Advanced Materials Research Vols. 133-134 (2010) pp 683-688

doi:10.4028/www.scientific.net/AMR.133-134.683

Numerical Simulation of the Seismic Response of a Mexican Colonial Model Temple Tested in a Shaking Table

CHAVEZ Cano M[1,a] and MELI R[1,b]

[1] Instituto de Ingeniería, Universidad Nacional Autónoma de México, Mexico City, Mexico

[a]mchavezc@iingen.unam.mx, [b]rmep@pumas.ii.unam.mx

Abstract With the main objective of providing basic information for calibration of analytical models and procedures for determining seismic response of historic stone masonry buildings, a shaking table testing program was undertaken at the Institute of Engineering of UNAM. A typical colonial temple was chosen as a prototype. The model was built at a 1:8 geometric scale. Increasing levels of seismic intensities were applied to the table. Main features of the measured response are compared in this paper to those computed though a nonlinear, finite element model; for the latter, a constitutive law corresponding to plain concrete was adopted for reproducing cracking and crushing of the irregular stone masonry, which could be considered as a conglomerate with low anisotropy. From the results of the analytical models, it was found that response is strongly governed by damping coefficient and tensile strength of masonry. Measured damping coefficients were found to significantly exceed those commonly used for modern structures. Observed damage patterns as well as measured response could be reproduced with a reasonable accuracy by the analytical simulation, except for some local vibrations, as those at the top of the bell towers.

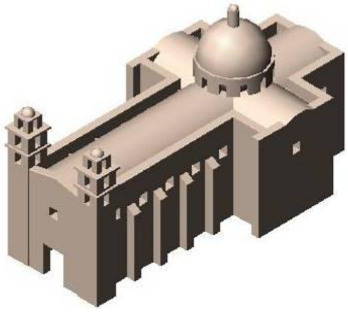
Keywords: Shaking table test, stone masonry, non linear analysis, finite element model, damping coefficient, tensile strength

Introduction

Historic masonry buildings are commonly structures of great size and with complex geometries; their material show a non linear behavior since early stages of loads; therefore they do not lend themselves to be studied by conventional finite element methods. On the other hand, sophisticated non linear methods developed in recent years require, for the characterization of the materials, of parameters that are not easily available. A shaking table test program has been performed at the Institute of Engineering of UNAM, pursuing the main objective of providing basic information for calibration of analytical models presently available for determination of the seismic response of ancient stone masonry structures. Additionally, the experimental research aimed at a better understanding of the seismic response and of the modes of failure of these buildings, as well as at evaluating the efficacy of typical techniques commonly used for reducing their vulnerability to severe earthquakes. A typical colonial temple was chosen as a prototype. Summarized in this paper are the results of a detailed non linear analysis that was carried out on a finite element model of the experimental structure, to ascertain main characteristics of its dynamic response and to compare them with those obtained in the shaking table tests.

Model Temple

Prototype temple is typical of central Mexico; being this a region where strong earthquakes are not very frequent, colonial architecture was not as robust as in other more seismically active areas; the temple is tall and with light buttressing (Fig. 1a). Temples of this kind are frequently damaged by earthquakes, but their collapse is uncommon. In order to limit the scale factor of the model and to ease its construction, the prototype was simplified by eliminating two bays, including the transept and its dome, but preserving all other characteristics (Fig 1b). The model tested in the shaking table was reduced at a scale 1:8, to fit the 4 by 4 m shaking table of the Institute of Engineering at UNAM (Fig 1c), which is biaxial (horizontal and vertical), and it can support a load up to 20 t. The dimensions of the model are: height of tower bells 3.5 m, thickness of vault 0.10 m, width of the walls 0.20 m and length 4 m. The model was built with the same materials as the prototype, in order to reproduce as close as possible its mechanical properties and the same modes of failure. It was not considered feasible to comply with all requirements of the dimensional analysis (Tomazevic and Velechovosky 1992) without altering the dynamic behavior; therefore, it was accepted that the model should be considered as a miniaturized prototype; extrapolation of its results to the prototype should be essentially made in a qualitative way. The time scale of the seismic input was reduced by the scale factor and the acceleration was increased by this same factor; in this manner the dynamic stresses are maintained the same than for the prototype, but the stresses due to the self weight of the building are reduced by the scale factor (eight time smaller in the model).



