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DYNAMIC ANALYSIS OF SLENDER STEEL DISTILLATION TOWERS

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ABSTRACT

A linear dynamic spectral modal analysis of two steel distillation towers was performed because of their high slenderness ratio as well as the high seismic risk of the construction sites. The new recommendations in the Chapter of Seismic Design of the Civil Works Manual, edited by the Federal Commission of Electricity (CFE, 1993a) were considered to evaluate the seismic response. Due to the particular geometry and load characteristics of the towers, 3D finite element models were elaborated to perform the dynamic spectral modal analysis. A comparison of five different modal combination criteria to estimate the peak response is presented in order to show how inappropriate selection of the modal combination rule could have a negative influence on the design. The peak response is only evaluated for testing condition plus earthquake loading combination.

KEYWORDS

Dynamic spectral analysis; modal combination; seismic response; distillation tower design.

INTRODUCTION

It is well known that for predicting a peak response of flexible structures from a response spectrum method it is necessary to define a rule for the modal combination. Recently, some authors (Maison *et al.*, 1983; Villaverde, 1984) pointed out that the existing conventional combination criteria can yield errors and may overestimate or underestimate the maximum response, with regard to more accurate results from a time history analysis. This is the case, when the criterion is applied to systems whose natural frequencies lie close to one another or if they have significant mode shapes with high frequencies. These conditions are more important in systems with torsional and translational effects, and with appendages.

In this paper, only a comparison of different modal combination rules is presented to emphasize the differences obtained for each procedure when applied to flexible slender structures as the distillation towers treated here. The geometry of the towers shows special dynamic characteristics due to their mass distribution, low damping, and a change of cross section in their highest portion. Thus, such characteristics led to investigate and verify the selected design rule compared with others. On the basis of this comparison some observations about the selected design rule are given.

MODAL COMBINATION RULES

Five different rules were applied to establish the variation of the peak response on the structural analysis of the towers, . The methods used for comparison were:

The absolute sum of the modal responses (ABS)

$$S_{\max} = \sum_{i=1}^{n} |S_i| \qquad (1)$$

The square root of the sum of the squares of the modal response (SRSS)

$$S_{\max} = \sqrt{\sum_{i=1}^{n} S_i^2} \qquad (2)$$

The absolute maxima of the modal responses plus the square root of the sum of the squares of the remaining modal responses (NRL)

$$S_{\max} = |S_j| + \sqrt{\sum_{i=1, i \neq j}^{n} S_i^2} \qquad (3)$$

where $S_j = \max$ for all i $|S_i|$

The average of the absolute maxima of the modal responses plus the square root of the sum of the squares of the modal responses (CFE)

$$S_{\max} = \frac{\sum_{i=1}^{n} |S_i| + \sqrt{\sum_{i=1}^{n} S_i^2}}{2} \qquad (4)$$

This rule is proposed in (CFE, 1993a) and in the following, it will be named CFE rule.

The complete quadratic combination (CQC)

$$S_{\max} = \sqrt{\sum_{i=1}^{n}\sum_{j=1}^{n} S_i \; S_j \; S_{ij}} \qquad (5)$$

in which,

$$S_{ij} = \frac{8\sqrt{\xi_i \xi_j}\,(\xi_i + \gamma \xi_j)\,\gamma^{3/2}}{(1 - \gamma^2) + 4\xi_i \xi_j (1 + \gamma^2) + 4\gamma^2(\xi_i^2 + \xi_j^2)} \tag{6}$$

where ξ_i and ξ_j are the critical damping coefficients for the ith mode, ω_j and ω_j are the frequencies with respect to the ith and jth modes, and $\gamma = \omega_j/\omega_i$.

In the four methods previously mentioned, S_i is the physical response due to mode i, and n is the number of considered modes. These rules were applied to the structural analysis of the distillation towers whose geometrical characteristics are presented as follows.

TOWERS GEOMETRY AND FINITE ELEMENT MODEL

Towers A and B are formed by two vertical elements that have a hollow circular section with different diameter. These elements are joined by mean of a reducer component. The towers are supported by a conical skirt. Figure 1 shows their global geometries, where dimensions are given in meters.

