Forced Vibration Testing and Finite Element Modeling of a Nine-Story Reinforced Concrete Flat Plate-Wall Building

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Tunnel form buildings, owing to their higher construction speed and quality, lower cost, and superior earthquake resistance over that of conventional reinforced concrete buildings, have been widely used for mass housing, urban renewal, and post-earthquake reconstruction projects all over the world as well as in Turkey. However, there have been few dynamic tests performed on existing buildings with this structural system. This study investigates the dynamic structural properties of a typical nine-story reinforced concrete flat plate-wall building by forced vibration testing and develops its three-dimensional (3-D) linear elastic finite element structural model. The finite element model that uses the modulus of elasticity for concrete in ACI 318 predicts the natural vibration periods well. Mode shapes are also in good agreement with the test results. Door and window openings in the shear walls, and the basement with peripheral wall emerge as modeling considerations that have the most significant impact on structural system dynamic properties. [DOI: 10.1193/091212EQS287M]

INTRODUCTION

The structural response of buildings and other structures when subjected to earthquake, wind, and similar dynamic excitations is controlled by their dynamic properties. Tests on existing structures in order to determine primarily their natural vibration periods (frequencies), natural modes of vibration, and modal damping capacities have been developed in the field of structural dynamics. Identification of these dynamic structural properties is invaluable for developing building codes, improving the modeling of structural systems, structural health monitoring, and seismic risk assessment of structures. For example, building codes provide empirical equations to estimate the fundamental vibration period of a structure, which is the key parameter in determining the minimum seismic design loads, and to estimate the associated damping ratio, which is particularly important if a time-history analysis is required for checking the design (note that damping cannot be computed directly from structural material properties or from finite element structural models). These empirical equations were derived from recorded structural responses during dynamic tests on existing structures (Goel and Chopra 1997, 1998, Satake et al. 2003).

Dynamic tests can be classified into three as forced vibration (Blume 1935, Hudson 1961, Kuroiwa 1967, