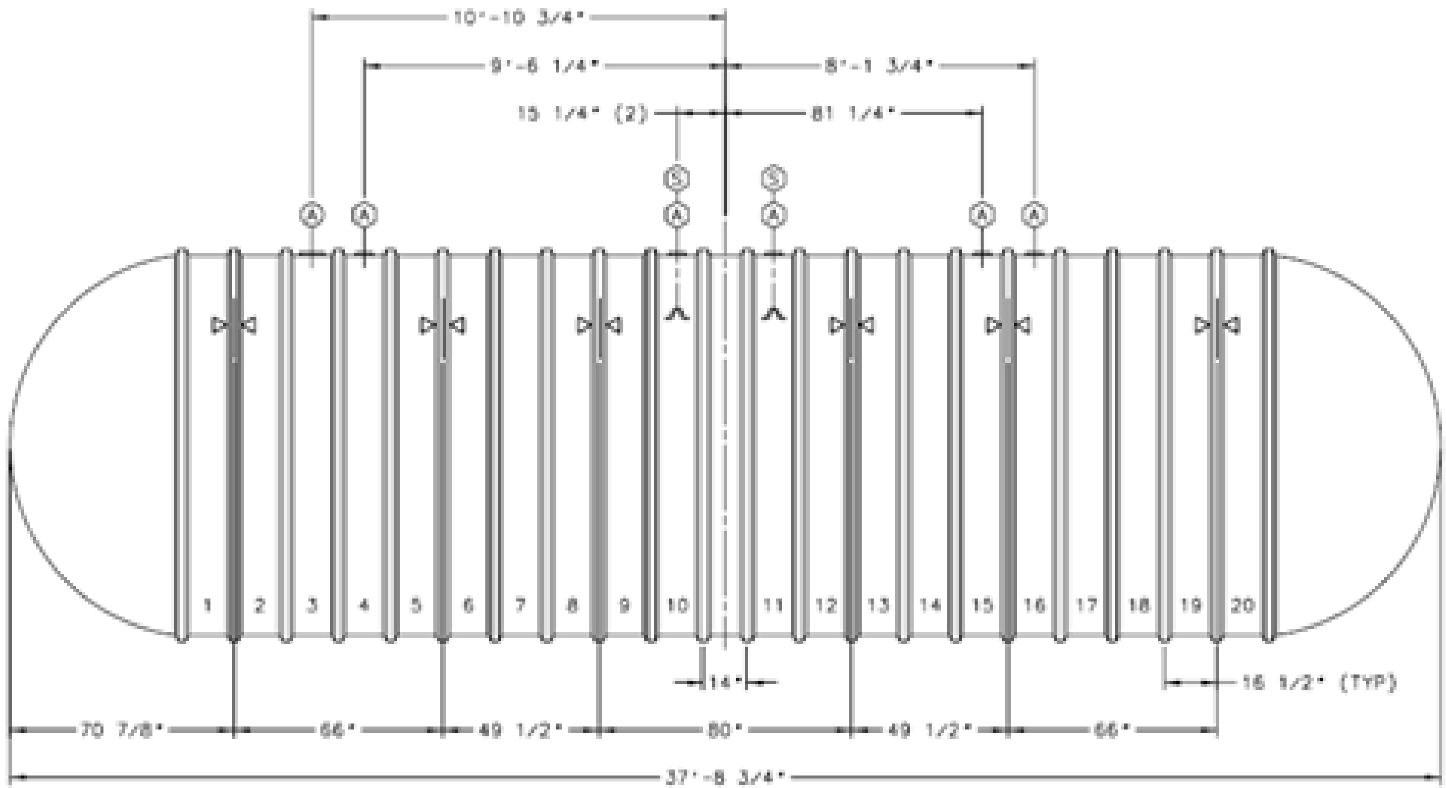
ASCE 7-10, Table 1.5-1, p. 2, Risk Category IV, Buildings or other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quanitity exceeds a theshold 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



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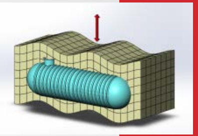
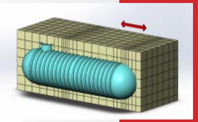
1) Axial Stress Due to P Waves and S Waves





Design Example

Set Values for Some Variables:

Tank inside diameter	Tank Capacity
$dia \equiv 120 \cdot in$	$Capacity \equiv 20000$
$R_T \equiv \frac{dia}{2}$	$R_T = 60 \cdot in$

These values are repeated in the calculations when they are first used for clarity. They are defined here in one place to simplify changes in the values of these particular variables.

$t_{sh} = 0.260 \cdot in$ Tank wall thickness, inches $\quad D_M := dia + t_{sh} \quad H := H_{bury} + \frac{dia}{2} \quad H = 10\,ft$

$L_{ss} = 27.4\,ft$ Tank straight shell length

$H_{bury} \equiv 5ft$ Tank burial depth (grade to top of tank).

$F_m := 0.9$ Long term axial tensile modulus retention factor

$F_{st} := 0.8$ Long term axial tensile strength retention factor

$\sigma_{Ault} := 10000 \cdot psi$ Tank wall ultimate axial tensile strength, psi. This value is known for tank wall by test data per CSI

$\sigma_{Hult} := 10000 \cdot psi$ Tank wall ultimate hoop tensile strength, psi. This value is known for tank wall by test data per CSI

$E_A := 900000 \cdot psi$ Axial tensile modulus, psi. This value is known for tank wall by test data per CSI

Calculations:

Shear and compression wave velocities

$\gamma_{soil} := 120 \cdot pcf$ Unit weight of soil, lb/ft^3 (crushed rock). Worse case compared to pea gravel.

$g = 32.17 \cdot \frac{ft}{s^2}$ Gravitational constant. Use 32.2 ft/sec^2 or 386.4 in/sec^2.

$\rho := \frac{\gamma_{soil}}{g} \quad \rho = 0.00018 \cdot lbf \cdot \frac{sec^2}{in^4}$ Unit mass of soil, $lb\text{-}sec^2/in^4$.

$G = (30000) \cdot psi$ Soil shear modulus at 5ft bury, psi. (crushed rock) $\quad k' := 0 .. last(G)$

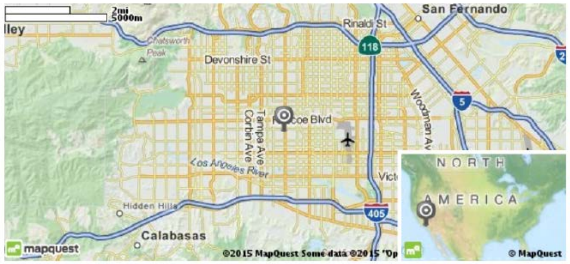
$C_s := \sqrt{\frac{G}{\rho}} \quad C_s = (1076) \cdot \frac{ft}{sec}$ Shear wave velocity, in/sec.

$\nu := 0.4$ Poisson's Ratio (Soil Mechanics by R. W. Lambe and R. V. Whitman, John Wiley & Sons, 1969)

$C_p := \left(2 \cdot \frac{1-\nu}{1-2 \cdot \nu}\right)^{0.5} \cdot C_s \quad C_p = (2636) \cdot \frac{ft}{sec}$ Compression wave velocity, in/sec.

Seismic Load

Report Title	Northridge Thu June 4, 2015 16:17:03 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.2131°N, 118.5369°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	IV (e.g. essential facilities)



http://earthquake.usgs.gps/us/application.php

$Latitude \equiv "36.23046\ N"$

$Longitude \equiv -118.5369$

$Site\_Class \equiv "D"$

$Loc \equiv "Northridge, CA"$

$S_S \equiv 190.5\%$

$S_L \equiv 61.4\%$

USGS-Provided Output

$S_S =$	1.905 g	$S_{MS} =$	1.905 g	$S_{DS} =$	1.270 g
$S_1 =$	0.614 g	$S_{M1} =$	0.922 g	$S_{D1} =$	0.614 g

Code $\quad Code := "ASCE\ 7\text{-}10"$

Location $\quad Loc = "Northridge, CA"$

Short Period $\quad S_S = 190.5 \cdot \%$

Long Period $\quad S_L = 61.4 \cdot \%$

Soil Profile Type used in Analysis $\quad Site\_Class = "D"$

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)

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".

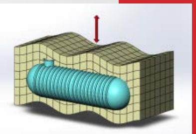
Design spectral response acceleration parameter at short periods

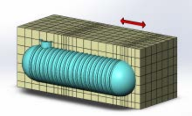
$S_{DS} = \frac{2}{3} \cdot S_{MS} = \frac{2}{3} \cdot (F_a \cdot S_S)$

Design spectral response acceleration parameter at a period of 1 sec

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \cdot (F_v \cdot S_1)$

1) Axial Stress Due to P Waves and S Waves





Design Example

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"

$$F'_a = \begin{pmatrix}1.6\\1.4\\1.2\\1.1\\1.0\end{pmatrix} \quad S'_S = \begin{pmatrix}0.25\\0.50\\0.75\\1.00\\1.25\end{pmatrix} \quad F'_v = \begin{pmatrix}2.4\\2.0\\1.8\\1.6\\1.5\end{pmatrix} \quad S'_L = \begin{pmatrix}0.10\\0.20\\0.30\\0.40\\0.50\end{pmatrix}$$

Determine F_a

$$F_a := \begin{cases} F'_{a_0} & \text{if } S_S \le S'_{S_0} \\ \text{linterp}(S'_S, F'_a, S_S) & \text{if } S'_{S_0} < S_S < S'_{S_{last(S'_S)}} \\ F'_{a_{last(F'_a)}} & \text{if } S_S \ge S'_{S_{last(S'_S)}} \end{cases}$$

$F_a = 1.00$

$S_S = 1.905$

Determine F_v

$S_L = 0.614$

$$F_v := \begin{cases} F'_{v_0} & \text{if } S_L \le S'_{L_0} \\ \text{linterp}(S'_L, F'_v, S_L) & \text{if } S'_{L_0} < S_L < S'_{L_{last(S'_L)}} \\ F'_{v_{last(F'_v)}} & \text{if } S_L \ge S'_{L_{last(S'_L)}} \end{cases}$$

$F_v = 1.50$

S_{DS} and S_{D1} per Section 11.4.4 (ASCE 7-10, page 65)

Design spectral response acceleration parameter at short periods

$S_{DS} := \frac{2}{3}\cdot\overrightarrow{(F_a \cdot S_S)}$ $\quad S_{DS} = 1.270$

Design spectral response acceleration parameter at a period of 1 sec

$S_{DL} := \frac{2}{3}\cdot\overrightarrow{(F_v \cdot S_L)}$ $\quad S_{DL} = 0.614$ $\quad F_v = 1.50$

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

For Nonbuilding Structures

$V_{Tank} := 0.3\cdot S_{DS}\cdot I_e\cdot 1lbf$ $\quad V_{Tank} = 0.571\cdot lbf$ **IBC/ASCE** $\quad W_p := 1\cdot lbf$

$V_{Tank} = 0.571\cdot lbf$ $\quad M := \frac{W_p}{g}$

$a_p := \frac{V_{Tank}}{M}$ $\quad a_p = 18.387\cdot\frac{ft}{s^2}$ $\quad V_{Tank} = 0.571\cdot lbf$ $\quad \frac{a_p}{g} = 57\cdot\%$

Seismic Stresses Due to Wave Propagation for No Slip Condition

$E_A = 900000\cdot psi$ — Axial tensile modulus of elasticity

$t_{sh} = 0.260\cdot in$ — Tank wall thickness, inches

$R_T = 5\,ft$ — Mean tank wall radius, in.

$\sigma_{bs} := \frac{E_A\cdot R_T\cdot a_p\cdot F_m}{C_s^2}$ $\quad \sigma_{bs} = (64)\cdot psi$ — Axial bending stress due to shear waves, psi.

$V_p := \frac{a_p\cdot 48\cdot\frac{in}{(sec)}}{g}$ Use this value unless otherwise specified by costumer — Maximum ground particle velocity due to shear waves, in/sec (see Yeh, "Seismic Anaylsis of Slender Buried Beams", Bulletins of the Seismological Society of America, Vol 64, No 5, pp 1551-1562)

$V_p = 27.43\cdot\frac{in}{sec}$ — Maximum ground particle velocity due to compression waves, in/sec

$\sigma_{ac} := \frac{E_A\cdot V_p\cdot F_m}{C_p}$ $\quad \sigma_{ac} = (702)\cdot psi$ — Axial tensile stress due to compression waves, psi.

$\sigma_{as} := \frac{E_A\cdot V_p\cdot F_m}{2\cdot C_s}$ $\quad \sigma_{as} = (860)\cdot psi$ — Axial tensile stress due to shear waves, psi.

$\sigma_{bc} := \frac{0.385\cdot E_A\cdot R_T\cdot a_p\cdot F_m}{C_p^2}$ $\quad \sigma_{bc} = (4.1)\cdot psi$ — Axial bending stress due to compression waves, psi.

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).

$\sigma_{A'} := \left[\left(\sigma_{as}^2 + \sigma_{bs}^2\right)^{0.5} + \left(\sigma_{ac}^2 + \sigma_{bc}^2\right)^{0.5}\right]$ — Combined axial stress due to seismic event, psi. Include long term property loss.

$\sigma_{A'} = (1565)\cdot psi$

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

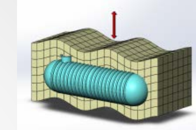
If the tank length L is less than or equal to 2*lm then the seismic design stress due to wave propagation is controlled by "Slippage". Calculate the value of lm.

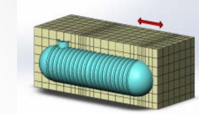
$D_M = 120\cdot in$ — Tank mean diameter

$\mu = 0.49$ — Coefficient of friction between tank and soil (see any soils reference)

$k_o = 0.7$ — Coefficient of lateral soil pressure, use 0.7 for ribbed shell wall (see any soils reference on soil rest pressure)

$H = 10\,ft$ — Distance from grade to tank centerline. Worst case (max σ_A) is at deepest bury of 7ft from grade to tank top.





Design Example

$P_r := \left(\frac{1+k_o}{2}\right)\cdot\gamma_{soil}\cdot H$ $\quad P_r = 7.1\cdot psi$ $\quad$ Average radial soil pressure on tank

$f := \pi\cdot dia\cdot P_r\cdot\mu$ $\quad f = 1307\cdot\frac{lbf}{in}$ $\quad$ Frictional force per unit length between the soil-tank interface.

$A := \pi\cdot D_M\cdot t_{sh}$ $\quad A = 98.2\cdot in^2$ $\quad$ Cross-sectional area of the tank shell wall

$P_r = 7.1\cdot psi$ $\quad H_{bury} = 5\,ft$ $\quad$ Average radial soil pressure on tank

$f = 1307\cdot\frac{lbf}{in}$ $\quad H_{bury} = 5\,ft$ $\quad$ Frictional force per unit length between the soil-tank interface.