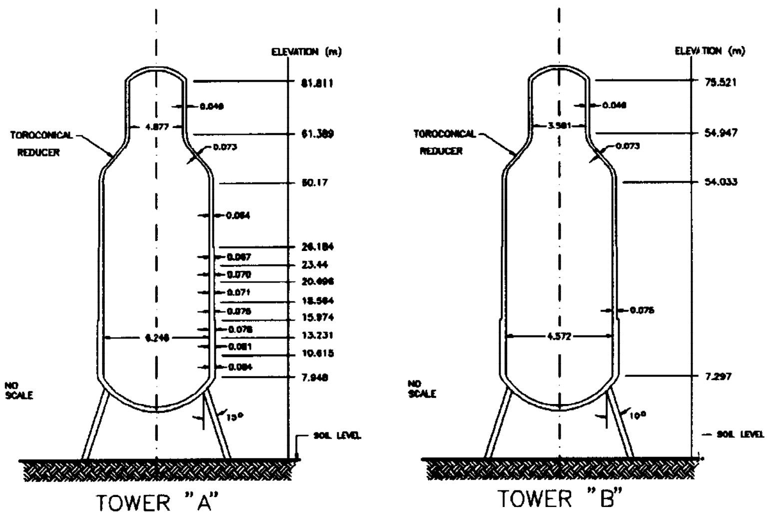

Fig. 1. Geometry of the two towers

The towers will be built of A-516 steel, grade 70 with modulus of elasticity = 187.3 Gpa, thermal expansion coefficient = $1.27 \times 10^{-5}\,K^{-1}$, and yield stress = 254.9 MPa.

A three-dimensional finite element model was adopted for spectral modal analysis of the tower A which is presented in Fig. 2. To consider all the particular loading and geometrical aspects, the FE models were elaborated with 3D shell elements. Since, the influence of soil-structure interaction was neglected, a rigid base was considered. The main considerations for the dynamic spectral modal analysis were:

- A directional combination of the responses to consider the effects of the ground motion in two orthogonal directions for each vibration mode.
- The modal combination was performed for twenty modes which consisted in flexural, torsional, cross oval deformation, and buckling shapes. A simplified combination of six flexural mode shapes was also performed.
- The comparison between modal combination criteria was made for the stresses on the shell.
- Mass distribution, geometry, and loads were symmetric about the vertical axis, except the case for the wind loads that had an irregular distribution about the cross section. However, this case was treated apart and it was not critical with respect to the seismic condition.

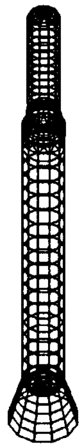

Fig. 2 Three-dimentional FE model

- For design purposes the critical load combination resulted from the testing condition plus earthquake loading. For this case the CFE modal rule was applied.

DYNAMIC PROPRIETIES AND REQUIRED LOAD COMBINATIONS

This kind of structure has a low energy dissipation system and for practical purposes low damping can be taken. Based on (CFE, 1993a), the critical damping factor $\zeta_e = 0.02$ was considered in this study. However, the case for damping factors $\zeta_e = 0.03$ and $\zeta_e = 0.05$ were also obtained to study the sensitivity of the influence due to this parameter in the dynamic performance on the towers. The associated design spectra for these cases are presented in Fig. 3. where a ductility factor Q = 3 was considered.

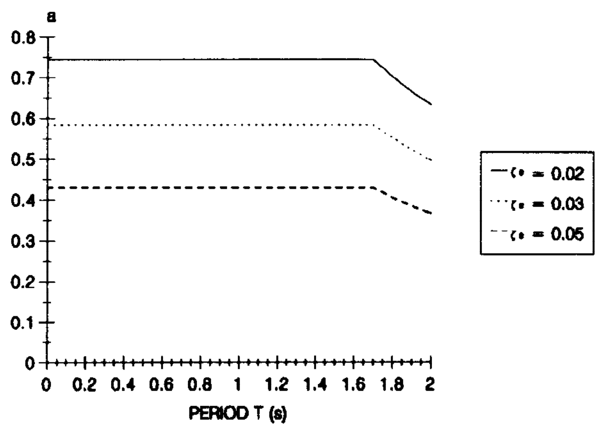

Fig. 3. Spectra design for earthquake loading

As it is required for design, two main load combinations were considered for analysis: operation and testing conditions. In table 1, the load cases for each combination are described.