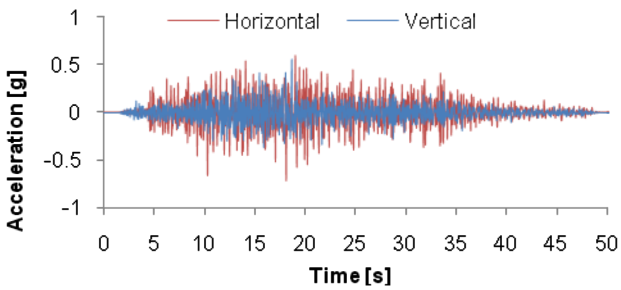
(a) Prototype

(b) Model built at 1:8 scale

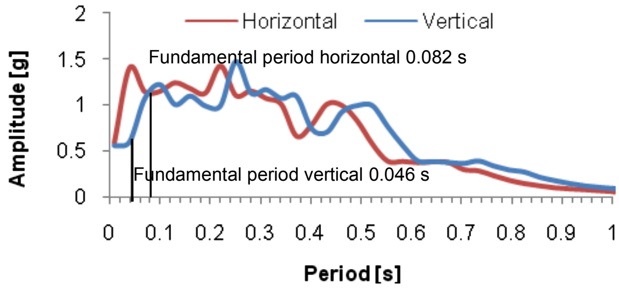
Figure 1: Prototype and model temples

Shaking Table Test

Acceleration time-histories applied to the table in the horizontal and vertical directions were derived from a strong motion record obtained in 1985 near the epicenter of a large magnitude earthquake (Ms 8.1). The seismic records were modified in their time scale in the order to enhance their effects in the model. The resulting time histories and their acceleration response spectra for 5% damping are shown in Fig. 2. Increasing fractions of the reference record, starting with 5 % and rising up to 60 %, were applied to the model. For each level of intensity, first only the horizontal component was applied, and then horizontal and vertical motions were simultaneously introduced.



(a) Acceleration time history



(b) Acceleration response spectrum

Figure 2: Base seismic input

A more complete description of the model and the experimental setup, as well as the results of the first stage of testing was presented in a previous SAHC Conference (Chávez and Meli 2008).

Numerical Simulation

Considering that the irregular stone masonry used in these temples is a conglomerate with low anisotropy, non linear analyses were performed with the constitutive model for plain concrete, as implemented in ANSYS version 11 (ANSYS 2007). The model predicts the failure of fragile materials, by using solid isoparametric tridimensional elements. Model material is initially linear elastic and is able to show tensile cracking and compressive crushing. The brittle behavior of conglomerate masonry is modeled according to the Willam-Warnke failure criterion (Willam 1975). A more detailed description of the model is found in (García 2007). Cracking is represented by introducing a plane of weakness in a direction perpendicular to that of the principal tensile stress; in other words, cracks develop in a perpendicular direction to the main stresses which exceed the tensile strength, producing a local redistribution of stresses. In this moment, the stiffness component normal to the crack plane is released, and the shearing stress along such plane decreases. On the other hand, crushing occurs when the failure surface is reached; then, stresses along that direction suddenly descend to zero. Plasticity is considered by the Drucker-Prager failure surface in which the yielding surface is defined through parameters representing cohesion and internal friction angle, as well as by a rule based on the dilatancy angle.

Material properties used in the numerical simulation are shown in Table 1. It must be recognized that some of the parameters had to be adjusted to avoid problems of convergence of the program and to obtain a better response of the model in terms of the pattern of cracking. The property influencing most the response was found to be masonry tensile strength which was taken as 0.08fm, Additionally, the two coefficients defining the shear transfer across the crack: (βt) when the crack is open, and (βc) when is closed, were taken as 0.6 and 0.8, respectively, and a third parameter related to stress relaxation across a tensile cracking (Tc) was taken as 0.7.

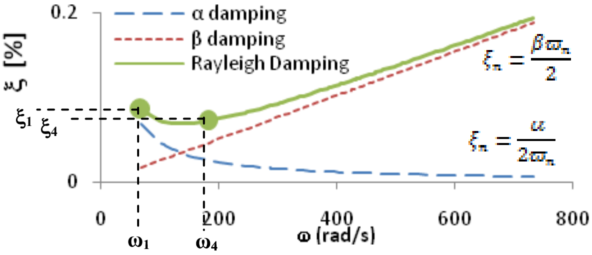
Table 1: Properties of materials

Properties	*Masonry*	
	Stone	*Brick*
Density [kg/m^3]	1830	1580
Young's modulus [MPa]	551	1472
Poisson's ratio	0.3	0.3
Compression Strength [MPa]	1.59	4.91
Tensile strength [MPa]	0.13	0.39

Considerable computational resources are needed to run a time-history non linear analysis of this kind. The model had 26403 solid elements and 35417 nodes; the imposed base motion was digitized by 5000 points at 0.01 s intervals; results files were generated only for a limited number of points, and did not exceed 12 Gb; execution time was between 190 y 220 hours, with a four core Intel Xeon equipment at 3.2GHz, and with 4 Gb RAM.