$\varepsilon_m := \frac{\sigma_{A'}}{E_A\cdot F_m}$ $\quad \varepsilon_m = (0.001932)\cdot\frac{in}{in}$ $\quad H_{bury} = 5\,ft$ $\quad$ Maximum long term soil strain

$lm := \frac{\varepsilon_m\cdot A}{f}\cdot E_A\cdot F_m$ $\quad lm = (118)\cdot in$ $\quad H_{bury} = 5\,ft$ $\quad$ Maximum long term slippage length

If the tank length L is less than 2*lm then the seismic design stress due to wave propagation would be:

$\sigma_{A'} = (1565)\cdot psi$ $\quad$ Only controls when L is less than 2*lm

where: $\quad L_{ss} = 328.5\cdot in$

$\sigma_A := \frac{f\cdot L_{ss}}{2\cdot A}$ $\quad \sigma_A = 2185\cdot psi$ $\quad 2\cdot lm = (235)\cdot in$

$\sigma_A = 2185\cdot psi$ $\quad H_{bury} = 5\,ft$ $\quad$ This is the controlling stress if L_j is $< 2\cdot lm$

$Control_m := if(L_{ss} < 2\cdot lm_m, \text{"slip"}, \text{"no slip"})$ $\quad Control = (\text{"no slip"})$ $\quad H_{bury} = 5\,ft$

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

Combining the stress equations gives:

$\sigma_{AA_m} := if(L_{ss} < 2\cdot lm_m, \sigma_A, \sigma_{A'})$ $\quad \sigma_{AA} = (\{1,1\})\cdot psi$

$$\sigma_{A_{k'}} := if\left[L_{ss} < 2\cdot lm_{k'}, \frac{f\cdot L_{ss}}{2\cdot A}, \left[\left[(\sigma_{as_{k'}})^2 + (\sigma_{bs_{k'}})^2\right]^{0.5} + \left[(\sigma_{ac_{k'}})^2 + (\sigma_{bc_{k'}})^2\right]^{0.5}\right]\right]$$

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

Note: Under a "No Slippage" condition, the σ_A stress does not vary by tank length

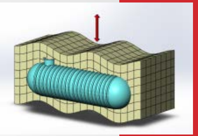
$L_{ss} = 328.5\cdot in$ $\quad 2\cdot lm = (235)\cdot in$ $\quad \sigma_A = (1565)\cdot psi$

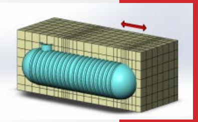
Seismic Stress due to Dynamic Soil Pressure

$K_c := 4.0$ $\quad K_s := 5.0$ $\quad$ Dynamic stress concentration factors, see Newmark & Rosenblueth, Fundamentals of Earthquake Engineering.

$\sigma_{\theta c} := K_c\cdot\rho\cdot C_p\cdot V_p$ $\quad \sigma_{\theta c} = (624)\cdot psi$ $\quad$ Maximum hoop stress induced by normal stress

$\sigma_{\theta s} := K_s\cdot\rho\cdot C_s\cdot V_s$ $\quad \sigma_{\theta s} = (319)\cdot psi$ $\quad$ Maximum hoop stress induced by shear stress





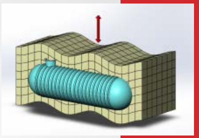
Summary of Axial Stresses vs. Shear Modulus (G_m)

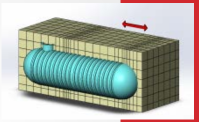
G (psi)	1625*	5000	30000	1625**	5000**	30000**
Burial Depth	**5 ft.**	**5 ft.**	**5 ft.**	**5 ft.**	**5 ft.**	**5 ft.**
a_p	57.1% g	57.1% g	57.1% g	57.1% g	57.1% g	57.1% g
V_p	48 in/s	48 in/s	48 in/s	9.8 in/s	9.8 in/s	9.8 in/s
L_{ss}	27.37 ft.	27.37 ft.	27.37 ft.	27.37 ft.	27.37 ft.	27.37 ft.
$2lm$	86.5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

- 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.

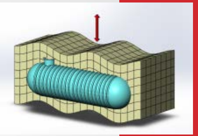
*For comparison to Xerxes method (slide 110)

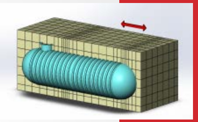
**Tripartite plot for Northridge Earthquake used for velocity





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

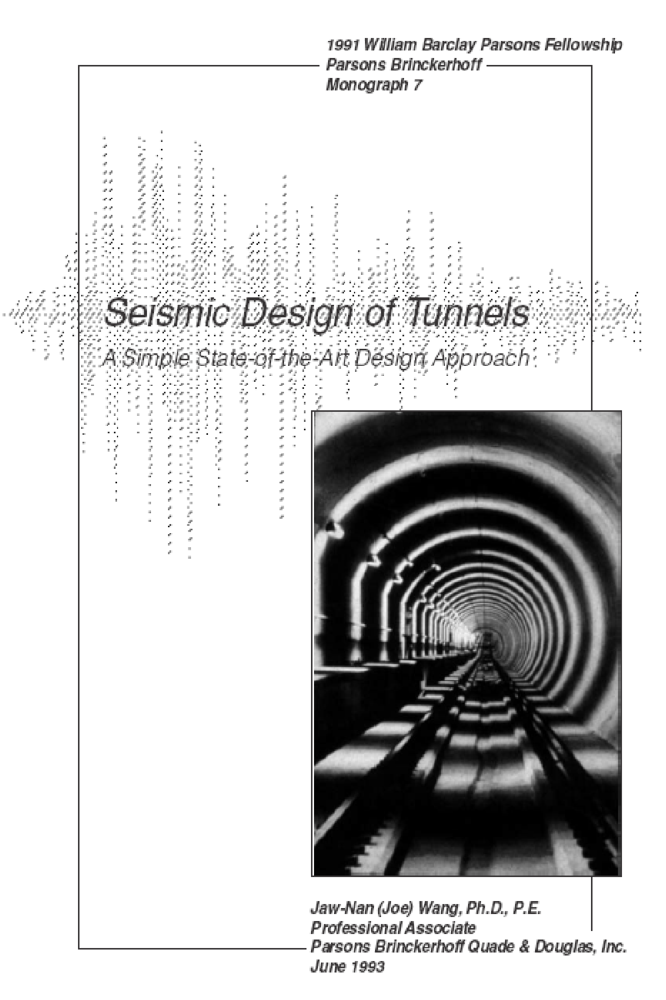




Ovaling/Racking Method

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

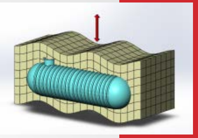
"Seismic Design of Tunnels" Monograph 7

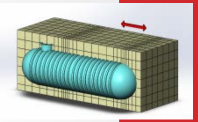

1991 William Barclay Parsons Fellowship
Parsons Brinckerhoff
Monograph 7

Seismic Design of Tunnels

A Simple State-of-the-Art Design Approach

Jaw-Nan (Joe) Wang, Ph.D., P.E.
Professional Associate
Parsons Brinckerhoff Quade & Douglas, Inc.
June 1993

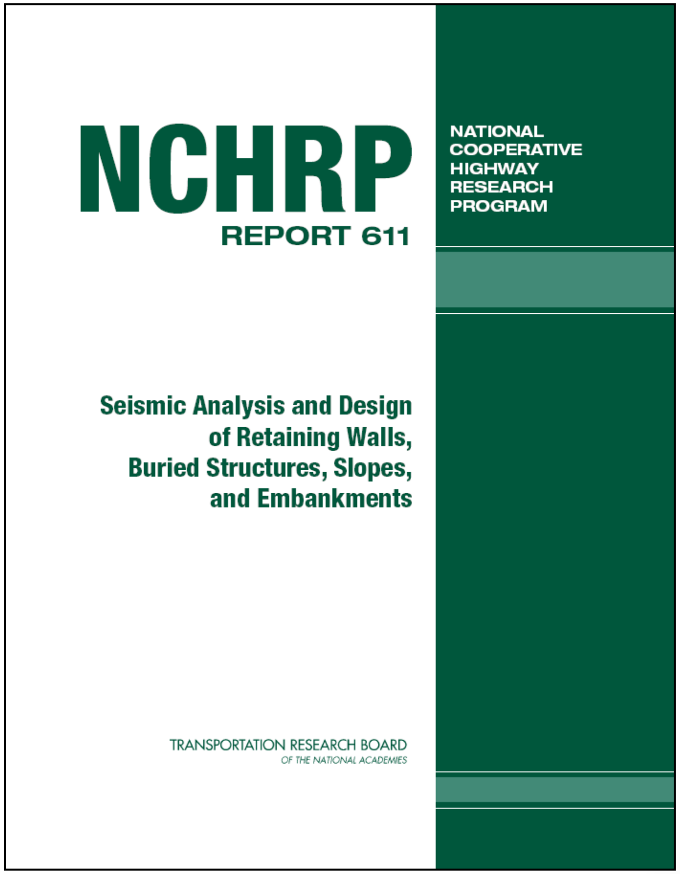




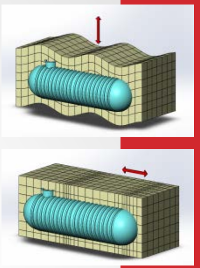
Ovaling/Racking Method

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

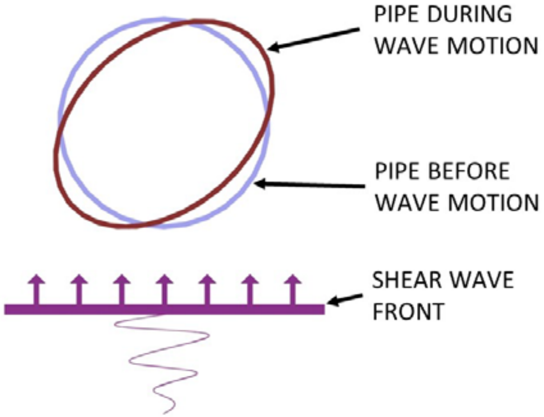


Can be downloaded at
http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_611.pdf

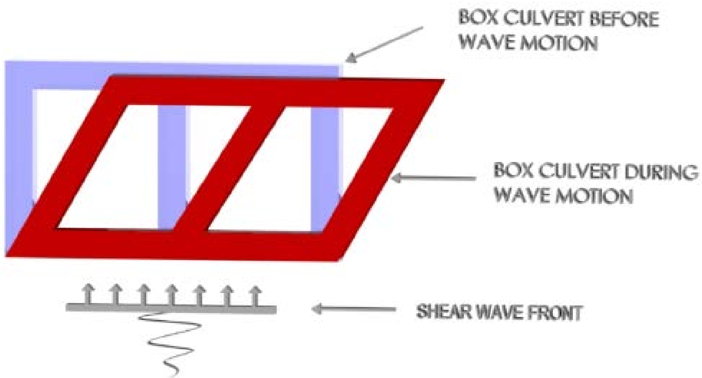


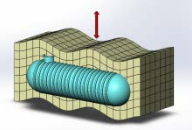
Discussion for:

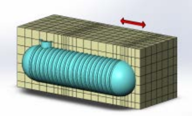
- **Circular Conduits and Tanks (Ovaling)**



- **Box culverts (Racking)**





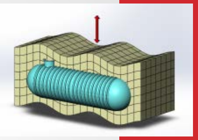


NCHRP Report 611, Chapter 9

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.



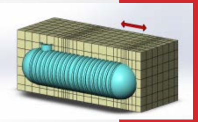
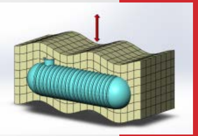


Table 9-1. Parameters used in the parametric analysis.

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

* Note: The Young's Modulus values used in this study are for parametric analysis purposes only.

2) Wang/NCHRP Method



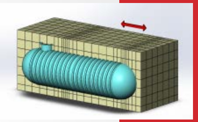


Table 9-2. Parametric Analysis Set 1—culvert lining properties (Reference Set).

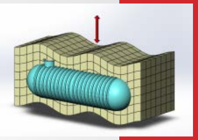
Culvert Properties	Rigid Culvert (Concrete Pipe)	Flexible Culvert (Corrugated Steel Pipe)
Culvert Diameter, ft	10	10
Young's Modulus, $E/(1-v^2)$, used in 2-D Plane Strain Condition, psi	4.0E+06	2.9E+07
Moment of Inertia I, ft^4/ft	0.025 ft^4/ft	0.00007256 ft^4/ft (=1.505 in^4/ft)
Sectional Area A, ft^2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

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

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

Cases Analyzed	Soil Cover H (feet)	Culvert Diameter d (feet)	Embedment Depth Ratio, H/d
Case 1	50	10	5
Case 2	30	10	3
Case 3	20	10	2
Case 4	10	10	1
Case 5	5	10	0.5
Case 6	2	10	0.2

2) Wang/NCHRP Method



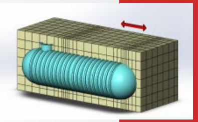


Table 9-4. Culvert lining compressibility and flexibility used in analysis.

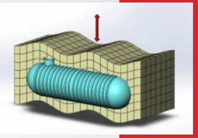
Properties	Rigid Culvert (Concrete Pipe)	Flexible Culvert (Corrugated Steel Pipe)
Compressibility Ratio, *C*	0.011	0.05
Flexibility Ratio, *F*	0.482	22.6

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

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

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.

2) Wang/NCHRP Method



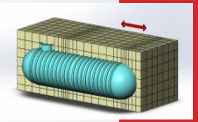
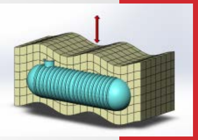


Table 9-6. Culvert diameter change—effect of interface slippage condition.

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

2) Wang/NCHRP Method



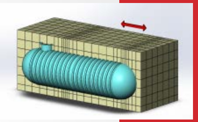


Table 9-8. Parametric analysis set 3—culvert lining properties.

Culvert Properties	Rigid Culvert (Concrete Pipe)	Flexible Culvert Aluminum CMP
Culvert Diameter, ft	10	10
Young's Modulus, $E/(1-v^2)$, psi	4.0E+06	1.0E+07
Moment of Inertia, ft^4/ft	0.2	0.00001168
Sectional Area, ft^2 per ft	1.333	0.01125
EI (lb-ft^2 per ft)	1.152E+08	1.682E+04
AE (lb, per ft)	7.678E+08	1.62E+07
Poisson's Ratio	0.3	0.3
Compressibility, C	0.005	0.256
Flexibility Ratio, F	0.060	411.7

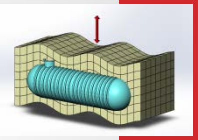
Note: Ground condition (firm ground with E_m = 3,000 psi, ν_m = 0.3).

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

Culvert Properties	Rigid Culvert (Concrete Pipe)	Flexible Culvert (Corrugated Steel Pipe)
Culvert Diameter, ft	5	5
Young's Modulus, $E/(1-v^2)$, psi	4.0E+06	2.9E+07
Moment of Inertia, ft^4/ft	0.025	0.00007256
Sectional Area, ft^2 per ft	0.67	0.02
EI (lb-ft^2 per ft)	1.44E+07	3.03E+05
AE (lb, per ft)	3.86E+08	8.35E+07
Poisson's Ratio	0.3	0.3
Compressibility, C	0.005	0.025
Flexibility Ratio, F	0.061	2.856

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

2) Wang/NCHRP Method



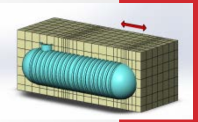


Table 9-9. Parametric analysis set 4—culvert lining properties.

Culvert Properties	Flexible Culvert (Corrugated HDPE)
Culvert Diameter, ft	5
Young's Modulus, $E/(1-v^2)$, psi	1.1E+05
Moment of Inertia, ft^4 per ft	0.0005787
Sectional Area, ft^2 per ft	0.0448
EI (lb-ft^2 per ft)	9.17E+03
AE (lb, per ft)	7.10E+05
Poisson's Ratio	0.45
Compressibility, C	2.927
Flexibility Ratio, F	94.424

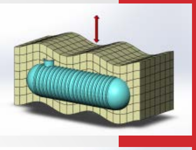
Note: Ground condition (firm ground with E_m = 3,000 psi, ν_m = 0.3).

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

Culvert Properties	Rigid Culvert (Concrete Pipe)	Flexible Culvert (Corrugated Steel Pipe)
Culvert Diameter, ft	10	10
Young's Modulus, $E/(1-v^2)$, psi	4.0E+06	2.9E+07
Moment of Inertia, ft^4/ft	0.025	0.00007256
Sectional Area, ft^2 per ft	0.67	0.02
EI (lb-ft^2 per ft)	1.44E+07	3.03E+05
AE (lb, per ft)	3.86E+08	8.35E+07
Poisson's Ratio	0.3	0.3
Compressibility, C	0.027	0.127
Flexibility Ratio, F	1.217	57.122

Note: ground condition (very stiff ground with E_m = 7,500 psi, ν_m = 0.3).

2) Wang/NCHRP Method



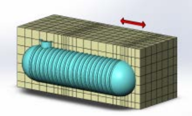


Table 9-11. Soil and structure parameters used in the analysis.

