STRUCTURAL BEHAVIOR OF PROCESS STEEL TOWERS SUBMITTED TO SEISMIC ACTIONS

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SUMMARY:
This research studied the structural behaviour of the steel towers submitted to seismic actions by numerical simulation using the seismic ground motions and the numerical results are compared with analytical methods such as beam and shell vibration theories. Consequently, this work is focused to know the structural behaviour, under seismic actions, of already existing steel tall towers located in high risk zones. The petrochemical tall towers for its geometry characteristics are slender structures that dissipative energy is less than the others such as buildings and also it damping is fewer. And it has carry out a procedure by numerical simulation who takes into a count the mechanic characteristics of the materials and the real geometrical of the existing steel towers. The numerical results have been compared with the seismic design CFE-2008 code. Finally, the results will be used to determine new general regulations to design the steel petrochemical towers in our country.

Keywords: Towers, tall towers, steel tall towers, structural response of towers, evaluation and retrofit.

1. INTRODUCTION

The growing need to satisfy the petrochemical oil industry demand has required in the last years the evaluation and retrofit of the existing facilities such as, petrochemical steel tall towers, and new facilities. Therefore, some tall towers have been placed in high seismic risk areas of Mexico. Then, this work is a part of the evaluation program of the earthquake risk of the oil industry. Consequently we are carrying out diverse studies about the seismic risk in all refinery of the country. In this article are presented some numerical results made about the process steel tower.

1.1. Problem statement

The petrochemical steel tall towers for its geometry are characterised as a slender structures that their dissipative energy is less than the others such as buildings and also it damping is fewer. Normally in the regulations such as CFE-2008, it is recommended a simplified method based in the beam theory for analysis and design, but this type of steel tower are considered and analysed as an axisymmetric cylindrical shell. For this reason in this research the towers subjected to seismic action are analysed by analytical and numerical methods using the FEM.

1.2. Objective and scope

The aim of this research is to know and distinguish the structural behaviour and response of the pressured petrochemical steel towers by numerical simulation under seismic actions, of already existing steel tall towers located in risk zones. Concerning to the real geometry it can observe that the union of the steel wall of the vessel with the skirt near to the bottom of the structure, modified the dynamical response of the tower and could be to reduce its structural carry capacity.

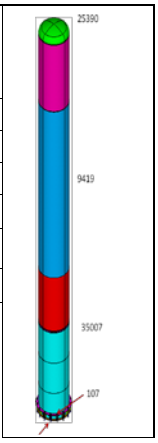
2. NUMERICAL MODELLING OF THE STRUCTURE STUDIED

Classical vibration shell theory of the axisymmetric structures together with numeric analysis by FEM has been used to different conditions: empty towers and internal pressure and hydrostatic cases to the tower structure are considered, in order to take into account the initial conditions and the earthquake action. To estimate the seismic response of the steel towers, real seismic record obtained at Minatitlan, Veracruz, Mexico was applied at the base of the structure, to carry out the analysis step by step.

The geometrical and mechanical properties of the steel tower are showed in table 2.1.

Table 2.1. Geometrical and mechanical properties of the steel tower

Steel thickness t=	15.875 12.7 10	First sector - body thickness (mm) Second sector and upper cap (mm) Skirt thickness of the wall (mm)
L=	28.207	Height of the tower (m)
E=	2.05946E+08	Young modulus of the steel (kN/m^2)
R=	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



2.1. Modelling of the Steel Process Tower

In this section it is studied the structural behaviour of the steel towers subject to the seismic action, the numerical simulation is carried out through of the, static, dynamic and step-by-step numerical analysis by FEM to obtain the seismic response, where the steel walls of the tower are modelled with solid elements solid185.

2.1.1. Linear Analysis and Meshing

The steel tower is considered as an axisymmetric structure, then is used a 3-D modelling with solid elements to represent the steel walls of the tower. For the meshing and numerical modelling it is employed the FE ANSYS 11 program. The element solid185 used for 3-D modelling is showed in figure 2.1 and is defined by eight nodes having three degree of freedom at each node: translation in the nodal x, y, and z directions. The element has large deflection, large strain capabilities.

