DYNAMICAL BEHAVIOUR OF STEEL TOWERS BY NUMERICAL SIMULATION

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ABSTRACT

The growing need to satisfy the national refined oil demand has required to evaluate and upgrading of existing facilities, such as steel towers for oil refining, and build new towers of process, as a result, some towers have been placed in facilities located in high seismic risk areas in the sud-east of Mexico. These tall petrochemical towers, by their geometrical characteristics are thin structures that dissipate less energy than others, such as buildings, as well as a lower damping. As a result, this research focuses on the study of the response and structural behavior by numerical simulation, of an existing process tower, pressurized, and submitted to seism, to carry out the numerical analysis has taken into account their actual geometrical and mechanical characteristics, in addition to using seismic records. The numerical results are compared with analytical methods such as, the vibration theory of beams and vibration theory of shells, given that the dynamical-modal response of the walls of steel tower is like at the thin cylindrical shells, so the local modes of the wall is represented by the contribution of higher modes as curve plates. An important aspect studied, is the boundary condition at the base of the tower, so the transmission of the mechanical elements is made by steel bars of anchorage, that is different to the ideal restrictions at the base, given that there is a slip in the interface by seismic action, which is modelling with contact elements to represent this slip by friction and contact, varying the level of effort in the steel bars. Finally, numerical results allowed warn that the theory of Euler-Bernoulli beams, recommended in design manuals is not a better choice for adequately represent the behaviour of these structures.

Keywords: Steel towers, numerical modelling, dynamical behaviour, evaluation and retrofit.

INTRODUCTION

1.1. Objective and scope

The aim of this research is to know and distinguish the structural behaviour and response of the pressurized petrochemical steel towers by numerical simulation, under seismic actions of steel tall

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towers already existing located in risk zones. Concerning to the actual geometry it can observe that the union of the steel wall of the vessel with the skirt near to the bottom of the structure and the steel anchors securing, modified the dynamical response of the tower and could be to reduce its structural carry capacity.

2. NUMERICAL MODELLING

The classical theory of beams and cylindrical shells together with numerical analysis using the FEM have been using for different conditions such as: case a. Empty tower case b. Pressurized tower (with hydrostatic internal pressure), in order to take into account the initial condition and the action of the earthquake. To estimate the seismic response of the steel tower, we used real seismic signals recorded in Minatitlan, Veracruz, which were applied at the base of the structure by analysis in time, step by step analysis.

2.1. Geometric properties and mechanical properties

The thickness of the tower walls vary along the height *L*, in Table 1 shows the geometric and mechanical characteristics of the structure.

Table 1: Geometric properties of the tower and mechanical properties of steel

Steel thickness t	15.875	First sector - body thickness (mm)
	12.7	Second sector and upper cap (mm)
	10	Skirt thickness of the wall (mm)
L	28.207	Height of the tower (m)
R	2.05946E+08	Young modulus of the steel (kN/m^2)
E	1.2955	middle radius of the tower (m)
ν	0.3	Poisson ratio of the steel
γ_s	76.910	Weight per unit volume of the steel kN/m^3
ρ	7.846	Mass per unit volume of the steel (kN/m^3)/g

2.2. Modeling processes steel tower

In this section it studies the structural behavior of a steel tower subject to the seismic action, employing dynamic analysis and step by step analysis through numerical modeling with FEM.