	Structural Configurations and Soil Properties
Case 1	10' x 10' Square Box, in Firm Ground (E_m = 3,000 psi, ν_m = 0.3)
Case 2	10' x 10' Square Box, in Very Stiff Ground (E_m = 7,500 psi, ν_m = 0.3)
Case 3	10' x 20' Rectangular Box, in Firm Ground (E_m = 3,000 psi, ν_m = 0.3)
Case 4	10' x 10' Square 3-Sided, in Very Stiff Ground (E_m = 7,500 psi, ν_m = 0.3)
Case 5	10' x 20' Rectangular 3-Sided, in Very Stiff Ground (E_m = 7,500 psi, ν_m = 0.3)

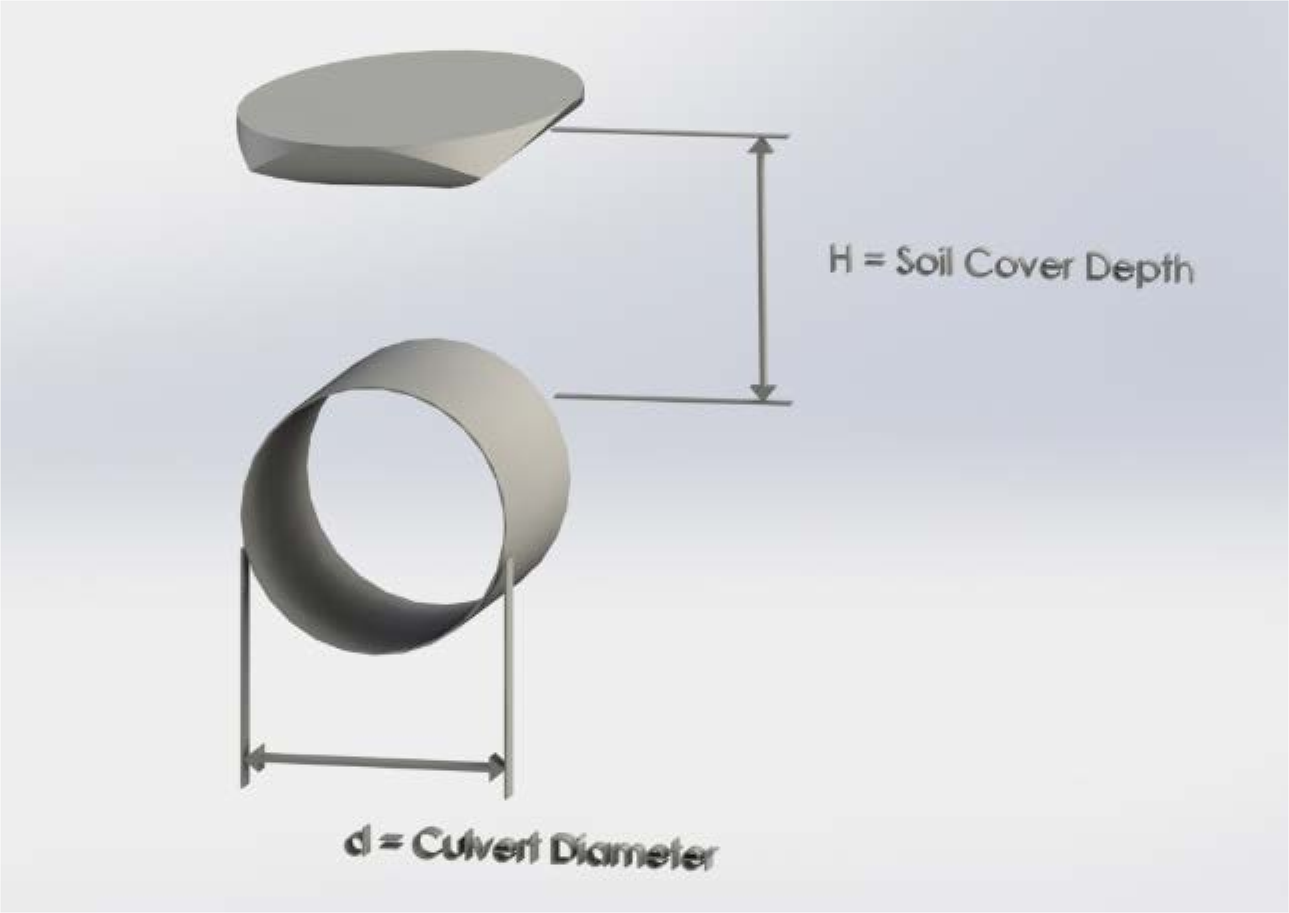
Note: For each case, the effects of culvert embedment depth (of 50 feet, 30 feet, 20 feet, 10 feet, and 5 feet, measured from ground surface to top of the culvert roof) were studied.

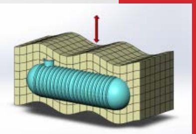
Table 9-12. Racking stiffness of culverts and flexibility ratios.

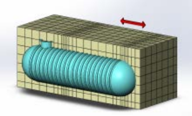
	Structural Racking Stiffness K_s (kips/ft)	Flexibility Ratio F_{REC}
Case 1	172	0.97
Case 2	172	2.4
Case 3	115	2.9
Case 4	57	7.3
Case 5	43	19.3

Embedment Depth Ratio (H/d)

H/d is used in studies by Wang.







Most FRP and Steel UST's for gasoline storage have H/d ratio from 0.26≤H/d≤1.75

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

*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).

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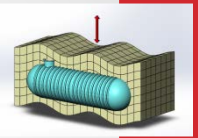
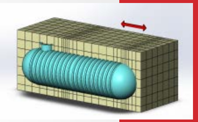
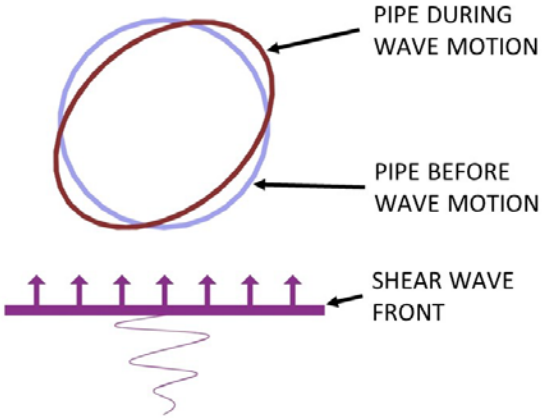
2) Wang/NCHRP Method



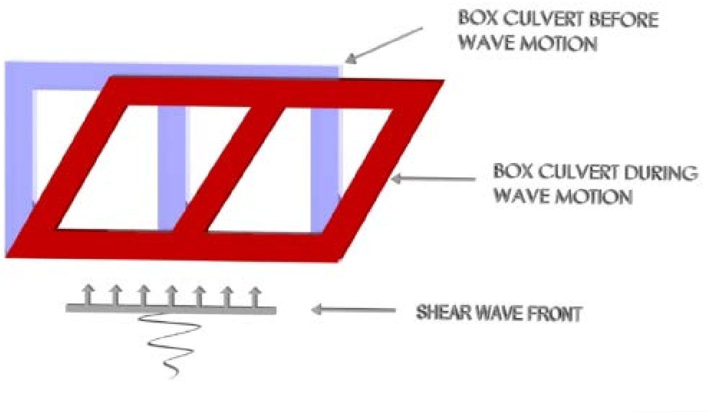


Wang shows that the results are almost unchanged for H/d > 1.0 and for H/d < 1.0 results change gradually.

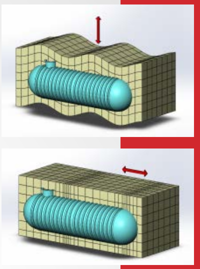
New method accounts for hoop thrust stress and diametrical bending stress for circular tanks and for racking for rectangular culverts.



Ovaling



Racking



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

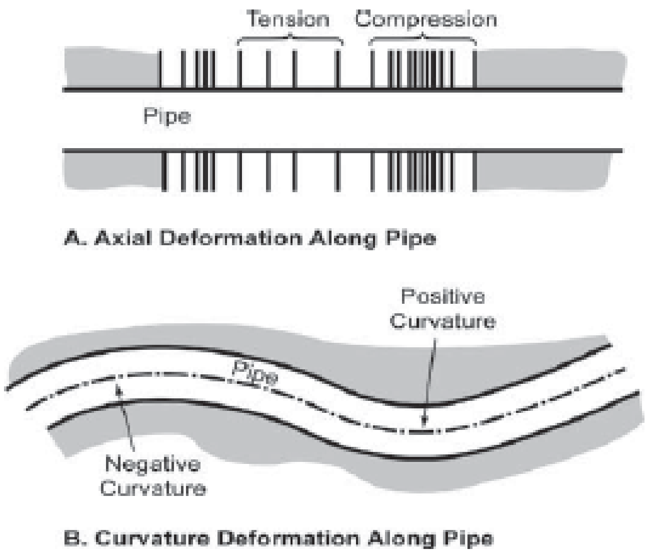


Figure 9-1. Axial and curvature deformations.

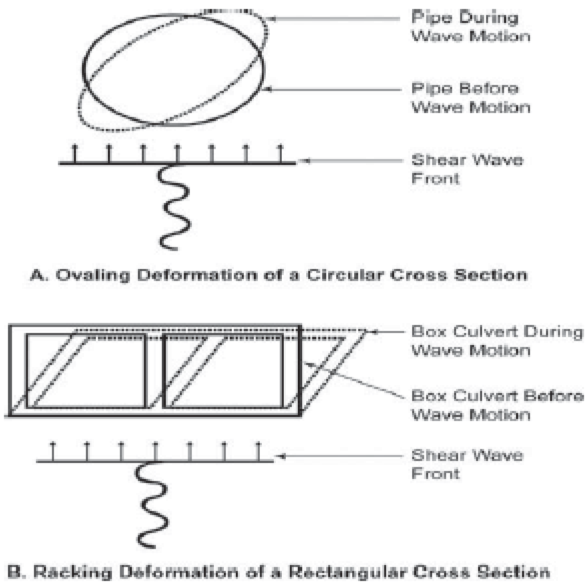
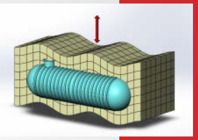
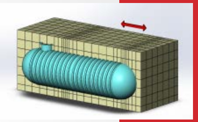


Figure 9-2. Ovaling and racking deformations.

*Numbers in parenthesis refer to appropriate section in Report 611





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

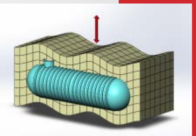
$$PS = \frac{P}{\Delta \cdot w} = \frac{EI}{0.149R^3 \cdot w} \text{ (psi/in)}$$

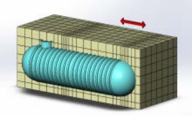
where w = width of section tested

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)

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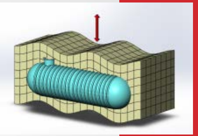
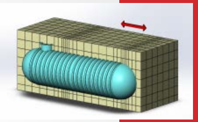
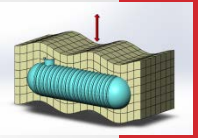




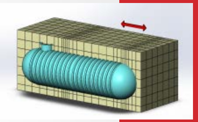
General Effects of Earthquakes and Potential “New” Failure Models

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

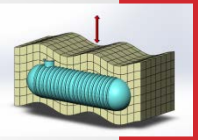
- Two different types of waves with two sub types
- Body Waves – within Earth’s crust
  - Longitudinal compressional: (P) *Push waves*
  - Transverse and shear: (S) *Shake waves*
  - Travel in any direction

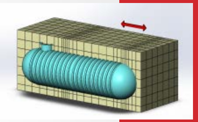


General Effects of Earthquakes and Potential "New" Failure Models



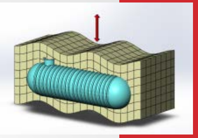
- Surface Waves – along Earth's surface
  - Rayleigh waves cause the ground to shake in an elliptical motion, with no transverse motion. *Ref. earthquake.usgs.gov*
  - Love waves have a horizontal motion that is transverse to the direction the wave is traveling. *Ref. earthquake.usgs.gov*
- **Unified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

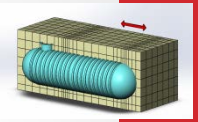




Rigid Culverts and Pipes

- 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





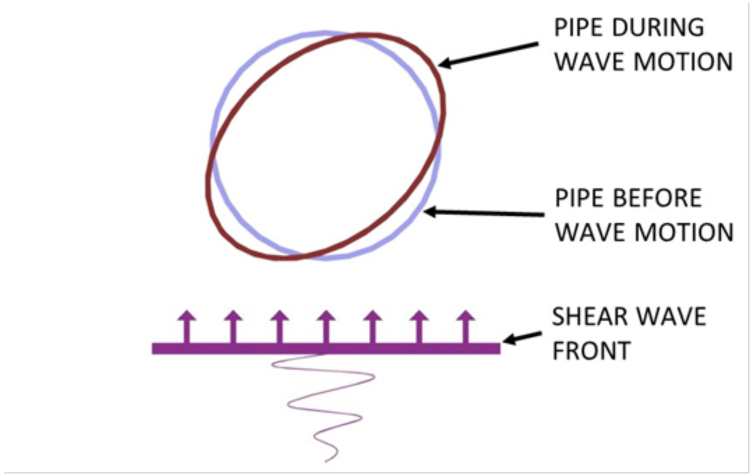
Principle Types of Transient Ground Deformations (TDA):

- Axial
- Curvature
- Ovaling or Racking

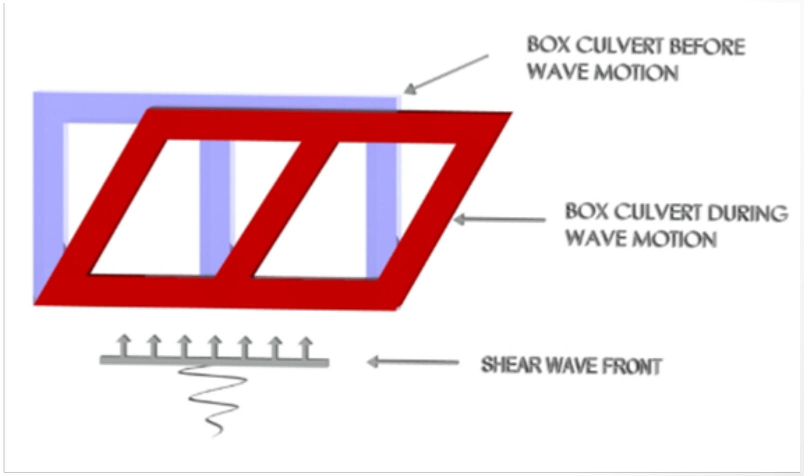
<u>Some Terminology</u>

PDA Peak Ground Deformation

TDA Transient Ground Deformation



Ovaling



Racking

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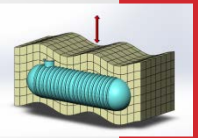
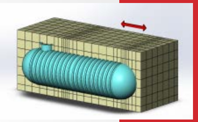
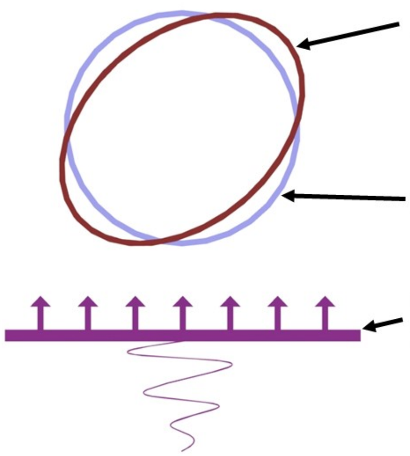
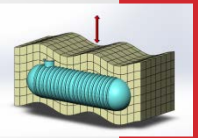


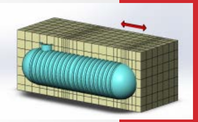


Focus of Chapter 9 (NCHRP 611)

- 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





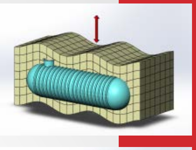


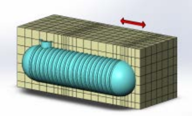
Focus of Chapter 9 (NCHRP 611)

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 <u>is much higher</u> in ***soft weak soil***

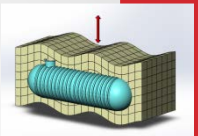
Evaluated using under two-dimensional plain strain condition per K. Ishihara

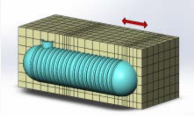




Ground Failure Modes (Ground Instability)

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



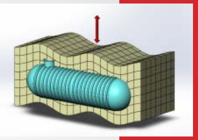


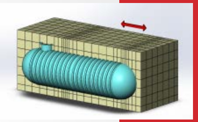
Permanent Deformation

- 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.