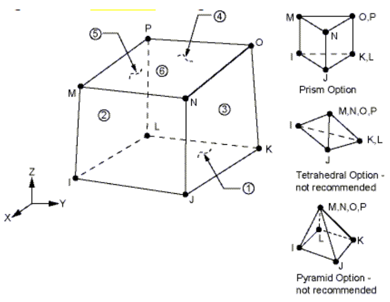


Figure 2.1. Solid element 185

2.1.2. Meshing and boundary conditions

The axisymmetric thin wall structures were modelled with solid elements; these elements take in a cont the mechanical properties such as: young modulus E_s and yielding f_y stress into the numerical model. The meshing of the numerical models was carried out using the mesh tool commands included in the ANSYS 11 program. The numerical model has a mesh of the steel wall with an aspect ratio a/b=1.57. The figures 2.2 to 2.5 and table 2.2 show the meshing for the two numerical models with solid element185 to carry the static, dynamical and transitory analysis (seismic analysis).

Table 2.2. Meshing with solid element solid185 and boundary conditions.

Height L (m)	Element	Boundary conditions	At the base	Nodes	Elements
28.207	Solid 185	Built	z=0	47786	58971
		Contact element	z=0	46043	59152

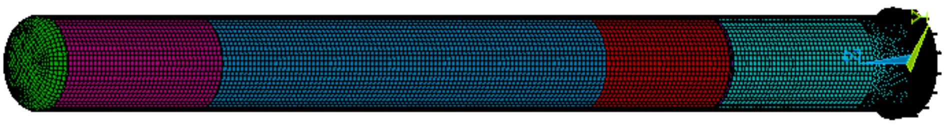
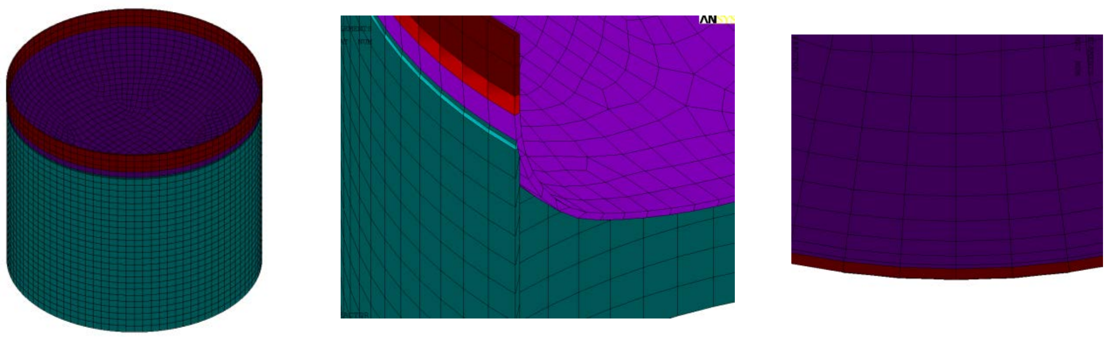


Figure 2.2. Steel tower complete model with element solid185



(a) (b) (c)

Figure 2.3. a. Steel skirt of tower b. Detail of the connection between the skirt and lower cap and c. Steel walls

