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COMPARATIVE STUDY OF LINEAR-ELASTIC AND NONLINEAR-INELASTIC SEISMIC RESPONSES OF FLUID-TANK SYSTEMS

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SUMMARY

This paper focuses on three typical water storage tanks, which were designed under the AWWA standards. All tanks are anchored cylindrical tanks on a rigid concrete foundation. Three time history records were scaled to design level ground motion for the specific location of the tanks using the value from the 2000 International Building Code. The computer models were run using linear elastic and nonlinear material and geometric properties. Results of this study show the substantial variation of values in base shear and overturning moment. The ratios of base shear between linear elastic and nonlinear properties ranged from 0.74 to 2.04, and overturning moments from 0.87 to 2.36, which differs from design standard reduction R-factored value of 4.5 for anchored tanks.

INTRODUCTION

Water storage tanks are important to the continued operation of water distribution systems in the event of earthquakes. Current knowledge about the behavior of liquid storage tanks is extensive, but many of the analytical and theoretical results are based on a number of simplifying conditions, including small deformation and linear elastic material assumptions. Typical tanks consist of thin wall, cylindrical shells of constant or varying wall thickness, a base plate, a flat or sloped roof, and roof support members. Water storage tanks in the U.S. are designed in accordance with the American Water Works Association (AWWA) Standard D100 (AWWA 1996 [1]) for welded steel tanks and D103 (AWWA 1997 [2]) for bolted steel tanks. Experiences in past earthquakes have shown tanks to be vulnerable to seismic damage. Recent earthquakes have resulted in various types of structural damage to both anchored and unanchored ground level water or oil storage tanks (ASCE 1991 [3]).

This paper focuses on three typical water storage tanks, which were designed under the AWWA standards in effect at the time of construction. All tanks are anchored cylindrical tanks on a

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rigid concrete foundation. Tank sizes and dimensions were selected to represent a range of configurations including a broad tank, with a height to diameter ratio of 0.25, a middle range tank with a ratio of 0.75, and a tall standpipe with a ratio of 3.75. All three tanks are located in south central Alaska. Three time history records were selected, and included the simulated 1964 Alaska earthquake, the 1940 El Centro earthquake (N-S component), and the 2002 Denali earthquake. The time history records were scaled to design level ground motion for the specific location of the tanks using the value from the 2000 International Building Code (ICC 2000 [4]), with details provided in a subsequent section of this paper.

The computer models were run using linear elastic and nonlinear material and geometric properties. Results were compared using the current AWWA standard calculations. Results of this study show how the assumptions and material properties selected have a large effect on the modeling outputs. In particular, values of base shear and overturning moment show substantial variation. The ratios of base shear between linear elastic and nonlinear properties ranged from 0.74 to 2.04, and overturning moments from 0.87 to 2.36, which differs from design standard reduction R-factored value of 4.5 for anchored tanks.

BACKGROUND

Water storage tank design considers the response of the tank and contents based on two response modes, a high frequency amplified response to the lateral motion of the tank structure and a portion of the contained fluid that moves with the shell structure, and a low frequency response of the liquid in a sloshing mode. In the available theoretical solution, it is assumed that the liquid is incompressible, inviscid, irrotational potential flow, and that all structural and liquid motions remain within the linear range. The determination of hydrodynamic pressure is based on the solution of the Laplace equation with appropriate boundary conditions. By separation of variables, two solutions to the velocity potential and resulting pressure can be found. This is the basis of the concept of dividing the hydrodynamic response into two parts, those representing the body terms, called the impulsive components, and those representing the surface wave terms, or the convective component of pressure (Housner [5]).

For a rigid tank, sloshing is due to a part of the liquid that moves as a surface wave, and is associated with the convective pressures. The modes are proportional to $J_1(\lambda_j, r/R)$ where J_1 is the first order Bessel function of the 1st kind, and $\lambda_j = jth$ zero root of the first derivative of the first order Bessel function of 1st kind. The first five values of λ are 1.8412, 5.3314, 8.5363, 11.706, and 14.8631. The frequency is given by:

$$f_j = \frac{1}{2\pi}\sqrt{\lambda_j \frac{g}{R}\tanh(\lambda_j \frac{H}{R})} \qquad \text{Eq. (1)}$$

The general frequency of the fluid/tank system for anchored tanks has been the subject of numerous studies (Clough [6] and Haroun [7]). Several different methods have been used. The combined system frequency can be expressed in the form:

$$f_{0m} = \frac{C_m}{2\pi H}\sqrt{\frac{E}{\rho_l}} \qquad \text{Eq. (2)}$$

The dimensionless frequency coefficient C_m depends on the tank proportions of height, radius, wall thickness, Poisson ratio, and relative mass density of contents and tank shell material. Tabular values of the frequency coefficients are available in the literature (Veletsos [8]) or graphically (Haroun [9]). The natural frequency of the fluid/tank system depends on the assumed deformed shape.

The design of water tanks for seismic considerations uses a simplified formula for base shear by the following formula in AWWA (AWWA 1996 [1]).

$$V_{ACT} = \frac{18ZI}{R_W}[0.14(W_s + W_r + W_f + W_1) + SC_1W_2] \qquad \text{Eq. (3)}$$

V_{ACT} is lateral shear; Z is the zone coefficient; I is the use factor; R_w is the force reduction coefficient; W_s is the total weight of tank shell and significant appurtenances; W_r is the total weight of the tank roof; W_f is total weight of the tank bottom; W_1 is the weight of effective mass of tank contents that moves in unison with the tank shell; S is the site amplification factor; W_2 is the weight of effective mass of the first mode sloshing of the tank contents, and C_1 is the coefficient related to the first sloshing period and tank geometry. The base shear is used for anchorage and connection design or sliding check. The overturning moment applied to the bottom of the tank shell due to seismic forces is determined in accordance with the following formula in AWWA (AWWA 1996 [1]).

$$M = \left[\frac{18ZI}{R_w}\right][0.14\{W_sX_s + W_rH_t + W_1X_1) + SC_1W_2X_2] \qquad \text{Eq. (4)}$$

M is overturning moment applied to the bottom of the tank shell; X_s is height from the bottom of the tank shell to center of gravity of the shell; H_t is total height of the tank shell; X_1 is height from the bottom of the tank shell to the centroid of lateral seismic force applied to W_1; and X_2 is height from the bottom of the tank shell to the centroid of lateral seismic force applied to W_2. The base moment is used to determine the uplift forces and the compressive forces acting on the tank shell near the base. Both the base shear and overturning moment are important variables in design practice.

