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A Comparison of Modelling the Seismic Response of a Solid-Stone Masonry Tower

S.E. Chidiac[1], Y. Lu[2], A.G. Razaqpur[3] and J.H. Rainer[1]

ABSTRACT

Heritage buildings built of solid stone-masonry were generally constructed without seismic considerations. Consequently, there is a need to assess the seismic adequacy of such structures in moderate to high risk earthquake zones. In this paper, two analytical methods, the stiffness method and the finite element method, were used to analyze the seismic response of a solid stone-masonry tower being subjected to the NBC 1990 equivalent lateral seismic force and to inertia forces from ground motion. Both the frame and the finite element model were calibrated against the measured natural frequencies and mode shapes of the tower.

The study shows that for the same return period, the equivalent static approach yields a base shear 1.5 times larger than the one obtained from the dynamic analysis when the differences in peak ground acceleration are accounted for. The overall dynamic response of the tower using the frame model did not agree with the one obtained from the finite element model. The frame analysis produced stresses that are significantly larger and displacements that are smaller than those obtained using the finite element method. Further, the locations of the maximum tensile stresses were different. The frame analysis was found inadequate for studying the structural characteristics of thick stone masonry tower.

INTRODUCTION

Many heritage buildings are exhibiting some form of deterioration caused primarily by the aging process and limited maintenance program. Although these buildings have so far withstood the various environmental actions, including earthquake, this does not assure their future performance, especially for the moderate to high seismic loading.

Given the complex geometry, connections and material properties of heritage buildings, various approximations in the modelling of the structure play an important role in obtaining reliable answers to the predicted structural behaviour. The adequacy of the various approximation methods such as the finite element and the stiffness method and of the

[1] Research Officer, Structures Lab., IRC, National Research Council Canada, Ottawa, Ont. K1A 0R6
[2] Graduate Student, Dept. of Civil and Environmental Engineering, Carleton University, Ottawa, Ont. K1S 5B6
[3] Associate Professor, Dept. of Civil and Environmental Engineering, Carleton University, Ottawa, Ont. K1S 5B6

different types of analyses such as static and dynamic analysis, need to be examined. This study investigates the differences in stresses and displacements of a stone masonry tower calculated by a) the equivalent static approach of the National Building Code of Canada (NBC 1990) and b) the dynamic analysis, and by the frame and the finite element methods.

DESCRIPTION OF TOWER

An unreinforced stone masonry tower with an iron frame roof is used for the comparison of modelling the seismic response of heritage buildings, Fig. 1. The tower, 83 m high, has a rectangular plan with two turrets symmetrically disposed at two corners. It is made up of various structural components such as multi-wythes walls of limestone and sandstone, brick walls and an iron frame roof of 35 m high. At its lower levels, the tower is connected on three sides to the main building. The tower has its footing resting on concrete flattened bedrock.

COMPUTATIONS OF SEISMIC FORCES

First, the application of NBC 1990 to this heritage building was examined. The tower was also subjected to artificially generated ground motions. Both analyses employ an annual probability of exceedance equal to 0.002.

Equivalent Lateral Seismic Forces

The National Building Code of Canada, NBC 1990, specifies a minimum seismic base shear force V, given by

$$V = (V_e/R)U \tag{1}$$

where R, the force reduction factor, is equal to 1.0 for unreinforced masonry, and U, the calibration factor, is equal to 0.6. V_e, the equivalent lateral force representing the elastic response, given by

$$V_e = vSIFW \tag{2}$$

in which v, S, I, F, and W are, respectively, the zonal velocity ratio, the seismic response factor, the importance factor, the foundation factor and the weight of all masses that induce inertia forces in the structure. From the geographical location of the tower and its measured frequency of 1.77 Hz, v=0.1 and S=2.0. Since the tower's footing rest on bedrock, F=1.0. The importance factor is set equal to 1.0 for this study.

For the static analysis, the base shear was distributed along the height of the building in accordance with NBC 4.1.9.1.(13):

$$F_x = V\left(W_x h_x \Big/ \sum_{i=1}^{n} W_i h_i\right) \tag{3}$$

in which h_i is the height above the base to level "i" and n the total number of storeys above the exterior grade.

Inertia Forces from Ground Motion

For the dynamic analysis of the tower, ground motion time histories that are representative of the site corresponding to annual exceedance probabilities of 0.002 were artificially generated (Atkinson, 1992). Two time records of ground motion, representing a high and a low frequency content were chosen. The former has a peak acceleration of 0.7 m/s^2 and duration of 2.7 s, the latter a peak of 0.33 m/s^2 and 12.0 s duration. The corresponding response spectra are shown in Figure 2. It is assumed that the ground motion will be transmitted to the building foundations without modification.