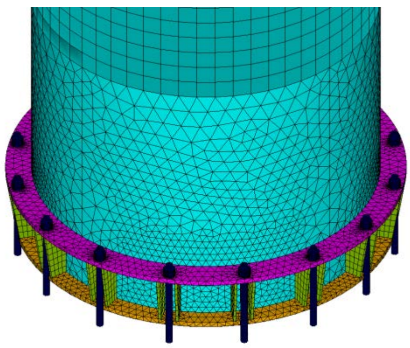


Figure 2.4. Lower segment at the level base of tower and the compression rings

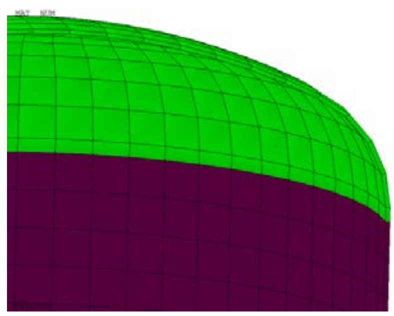


Figure 2.5. Upper cap and segment of the cylindrical wall of the steel tower

2.2. Dynamical modal analysis of the Steel Process

Classical Euler-Bernoulli beam theory and shell vibration theory for axisymmetric structures used to obtain the dynamic parameters such as; natural periods and modal configuration modes of the steel tower, these analytical values are compared with the numerical results to validate the modelling for elastic range.

2.2.1. Analytical model (Euler-Bernoulli beam)

The dynamical analysis of these types of structures require take into a count other structural aspect than the buildings. These structures are idealized as Euler-Bernoulli beam, based on the bending theory of beams, then it is supposed small displacement, the shear effect is ignored and the deflexion is presented on the plane. Consequently, the steel tower is analyzed with Euler-Bernoulli beam theory see figure 2.6, the transversal section is considered like a tube submitted to the dynamic action $f = f(x,t)$, $v = v(x,t)$ is defined as the lateral displacement and $\mu = \mu(x)$ is the mass per unit of long, then the dynamic differential equation of the system is:

$$\frac{d^2}{dx^2}\left(EI\frac{d^2v}{dx^2}\right) = -\mu\frac{d^2v}{dt^2} + f \tag{2.1}$$

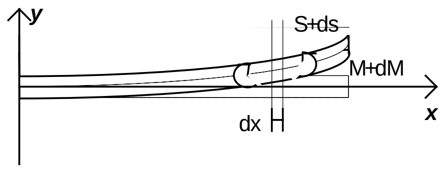


Figure 2.6. Analytical model (Euler-Bernoulli beam)

Finally, considering a free vibration, the equation 2.1 arrives to the follow differential equation 2.2:

$$\frac{d^2}{dx^2}\left(EI\frac{d^2}{dx^2}\varphi\right) = \omega^2\mu\varphi \tag{2.2}$$

The solution of the equation 2.2 together with the boundary condition, cantilever beam, to give the natural frequencies of each one of the i-th vibration modes and its eigenvectors, were EI is the flexural rigidly, then the general solution is:

$$\varphi(x) = C_1\,cosh\frac{\lambda x}{L} + C_2 senh\frac{\lambda x}{L} + C_3 cos\frac{\lambda x}{L} + C_4 sen\frac{\lambda x}{L} \tag{2.3}$$

were: $\lambda = L^4\sqrt{\frac{\mu\omega^2}{EI}}$ and the boundary conditions used for this study are built-free:

$$\varphi(0) = \varphi'(0) = \varphi''(\mathrm{L}) = \varphi'''(\mathrm{L}) = 0 \tag{2.4}$$

$$\lambda_1 = 1.8751, \lambda_2 = 4.69409, \lambda_3 = 7.85475, \lambda_4 = 10.9955, \lambda_5 = 14.13716 \tag{2.5}$$

and for the upper modes it is considered $\lambda = (2n-1)\pi/4$. The table 2.3 presents the circular frequencies for the first five modes.