FEA MODEL AND ANALYSIS METHODS

The model considers the tank roof system to be represented by shell and beam elements, which are placed in the radial and circular directions. The tank wall is modeled by shell elements. The contents are represented as three-dimensional contained fluid elements which are not attached to the shell elements at the wall boundary, but have separate coincident nodes that are coupled only in the direction normal to the interface. The relative movements in the tangential and vertical directions are allowed to occur. The fluid element nodes at the base are allowed to move horizontally, while the shell wall is fixed around the perimeter.

The fluid element selected is used to model fluids contained within tanks having no net flow. The fluid element is particularly well suited for calculating hydrostatic pressures and fluid/solid interactions. Fluid elements are rectangular (brick shaped) whenever possible, as results are known to be of lower quality for some cases using non-rectangular shapes. The bulk modulus K, 2,068 MPa, is taken to be characteristic of the water rather than infinite (incompressible), since it is used for shear stability as well as for compressibility effects.

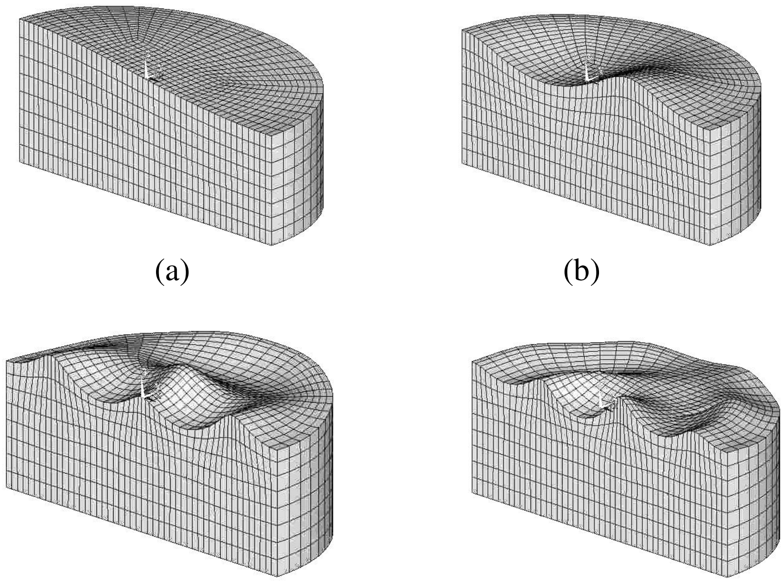
Because of the system symmetry with only one horizontal component of ground motion concerned, one half of the tank is modeled. The tank has a radius R, total height H, constant wall thickness t_s, constant base thickness t_b, and is filled with water of density ρ_l to a depth h. The tank is covered with a roof, supported by framing elements and a center column. Material properties include density of the steel, ρ_s, Young's modulus of elasticity, E_s and tangent modulus E_T for nonlinear analysis. Dynamic input is aligned in the horizontal direction. The ground acceleration time history occurs at the base of the tank/soil interface. Values for the three tank models are summarized in Table 1.

Table 1 Tank and Material Properties

Parameter	Tank A – Broad	Tank B- Medium	Tank C - Tall	Units
Height, H	8.44	8.44	23.86	m
Liquid Depth, H_t	7.44	7.44	22.86	m
Radius, R	9.06	4.98	3.05	m
Wall Thickness, t_s	4.76	4.76	9.52	mm
Roof Thickness, t_r	4.76	4.76	9.52	mm
Youngs Modulus, E	200,000	200,000	200,000	MPa
Tangent Modulus, E_T	20,000	20,000	20,000	MPa
Poisson Ratio, v	0.3	0.3	0.3	
Steel Density, ρ_s	7.83	7.83	7.83	kg /m^3
Liquid Density, ρ_l	1.00	1.00	1.00	kg /m^3

In addition to linear elastic model assumptions, a large deformation and an elasto-plastic stress-strain curve was assumed for the tank shell. Structural steel exhibits a linear stress-strain relationship up to the yield point (250 MPa in this study) beyond which the relationship becomes plastic and nonlinear. This has been represented by a bilinear kinematic hardening model such that the Bauschinger effect is included. The resulting behavior is non-conservative and path dependent. The sequence of applying the loads and the resulting plastic response affects the final solution.