Analyses were run for four sets of base motion histories recorded at the base of the model which was tested in the shaking table, corresponding to 40 and 60 per cent of the full scale of the selected record, and for each intensity to the effect of the horizontal component and to the combined effect of horizontal and vertical component.

Rayleigh's Damping Damping coefficients corresponding to the significant modes of vibration were derived from the measured response of the model under white noise signals applied before starting the time-history earthquake motions. The shape of the ratios of spectral amplitudes between the acceleration recorded at the roof and the basement, in the neighborhood of the peak at the fundamental frequency of vibration of the structure was used to apply the procedure proposed by (Rinawi and Clough, 1992), by which the theoretical transfer function of a single degree of freedom system (SDOF) is fitted to the experimental shape of the same function; the identification of the damping coefficient is based on the amplitude of this function for the vibration mode under consideration. Rayleigh damping is defined through parameters α y β, which are shown in Fig. 3, and are derived by solving a pair of simultaneous equation in which damping ratios ξ_m and ξ_n are associated to their respective frequencies, (ω_m, ω_n). In Fig. 3 the two frequencies correspond to the fundamental horizontal and vertical modes of vibration of the physical model.



Direction	Frequency Exp	Frequency Analy	ξ	α	β
	[Hz]		[%]	[1/s]	[s]
Transverse	12.8	12.3	0.088	9.31	0.0005
Vertical	24.1	21.6	0.069		

Figure 3: Parameters of Rayleigh's damping

Comparison of Analytical and Experimental Results

Cracking Patterns ANSYS "Concrete Plot" command draws small circles in finite elements where cracking has occurred, and small octagons where concrete has crushed. Cracking patterns of the experimental and the analytical models are shown in Fig. 4 for the most intense ground motion applied (HV 60%). The amount of cracking is clearly greater in the analytical model; nevertheless the affected elements and cracking patterns are quite similar: horizontal cracks in the bell towers, separation of the lower part of the towers from the façade, and longitudinal cracking of the vault. The differences in the amount of cracks can be attributed, to a great extent, to the fact that in the physical model cracks were identified after the test has finished, when most cracks were closed and was difficult to identify their traces, especially of those at the interface between mortar and stones.

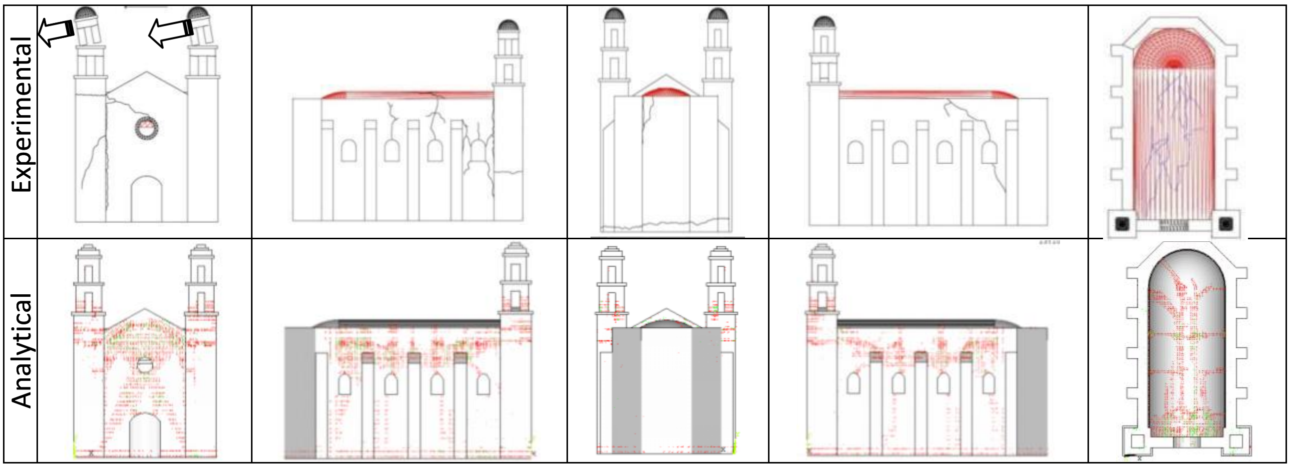


Figure 4: Comparison of analytical and experimental damage patterns for Cal 60%HV

Accelerations and Displacements Maximum horizontal accelerations and displacements, measured and computed at the top of the vault, are compared in Table 2 for several intensities of the applied motion. The complete time histories of acceleration and of displacement for the same point are compared in Fig. 5. As it can be appreciated, computed accelerations were consistently smaller than those measured, whereas displacements showed a better fit. In general terms, it can be concluded that the analytical simulation was able to reproduce with a reasonable approximation the measured response. It must be realized that very small maximum lateral displacements, of a few millimeters, and the very small amplification of the maximum ground amplification are originated by the high modal frequencies of the structures and of the applied ground motion.