Table 2.3. Analytical solution $\boldsymbol{\lambda_i}$ and $\boldsymbol{\omega_i}$ (natural circular frequencies) for first fives modes

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

2.2.2 Analytical model (vibration of the cylindrical shells)

The other approach applied to know the natural frequencies and periods is the axisymmetric vibration shell theory for the cylindrical shells. Then the expression employed in the work to obtain the dynamical parameters of the tower is the cubic equation, Warburton, (1976), These cubic equations, for the case, built at the base and free at the upper of the tower, depend of the adimensionales factor Δ of frequency; the roots define the natural frequencies of vibration of the cylindrical shells, Sánchez et al, (2001).

$$\Delta^3 - K_2 \Delta \Delta^2 + K_1 \Delta - K_0 \tag{2.6}$$

$$\text{were: } \Delta = \rho R^2 (1 - \nu^2) \omega^2 / E \tag{2.7}$$

$$f = \frac{\omega}{2\pi R} \sqrt{\frac{E\Delta}{\rho(1-\nu^2)}} \tag{2.8}$$

were, ***f*** is the natural frequency, R is the middle radius of the tower, t is the thickness of the shell steel wall, ω is the natural circular frequency, E is the Young modulus of the steel, ρ is the mass per unit volume of the steel, ν is the Poisson ratio of the steel, n = number of the tangential wave and m = number of the semi-wave axial

2.3. Seismic Records

The seismic step-by-step analyses were carried out employing a seismic record obtained at Minatitlan, Veracruz, Mexico, the 24 October 1980, the figure 2.7 shows the acceleration record and response spectra. The real seismic record was applied at the base of the models.

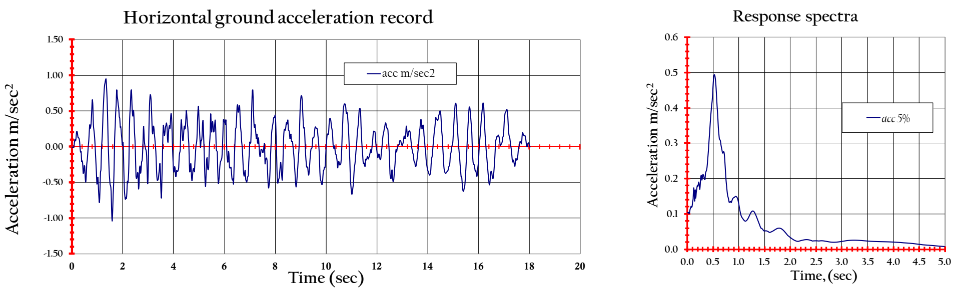


Figure 2.7. Horizontal ground acceleration record and response spectra, Minatitlan, Veracruz

3. NUMERICAL RESULTS

In this section are presented the numerical results for three cases: a. steel tower with rigid base and thin wall constant along of the height L, b. tower with rigid base, and thin wall variable along of the height L (real structure) and the third case c. tower with contact element at the base and thin wall of steel variable along of the height L. Firstly was carried out the dynamical analysis with the aim to know the dynamical parameters (natural periods and modal configuration) and validate the 3D numerical models and finally it made the time history analysis (transitory analysis).

3.1. Analytical results as Euler Bernoulli beam (case a)

Table 3.1. Analytical results as Euler-Bernoulli beam (frequencies and periods)

mode	ω_i	ω_i	T_i (sec)	f_i hertz
1	$\omega_i = (\lambda_i)^2 \sqrt{\frac{EI}{\overline{m}L^4}}$	20.103	0.3126	3.199
2		125.994	0.0499	20.053
3		352.787	0.0178	56.148
4		691.318	0.0091	110.027
5		1141.906	0.0055	181.740

The analytical results obtain as the Euler Bernoulli beam are presented in the table 3.1 for first five modes.

3.2. Analytical results as cylindrical shell (case a)

The numerical results for the first ten modes obtain with the axisymmetric vibration shell theory for the cylindrical shells are presented in the table 3.2 and in the figure 3.1 is displayed the period curve of the one to 30 tangential modes.