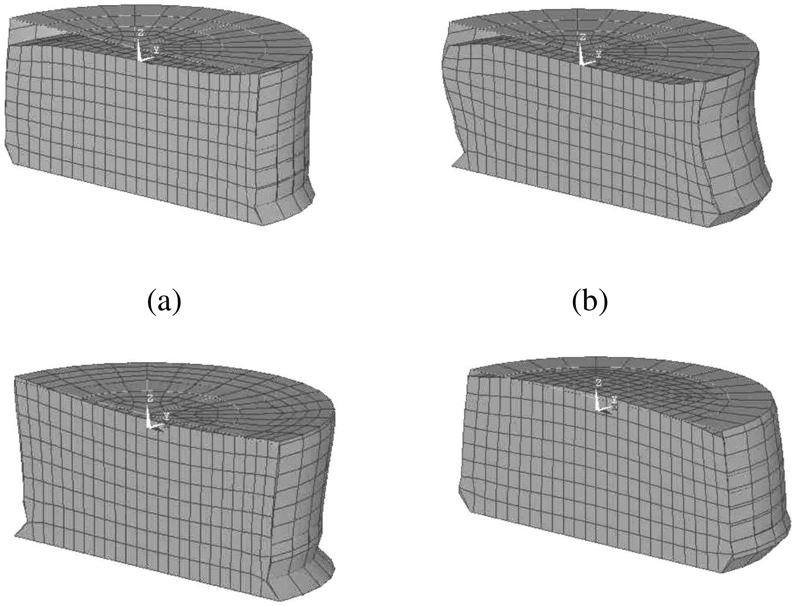
To validate the FEA models, comparison of the natural periods and mode shapes for both fluid-sloshing and fluid/tank system were extracted from the FEA models and compared with linear theory results or the approximate solutions from equations (1) and (2). Only a portion of the comparison results are presented in the paper. Using Tank A as an example, Figure 1 shows the first four sloshing mode shapes, which is consistent with the theoretically predicted cosine wave shapes on the fluid face. Figure 2 shows the 1st and 2nd lateral, 1st vertical and 1st radial mode shapes of fluid/tank system, which is consistent with the analytical prediction. The comparison of natural periods of Tank A with theory and analytical results is listed in Table 2. Table 3 lists the comparison results of natural periods of Tank A, B and C fluid/tank systems with the available approximate analytical results. The compatible solutions demonstrate the validity of the FEA models.



(a) (b)

(c) (d)

Figure 1 Sloshing Mode Shapes of Tank A. (a) 1st mode, T=4.812 s, (b) 2nd mode, T=2.721 s, (c) 3rd mode, T=2.092 s, (d) 4th mode, T=1.961 s.



(a) (b)

(c) (d)

Figure 2 Natural Mode Shapes of Fluid/Tank System -Tank A.
(a) 1st lateral mode, T=0.1443 s, (b) 2nd lateral mode, T=0.081 s,
(c) 1st vertical mode, T=0.1387 s, (d) 2nd vertical mode, T=0.01 s.

Table 2 Comparisons of Natural Periods of Tank A: FEA vs. Theoretical or Approximate Solutions

	Fluid-Sloshing Natural Periods (seconds)			
	1st mode	2nd mode	3rd mode	4th mode
FEA	4.815	2.721	2.092	1.961
Linear Theory	4.678	2.615	2.067	1.765
	Fluid/Tank System (seconds)			
	Lateral		Vertical	
	1st mode	2nd mode	1st mode	2nd mode
FEA	0.1443	0.0810	0.1387	0.01
Approximate Analysis	0.1446	0.0781	0.1498	-

Table 3 Comparison of Fluid/tank System Natural Periods of the Three Modes: FEA vs. Approximate Analytical Solutions

		Fluid/Tank System Natural Periods (second)		
		Lateral		Vertical
		1st mode	2nd mode	1st mode
Tank A	FEA	0.1443	0.0810	0.1387
	Approximate Analysis	0.1446	0.0781	0.1498
Tank B	FEA	0.097	0.048	0.093
	Approximate Analysis	0.099	0.045	-
Tank C	FEA	0.392	0.0897	0.168
	Approximate Analysis	0.370	0.083	-

SEISMIC GROUND INPUT

The time history ground inputs were obtained from published records of the 1940 El Centro and the 2002 Denali earthquakes. The 1964 Alaska earthquake was a synthetic record generated using a model representation of the area and the probable frequency range and soil conditions at the site (Papageorgiou [10]). A Design Response Spectrum (DRS) was developed using procedures in the International Building Code (ICC 2000 [4]) for the location of Tank C. Using IBC (International Code Council 2000) site coefficients F_a= 1.0, F_v=1.53 based on an assumed granular soil base material resulted in DRS acceleration at short periods of 0.86g. 5% damping was used to create the Acceleration Response Spectrum (ARS) for three ground inputs shown in Figure 3. To generate design level earthquake inputs a reasonable approach is to keep the acceleration response spectrum value inputs at the same level of the DRS value at the fundamental natural frequency of the fluid/tank system. The three tanks have separate fundamental periods in a range of 0.097 sec. to 0.392 sec. while the ARS curves fluctuate. In order to match the peak values between the ARS's and DRS, the recorded ground input time histories were adjusted so that the average ARS values in the range of 0.1 to 0.7 seconds match the peak value of the IBC DRS acceleration, i.e. 0.86g. Also shown in Figure 3 as represented by the vertical lines are the tank fluid-structure natural periods. Figure 4 shows the resulting three time histories used in the analysis and corresponding Fourier amplitude spectra. Damping ratio of 5% critical was used in all time history analyses to include the effects of soil/structure interaction.