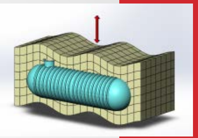


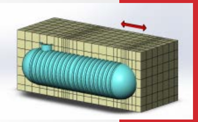
General Methodology

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)

http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf **(28)**

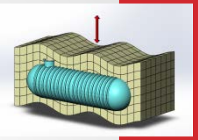
a) Seismic force for rigid non-building structures

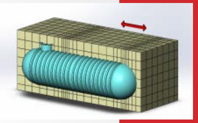
$V_{Tank} = 0.3 \cdot S_{DS} \cdot I_e \cdot 1.lbf$ (ASCE 7−10† 15.4-5)

S_{DS} and S_{D1} — Ref 11.4.4 ASCE 7-10 p. 65 (1)

$I_e = 1.5$ — 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

$$M = \frac{1lbf}{g}$$

c) $$\mathrm{V_P} = \frac{a_P \cdot 48^{\mathrm{in}}/_{\mathrm{sec}}}{\mathbf{g}}$$

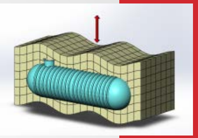
(Ref Earthquake Spectra: Northridge Earthquake of January 17, 1994

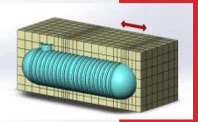
Reconnaissance report-Vol 1 (15) or use site specific values if available or use response spectra)

OR

$$\mathrm{V_P} = a_p \cdot \frac{25\frac{cm}{s}}{g}$$

From Northridge Response Spectra (see slides 18 and 19)





Determine Seismic Demands (9.4)

Step 1: Maximum Free Field Strain

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

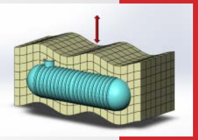
where C_{SE} = effective shear wave velocity

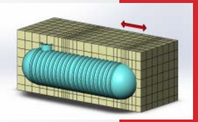
Shear Wave Velocity

$$V_s = \frac{V_P}{\sqrt{\frac{(1-2\nu_m)}{2(1-\nu_m)}}}$$

where V_P = velocity from compression wave

ν_m = Poisson's ratio for surrounding soil





Determine Seismic Demands

Step 1 continued:

Alternate

$$\gamma_{max} = \tau_{max} / G_m$$

$$\tau_{max} = \left(\frac{PGA}{g}\right) \sigma_v R_d$$ PGA Particle Ground Acceleration (a_p)

$$\sigma_v = \gamma_t \cdot (H_{TOP} + d)$$ (Total over burden pressure at invert)

$$R_d = 1.0 - .00233 \cdot z$$ *z<30 ft.*

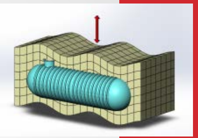
G_m = effective shear modulus (From tables in K. Ishihara text or other appropriate source)

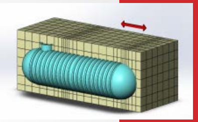
or use **"SHAKE"** program analysis

$G_m = G_o$ for these calculations

(Reduce to include strain reduction per Ishihara (11))

Determine Seismic Demands





Step 2: Maximum free field diameter change

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

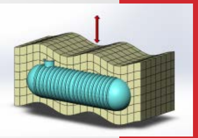
If hole cavity in soil is considered (yes for this case)

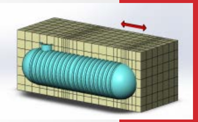
$$\Delta_{D_{EQ}} = \pm 2\gamma_{max}(1 - \nu_m)D$$

ν_m = Poisson's ratio for surrounding soil

D = diameter

Good for flexible conduits in competent ground.





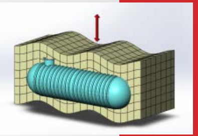
Alternate γ_{max} based on FEA studies by Wang (1993 and NCHRP Report 611 2008)

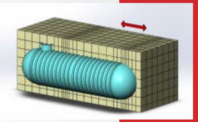
Basis: define relative stiffness between circular lining and surrounding ground.

Compressibility Ratio C

$$C = \frac{E_m(1-\nu_m{}^2)R_T}{E_A \cdot t_{eff}(1+\nu)(1-2\nu)} = \underset{\text{Geometry}}{\left(\frac{R_T}{t_{eff}}\right)} \underbrace{\left(\frac{E_m}{E_A}\right)\left(\frac{1-\nu_m{}^2}{(1+\nu)(1-2\nu)}\right)}_{\text{Stiffness Ratio for Tank/Soil}}$$

(Definitions on next slide)





Alternate γ_{max} based on FEA studies by Wang (1993 and NCHRP Report 611 2008)

Flexibility Ratio F

$$F = \frac{E_m(1-\nu_m{}^2)R_T{}^3}{6EI\ (1+\nu)} = \underbrace{\left(\frac{2R_T{}^3}{t_{eff}{}^3}\right)}_{\text{Geometry}} \underbrace{\left(\frac{E_m}{E}\right)\left(\frac{1-\nu_m{}^2}{(1+\nu)}\right)}_{\text{Stiffness Ratio for Tank/Soil}}$$

where EI = flexural rigidity of pipe/tank

ν = Poisson's ratio of pipe/tank

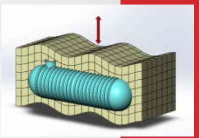
E_m: strain compatible elastic modulus of surrounding soil (soil report or estimate from available literature) e.g. K. Ishihara, etc. (11)

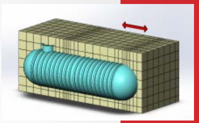
ν_m = Poisson's ratio of surrounding soil

R_T = tank or pipe radius

Rigid Ring F<1

Flexible Ring F>1





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

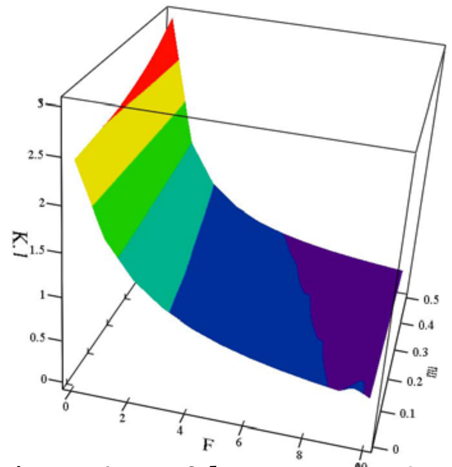
Change in Diameter

$$\Delta D_{EQ} = \pm \frac{1}{3}\left(k_1 F_{\gamma max} D\right)$$

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

Max hoop thrust

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

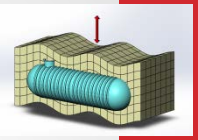


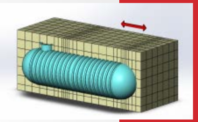
(3D Plot of k_1 vs. F and ν_m)

Max Hoop Bending

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

$$= \mathrm{R} \cdot T_{max}$$





No Slip Condition

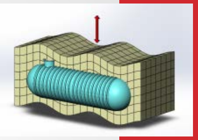
$$T_{max} = k_2 \left[\frac{E_m}{2 \cdot (1 - 2\nu_m)} \right] R\gamma_{max}$$

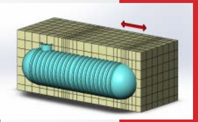
where:

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

$$C_T[-8\nu_m + 6\nu_m{}^2 + 6 - 8\nu_m]$$

2) Wang/NCHRP Method

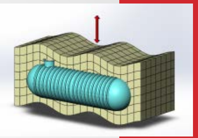


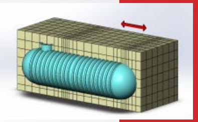


"In most cases the condition at the interface is between slip and no slip," p. 111. According to Fahimifar and Vakilzadeh in *Numerical and Analytical Solutions for Ovaling Deformation in Circular Tunnels Under Seismic Loading* *(8)*,

> "Note that no solution is developed for calculating diametric strain and maximum moment under no-slip condition. It is recommended that the solutions for fullslip 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.

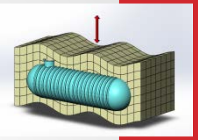


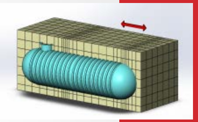


Summary for Horizontal Seismic Stresses per Wang/NCHRP Method for Hoop Load

G	1625**	5000	10000	30000
E	**4549**	**14000**	**28000**	**84000***
$\gamma\%$	.913%	.52 in/m	0.368%	.212%
Δ_D	$\sim 1\,{}^{5}/_{16}$ in	$\sim {}^{3}/_{4}$ in	$\sim {}^{1}/_{2}$ in (+)	$\sim {}^{5}/_{16}$ in
σ_T	4	2	1.5	1.0
σ_m	623	356	252	145
σ_{Total}	627	358	254	146

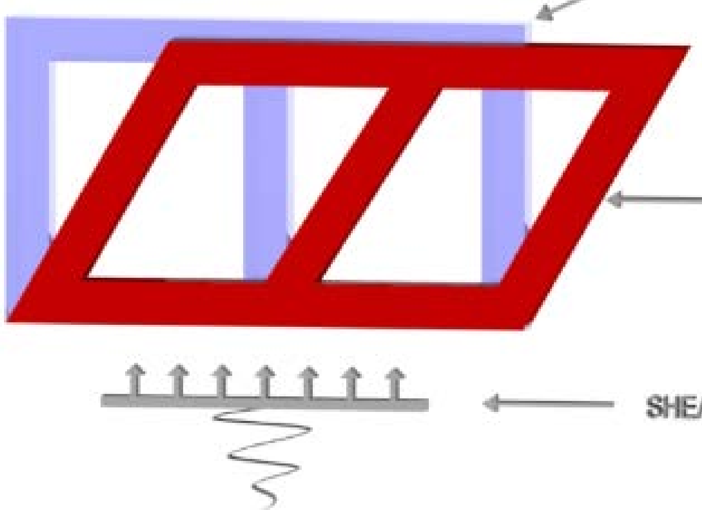
All stresses and moduli values are in psi.
*E = 2(1+v)·G. Max value ~29000 per Bowles (6)
v = 0.4
**Xerxes Patent value corresponds with E = 4549 psi



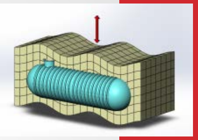


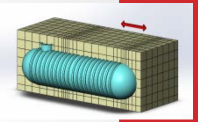
Racking for Rectangular Conduits (9.5.2)

Racking: differential sideways movements between top and bottom.



Results in differential inertial strain.





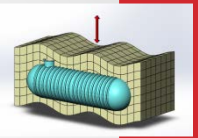
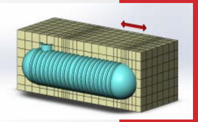
Method for Rectangular Structure

Step 1: Estimate γ_{max} - free field ground strain

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

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

$$V_s = \frac{V_P}{\sqrt{\frac{(1-2\nu_m)}{2(1-\nu_m)}}}$$ shear wave velocity

Method for Rectangular Structure

OR

$$\gamma_{max} = \frac{\tau_{max}}{G_m}$$

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

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

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

$R_d = 1.0 - 0.00233 \cdot z$ z<30 ft.*

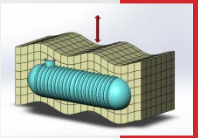
G_m = effective shear modulus

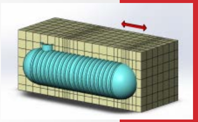
Determine differential free-field relative displacements $\Delta_{free-field}$ at top and bottom elevations.

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

$H = H_{Top}$ and H_{Bottom}

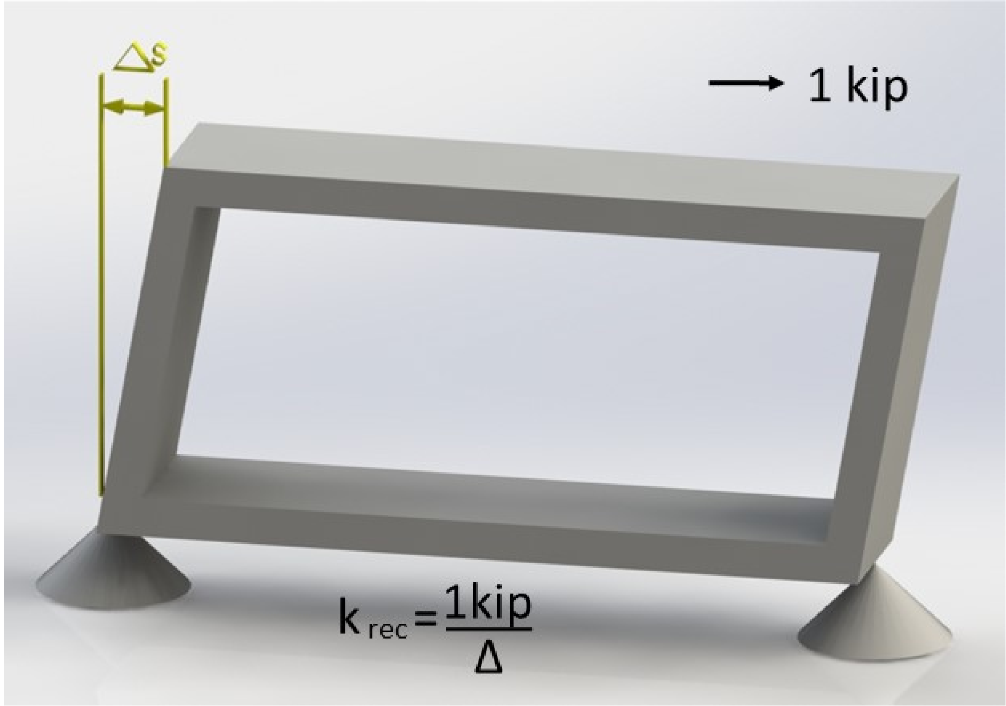
*For design > 30 ft. refer to Report 611 (4)



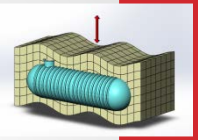


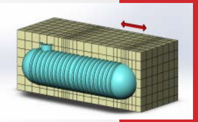
Method for Rectangular Structure

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.



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

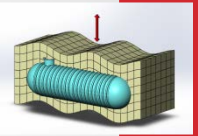
$k_{REC} =$ racking stiffness $= \frac{P(1kip)}{\Delta_{TOP}}\left(\frac{kip}{in}\right)$

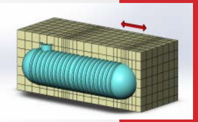
Step 4: Racking Ratio

$$R_{REC} = \frac{2F_{REC}}{1 + F_{REC}}$$

Ratio of actual racking to free-field racking.