Table 3.2. Analytical results (frequencies and periods)

mode	f hertz	ω omega = $2\pi/T_1$	T (sec)
1	3.2569	20.4639	0.307
2	6.3591	39.9557	0.1573
3	17.7423	111.4784	0.0564
4	33.9972	213.6113	0.0294
5	54.9707	345.3922	0.0182
6	80.6335	506.6364	0.0124
7	110.9754	697.2808	0.009
8	145.9922	917.2981	0.0068
9	185.6814	1166.6735	0.0054
10	230.0417	1445.3979	0.0043

axial harmonica m=	1
circumferential harmonica n=	30
Boundary conditions	Built – free

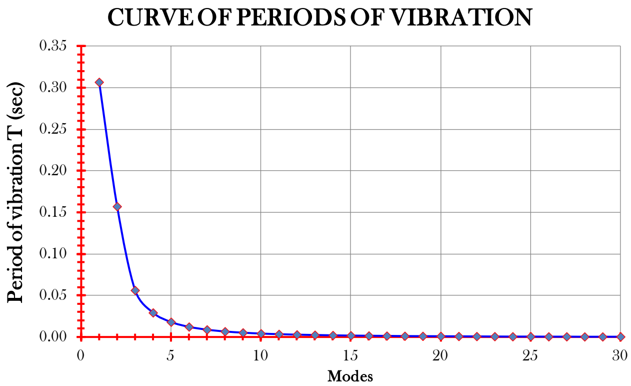


Figure 3.1. Curve of natural periods T *vs* tangential modes n, built- free case

3.2. Numerical results of the dynamic modal analysis

The numerical results of the dynamical modal analysis were carried out for the three cases: **a.** tower with rigid base and thinness wall constant along of the height L, **b.** tower with rigid base, and thinness wall variable along of the height L and the third case **c.** tower with contact element at the base and thinness wall of steel variable along of the height L (real structure) with the aim to validate the 3D numerical models and know the dynamical parameters.

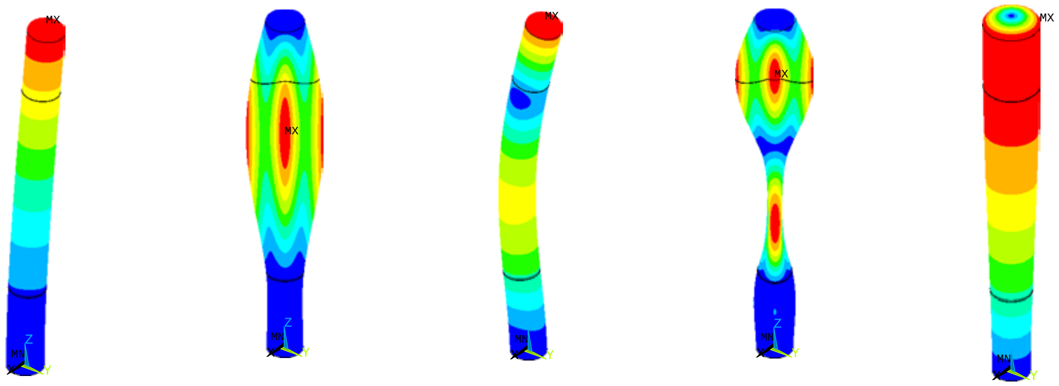
3.2.1. Case a – Empty tower with rigid base and thinness wall constant along of the height L

Table 3.3. Frequencies and periods

mode	f hertz	T_1 num (seg)	mode	f hertz	T_1 num (seg)
1	2.7798	0.35974	6	16.7840	0.05958
2	2.7798	0.35974	7	24.9782	0.04003
3	16.5579	0.06039	8	24.9792	0.04003
4	16.5583	0.06039	9	25.8896	0.03863
5	16.7840	0.05958	10	41.2141	0.02426

3.2.2. Case b - Empty tower with rigid base, and thinness wall variable along of the height L

The numerical results for case b are showed in figure 3.2 and table 3.4.