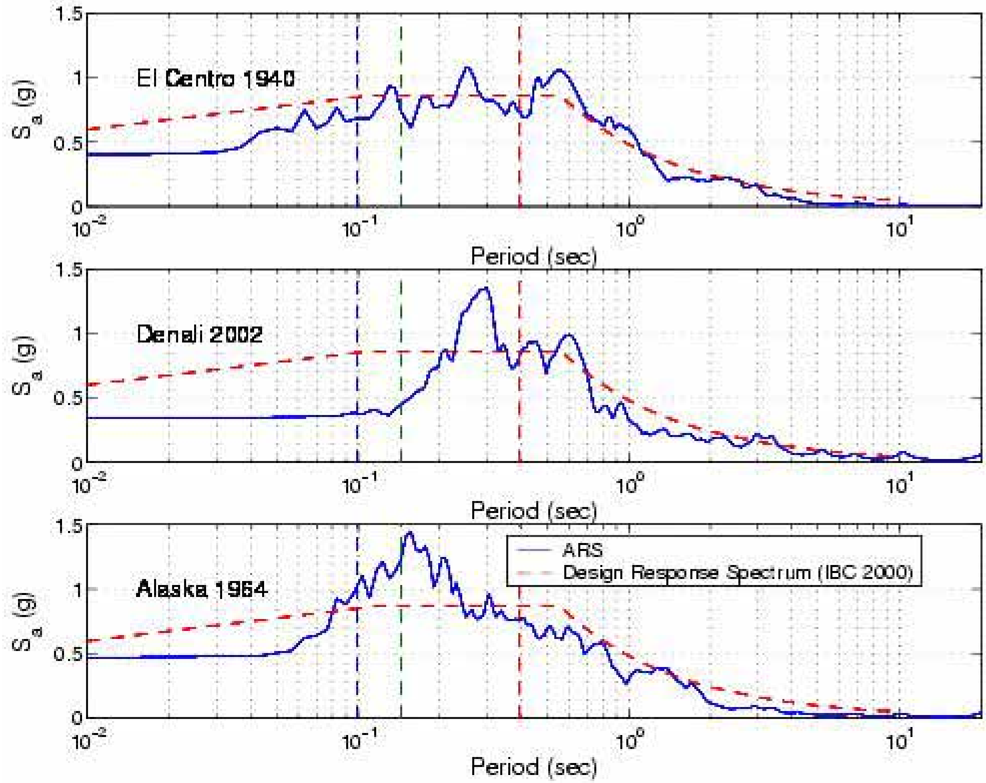


Figure 3 Scaling of Earthquake Records Based on IBC 2000 Design Response Spectrum.

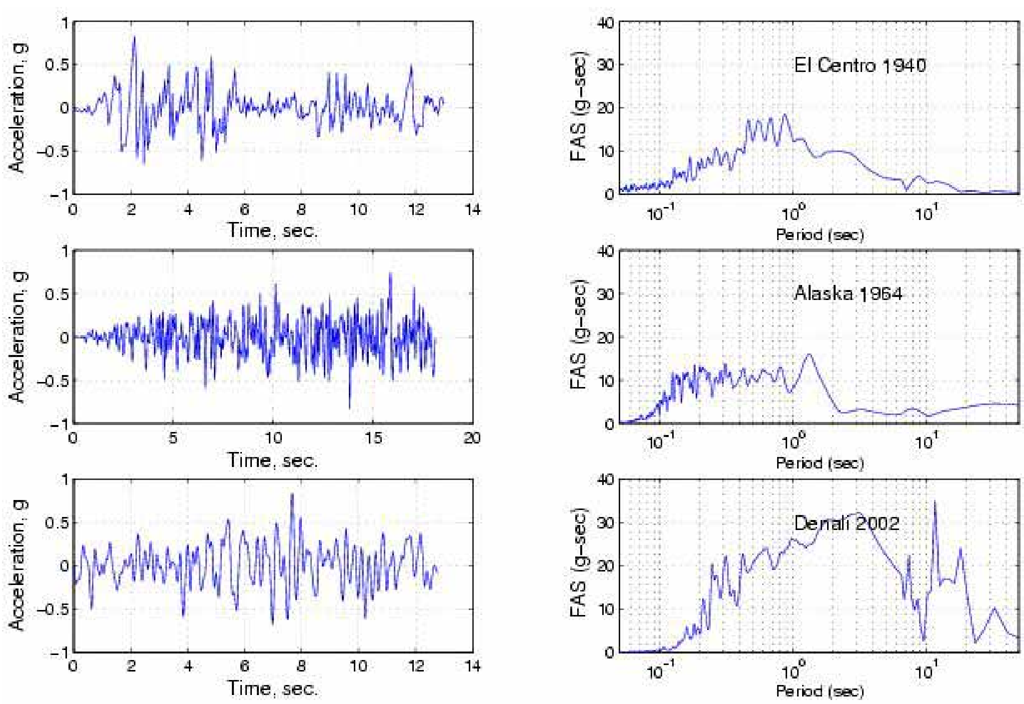


Figure 4 Scaled Earthquake Ground Acceleration Time Histories and Corresponding Fourier Amplitude Spectra.

LINEAR AND NOLINEAR TIME HISTORY ANALYSIS RESULTS

The results of the model analysis including base shear and overturning moment time histories are summarized below.

Base Shear

Figure 5 - 7 (a) to (c) show the linear and nonlinear time histories of total base shear for Tanks A, B and C, respectively. The results show similar linear and nonlinear responses for the first two (2) seconds of the record, after which the nonlinear large deformation response values in Tank A and B are considerably reduced, possibly reflective of yielding. Also noted is that the responses are controlled by the first natural periods (0.144, 0.097 and 0.392 sec., respectively, for Tanks A, B and C) in all cases. Maximum base shears V_{max} picked up from time history curves in both linear and nonlinear cases are also indicated in each figure. The ratios of the maximum values of base shear for the linear elastic assumption to that of the nonlinear, large deformation model are in the range of 0.74 to 2.04. These ratios are much smaller than that suggested in AWWA (AWWA 1996 [1]), the reduction factor R_w of 4.5.

Broad Tank – Tank A

El Centro earthquake: The ARS value of the scaled El Centro earthquake time history is about 80% of the DRS level at the natural period of Tank A, which can be seen in Figure 3. However, from Figure 5 (a), it can be seen that the difference in base shears between linear elastic small deformation and nonlinear large deformation results starts to appear at about 2 seconds after the beginning of the ground shaking, which implies the possible yielding may occur at about 2 seconds. The overall response is dominated by the first natural period (0.144 seconds), typical for the linear solution. The resulting ratio of linear to nonlinear maximum base shear is 2.04.