Method for Rectangular Structure





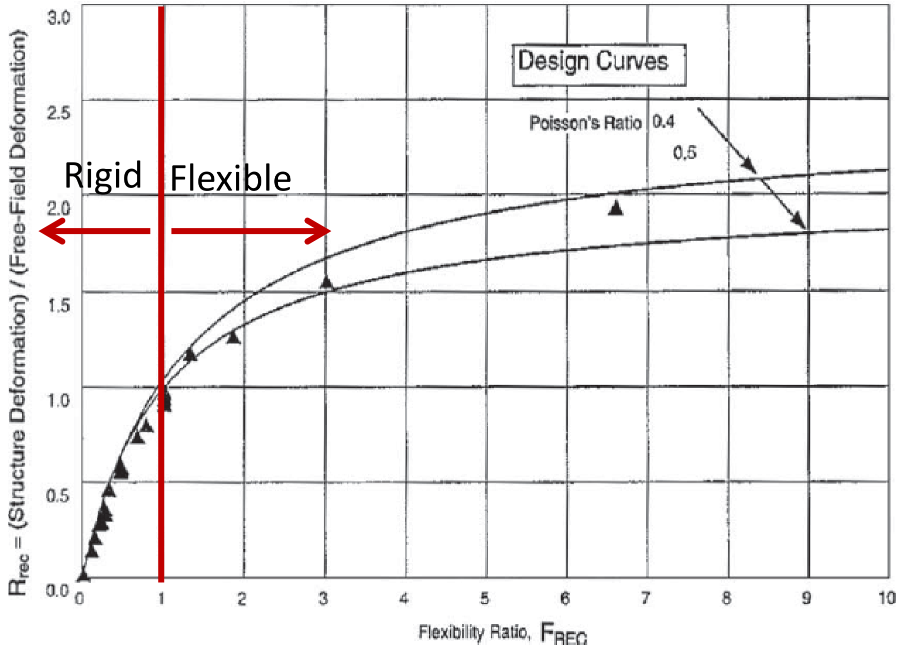
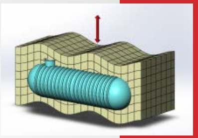
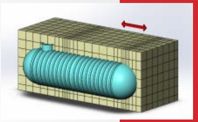


Figure 9-9. Racking ratio between structure and free-field.

F = 1	Ground and structure have same distortion
F→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





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

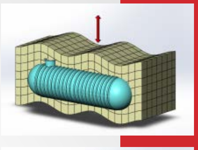
$$\left.\begin{array}{l} M = \text{moment} \\ T = \text{thrust} \\ V = \text{shear} \end{array}\right\} \text{By imposing } \Delta_R \text{ on the structure.}$$

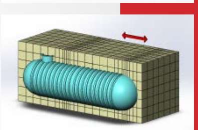
$$M = M_{unit\ load} \cdot \frac{\Delta_R}{\Delta_{unit\ load}}$$

$$T = T_{unit\ load} \cdot \frac{\Delta_R}{\Delta_{unit\ load}}$$

$$V = V_{unit\ load} \cdot \frac{\Delta_R}{\Delta_{unit\ load}}$$

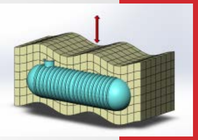
2) Wang/NCHRP Method

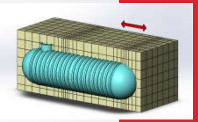




Seismic effects must be added to other load cases to obtain total stress normal load effects

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.

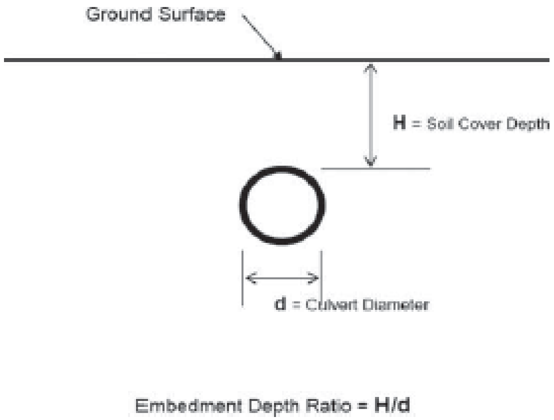


Figure 9-16. Definition of embedment depth ratio.

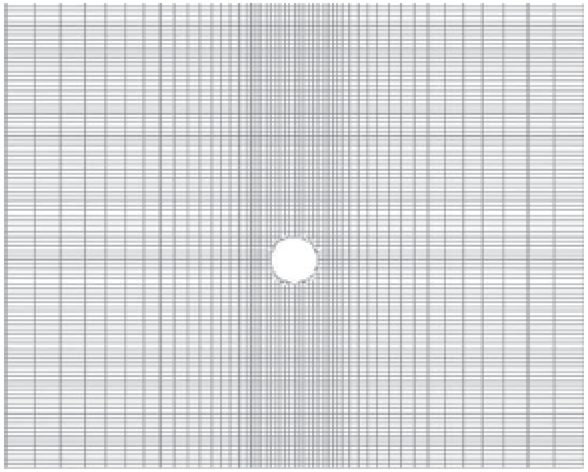


Figure 9-11. Case 1 finite difference mesh (soil cover = 50 feet).

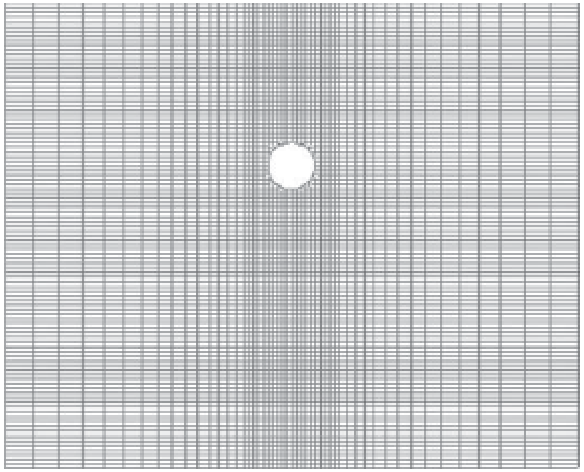
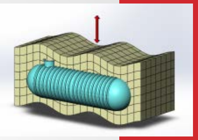
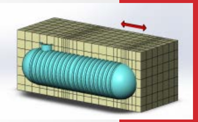


Figure 9-12. Case 2 finite difference mesh (soil cover = 30 feet).

FEA Model





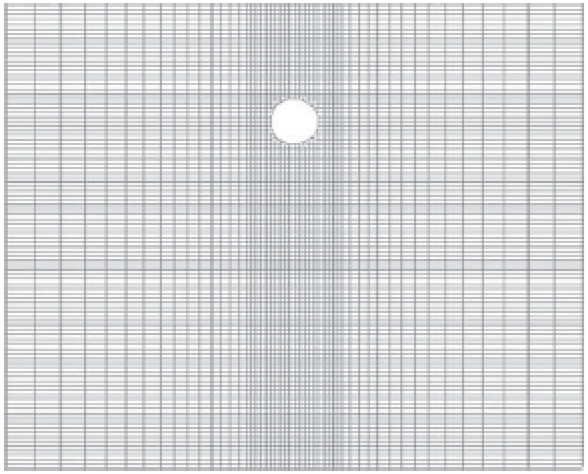


Figure 9-13. Case 3 finite difference mesh (soil cover = 20 feet).

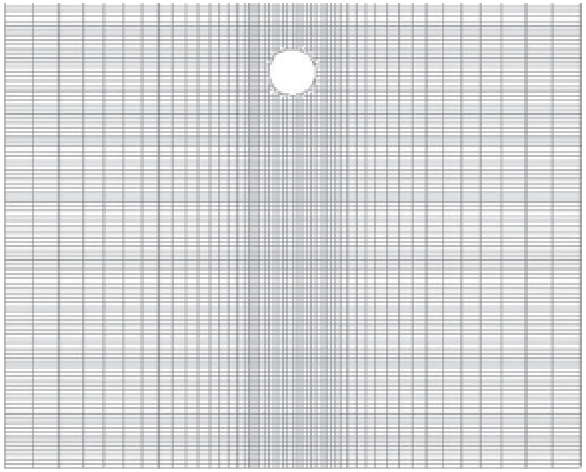


Figure 9-14. Case 4 finite difference mesh (soil cover = 10 feet).

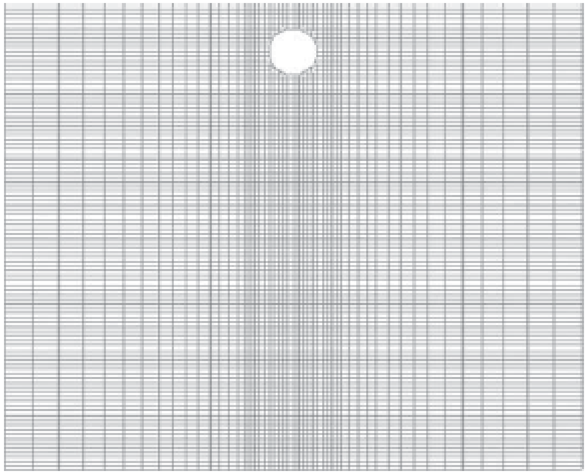
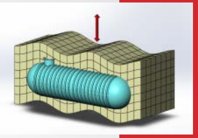
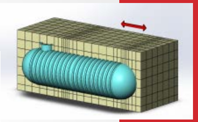


Figure 9-15. Case 5 finite difference mesh (soil cover = 5 feet).

2) Wang/NCHRP Method

FEA Model





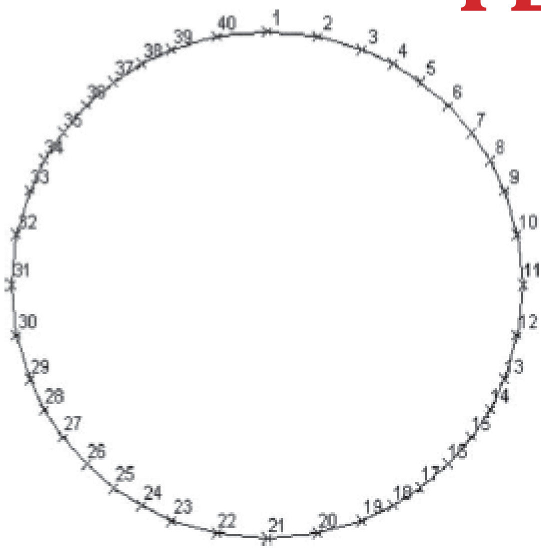


Figure 9-17. Culvert beam element number.

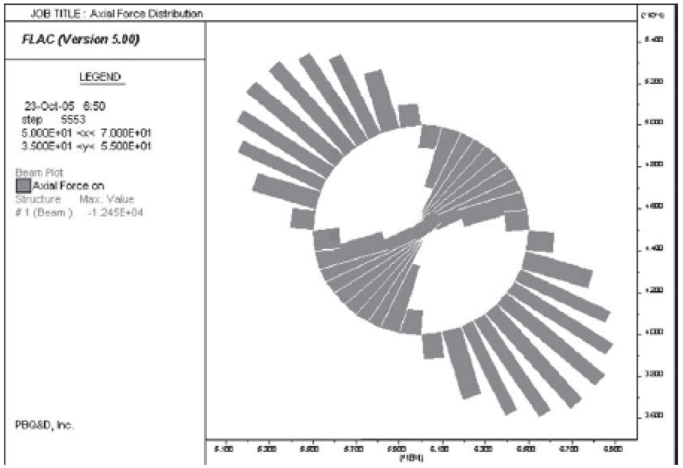


Figure 9-19. Culvert lining thrust/hoop force distribution (for flexible culvert in Set 1, Case 1 geometry).

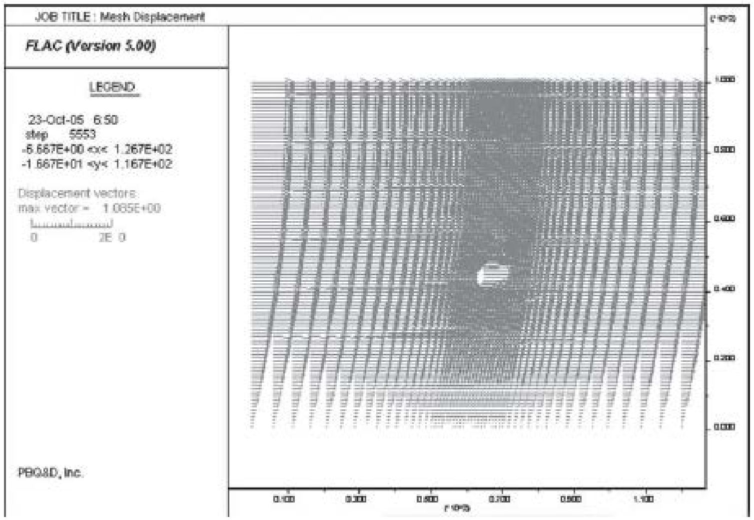


Figure 9-18. Soil deformations subjected to pseudo lateral acceleration of 0.3g.

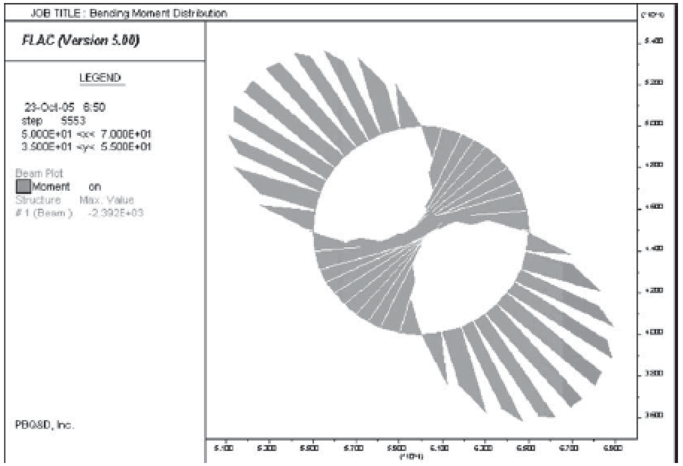
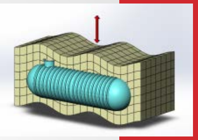
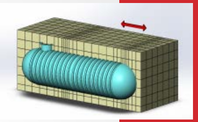


Figure 9-20. Culvert lining bending moment distribution (for flexible culvert in Set 1, Case 1 geometry).





FEA Model

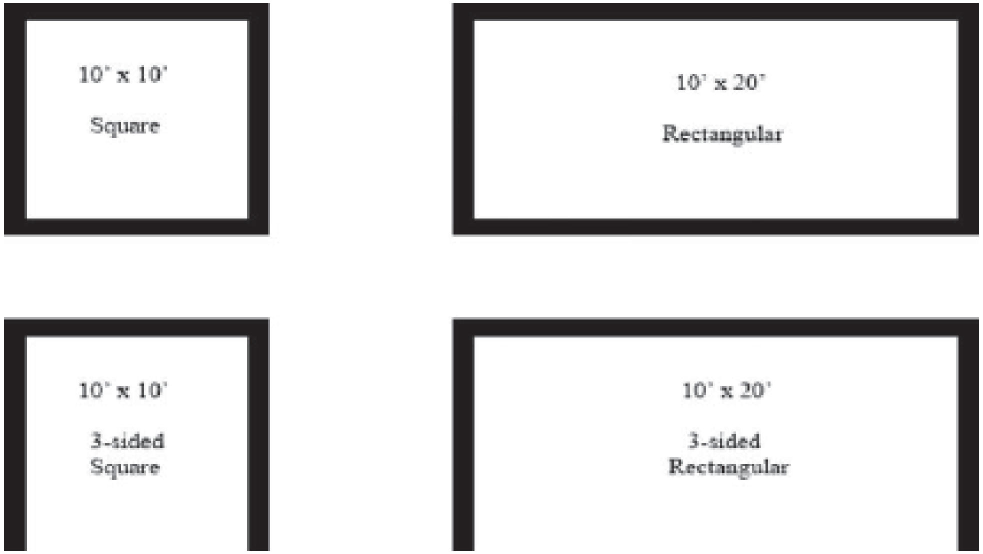
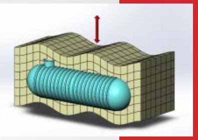
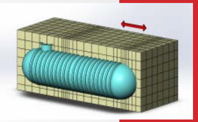


Figure 9-37. Various concrete box culvert sectional shapes and sizes used in the parametric analysis—Set 6.