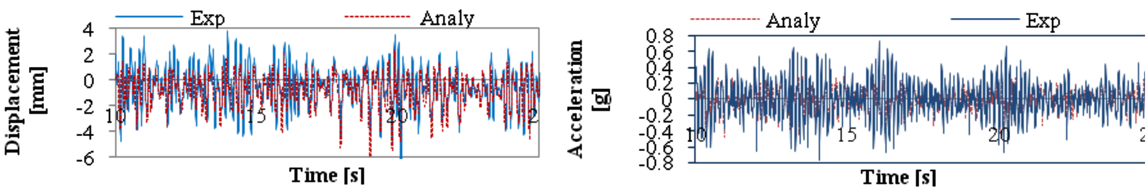


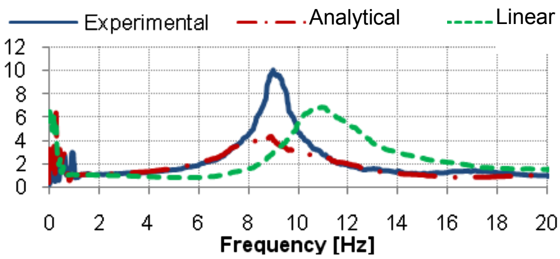
Figure 5: Comparison of analytical and experimental results 40%

Amplification of the acceleration measured at the roof level is moderate, and increases with the intensity of the applied motion at the table; non linear model predicts a significantly smaller amplification than the linear model.

Table 2: Maximum displacements and relative accelerations at the roof

Intensity	*Displacement [mm]*			*Relative acceleration [g]*			*Amplification acceleration*		
	Exp	*Analy*	*Linear*	*Exp*	*Analy*	Linear	*Exp*	*Analy*	Linear
H= 0.33g	1.74	2.12		0.47	0.39		1.53	1.27	
H= 0.33g; V= 0.26 g	1.84	1.75	0.93	0.53	0.42	0.62	1.51	1.27	1.9
H= 0.45g	5.83	5.16		0.76	0.49		1.88	1.27	
H= 0.45g; V= 0.37g	6.6	5.68		0.8	0.49		1.86	1.28	

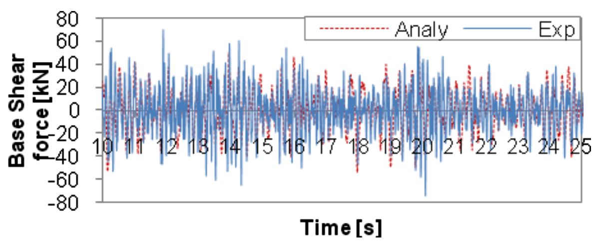
Natural Periods of Vibration Fundamental periods of vibration, computed from the eigenvalue analysis of the numerical model, were 0.082 [s] and 0.046 [s], for the horizontal and vertical vibration, respectively; they were very close to those derived from white noise records applied to the table before starting the tests (0.08 [s] and 0.04 [s], respectively). Modal frequencies for the analytical and physical models were determined from spectral ratios of Fourier spectra amplitudes of the applied strong motions. Spectral ratios for the motion at the centre of the vault to that at the table allowed the identification of the fundamental modes. Fourier spectra from the measured and computed records are shown Fig 6, along with frequencies derived from spectral ratios for different intensities of the applied motion.



Test	*Intensity [g]*	*Fourier Spectrum*		*Transfer functions Vault/base*	
		Exp	*Analy*	*Exp*	*Analy*
		Frequency [Hz]			
40%H	H= 0.33	8.03	7.84	9.55	8.78
40%HV	V= 0.26	7.9	7.84	9.02	8.95
60%H	H= 0.45	7.37	6.45	7.70	7.09
60%HV	V= 0.37	6.38	5.85	7.07	7.09

Figure 6: Analytical vs. experimental transfer functions vault/base, and frequencies for CAL 40%