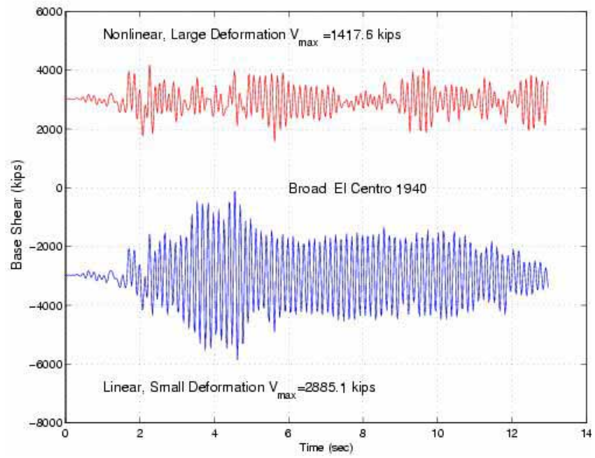
Denali earthquake: The ARS value of the scaled Denali earthquake time history is about 50% of the DRS level at the natural period of Tank A. However, the apparent yielding still possibly occurs before 2 seconds. The ratio of linear to nonlinear peak base shears is 1.16.

1964 Alaska earthquake: The ARS value of the scaled Alaska 1964 earthquake time history is about 140% of DRS level at the natural period of Tank A (see Figure 3). The base shear results show a ratio of 1.70, which is still much smaller than the reduction factor, R_w. Similar yielding after about 2.5 seconds is possible, which can be seen from Figure 5 (c).

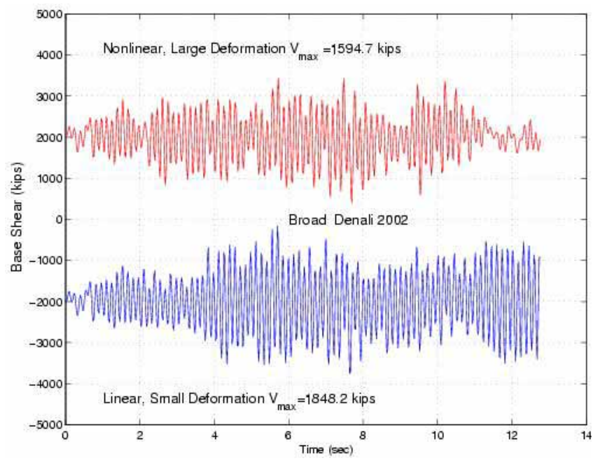
Among the three linear results, the differences are due to variations in input. The greatest base shear occurs with the 1964 Alaska earthquake, due to the higher spectrum value around the natural period of this tank. Base shear values for the linear model are proportional to the acceleration response spectrum. Values for the nonlinear assumptions are not predictable, with the ratio of linear to nonlinear response in the range of 1.16 to 2.04.

Middle Tank – Tank B

El Centro earthquake: The fundamental period of tank B is shorter than Tank A due to the wall thickness proportions. For the El Centro earthquake, the ARS is less than the DRS value, at 80% level, which can be seen in Figure 3. Again, yielding is possible after about 1.5 seconds into the time history input, although the value is small, which can be seen from Figure 6(a). The peak value of base shear for the nonlinear model is greater than the linear model, possibly due to the large deformation assumption. Base shear would be expected to be small after yielding.



(a)



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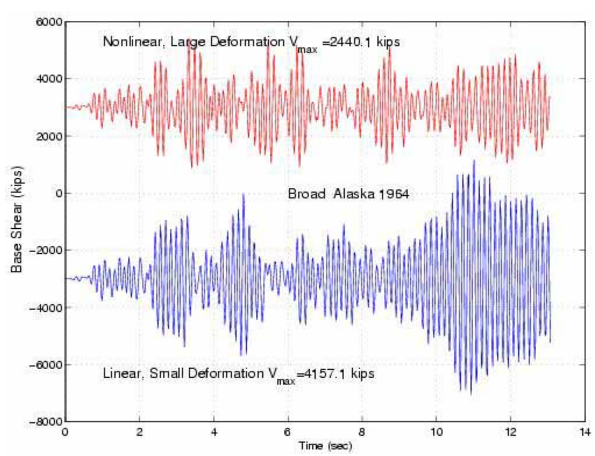
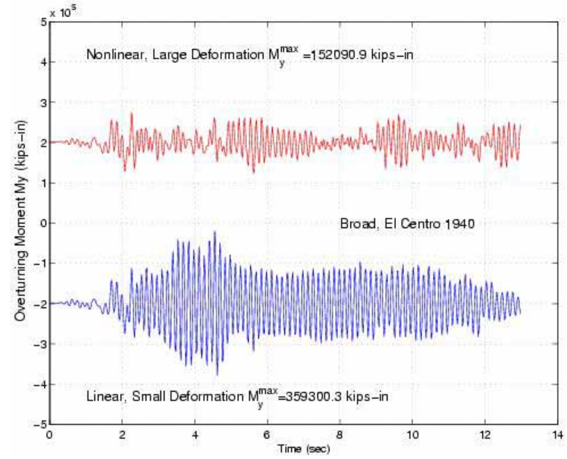


(c)



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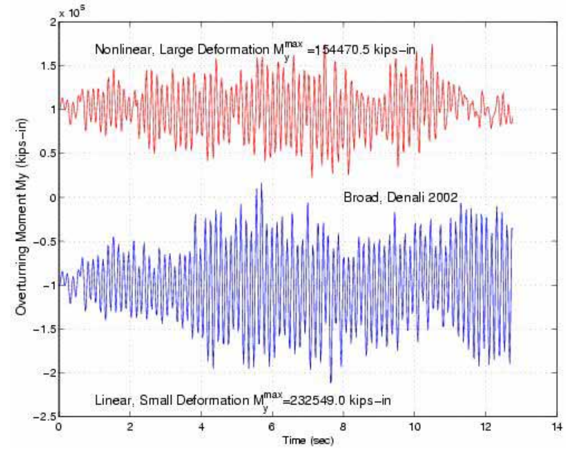
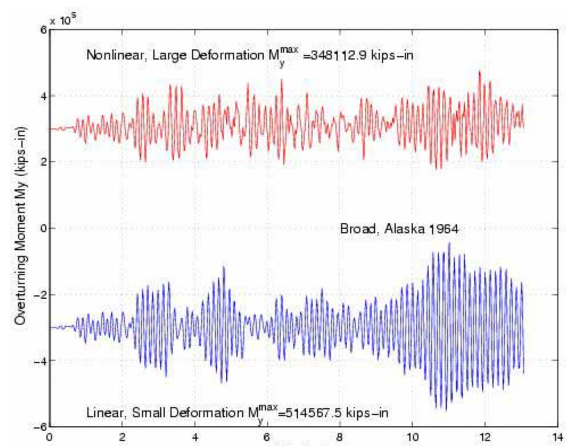


(e)



(f)

Figure 5 Nonlinear and Linear Base Shear and Overturning Moment Time Histories of Tank A ("Broad").