The BHP load consists on a column of the distillation liquid retained in the bottom of the vessel, and the FHP consists on the filling of water in the tower, both cases with the towers in erected position. The FHP is critical not only for the pressure originated on the wall but also due to the additional inertial mass in seismic response.

seismic forces (S) and wind external pressure (W) were considered additionally to each general condition. The wind loading was not critical compared with seismic effects. The critical load combination resulted for the testing condition plus Earthquake loading. The results presented herein are only for this critical load combination (TP + FHP + DLW + 0.5S).

Table 1. General loading conditions and load cases

Accidental Loads			
Seismic Forces (S)		Wind external pressure (W)	
OPERATION		TESTING	
Operation internal Pressure	OP	Test internal Pressure	TP
Bottom hydrostatic load	BHP	Full hydrostatic load	FHP
Operation temperature	OT	Ambient temperature	AT
Dead load including the distillation liquid weight	DLL	Dead load including the water weight	DLW

MECHANICAL STRESS RESULTS

The modal analysis provided twenty modes with well separated frecuencies. Therefore, there were not coupled modes. It could be observed that the highest modes had very small participation factors. To demostrate that the highest modes were not significant in the dynamic response of the towers, the modal combinations were determined for combining twenty modes and for only six flexural modes. In table 2 the frequencies for the twenty modes are presented. From this, it is shown that the flexural mode shapes were the most significant for the dynamic behavior of the towers.

Table 2. Dynamic parameters from Modal analysis

MODE NUMBER	FREQUENCY (CYCLES/SEC)	PERIOD (SEC)	MODE SHAPE	PARTICIPATION FACTOR	
1	5.452653E-01	1.833970E+00	Flexion	3.2843E+00	2.5152E+05
2	5.452653E-01	1.833970E+00	Flexion	-4.3012E+01	1.9205E+04
3	2.670821E+00	3.744167E-01	Cross oval deformation	3.5569E-06	1.0439E-02
4	2.670823E+00	3.744164E-01	Cross oval deformation	-8.4937E-06	-4.7552E-03
5	2.733464E+00	3.658361E-01	Flexion	1.2746E+01	-4.2073E+04
6	2.733464E+00	3.658361E-01	Flexion	-2.0669E+01	-2.5946E+04
7	4.768439E+00	2.097122E-01	Cross oval deformation	-1.0984E-04	-5.2939E-02
8	4.768449E+00	2.097118E-01	Cross oval deformation	-1.4908E-07	4.1620E-04
9	5.629666E+00	1.776304E-01	Torsion	1.0707E-04	-3.7934E-02
10	6.281162E+00	1.592062E-01	Flexion	1.1817E+01	-1.5973E+04
11	6.281162E+00	1.592062E-01	Flexion	1.1493E+01	1.6422E+04
12	6.351408E+00	1.574454E-01	Cross oval deformation	-3.3127E-05	1.5851E-02
13	6.351410E+00	1.574454E-01	Cross oval deformation	-4.2783E-08	3.4441E-04
14	6.866529E+00	1.456340E-01	Buckling	2.0273E-04	1.9576E-01
15	6.866529E+00	1.456340E-01	Buckling	-1.4328E-04	-1.6113E-02
16	7.244038E+00	1.380446E-01	Buckling	-3.2150E-05	-2.7632E-02
17	7.244040E+00	1.380445E-01	Buckling	1.4659E-04	1.4834E-01
18	7.389283E+00	1.353311E-01	Buckling	-3.3207E-04	-2.4511E-01
19	7.389284E+00	1.353311E-01	Buckling	-7.6254E-04	-7.5903E-01
20	8.246180E+00	1.212683E-01	Axial deformation	-3.3011E-04	-3.8066E-01

In Figs. 4, 5, and 6 the different modal combinations are superimposed. They are shown as stress magnitudes. The ABS rule represents the maximum of the peak responses, as this is the upper bound response. The CFE rule shows similar behavior as for the NRL rule, and the CQC modal combination had almost the same magnitud as the SRSS rule.