ANALYTICAL MODELS

Given the complex geometry, connections and material properties of the tower, an attempt was made to minimize the approximation errors due to idealizations by first generating the analytical models according to the exact geometry of the tower and then by calibrating the first dynamic mode computed analytically against measured values from ambient vibration tests.

To assess the response of the tower, a frame analysis and a finite element analysis (FEA) were employed. The tower was assumed to deform linearly and elastically, and was subjected to both the equivalent static forces and the inertia forces due to ground motion in the two orthogonal directions North-South and East-West. For both static and dynamic analyses, the same mathematical model and boundary conditions are employed. For the dynamic analysis, 3% damping ratio was assumed. Only the results for the stone portion of the tower are presented.

Frame Model

The frame model of the structural members representing the tower consists of 256 3-D brace elements, 305 beam elements, 160 rigid beam elements and 115 panel elements. The panel elements were used to model the masonry walls except for the turrets and the walls with openings. The two wythes of the masonry walls were merged and modeled as a single structural element. The turrets were modeled as columns of appropriate section properties and were connected to the tower at the diaphragm level using rigid links. The steel roof was modeled as a space frame, and the in-plane stiffness of both the wood and copper sheathing located on top of the steel roof were ignored. The intersecting boundaries between the tower and the building were modeled as fixed. Computer program ETABS (1991) was employed to calculate the dynamic behaviour of the tower.

Table I: Computed base shear (MN) for a 500 return period seismic motion.

Equivalent static analysis using NBC - 1990	Dynamic Frame Analysis			
	High frequency content		Low frequency content	
	E-W	N-S	E-W	N-S
10.9	4.52	4.71	2.37	2.62

Table II: Maximum horizontal displacement from Frame Analysis for a 500 return period seismic motion in E-W direction.

Elevation (m)		Structural member	Equivalent Static (mm)	Dynamic Analysis (mm)	
From	To		NBC 1990	High frequency	Low frequency
22.4	26.465	Tower	0.3	0.3	0.1
26.465	31.19	Tower	1.9	1.3	0.6
31.19	40.79	Tower	8.9	4.5	2.4
40.79	46.57	Tower	12.6	6.4	3.4
46.57	48.95	Tower	13.3	6.7	3.6

Table III: Maximum horizontal displacement from Frame Analysis for a 500 return period seismic motion in N-S direction.

Elevation (m)		Structural member	Equivalent Static (mm)	Dynamic Analysis (mm)	
From	To		NBC 1990	High frequency	Low frequency
26.465	31.19	Tower	3.0	1.1	0.7
31.19	40.79	Tower	9.8	3.5	2.1
40.79	46.57	Tower	13.7	4.9	3.0
46.57	48.95	Tower	14.5	5.3	3.2

Table IV Maximum tensile stresses computed from Frame Analysis for a 500 return period seismic motion in E-W direction.

Elevation (m)		Structural member	Equivalent Static (MPa)	Dynamic Analysis (MPa)	
From	To		NBC 1990	High frequency	Low frequency
11.785	22.4	Tower	2.71	3.06	2.94
22.4	26.465	Tower	4.05	1.86	1.38
26.465	31.19	Tower	1.08	0.57	0.52
31.19	40.79	Tower	2.77	1.02	0.66
40.79	46.57	Tower	1.36	0.47	0.25
46.57	48.95	Tower	1.61	0.63	0.56

Table V: Maximum tensile stresses computed from Frame Analysis for a 500 return period seismic motion in N-S direction.

Elevation (m)		Structural member	Equivalent Static (MPa)	Dynamic Analysis (MPa)	
From	To		NBC 1990	High frequency	Low frequency
22.4	26.465	Tower	3.01	1.34	1.08
26.465	31.19	Tower	1.40	0.64	0.56
31.19	40.79	Tower	4.79	0.86	0.23
40.79	46.57	Tower	1.37	0.50	0.34
46.57	48.95	Tower	2.21	1.23	0.87

The computed values for the base shear from the static and the dynamic analyses are given in Table I. The computed maximum horizontal displacements and maximum tensile stresses obtained for the 500 year return period from static and dynamic analyses are given in Tables II to V.

Finite Element Model

The finite element model of the tower was constructed using four different element types, with a total of 7073 finite elements and 6907 nodes. The composite shell element was used to model the two-wythe masonry wall and the wood and copper sheathing on top of the steel roof. The thick shell element was used to model the concrete floors located at the bottom portions of the tower. The steel roof structure was modeled using both beam and truss elements. Finite element computer program AFEMS (1992) was employed to perform the linear elastic dynamic analysis. The intersecting boundaries between the tower and the building were modeled as fixed.