Mode 1; T_1= 0.33882 | Mode 3; T_3=0.06053 | Mode 4; T_4=0.05798 | Mode 6; T_6=0.04018 | Mode 8; axial mode T_8=0.03730

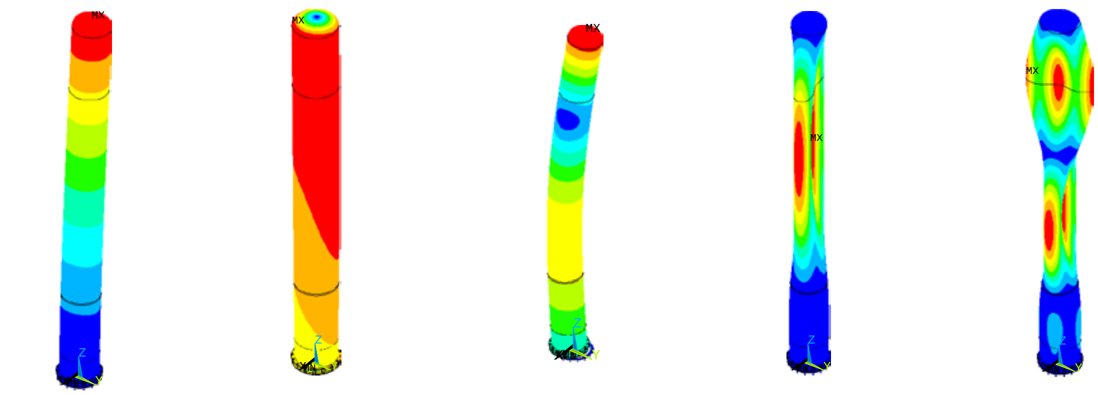
Figure 3.2. Natural periods T and modal configuration

Table 3.4. Frequencies and periods

mode	fi hertz	T_i num (sec)
1	2.95141	0.33882
2	2.95141	0.33882
3	16.52073	0.06053
4	16.52111	0.06053
5	17.24739	0.05798
6	17.24739	0.05798
7	24.89060	0.04018
8	24.89158	0.04017
9	26.80833	0.03730

3.2.3. Case c- Pressurized tower with contact element at the base and variable thinness of the wall of steel along of the height L (real structure)

The numerical results for case c are showed in figure 3.3 and table 3.5.



Mode 1; T_1= 0.38067 | Mode 3; T_3=0.08084 | Mode 4; T_4=0.07324 | Mode 6; T_6=0.04756 | Mode 8; T_8=0.03572

Figure 3.3. Natural periods T and modal configuration

Table 3.5. Frequencies and periods

mode	f_i hertz	T_i num (sec)
1	2.62695	0.38067
2	2.62777	0.38055
3	12.37044	0.08084
4	13.65442	0.07324
5	13.66067	0.07320
6	21.02805	0.04756
7	21.02973	0.04755
8	27.99887	0.03572

3.3. Comparison between analytical and numerical results of the dynamical analysis

The figure 3.4 shows the comparison between analytical and numerical results, in this curve it can see that among beam and shell analytical theories for the first period T_1 the values are almost the same for empty conditions, while for the numerical results these values are about of the 8, 14 and 20% greater than the analytics. This difference corresponds to levels of loads (mass), empty and pressurized towers and also the variation of the thickness of the wall t along of the height L, but for the higher modes the results tend to approximate them. Finally this comparison validated the numerical modelling.

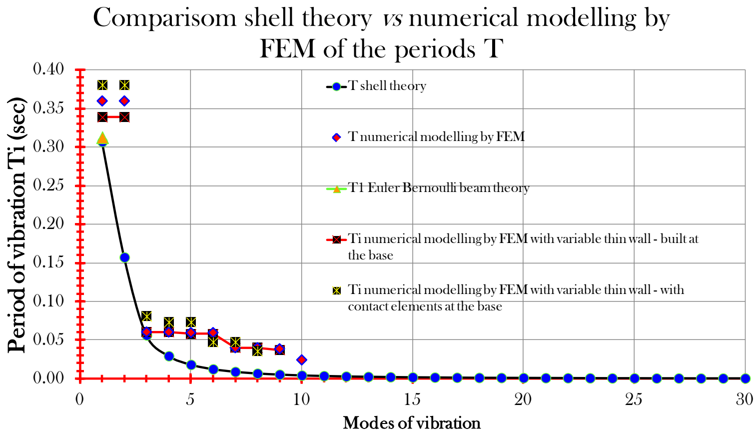


Figure 3.4. Comparison between analytical and numerical natural periods T *vs* modal configuration