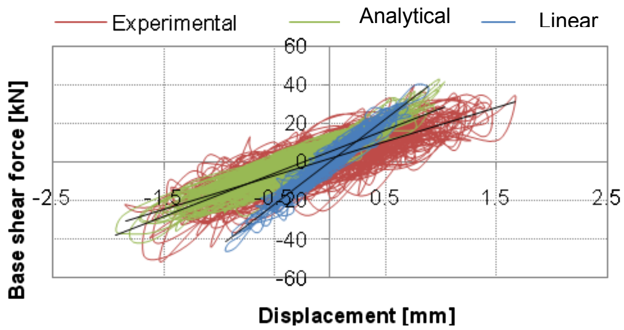
Applied Base Shear Force and Hysteretic Behavior Lateral inertia forces were computed from the accelerations measured or computed at different points of the structure; from them, maximum base shear force and the corresponding base shear coefficient were obtained. Results are shown in Fig. 7; as it can be appreciated, the time histories of the applied base shear are very similar, even if experimental base shear is consistently slightly smaller than that derived from the analytical model, probably because the former was estimated from a reduced number of points where accelerometer were placed. The simultaneous application of vertical and horizontal motions produced a slight increase of the base shear force. The maximum applied base shear coefficient was 0.58.



Intensity [g]	*Base shear [kN]*		*Seismic coefficient*	
	Exp	*Analy*	*Exp*	*Analy*
H = 0.33	43.08	41.98	0.35	0.34
H = 0.33; V = 0.26	51.74	42.70	0.41	0.34
H = 0.45	68.14	52.81	0.54	0.42
H = 0.45; V = 0.37	72.10	52.38	0.58	0.42

Figure 7: Analytical vs. experimental base shear

Hysteretic loops relating the lateral shear to lateral displacement at the top of the roof are shown in Fig. 8. Loops are rather narrow, except near the maximum intensity. A secant stiffness was obtained from each loop and its variation with the corresponding shear force was determined. As it can be appreciated from the figure, the softening of the behavior is greater when both components of the motion are simultaneously applied. Differences between results of analytical and experimental analyses are rather significant; nevertheless, trends are quite similar. It can be also observed that the stiffness obtained by the non linear numerical model is approximately 30% greater than the one derived from the measured response, whereas that of the linear elastic model is almost 2.5 times greater than the experimental one.



Intensity	*Analytical [kN/mm]*		*Linear*	*Experimental [kN/mm]*	
	H	*HV*		*H*	*HV*
H= 0.33g V= 0.26 g	21.3	23.0	44.2	16.3	18.0
H= 0.45g V=0 .37g	13.4	10.5		11.1	8.5

Figure 8: Stiffness and hysteresis analytical and experimental

Conclusions

Numerical models used for the simulation of the physical test were rather crude considering the complex behavior of an irregular material like stone masonry. Furthermore, some of the parameters of the numerical model were not directly derived by testing the material itself, but from what proposed by other authors; also, to some extent they were adjusted to obtain a better fit between the experimental and analytical results. From parametric analysis varying significant material properties, it was found that the response of the structure and the patterns of cracking are very sensitive to the tensile strength assumed in the numerical model and to the amount of internal damping.

Despite of the above mentioned limitations, it is considered that the numerical simulation was able to reasonably reproduce the response of the model even at very advanced stages of damage. This stands for time histories of acceleration and displacements at most significant points of the structure, and for the pattern of damage at different levels of applied ground motion intensity.

Natural periods of vibrations and their variation with the intensity of the motion were closely reproduced, and the shapes of the hysteretic loops were rather similar.

Former statements regarding the accuracy of the numerical simulation do not equally stand when the local response of structural elements showing large amplifications, as the bell towers are studied. For a better prediction of their response, it is likely that most detailed models be required.

References

[1] Chávez, M and Meli, R (2005). "Shaking table testing of a typical Mexican colonial temple," in *6th. International Conference of Structural Analysis of Historical Constructions,* 2-4 July 2008, Bath, UK. 825-832.

[2] García, N (2007). "Funcionamiento y seguridad estructural de los templos conventuales del siglo XVI en México." *UNAM*. PhD. Thesis.

[3] *Release 11.0, Documentation for ANSYS*, ANSYS, 2007.

[4] Rinawi, A, and Clough, R (1992). "Improved amplitude fitting for frequency and damping estimation," in *Proc. 10th Int. Modal Analysis Conference. Soc. for Exp. Mechanics*, Bethel, Conn, 893-898.

[5] Tomazevic, M, and Velechovosky, T (1992). "Some aspects of testing small-scale masonry building models on simple earthquake simulators." *Earthquake Engineering and Structural Dynamics*, 21, 945-963.

[6] Willam, K J, and Warnke, E P (1974). "Constitutive model for triaxial behavior of concrete," in *Seminar on Concrete Structures Subjected to Triaxial Stresses, International Association of Bridge and Structural Engineering Conference*, Bergamo, Italy, 174.