The results of the finite element analysis in the form of maximum horizontal displacements and maximum tensile stresses at various heights of the tower for the 500 year return period and from static and dynamic analyses are given in Tables VI to IX.

Table VI: Maximum horizontal displacement from FEA for a 500 return period seismic motion in E-W direction.

Elevation (m)		Structural member	Equivalent Static (mm)	Dynamic Analysis (mm)	
From	To		NBC 1990	High frequency	Low frequency
11.785	26.465	Tower	2.13	0.7	0.8
11.785	22.4	Tower	0.47	0.2	0.2
22.4	26.465	Tower	1.93	0.6	0.7
22.4	26.465	Turret	2.07	0.9	0.9
26.465	31.19	Tower	5.60	1.4	2.0
26.465	31.19	Turret	4.60	1.5	2.0
31.19	48.95	Tower	24.53	5.6	10.0
31.19	40.79	Turret	15.33	3.0	6.2
40.79	46.57	Turret	22.07	5.0	9.0
46.57	48.95	Turret	24.27	5.7	10.0

COMPARISON OF ANALYTICAL RESULTS

The computed values from the frame analysis given in Table I shows that the analysis specified by the NBC 1990 produces a base shear 2.5 and 4.5 times larger than the ones obtained from the dynamic analysis for the high and low frequency content, respectively.

Tables II and III show that the maximum horizontal displacements exhibit the same trend for both static and dynamic frame analysis. It also shows that the ratio of the horizontal displacement obtained using static analysis to the ones obtained using the dynamic analysis varies on the average from 2.3 to 4.2 for the high and low frequency content, respectively. On the other hand, the maximum tensile stresses obtained from static analysis are not in the same location as those obtained from the dynamic frame analysis, Tables IV and V. The average ratios for the maximum tensile stresses values obtained using the static analysis to the dynamic analysis is 2.0 and 3.9 for the high and low frequency, respectively.

Table VII: Maximum horizontal displacement from FEA for a 500 return period seismic motion in N-S direction.

Elevation (m)		Structural member	Equivalent Static (mm)	Dynamic Analysis (mm)	
From	To		NBC 1990	High frequency	Low frequency
11.785	26.465	Tower	11.67	2.4	2.1
11.785	22.4	Tower	6.80	1.6	1.2
22.4	26.465	Tower	11.07	2.3	2.0
22.4	26.465	Turret	12.93	2.8	2.4
26.465	31.19	Tower	17.60	3.1	3.1
26.465	31.19	Turret	18.13	3.2	3.4
31.19	48.95	Tower	43.40	6.0	8.7
31.19	40.79	Turret	32.87	4.4	5.9
40.79	46.57	Turret	41.27	5.6	8.2
46.57	48.95	Turret	44.40	6.1	8.9

Table VIII Maximum tensile stresses from FEA for a 500 return period seismic motion in E-W direction.

Elevation (m)		Structural member	Equivalent Static (MPa)	Dynamic Analysis (MPa)	
From	To		NBC 1990	High frequency	Low frequency
11.785	26.465	Tower	0.21	0.13	0.18
11.785	22.4	Tower	0.31	0.2	0.31
22.4	26.465	Tower	0.38	0.33	0.53
22.4	26.465	Turret	0.47	0.18	0.28
26.465	31.19	Tower	0.37	0.24	0.44
26.465	31.19	Turret	0.75	0.31	0.47
31.19	48.95	Tower	2.42	1.31	1.41
31.19	40.79	Turret	0.81	0.38	0.46
40.79	46.57	Turret	0.43	0.21	0.26
46.57	48.95	Turret	0.40	0.22	0.18

The results of the static and dynamic finite element analysis given in Tables VI to IX exhibit the same trend for both the maximum horizontal displacement and the maximum tensile stresses. For the displacement, an average ratio of 3.6 and 4.1 are found between the static and dynamic results for the high and low frequency content, respectively. And an average ratio of 2.1 for the tensile stresses are found between the static and dynamic for both high and low frequency content. The location of the maximum stresses also changed from the dynamic frame analysis to the finite element analysis. The latter is believed more accurate due to large openings found in the wall.

Table IX Maximum tensile stresses from FEA for a 500 return period seismic motion in E-W direction.