Design Example (Tank/Pipe)





Design Example Wang / RCHRP Method

$G = (5000)\,psi$ $E_m = 7500\,psi$

Maximum Ovaling

$$\Delta_D := \frac{1}{3} \cdot K_{1T} \cdot F_T \cdot \gamma_{max} \cdot dia \qquad \Delta_D = (0.75) \cdot in$$

Modulus Range (E_m) Ref Bowles fth ed pg 125

Loose Gravel	Dense Gravel
7250 to 21700 psi	14500 to 29000 psi

Rib EI

$I_{rib} = 3.399 \cdot in^4$ $S = \begin{pmatrix} 2.07 \\ 4.82 \end{pmatrix} \cdot in^3$ $A_{p\_tot} = 5.6 \cdot in^2$ $E_m = 7500\,psi$

Axial Stress

$$T_{max} := \frac{1}{6} \cdot K_{1T} \cdot \frac{E_m}{(1. + \nu)} \cdot R_T \cdot \gamma_{max} \cdot spa_{rib} \qquad T_{max} = (12.25) \cdot lbf$$

Maximum Thrust under Full Slip

$$\sigma_{thrust} := \frac{T_{max}}{A_{p\_tot}} \qquad \sigma_{thrust} = (2.19) \cdot psi$$

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

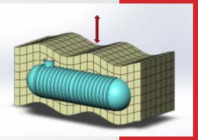
Maximum Bending Moment in shell / rib wall

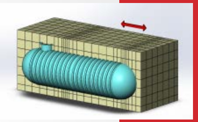
$$M_{max} := \frac{1}{6} \cdot K_{1T} \cdot \frac{E_m}{(1. + \nu)} \cdot R_T^2 \cdot \gamma_{max} \cdot spa_{rib} \qquad M_{max} = (735) \cdot lbf \cdot in$$

Maximum Bending Stress in shell / rib wall

$ii := 0$ $jj := 0 .. last(S)$ $jj = \begin{pmatrix} 0 \\ 1 \end{pmatrix}$

$$\sigma_{Tk_{jj,ii}} := \frac{M_{max_{ii}}}{S_{jj}} \qquad \sigma_{Tk} = \begin{pmatrix} 356 \\ 153 \end{pmatrix} \cdot psi \quad \begin{pmatrix} \text{"top"} \\ \text{"bottom"} \end{pmatrix} \qquad S = \begin{pmatrix} 2.07 \\ 4.82 \end{pmatrix} \cdot in^3$$

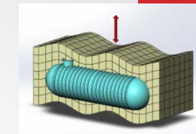


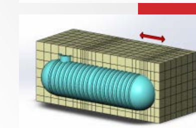


0) Historical background, some seismic information, shear modulus, and seismic spectra
1) Axial stress due to P waves and S waves
2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts

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

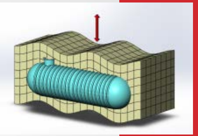
Granted Patent

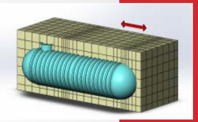
Patent No.	US 6,397,168 B1
Date of Patent	May 28, 2002

Seismic Evaluation Method for Underground Structures.

A brief summary of some pertinent points follow.

Based on FEA





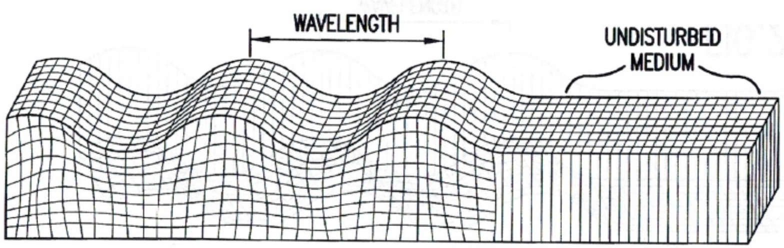


FIG.3(A)

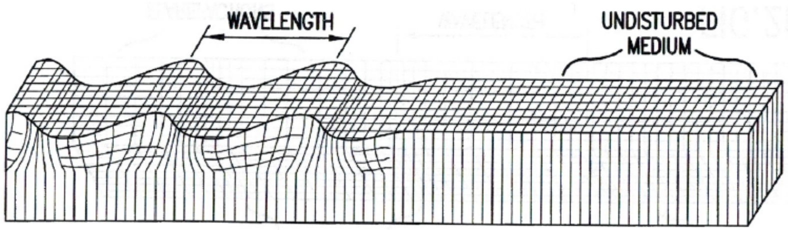


FIG.3(B)

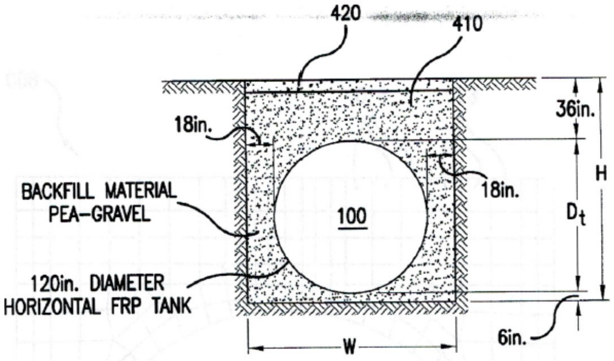


FIG.4

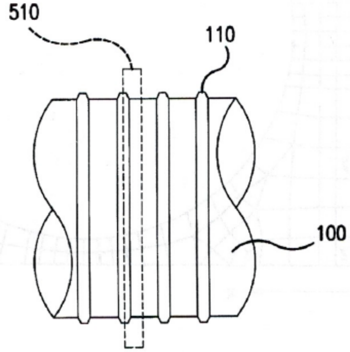
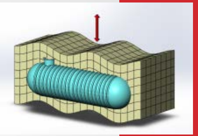
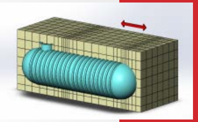


FIG.5





FEA Soil Mesh

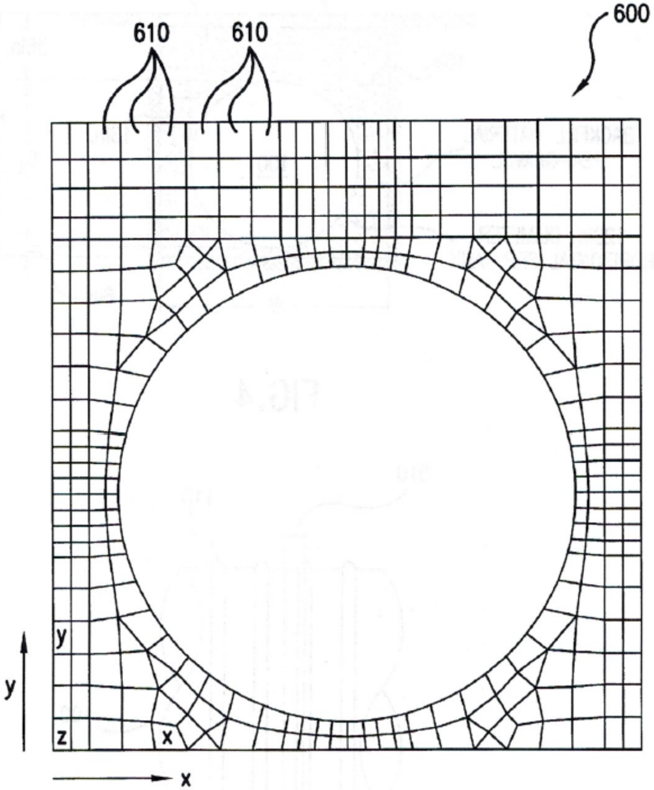


FIG.6

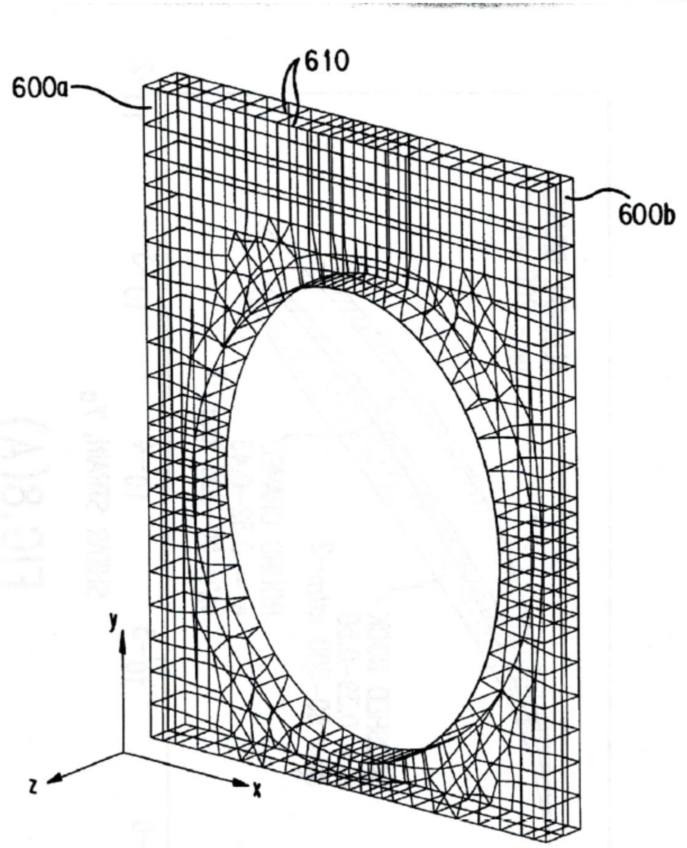
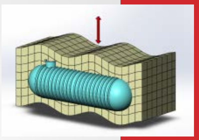
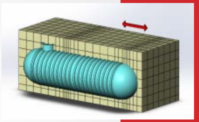


FIG. 7





FEA Ring Mesh

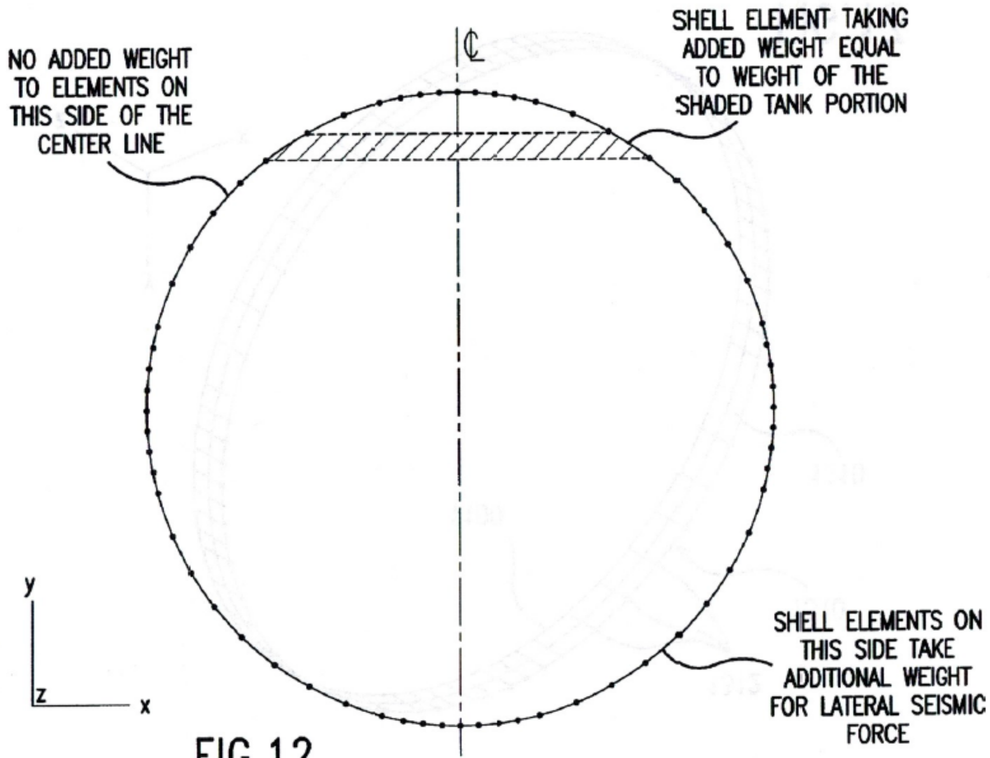
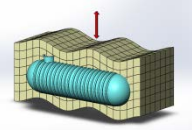
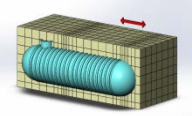


FIG.12

See slide 107 for rib cross section.

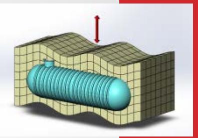


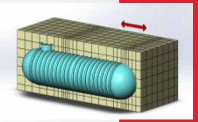


Xerxes Method

- 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

3D Brick Element

- 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.

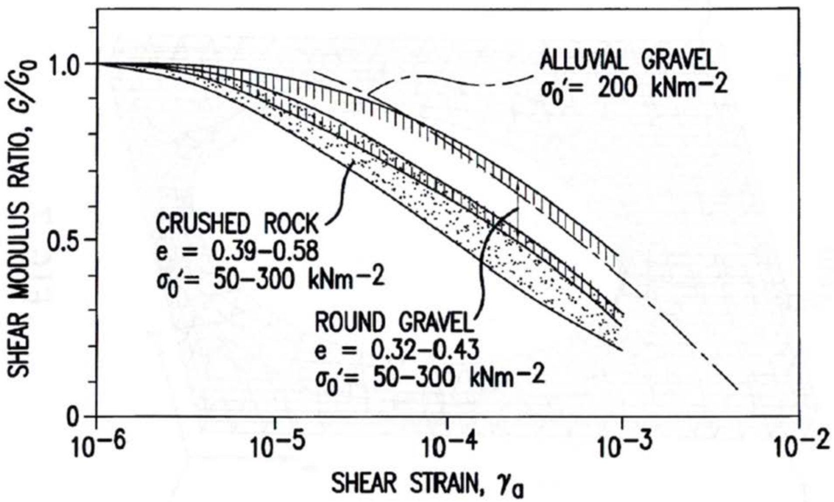


FIG.8(A)

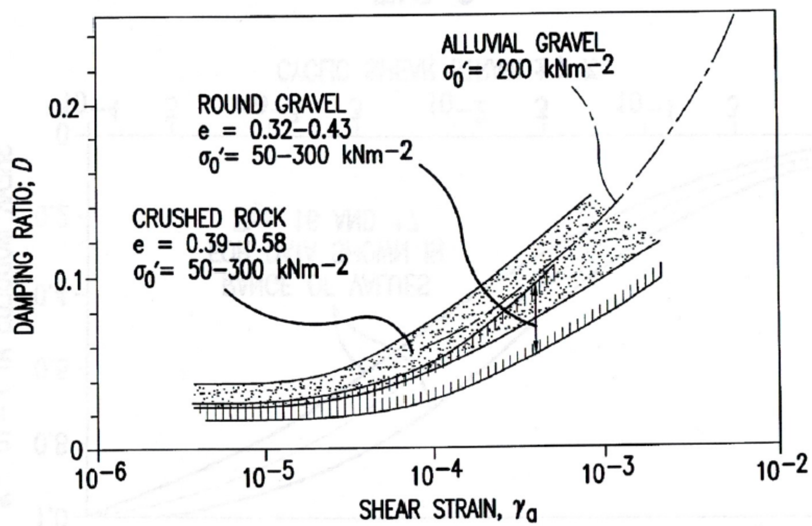
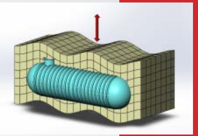
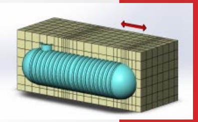
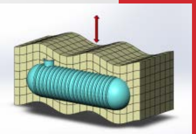
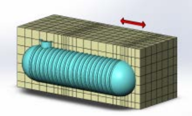


FIG.8(B)









Xerxes Method

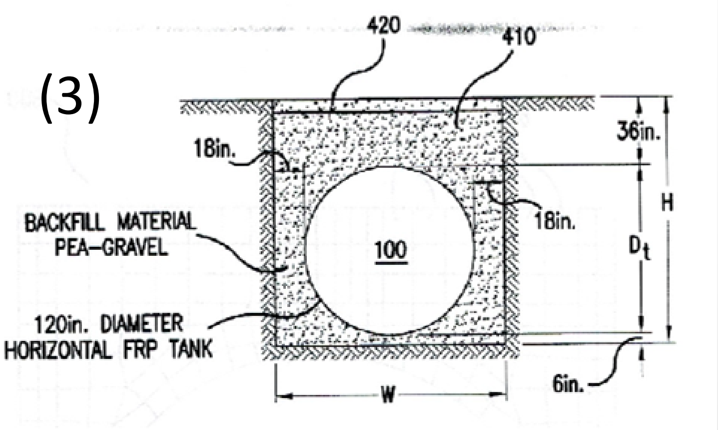
- Initial shear modulus by Kokusho and Esashi gives, (from Table I):

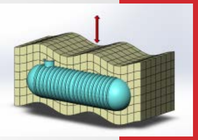
$$G_o = \frac{8400(2.17-e)^2(\sigma_0)^{0.60}}{(1+e)} \text{ (kPa)} \quad (1) \qquad \text{(Table I, slide 14)}$$

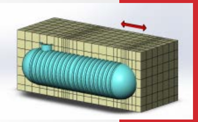
- Apply factor G/G_0 from shear modulus curve using $G/G_o = 0.15$ gives

$$G_o = \frac{1260(2.17-e)^2(\sigma_0)^{0.60}}{(1+e)} \text{ (kPa)} \quad (2)$$

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





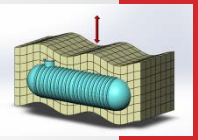


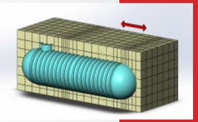
Xerxes Method

- Confining pressure is taken as ${}^{3}/_{8}\,\gamma H$
- Void ratio assumed to be $e = 0.4$ and $\gamma_{soil} = 120pcf$
- 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 $E = 900{,}000\,psi, \nu = 0.3,$ $\gamma_{FRP} = 0.061\,pci$

Metric Evaluation

*These values confirmed by independent check see Appendix A, slide 136.