2.2.1. Linear analysis and meshing

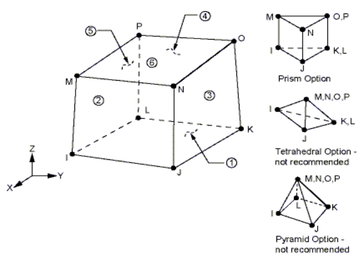
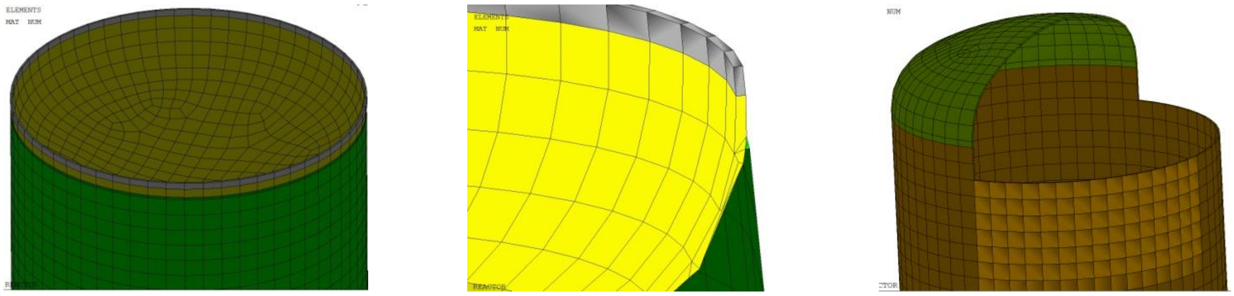


Figure 1: Solid element 185.

The steel tower is considered an axisymmetric structure, so to represent the structure was built a 3D numerical model with solid elements representing axisymmetric structure walls. The numerical analyses were made by using ANSYS program 14. The solid element used to carry out the study of the tower, is solid 185 as shown in Figure 1, which is defined by eight nodes with three degrees of freedom at each node (translational displacements in three orthogonal directions u_x, u_y, u_z), also this element is able to take large deformations.

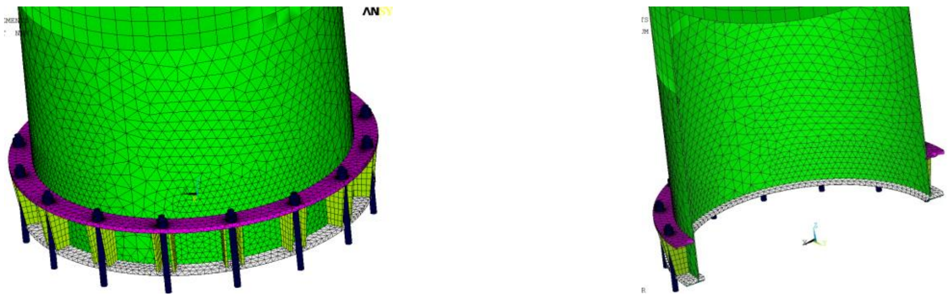
2.2.2. Meshing and boundary conditions

For the numerical modeling is created a meshing of the steel walls with solid elements 185, the details for the two models used in the study; dynamic analysis and transient seismic analysis are shown in table 2 and in figures 2, 3. The numerical model has a mesh with aspect ratios a / b = 1.57.



(a) (b) (c)

Figure 2: a. Steel skirt tower, b Detail of the connection between the skirt and the lower body of the tower, c Upper zone, body cap and the steel walls.



(a) (b)

Figure 3: Connection between the skirt and the foundation with rings, stiffeners and anchors securing the structure to the foundation.

Table 2: Meshing of structure with 185 solid elements and boundary conditions

Height L (m)	Element	Boundary conditions	at the base	Nodes	Elements
28.207	Solid 185	built-in	z=0	47786	58971
		contact elements	z=0	46043	59152

2.2.3. Dynamic analysis of tower

The modal dynamic analysis employed, are based primarily on the classical theory of vibrations of the Euler-Bernoulli beam with continuous mass (recommended by MDOC-CFE-2008) and the second method used is that of the theory of vibrations of axisymmetric structures with thin wall "cylindrical shells". Through applying these theoretical methods, the dynamic parameters obtained such as natural periods and mode shapes of the steel tower and compared with the results of numerical models to validate latter.

2.2.4. Analytical model (Euler-Bernoulli)

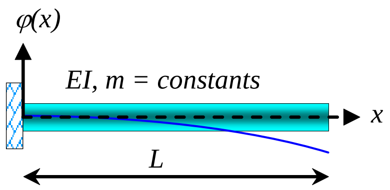


Figure 5: Analysis model (Euler-Bernoulli).