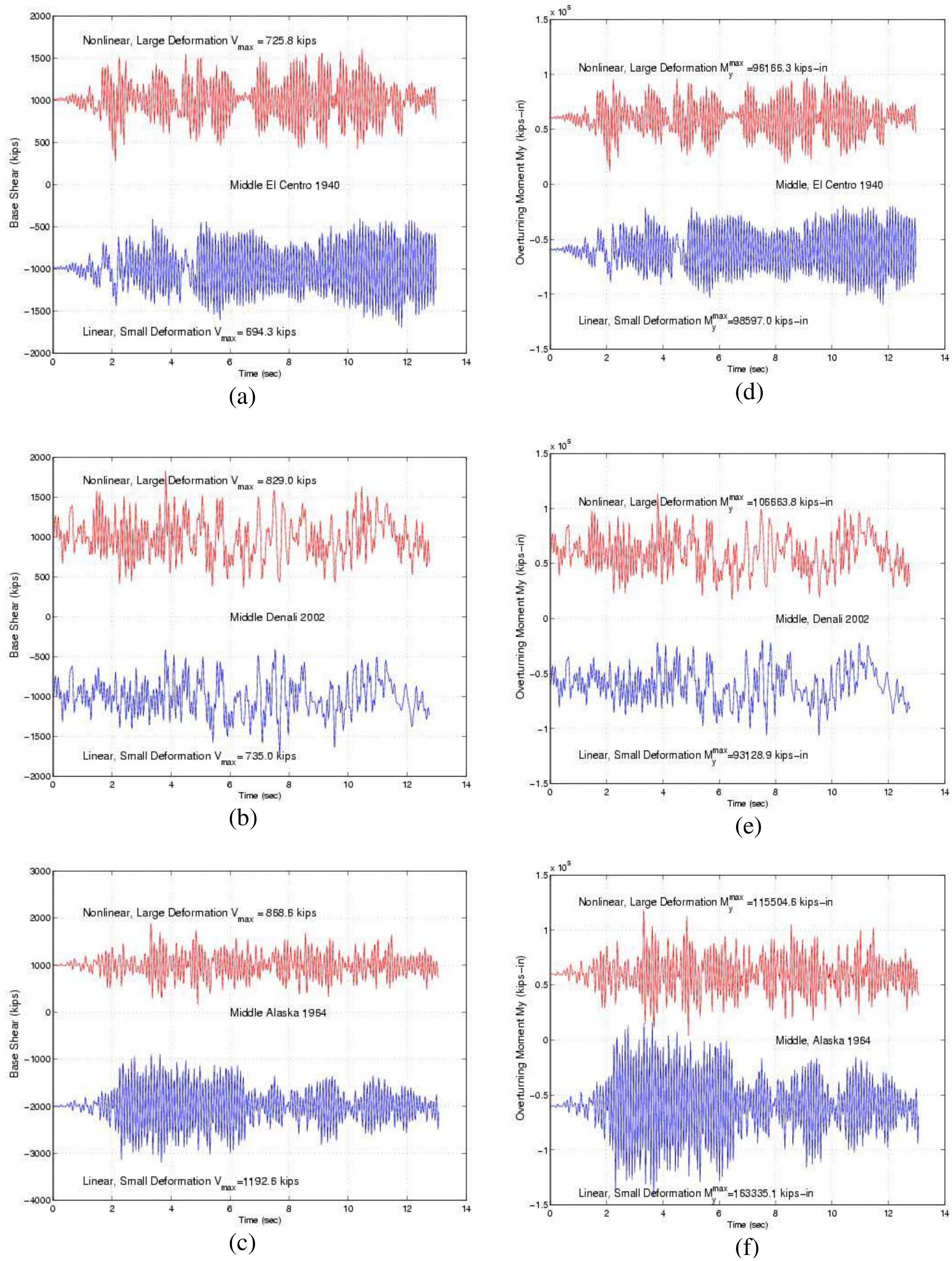


Figure 6 Nonlinear and Linear Base Shear and Overturning Moment Time Histories of Tank B ("Middle").