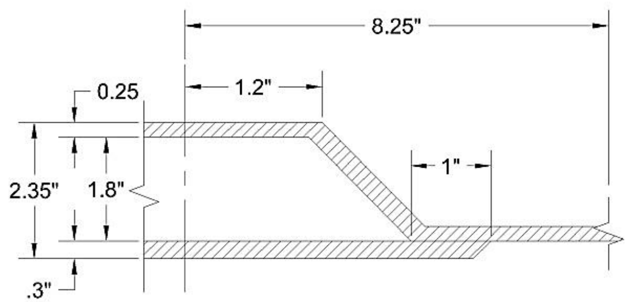




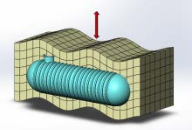
Section Properties (for Half Rib)

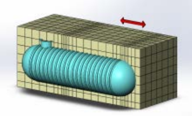
Rib space $= 16\frac{1}{2}\ in$

Reported in patent	Calculated
$A = 3.25in^2$	$A = \frac{5.6in^2}{2} = 2.8in^2$
$I = 4.0in^4*$ (use $1.75in^4$)	$I = \frac{3.4in^4}{2} = 1.7in^4$
$S = 2.35in^3$	$S = 2.07in^3$
$EI = 1{,}575{,}000\ lbs - in^2$	$EI = \frac{3{,}059{,}400}{2} = 1{,}529{,}700\ lbs - in^2$
$^{EI}/_{in} = 190{,}909\ \frac{lbs-in^2}{in}$	$^{EI}/_{in} = 185{,}424\ \frac{lbs-in^2}{in}$



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





Xerxes Method

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

Horizontal Analysis

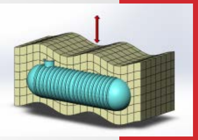
Peak Acceleration = 1.78 g @ 8.36 s

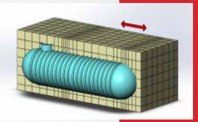
Peak Velocity = 47.37 in/s @ 7.92 s

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.

Xerxes Method

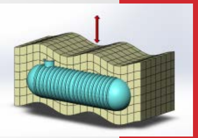
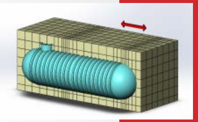
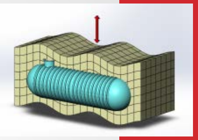
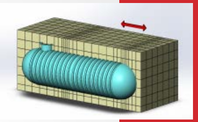



Table IV.

	With HORIZONTAL soil displacements																
	Column	Model	Figure	Time (s)	Tank Deformation Δ (in)	S_{11}	S_{22}	Fa	Fb	Soil Movement Δx	Condition	E_{top} (psi)	$E_{Mid\text{-}Height}$ (psi)	E_{bottom} (psi)	From Top to Mid-Height	From Mid-Height to Bottom	Avg E (psi)
*	10	U10b1	14A	7.96	<0.04	-67	-37	34	38	0.05	Empty	1008	6320	6320	varied	constant	4549
*	11	U10b2	14B	8.00	0.85	-110	-74	26	400	0.98	Empty	100	100	100	constant	constant	100
*						148	100										
	11	U10c1	14C	7.96	0.03	-129	-61	-75	29	0.0535	Full of Gasoline	1008	6320	6320	varied	constant	4549
	11	U10c2	14D	8.00	0.3	286	-144	52	389	0.828	Full of Gasoline	100	100	100	constant	constant	100
							206										
	12	U10c4	14E	8.62	3.87	-595	-358	220	1555	6.71	Full of Gasoline	10	10	10	constant	constant	10
							279										
	12	U10c5	14F	7.96	0.02	-139	-68	-76	23	0.03	Full of Gasoline	5000	5000	5000	constant	constant	5000
	13	U10c6	14G	7.96	0.05	-129	-79	-37	58	0.104	Full of Gasoline	1000	1000	1000	constant	constant	1000
	13	U10c7	14H	8.58	0.91	-341	-254	63	692	2.47	Full of Gasoline	30	30	30	constant	constant	30
							190										
	14	U10c8	14I	7.98	0.08	151	103	31	154	2.95	Full of Gasoline	300	300	300	constant	constant	300
						Min	Max	for range									
						-595	1555	above									

* Compare to previous methods

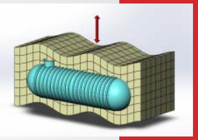


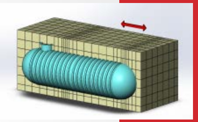


Xerxes Method

Table V.

Column	Model	Figure	Time (s)	Δ (in)	S_{11}	S_{22}	Fa	Fb	Δx	Condition	E_{top} (psi)	$E_{Mid\text{-}Height}$ (psi)	E_{bottom} (psi)	From Top to Mid-Height	From Mid-Height to Bottom	Avg E (psi)
14	V10c1	14J	8.52	0.05	-286	-167 27	158	32	0.046	Full of Gasoline	1008	6320	6320	varied	constant	4549
15	V10c2	14K	8.54	0.55	-971	-842 531	-116	531	0.804	Full of Gasoline	100	100	100	constant	constant	100
15	V10c3	14L	8.52	0.47	-1146	-990 610	-179	680	0.504	Full of Gasoline	1008	6238	100	varied	constant	2449
15	V10c4	14M	8.58	0.95	-82 432	226 351 368	-62	868	2.9	Full of Gasoline	10	10	10	constant	constant	10
16	V10c5	14N	8.52	0.04	-311	-224	-161	35	0.046	Full of Gasoline	5000	5000	5000	constant	constant	5000
16	V10c6	14O	8.52	0.1	-585	-508 201	-158	83	0.127	Full of Gasoline	1000	1000	1000	constant	constant	1000
17	V10c7	14P	8.56	0.69	573	-272 515	71	622	1.43	Full of Gasoline	30	30	30	constant	constant	30
17	V10c8	14Q	8.52	0.46	-829	-726 382	-131	212	0.333	Full of Gasoline	300	300	300	constant	constant	300
17	V10b1	14R	8.52	<0.04	-253	-196 27	126	23	0.044	Empty	1008	6320	6320	varied	constant	4549
18	V10b2	14S	8.54	0.47	-776	-694 410	113	371	0.768	Empty	100	100	100	constant	constant	100
18	V10b3	14T	8.52	0.4	-1015	-870 492	-190	655	0.44	Empty	1008	6238	100	varied	constant	2449
					Min	Max	for range									
					-1146	868	above									





Comparison of Three Methods Using Lowest Shear Modulus

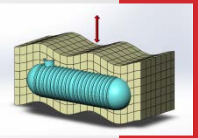
Note that Xerxes method does not include sloshing. This must be included.

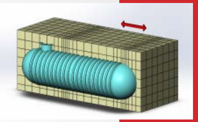
	Historical Axial Method	Wang/NCHRP	Xerxes
E	4549	4549	4549
G	1625	1625	1625
$\boldsymbol{\sigma_A}$	2185	NR	NR
$\boldsymbol{\sigma_H}$	114**	627*	148*
Δ_D	NR	$\sim 1\ ^5/_{16}$ in	~ 1 in
Ref	Slide 45	Slide 83	Slide 110

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.

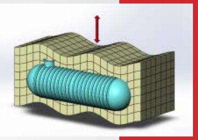


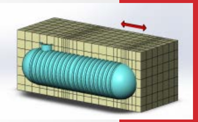


0) Historical background, some seismic information, shear modulus, and seismic spectra
1) Axial stress due to P waves and S waves
2) Wang (23) method (NCHRP) (4) transverse loads on circular conduits and box culverts
3) Xerxes (20) patent (reduced shear modulus) with transverse loads on FRP UST's

4) Sloshing

5) Liquefaction
6) Buckling of soil surrounded tubes



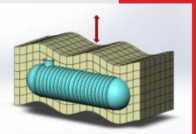


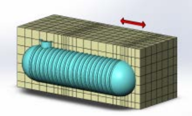
Sloshing

- 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…”





Sloshing

First Mode Period (convective) sloshing

$$\lambda = \sqrt{3.16g \cdot tanh\left(3.16 \cdot \frac{d_{fld}}{L'}\right)}$$

ACI 350.3, Eqn 9-13, page 36 (2)

where

$$d_{fld} = \text{depth of fluid}$$

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

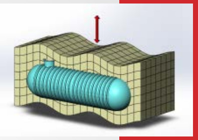
ACI 350.3, Eqn 9-14, page 36 (2)

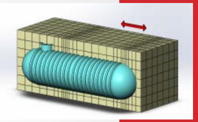
$$L' = \text{effective tank length}$$

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

$$W_c = 0.264\left(\frac{L}{d_{fld}}\right) \cdot tanh\left[3.16\left(\frac{d_{fld}}{L'}\right)\right] \cdot Wt_{fld}$$

Sloshing





Fundamental period ASCE 7-10 pg. 152 (1)

$$T_S = \frac{S_{D1}}{S_{DS}} s$$

Seismic Response Coefficient

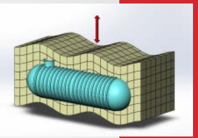
ACI 350.3, Table 4.1.1(b), page 20 (3)

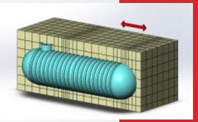
$$C_{c_k} = if\left[\frac{T_{c_k}}{s} \le \frac{1.6s}{T_S}, \frac{1.5 \cdot S_{D1}}{T_{c_k} \div s}, \frac{2.4 \cdot S_{DS}}{\left(T_{c_k} \div s\right)^2}\right]$$

Convective Component $R_c = 1.0$

ACI 350.3, Section 9.4.2, page 40 (3)

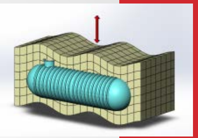
$$V_c = \left(C_c I_e \cdot \frac{W_c}{R_c}\right)$$

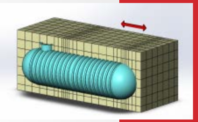




Method for Sloshing Calculations

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

Sloshing Wave Height

$\delta'_s := \overrightarrow{(0.42 \cdot L_{ss} \cdot I_e \cdot S_{ac})}$ $\quad \delta'_s = 52.9 \cdot in$

Per ASCE 7-10, pg 153, Eqn. 15.7-13.

Fluid Depth

$d_{fld} = 114 \cdot in$ The worst case fluid depth

$G = 1625 \cdot psi$

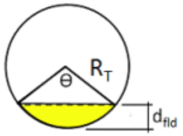
$I_e = 1.5$

Tank Radius

$R_T = 60 \cdot in$

Inscribed Angle

$\theta := 2 \cdot acos\left(\frac{R_T - d_{fld}}{R_T}\right)$ $\quad \theta = 308 \cdot deg$



Area of a Segment of a Circle

$A_{seg\_cirl} := \frac{R_T^2}{2}(\theta - \sin(\theta))$

$A_{seg\_cirl} = 77.1 ft^2$ See any Geometry Text

Area of a Segment of a Circle

$A_{seg\_cirl} = 77.1 ft^2$

Straight Wall Length

$L_{ss} = 27 ft$

Fluid Volume in Straight Section

$Vol_{strt} := A_{seg\_cirl} \cdot L_{ss}$ $\quad Vol_{strt} = 15783 \cdot gal$

Fluid Volume in End Caps

$Vol_{EC.} = 3890 \cdot gal$

Total Fluid Volume

$Vol := Vol_{strt} + Vol_{EC.}$ $\quad Vol = 19673 \cdot gal$

for straight shell length $=$ $L_{ss} = 328 \cdot in$

Weight of Fluid

$G_{fld} = 1.0$ $\quad \gamma_{wat} = 0.03612 pci$ $\quad Vol = 19673 \cdot gal$

$Wt_{fluid} := G_{fld} \cdot \gamma_{wat} \cdot Vol$ $\quad Wt_{fluid} = 164.1 \cdot kip$

Sloshing mass is computed per ACI 350.3, Eqn 9-16, pg 48

$W_c := \overrightarrow{\left[0.264\left(\frac{L_{ss}}{d_{fld}}\right) \cdot \tanh\left[3.16\left(\frac{d_{fld}}{L_{ss}}\right)\right] \cdot Wt_{fluid}\right]}$ $\quad W_c = 99.8 \cdot kip$

Convective Component

$V_c := \overrightarrow{\left(\frac{S_{ac} \cdot I_e}{1.5} \cdot W_c\right)}$ $\quad V_c = 25.5 \cdot kip$

$V_{c\_Horz} := \frac{S_{ac} \cdot I_e}{1.5} \cdot F_{T\_max}$ $\quad V_{c\_Horz} = 13 \cdot kip$