The analysis of these structures requires taking into account other parameters than the urban buildings. Thus this structure is idealized as Euler-Bernoulli beam (figure 5), where it is assumed small displacements, the effect of shear is discarded and the deflection occurs in the plane. So that the tower cross section is regarded as a tubular section subjected to the action dynamics f=f (x, t), v=v(x, t) is the lateral displacement and the mass per unit length $\mu = \mu\ (x)$, so that the equation of motion for the system is the equation1;

$$\frac{d^2}{dx^2}\left(EI\frac{d^2v}{dx^2}\right) = -\mu\frac{d^2v}{dt^2} + f \qquad (1)$$

also, when generating free vibrations, *f=0*, normal modes of vibration appear to take the form $v(x,t) = \varphi(x)sen(\omega t + \epsilon)$, substituting this expression in Equation 1 is obtained as follows, Equation 2.

$$\frac{d^2}{dx^2}\left(EI\frac{d^2}{dx^2}\varphi\right) = \omega^2\mu\varphi \qquad (2)$$

The solution of the equation 2 together with the boundary conditions of the *cantilever beam* provides the natural frequencies of vibration of the *i-th* modes and their configuration, where EI is the bending stiffness, and then the frequency expression is, $\omega_i = (\lambda_i)^2\sqrt{\frac{EI}{\bar{m}L^4}}$; were: $\lambda = L^4\sqrt{\frac{\mu\omega^2}{EI}}$; the roots for five first modes are shown in table 3.

Table 3: Natural frequencies of vibration of the first five modes

Mode	λ_i	ω_i

1	1.875	
2	4.69409	
3	7.85475	$\omega_i = (\lambda_i)^2 \sqrt{\frac{EI}{\overline{m}L^4}}$
4	10.9955	
5	14.13716	

2.2.5. Analytical model of a cylindrical shell (thin Structure Theory)

In the literature are different theories which differ from each other in terms of the differential equations describing the kinematic behavior of thin wall curved elements, among the most used theories are those of A. E. Love, W. Flügge, L. H. Donnell, the differences between these theories is mainly due to the assumptions made about the smaller terms, and the of upper order that are retained for consideration in the analysis. Donnell's theory is the simplest, and Flügge theory generally has higher approximation. Sometimes these theories presented results which are significantly different. However, for the intervals which are used in engineering, these theories become very similar. Each of the known theories describing the movement of the shells in terms of differential equations, and inertia terms associated with each of the three mutually orthogonal displacements for a cylindrical shell. If the spatial dependence of each of the strains could be estimated, then the natural frequency of the plate could be reduced to the solution to a cubic characteristic polynomial, and the relative amplitude of the three displacements could be found by the approach of a matrix 3 by 3, representing the linear simultaneous equations. Based on the above, the second method used to determining the frequencies and periods was of the thin shell structure theory. Then the expression used to calculate the natural frequencies and vibration modes is a cubic equation (Warburton, 1976), for the boundary condition of the tower, fixe at the base and free at the top. The cubic equation is expressed in terms of the dimensionless frequency "**Δ**", see equation 3.

$$\Delta^3 - K_2\Delta\Delta^2 + K_1\Delta - K_0 = 0 \qquad (3)$$

where: $\Delta = \rho R^2(1-\nu^2)\omega^2/E;\quad f = \frac{\omega}{2\pi R}\sqrt{\frac{E\Delta}{\rho(1-\nu^2)}};\quad T = {}^{1}/_{f}$