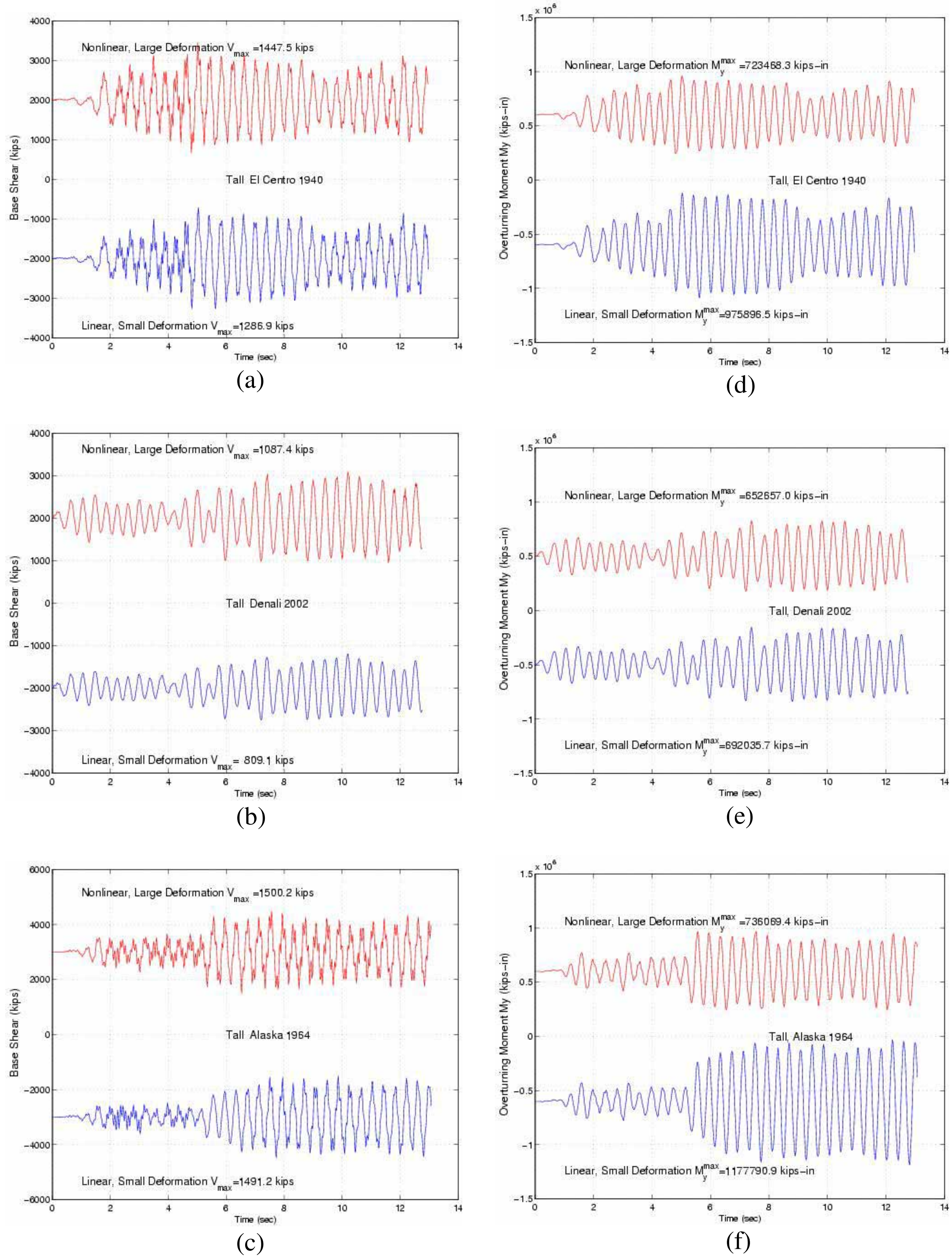


Figure 7 Nonlinear and Linear Base Shears and Overturning Moment Time Histories of Tank C ("Tall").

Denali earthquake: For the Denali input, the ARS is less than the DRS by as much as 50% at the fundamental period. The base shear time histories coincide each other (linear and nonlinear) up to 1.5 second and no significant difference exists in the entire seismic duration, indicative of no significant yielding. The nonlinear model shows a larger base shear, possibly due to the large deformation assumption. The ratio is less than one due to the lower input ground motion.

1964 Alaska earthquake: For the 1964 Alaska earthquake, the ARS value in the vicinity of the tank fundamental period is greater than the design response spectrum, at about 120% level (see Figure 3). The time history of base shear (see Figure 6 (c)) shows the probable effect of yielding after approximately 2 seconds. The ratio between ARS and DRS is comparable, with the resulting peak base shear ratio of maximum time histories of 1.37.

Tall Tank – Tank C

For the El Centro, Denali and 1964 Alaska earthquakes, the ARS values at the tank fundamental period are approximately 80-90% of the DRS, which can be seen in Figure 3. The results show similar base shear response, and no significant yielding occurs (see Figure 7 (a)–(c)). The ratios of linear to nonlinear maximum base shear are 0.74-0.99, which means the base shear values are comparable or lower. The differences in the maximum values are likely due to the large deformation assumption, indicating the importance of including this in the model.

Table 4 shows the comparison of FEA base shear results with calculated results from the current AWWA design code for the three tanks under each time history input. The ratios of linear and nonlinear base shears from the FEA analysis are shown in the parentheses, with the range of 0.74-2.04. The AWWA adjusted base shears, without considering factors I and R_w, are compatible with the linear solutions if the ground inputs are at the level close to the design earthquakes at the tank fundamental periods. However, the base shears computed using the reduction factor, R_w (AWWA D100-96) are much lower than results from the nonlinear analysis, which indicates that tanks designed based on AWWA would not provide enough safety factor in a design level earthquake, particularly for the base anchorage and piping connections.

Overturning Moment

Time history results for overturning moments for Tank A, B and C under the three scaled earthquake inputs are shown in Figure 5, 6 and 7 (d)-(f), respectively. Table 5 summarizes the values for each tank and event, along with the calculated values from the AWWA code (indicated by "AWWA D 100-96" in Table 5) and adjusted value without considering factors I and R_w (indicated by "AWWA adjusted"). The numbers in the parentheses are ratios of linear to nonlinear results from the FEA models. The overall trends of ratios of linear to nonlinear overturning moment solutions are similar to that of base shear. The ratios in overturning moments are slightly higher with the base shears in most of cases. The range of the ratios is 0.87 to 2.36 for all scaled events, which are still lower than R_w/I ratio of 3.6.