For meridional stresses a variation in the results between the different combining rules was observed, specially in the skirt and reducer elements. Here, an 20% reduction of stresses between CQC and CFE rules is observed. On the other hand, for the hoop stresses no variation was found except at the reducer region where the responses presented significant changes reducing the stresses in about 20%.

As a result from the comparison, the suggestions made by (Maisson *et al.*, 1983, Villaverde, 1984, EMRC, 1993) that the CQC modal combination is recommended for general use is confirmed and it can be very convenient for design purposes. However, CFE rule gives more conservative values.

Thus, the towers presented here were designed using the CFE combination as a better conservative design, because these structures will be exposed to a high risk earthquake loading. Therefore, the CFE is a good choice to obtain conservative designs, although CQC could be used if the economical advantages are the ruling factor. It would be interesting to perform a time history analysis to demonstrate in detail the convenience of using the CQC method for this type of towers with similar characteristics.

Finally, Fig. 8 presents stresses obtained from CFE combination as a function of different damping values mentioned previously. It is observed that a small change in the damping can influence in a significant manner the response magnitudes. Therefore, an actual damping value obtained from an experimental procedure could give a better estimation of the seismic forces.

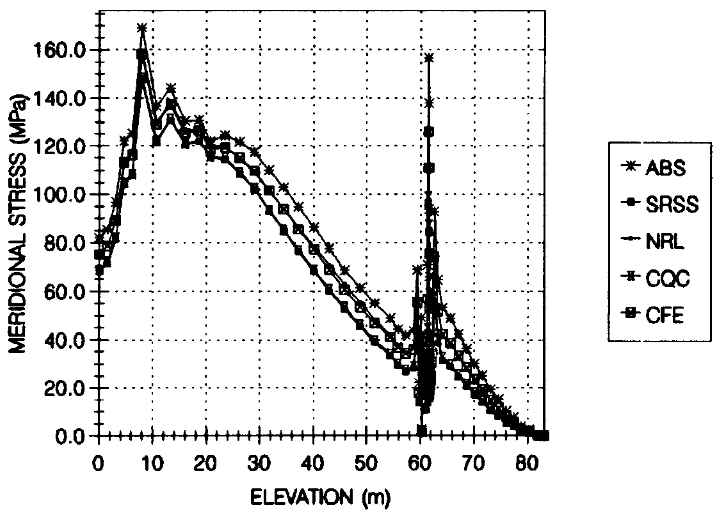

Fig. 4. Peak responses comparison for meridional stresses (tower A)

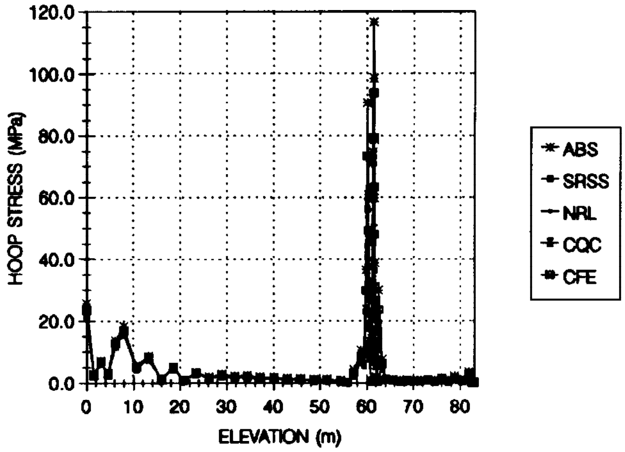

Fig. 5. Peak responses comparison for hoop stresses (tower A)