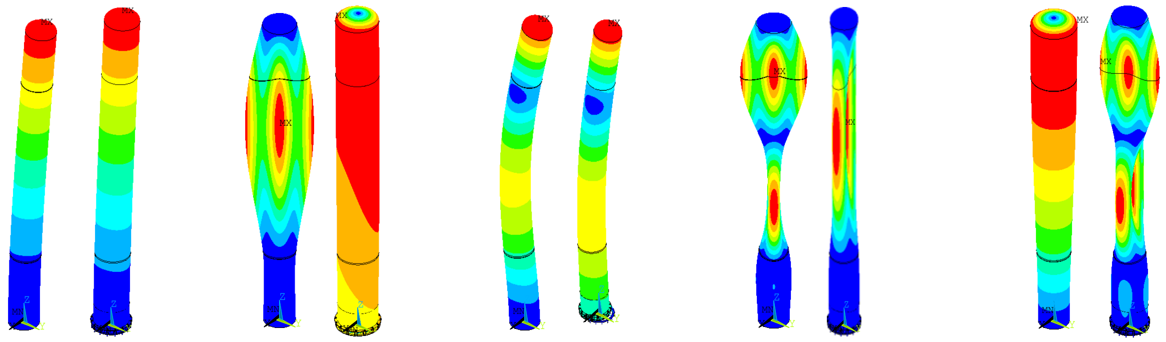
wherein, $\boldsymbol{f}$ is the natural frequency, $\boldsymbol{R}$ the radius of the cylinder, $\boldsymbol{t}$ the thickness of the plate, and $\boldsymbol{\omega}$ circular frequency, $\boldsymbol{L}$ is the axial length of the cylinder, $\boldsymbol{E}$ is the modulus of elasticity of material, $\boldsymbol{\rho}$ is the density of the material, $\boldsymbol{n}$ = number of circumferential waves, and $\boldsymbol{m}$ = number of half-wave axial. K_0, K_1 and K_2 depend on $\boldsymbol{m}$ and $\boldsymbol{n}$ (Sanchez et al, 2001).

3. NUMERICAL RESULTS

In this section are present numerical results for three cases studied: **a**. rigid base tower with constant wall thickness t on high L, **b**. rigid base tower with variable thickness t, on all walls, see Table 1 (actual structure) and case **c**. tower with contact elements on the base and walls of variable thickness t at the structure (actual structure). Firstly, modal dynamic analysis was performed with in order to know the dynamic parameters (natural periods and mode shapes), also to validate the 3D numerical models and finally was carried out the time history analysis (transient analysis).

3.1. Analytical results as Euler Bernoulli beam (*case a*) and cylindrical shell (*case b*) and numerical results of the modal dynamic analysis

In Table 4 are shown the analytical results, as Euler Bernoulli beam (case a), and cylindrical shell (case b) for n=30 and m=1, the tangential and axial modes, for boundary conditions built–free. Also in the figures 6, 7 show numerical results of the modal dynamic analysis for the three cases of study: case **a**, case **b** and case **c**.



a b	*a b*	*a b*	*a b*	*a b*
Mode 1	Mode 3	Mode 4	Mode 6	Mode 8
Case a. - T_1= 0.33882	T_3=0.06053	T_4=0.05798	T_6=0.04018	T_8=0.03730
Case b. - T_1= 0.38067	T_3=0.08084	T_4=0.07324	T_6=0.04756	T_8=0.03572

Figure 6: Case b. and case c (*actual structure*), modal configuration.

Table 4: Frequencies and periods of the analytical and numerical results

	Analytical results				**Numerical results**					
Tangential	**As beam Case a**		**As cylindrical shell Case b**		**Case a**		**Case b**		**Case b**	
modes ***n***	f_i *hertz*	T_i *(sec)*	f_i *hertz*	T_i *(sec)*	f_i *hertz*	T_i *(sec)*	f_i *hertz*	T_i *(sec)*	f_i *hertz*	T_i *(sec)*
1	3.199	0.3126	3.257	0.307	2.7798	0.35974	2.95141	0.33882	2.6270	0.38067
2	0.0499	125.994	6.359	0.1573	2.7798	0.35974	2.95141	0.33882	2.6278	0.38055
3	0.0178	352.787	17.743	0.0564	16.5579	0.06039	16.52073	0.06053	12.3704	0.08084
4	0.0091	691.318	33.997	0.0294	16.5583	0.06039	16.52111	0.06053	13.6544	0.07324
5	0.0055	1141.91	54.971	0.0182	16.7840	0.05958	17.24739	0.05798	13.6607	0.07320
6	-	-	80.634	0.0124	16.7840	0.05958	17.24739	0.05798	21.0281	0.04756
7	-	-	110.975	0.009	24.9782	0.04003	24.89060	0.04018	21.0297	0.04755
8	-	-	145.992	0.0068	24.9792	0.04003	24.89158	0.04017	27.9989	0.03572