DISCUSSION AND CONCLUSIONS

An accurate prediction of the base shear and overturning moment is essential in determining the safety of tanks against shell buckling and uplift. Comparing the linear elastic theory with rigid

Table 4 Comparisons of Base Shear Analysis Results (kN)

	Tank A					
	El Centro		Denali 2002		Alaska 1964	
Nonlinear model	6,305	(2.04)[2]	7,093	(1.16)	10,854	(1.70)
Linear model	12,833		8,221		18,491	
AWWA D100-96	3,238					
AWWA adjusted [1]	11,657					
	Tank B					
	El Centro		Denali 2002		Alaska 1964	
Nonlinear model	3,228	(0.96)	3,687	(0.89)	3,864	(1.37)
Linear model	3,088		3,269		5,305	
AWWA D100-96	1,238					
AWWA adjusted	4,458					
	Tank C					
	El Centro		Denali 2002		Alaska 1964	
Nonlinear model	6,438	(0.89)	4,837	(0.74)	6,673	(0.99)
Linear model	5,724		3,599		6,633	
AWWA D100-96	1,944					
AWWA adjusted	6,998					

[1] The AWWA adjusted values excludes for the use factor, *I*, and the reduction factor, R_w.
[2] The numbers in paraphrases are ratios of peak base shears of linear to nonlinear results.

Table 5 Comparisons of Overturning Moment Analysis Results (kN-m)

	Tank A					
	El Centro		Denali 2002		Alaska 1964	
Nonlinear model	17,183	(2.36)[2]	17,452	(1.51)	39,330	(1.48)
Linear model	40,593		26,273		58,135	
AWWA D100-96	10,338					
AWWA adjusted [1]	37,215					
	Tank B					
	El Centro		Denali 2002		Alaska 1964	
Nonlinear model	10,865	(1.03)	12,051	(0.87)	115,505	(1.41)
Linear model	11,139		10,522		163,335	
AWWA D100-96	3,677					
AWWA adjusted	13,238					
	Tank C					
	El Centro		Denali 2002		Alaska 1964	
Nonlinear model	81,736	(1.35)	73,737	(1.06)	83,161	(1.60)
Linear model	110,256		78,186		133,066	
AWWA D100-96	21,701					
AWWA adjusted	78,123					

tank assumption, the seismic response of a flexible tank may be substantially greater than that of a similar excited rigid tank. However, if yielding occurs in a large area of tank shell and affects the overall behavior changing from linear elastic to nonlinear, the nonlinear response, particularly the base shear and overturning moment, may be lower than that predicted by linear theory. Combining the effects of flexibility of the tank shell and the nonlinear behavior for tanks with different aspect ratios complicates the comparison of results. The discussion and conclusion from the limited cases in this study are as follows:

1. The modeling approach of using FEA allows engineers the option of evaluating the seismic response of liquid storage tanks more accurately than by the use of simple design formulas, and more efficiently than by a more complicated modeling application. Comparison of natural periods and mode shapes of fluid sloshing and fluid/tank system support the validity of the FEA models.

2. Linear base shear and overturning moment time histories from FEA models show a strong correlation with the fundamental periods of tanks in all cases, implying that the responses are governed by the first lateral modes, which is consistent with other published results. Although the nonlinear solutions are path dependent and not predictable, apparent "periods" are still close to the fundamental natural periods of fluid/tank systems.

3. Overall linear elastic small deformation solutions for base shear and overturning moment are close to the AWWA results if the I and R_w factors are excluded. The average FEA linear solution for base shears selected from earthquake events with compatible ARS and DRS values is 1.05 times AWWA solution excluding I and R_w factors. This implies that the FEA linear solutions for base shears in these few cases are slightly higher than theoretical solutions from which the AWWA equations are based. The difference in the FEA linear models and AWWA based theory is that the FEA models includes the effects of tank flexibility, even with the use of small deformation assumptions, while the theory behind AWWA is based on rigid tank assumptions. A similar comparison for overturning moments gives a higher ratio of 1.34. This may imply that tank flexibility plays a more significant role in the overturning moment that is applied to the bottom of the tank shell.

4. The ratios of base shear between linear elastic and nonlinear large deformation models, 0.74-2.04, are less than suggested, $R_w = 4.5$ in AWWA.

5. In all cases studied, for nonlinear analysis, there is no obviously inelastic buckling near the tank base due to the overturning moment applied on the tank shell. The ratios of overturning moments between linear elastic and nonlinear large deformation models, 0.87-2.36, are also less than suggested R_w of 4.5 in AWWA. The reason might be that the yielding of the tank shell has a more significant effect on the resulting overturning moment than on the base shear.

6. The tall tank is more vulnerable to seismic events than one that is broad and flat. Wall flexibility affects the response of tall tanks significantly and the possibility of complete overturning is greater. While base shear dominates the response of broad tanks, foundation overturning moment is more important for tall tanks. For tall tanks, the flexibility affects the impulsive action more than the convective action, resulting in a greater effects on base shear and overturning moment. This may explain why the nonlinear base shear is higher than the linear base shear results for Tank C. Moreover, since the height-to-diameter ratio is a

constant for the convective liquid portion for tall tanks, a constant ratio may be used to determine the convective liquid portion of the tank contents. This leads to a higher impulsive centroid in tanks with the same diameter but greater height-to-diameter ratio. The effects of flexibility of tank wall on the overturning moment applied on the tank shell at the bottom would be more significant than on the base shear. On the other hand, the contribution of the higher impulsive modes to the overall response may be substantial in these cases, which can be seen from figure 7 (a) and (c). It would be useful to further study tall tanks with height-to-diameter ratios around 0.75-2 to gather more results.

7. The above discussions and conclusions are based on a limited number of tanks, seismic inputs, and ground motion levels. More general conclusions must be based on a more complete data base and further studies.
8. The FEA models developed in this study are reasonable to represent tank performance under seismic events. Application of advanced options such as the use of base contact elements, nonlinear material properties, and large deformation analysis, allows future study into liquid storage tank behavior and more detailed evaluation of other complex factors involved in the behavior of liquid storage tanks, such as the hydrodynamic pressure, axial and hoop stresses and strains, inelastic buckling, water surface displacements and shell wall deformations. Future research will also permit a greater understanding of the response of tanks under various seismic loadings, and permit resources to be directed to areas that may have the greatest risks. Refinements in standard procedures for estimating site-specific earthquake conditions will permit performance-based design of tanks and other lifeline structures.

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