Fluid Depth

$d_{fld} = 114 \cdot in$ $\quad I_e = 1.5$

Tank Straight Wall Length where Worst Case Convective Component Occurs

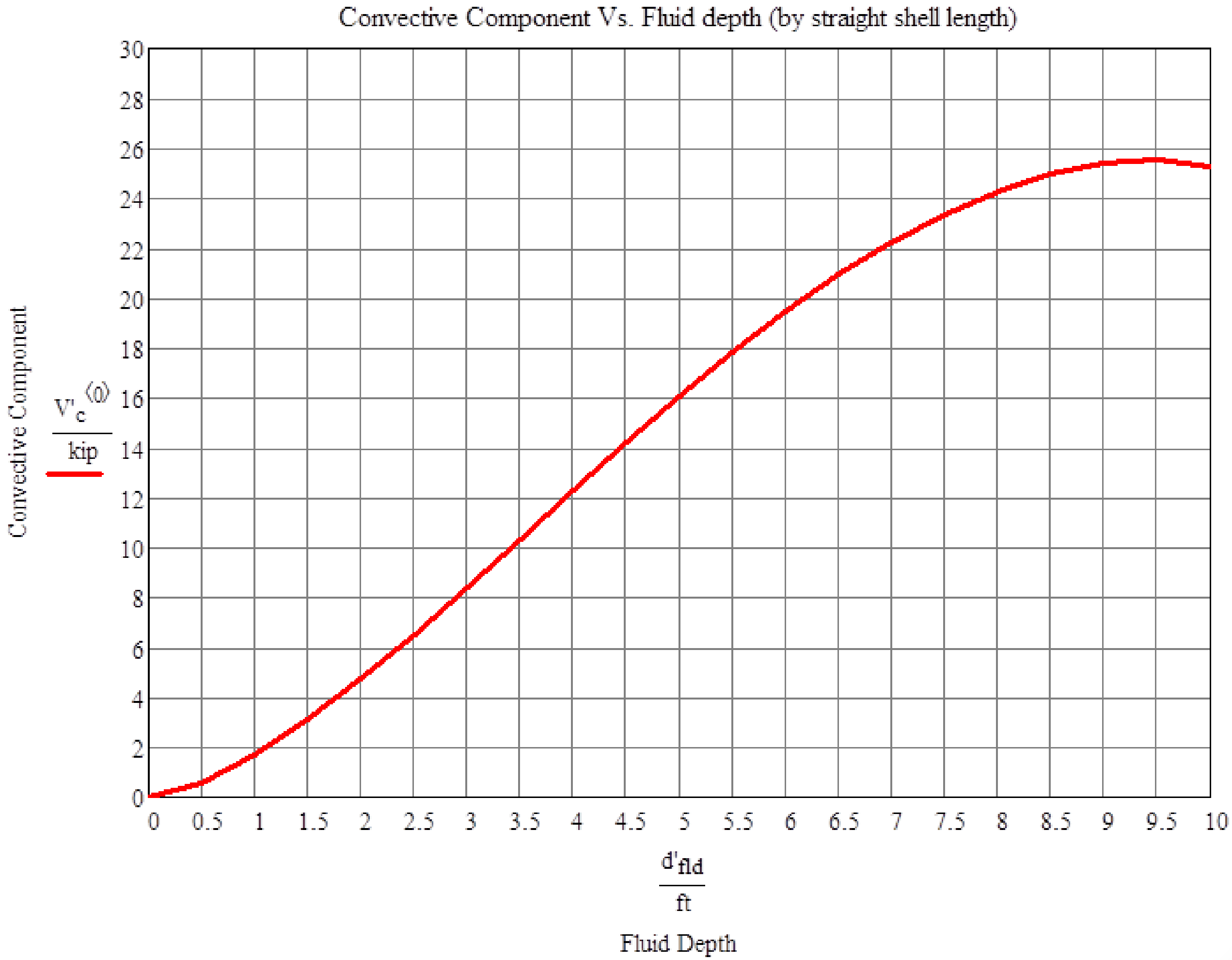
$L_{Vc\_max} := L_{ss}$ $\quad L_{Vc\_max} = 328 \cdot in$

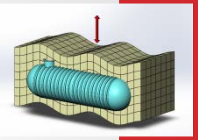
Actual Tank Straight Wall Length

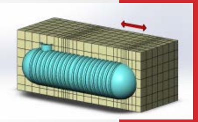
$L_{ss} = 328 \cdot in$

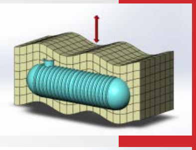
Example

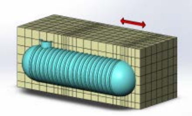
Sloshing force = f (fluid height)











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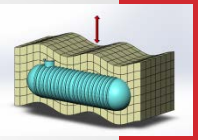
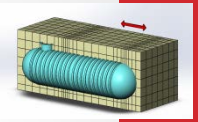


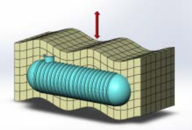


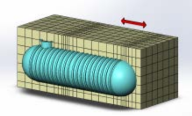
Liquefaction – Flotation

- 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

Example of Flotation Due to Liquefaction

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





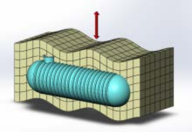
Liquefaction

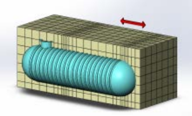
Reference Lambe and Whitman, Soil Mechanics p. 445 (13)

- 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)

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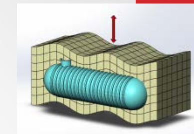
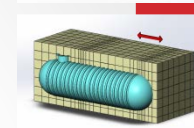
5) Liquefaction

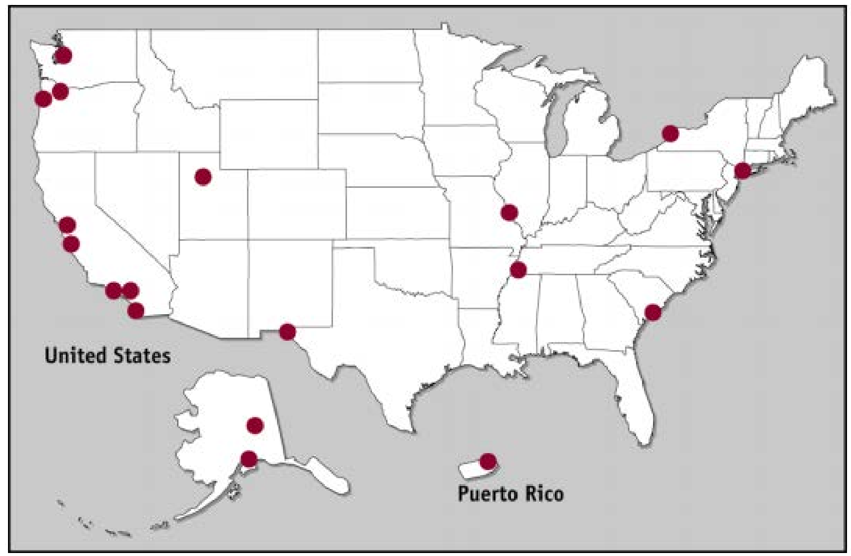




Four conditions must be present for liquefaction to occur

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.

Possible Locations in Continental US Where Liquefaction can Occur



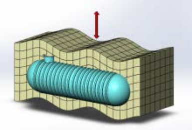
Reference M. Power and T. Holzer, Liquefaction Maps, ©1996, pg. 1 (21)
Can be found at http://www.atcouncil.org/pdfs/atc-35.pdf

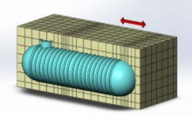
Probability of these four occurring simultaneously is very low.

One study based on USGS data for p 6.7 EQ resulted in $P_{LIQF} = .001995\%$ or one installation 1in 50125 tanks.

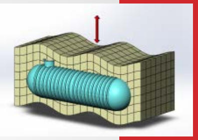
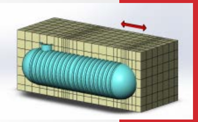
Flotation has occurred in New Zealand during Christchurch earthquake (2011)

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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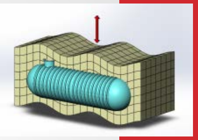
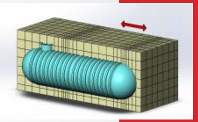




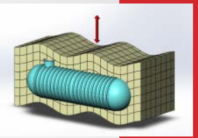
Buckling of Soil Surrounded Pipes and Tanks

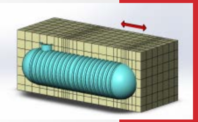
- Initial work done by Ulrich Luscher in 1966 – Buckling of Soil Surrounded Tubes. J. Soil Mechanics and Foundation. ASCE 92 (6) (14)
  - Valid for long pipes without end loading i.e. for hoop load only.
    - Not valid for short cylindrical tanks with axial load.
- Excellent review by Ian D. Moore, 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

Buckling force $N_{\theta-CR}$ for no slip condition

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

Buckling of Soil Surrounded Pipes and Tanks





Where

n = buckling mode (use 2 for flexible pipe)

EI = flexural rigidity of tank or pipe

R = radius

G_m = shear modulus of soil

ν_m = Poisson's ratio of soil

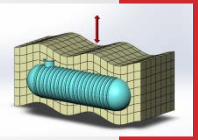
For slip condition

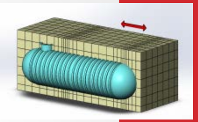
$$N_{\theta-CR} = (n^2 - 1)\frac{EI}{R^2} + 2G_m R\left[\frac{1}{2n(1-\nu_m)-(1-2\nu_m)}\right] \quad \text{(b)}$$

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

- 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.



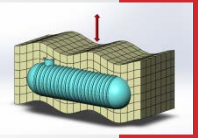


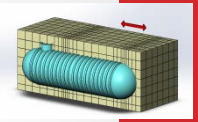
Buckling of Soil Surrounded Pipes and Tanks

- 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.

$$F_{s_{Tank}} = \frac{1}{\frac{1}{F_{s_\theta}} + \frac{1}{F_{s_{AX}}}}$$

- Generally requires testing to validate.





Two possible axial buckling equations are:

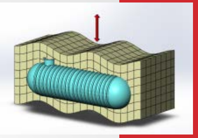
- Roark's Formulas for Stress and Strain 6th ed., Table 35 Case 15, p. 689 (25)

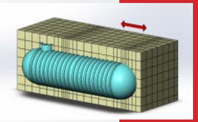
$$\sigma' = \frac{1}{\sqrt{3}} \cdot \frac{E_{pipe}}{(1-\nu^2)} \cdot \frac{t_{eff}}{R} \qquad \text{(a)}$$

where

E_{pipe} is modulus of pipe material
ν is Poisson's ratio of pipe material
t_{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)

$$\sigma_{CR-AX} = \frac{2\sqrt{3}C\sqrt{D_x A_\theta}}{R} \quad \text{(b)}$$

$$D_x = \frac{E_x t^3}{12(1-\nu^2)} = \frac{\left({}^{EI}/_{in}\right)}{(1-\nu^2)}$$

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

$$C = k_o \cdot k_n \cdot k_s$$

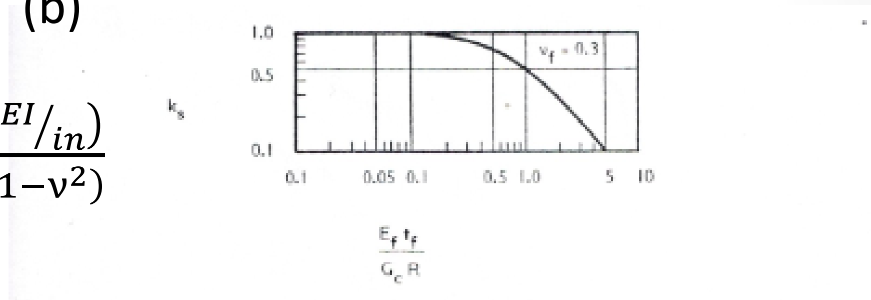


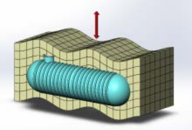
Fig. 9-26 REDUCTION FACTOR FOR SHEAR DEFORMATION IN BUCKLING OF LONGITUDINALLY COMPRESSED LONG SANDWICH CYLINDER WITH ISOTROPIC FACINGS AND SHEAR FLEXIBLE CORE (Source 9.9)

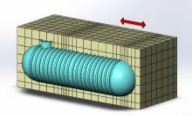
$$k_o = \frac{1}{3\sqrt{1-\nu^2}} \cong 0.6 \text{ for } \nu = 0.3$$

k_n = knockdown factor for imperfections

$\sim 1.53\text{-}0.477 \log\frac{R}{t} \leq 0.21$

k_s = shear reduction factor (see Fig 9-26)





When to Use which Method for Computing Buckling FS

<u>For no ground water</u>

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

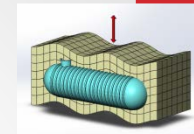
Use interaction equation

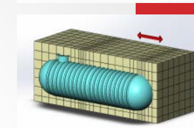
For liquefaction

Check hoop and axial buckling (equations a or b) in a heavy fluid

Use interaction equation

Summary





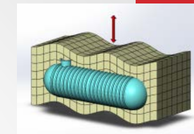
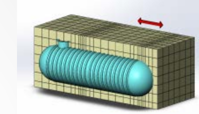
A. Studies show that soil shear modulus G_m is reduced with increasing strain during and earthquake. This was not considered in previous method of analysis for axial stress.

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.

Summary

C. Comparison of methods

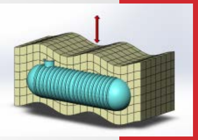
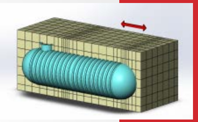
	Historical Axial Method	Wang/NCHRP	Xerxes
E	4549	4549	4549
G	1625	1625	1625
$\boldsymbol{\sigma_A}$	2185	NR	NR
$\boldsymbol{\sigma_H}$	114**	627*	148*
$\boldsymbol{\Delta_D}$	NR	$\sim 1\ {}^{5}/_{16}$ in	~ 1 in
Ref	Slide 45	Slide 83	Slide 110

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.

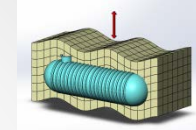
Summary

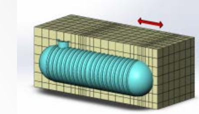
D. Sloshing should be considered and added to any method used.

    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

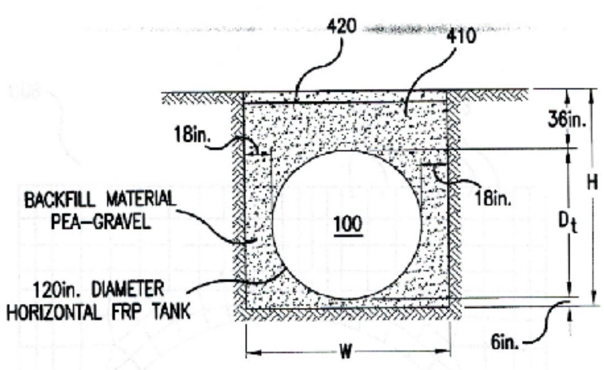
Verify Values for G and E Used in Xerxes Patent

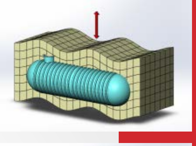
$A := 8400$ $\gamma_{pg} := 120pcf$ $n_1 := 0.6$ $e' \equiv 0.4$ $\nu := 0.4$ $F(e) := \frac{(2.17 - e')^2}{1 + e'}$ $F(e) = 2.238$

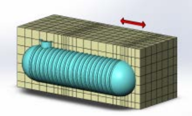
Calculate using metric units for from Grade to Springline

Depth below grade (ft)	Confining Pressure	Shear Modulus	Factor by 15% per Fig 8 in patent
	$\sigma_o := 0.375H' \cdot ft \cdot \gamma_{pg}$	$\overrightarrow{G'_{rg} := \left[A \cdot F(e) \cdot \left(\frac{\sigma_o}{kPa}\right)^{n_1}\right] kPa}$	$G_{rg} := 15\% \cdot G'_{rg}$
$H' \equiv \begin{pmatrix} 0.01 \\ 0.375 \\ 1 \\ 3 \\ 8 \end{pmatrix} ft$	$\sigma_o = \begin{pmatrix} 0 \\ 0.8 \\ 2.2 \\ 6.5 \\ 17.2 \end{pmatrix} \cdot kPa$	$G'_{rg} = \begin{pmatrix} 1.9 \\ 16.5 \\ 29.8 \\ 57.6 \\ 103.7 \end{pmatrix} \cdot MPa$	$G_{rg} = \begin{pmatrix} 282 \\ 2481 \\ 4469 \\ 8639 \\ 15562 \end{pmatrix} kPa$

	Convert to psi	Compute Modulus	
		$E := 2(1 + \nu) \cdot G_{rg}$	
$H' = \begin{pmatrix} 0.01 \\ 0.375 \\ 1 \\ 3 \\ 8 \end{pmatrix} ft$	$G_{rg} = \begin{pmatrix} 41 \\ 360 \\ 648 \\ 1253 \\ 2257 \end{pmatrix} psi$	$E = \begin{pmatrix} 115 \\ 1008 \\ 1815 \\ 3509 \\ 6320 \end{pmatrix} psi$	**0.375 ft** **Springline (8ft)**







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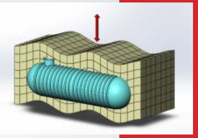
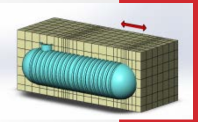

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