3.2. Comparison between theoretical and numerical results of the dynamic analysis

Figure 7 shows the comparison between the theoretical results, like beam and axisymmetric shell structure, and the numerical results obtained from simulations with the finite element method of the dynamic analysis, in this graph shows curves *vs*. periods $\mathbf{T_i}$ vibration and modes. It can observe that for the case of the empty condition of variation between the numerical results are in the order of 8, 14 and 20% higher than the theoretical results; this difference could be attributed to the mass distribution at the top of the tower, the boundary conditions and the variation in wall thickness of the structure. However, for the higher modes of vibration periods tend to approach. Finally this comparison has served to validate the numerical models and to perform other analyzes in time.

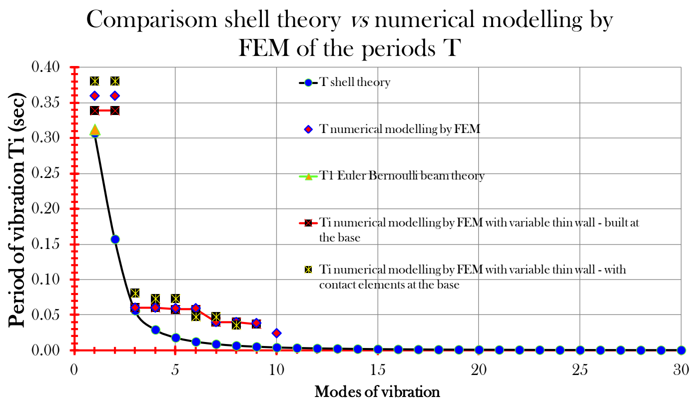


Figure 7: Comparison between theoretical and numerical results.

3.3. Time history analysis

Before carrying out the transient analysis, firstly, the container is pressurized (*p_i=0.3434Mpa*), at the second is applied the hydrostatic pressure of the liquid *p_h=0.0466Mpa*, see Figure 14.c, and finally the transient analysis is carried out with records obtained for the Minatitlan earthquake of October 24, 1980; the Figure 8 shows the acceleration record employed and its response spectrum.

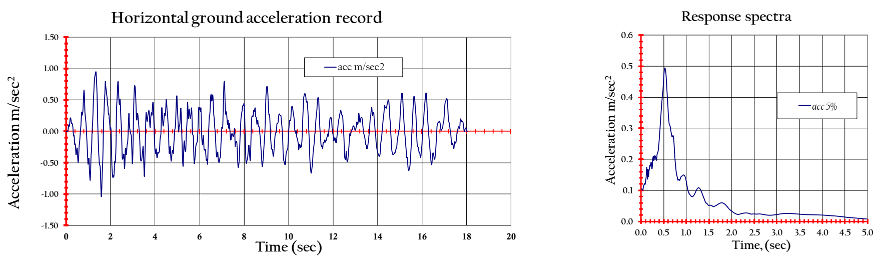


Figure 8: Horizontal ground acceleration record and response spectra.