Elevation (m)		Structural member	Equivalent Static (MPa)	Dynamic Analysis (MPa)	
From	To		NBC 1990	High frequency	Low frequency
11.785	26.465	Tower	0.84	0.22	0.24
11.785	22.4	Tower	1.26	0.33	0.31
22.4	26.465	Tower	1.89	0.45	0.44
22.4	26.465	Turret	0.84	0.22	0.24
26.465	31.19	Tower	1.23	0.25	0.26
26.465	31.19	Turret	0.95	0.28	0.29
31.19	48.95	Tower	2.88	1.4	0.83
31.19	40.79	Turret	0.57	0.32	0.28
40.79	46.57	Turret	0.51	0.24	0.21
46.57	48.95	Turret	0.67	0.57	0.43

The above comparison are based on the NBC 1990 values and dynamic analysis using artificially generated earthquakes for a 500 year return period. The ground motions have, however, different peak accelerations {0.033 g for the low frequency record and 0.07 g for the high frequency one}, and these in turn differ from the zonal acceleration of 0.1 g implied by the NBC 1990. Taking the above results and scaling them by these respective acceleration values, one can observe that for comparable peak accelerations, for this structure, the code provides base shears, deformations and stresses that are in reasonable agreement and sometimes slightly larger than those from the dynamic analyses.

CONCLUSION

The equivalent static approach yields a base shear 2.5 and 4.5 times larger than the one obtained from the dynamic analysis for the 500 year return period seismic ground motion. When the differences in peak ground acceleration are accounted for, however, the static approach gives values only 1.45 and 1.65 times larger than the dynamic analysis.

The overall dynamic response of the frame model did not agree with the finite element results. Only the results of the finite element analysis are found to properly represent the dynamic behaviour of the tower. The frame analysis, primarily designed to model the response of modern constructions i.e. beams, columns, shear walls and rigid diaphragm, is found inadequate for studying the structural characteristics of thick stone masonry tower.

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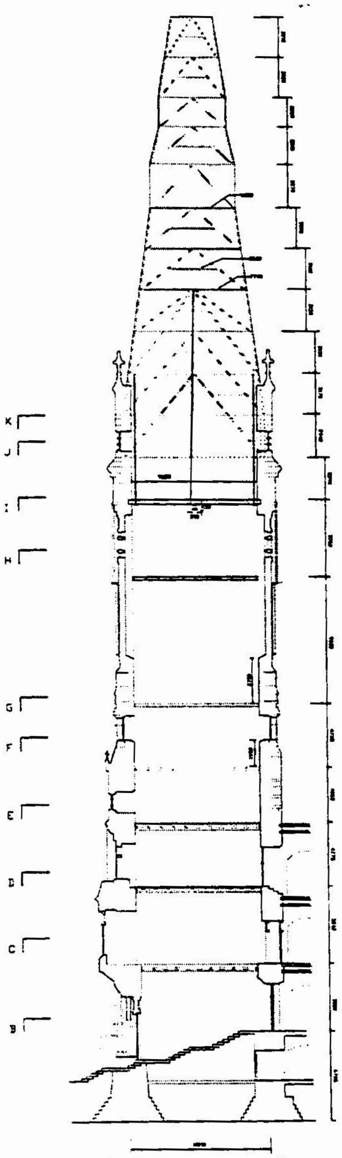

Fig. 1: Elevation view of the tower.

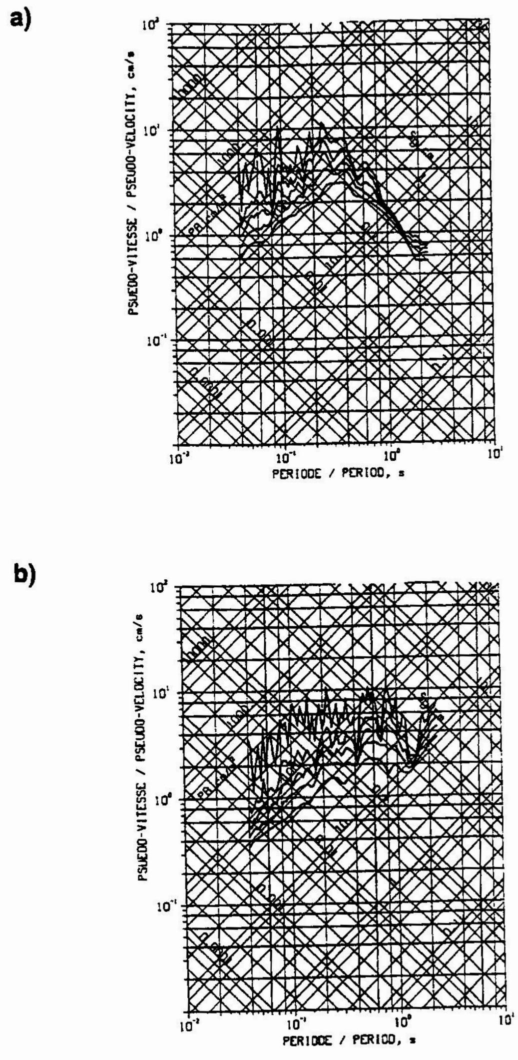

Fig. 2: Response spectra of the ground motion with a) high and b) low frequency content (Damping ratio = 0, 2, 5, 10, 20% of critical)