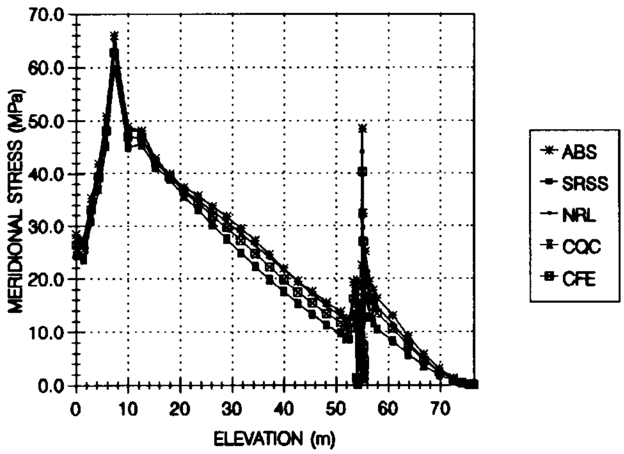

Fig. 6. Peak responses comparison for meridional stresses (tower B)

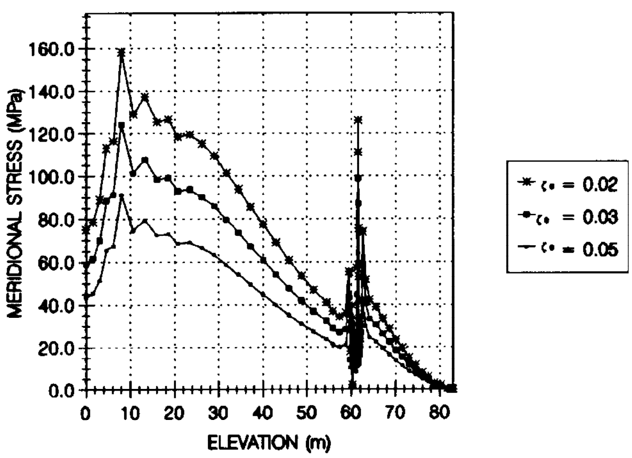

Fig. 7. CFE responses comparison for different damping factors (tower A)

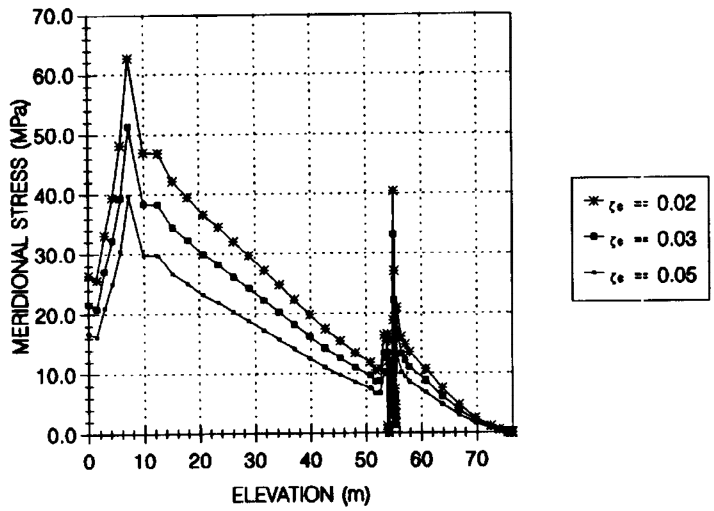

Fig. 8 CFE responses comparison for different damping factors (tower B)

CONCLUSIONS

A dynamic analysis of two metallic distillation towers was performed using a spectral response method. A comparison of five different modal combination criteria to estimate the peak response was presented in order to show how an inappropriate selection of the modal combination rule could influence the design of structures such as the towers treated here. The main objetive of the study was to investigate the differences obtained for each procedure and verify the selected design rule compared with others.

The responses obtained by combining twenty modes and six flexural modes, treated separately, show that the peak response can be obtained considering only the flexural modes for similar towers.

From the comparison of modal combination criteria, it is demonstrated that the CFE rule is a good choice to get conservative designs. The CQC method could be an appropriate option to desing towers with similar dynamic behavior if economical advantages are required. However, a time history analysis should be done to demostrate that CQC method is a convenient choice.

As shown in Fig. 8, it is important to know with accuracy the actual damping factor in order to determine a better estimation of the responses since this parameter has an important influence. An experimental procedure should be used to obtain this value.

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