3.3.1. Analysis step by step (transient analysis)

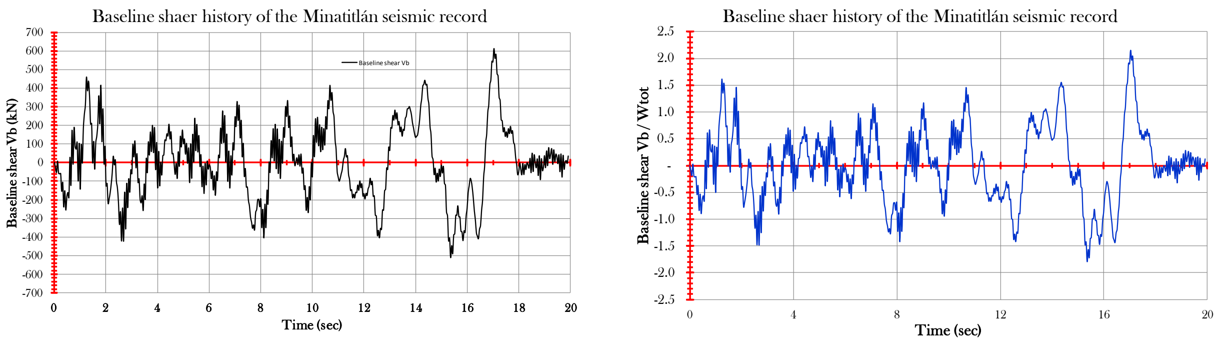
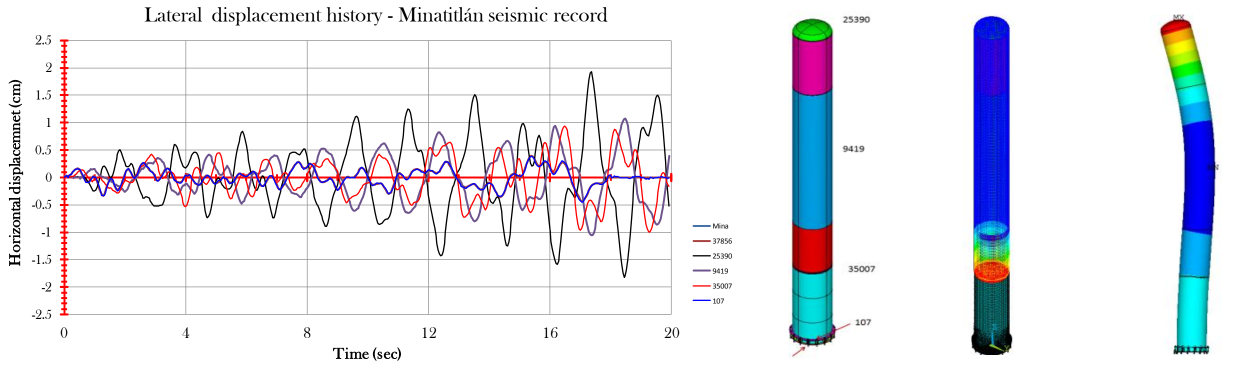


Figure 9: Base shear V_B History. **Figure 10: History of the relationship V_v/W.**

The figures 9 to 11 illustrate the history of basal shear response, Vb, and the normalized response to the total weight of the structure (V_b/W), as well as the history of the lateral displacements of the top of the tower, obtained with the Minatitlan seismic recording. The maximum excitations occur at the times: 1.24, 2.68, 8.12, 10.68, 12.6, 14.36, 15.36 and 17.04 seconds, and the ratio (V_b/W) is about 1.5 to 2 times the seismic coefficient. Figure 11 illustrates the history of the lateral peak displacement ux; it is observed that the maximum displacement of 1.93cm is presented to 17.36 sec. (black line, node 25390).



a. b. c. pressurized d. 17:32sec

Figure 11: History of the maximum displacements u_x.

4. CONCLUSIONS

Regarding the results of the numerical dynamic analysis and of the theories of vibration of shells, the fundamental periods T1 obtained numerically, showed be greater in 17, 11 and 24% compared to those obtained analytically. It was observed that this difference results from the variation of mass for the three cases studied, empty tower and pressurized, due to variation of the thickness of the walls of the tower, which influence the results. This difference was reduced for the periods exceeding the 14 mode. It is important to note that there is a good correlation between the theoretical and numerical models developed in this research, since the results were satisfactory for a mesh not extremely discrete, since solid elements are used. Thus, through using these models is possible visualized in detail the structural response, given that the actual structure shows a complexity in their connections between the walls and the base of the tower with anchors. With respect to the transient analysis, it was possible to study the structural response of the tower through the lateral movement history, and it is possible to identify the sequence of strain in the most intense phase corresponding to the range of time between 17:04 to 17: 40 seconds.

Finally, numerical results allowed warn that the theory of Euler-Bernoulli beams; recommended by design manuals do not adequately represent the behavior of these structures. Hence it is recommended to carry out dynamic analysis, considering these towers as axisymmetric structures, and that through them you can observe their behavior, such as, their modal configurations, which can help us identify possible failure by buckling, in some parts of the walls of vessel, as well as carrying out geometric nonlinear analysis and material.

5. ACKNOWLEDGMENTS

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