3.4. Time history analysis (Numerical results in 3D of the towers)

3.2.1 Initial conditions

Before to carry out the transitory analysis, in the first step of analysis was applied an internal pressure of the 0.3434Mpa (3.5Kg/cm^2 design pressure) more a hydrostatic pressure of the liquid of the 0.0466Mpa at the lower cap of the tower, see figure 3.4.

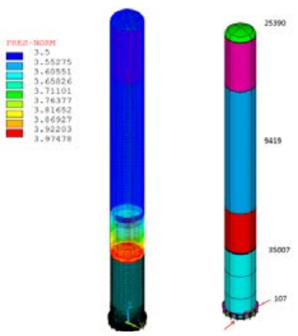


Figure 3.5. Initial condition

3.2.1 Time history analysis (transitory analysis)

The figures 3.6 and 3.7 illustrate the baseline shear history response of the tower with the Minatitlán seismic ground acceleration record used for this analysis. In these figures it can see that the maximal excitations are presented in different times such as; at 1.24, 2.68, 8.12, 10.68, 12.6, 14.36, 15.36 and 17.04, this represent relationship $\mathbf{V_B/W_{tot}}$ about the 1.5 at 2 times.

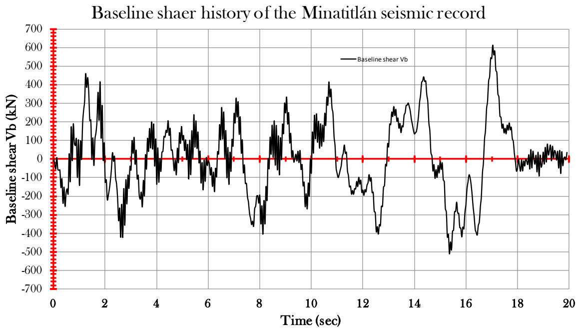


Figure 3.6. $\mathbf{V_B}$ shear response history

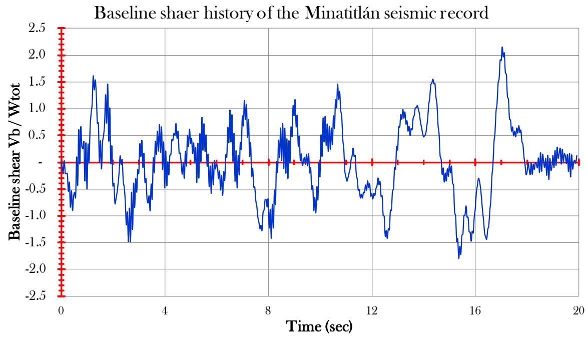


Figure 3.7. $\mathbf{V_B}$ $/W_{tot}$ response history

The figures 3.8 and 3.9 illustrate the lateral displacement history response of the tower. In figure 3.8 it can see the maximal lateral displacement of the 1.93cm at 17:36 sec at the top of the tower (node 25390), and the figure 3.9 presents the sequence of the lateral displacement of the tower for the interval 17:04 to 17:40 sec.

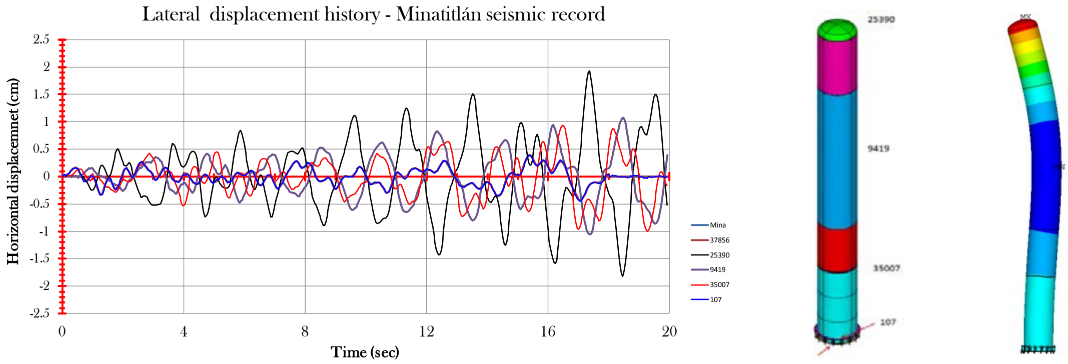


Figure 3.8. Displacement history response

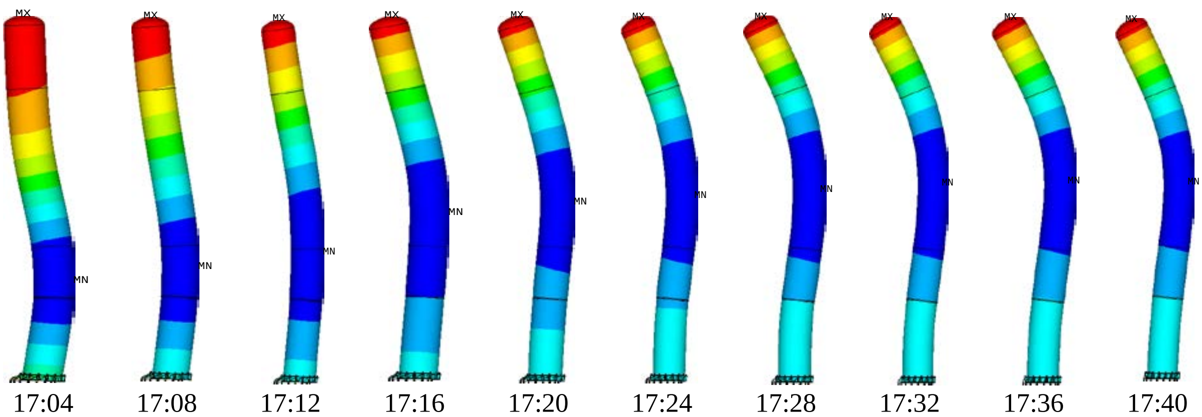


Figure 3.9. Sequence of the displacement of the tower for the 17:04 to 17:40 sec

4. CONCLUSIONS

It is studied the structural behaviour of the steel towers submitted to seismic actions by numerical simulation using the seismic ground motions and the numerical results are compared with analytical methods such as beam and shell vibration theories.

Concerning to dynamical analysis comparison it found that among beam and shell theories for the fundamental period T_1 the values are almost the same for empty conditions, while for the numerical results these values are about of the 8, 14 and 20% greater than the analytics. This difference is consequence to levels of mass, (empty and pressurized towers) and also the variation of the thickness of the wall t along of the height L, but for the higher modes the results tend to approximate them.

Relating to the modelling, the size of the models with solid elements is not so bigger and the results are considered acceptable with respect to the analytical values; the advantage of these models is that it can to visualize in detail the mechanical results of all structure given that the complexity of the connections between the components.

By the transitory elastic analysis was studied the structural response history as well as the lateral displacement pattern of the tower and it can see the sequence of deformation of the all tower in the intense phase of the response in the interval 17:04 to 17:40 sec.

Finally, the numerical results have permitted to visualize that the beams theory does not represent the real behaviour of this type of structures. Therefore is recommended to analyse these structures as axisymmetric shells. From elsewhere it is proposed to carry out the buckling analysis and the non-linear step-by-step analysis to know the plastic region of the structure.

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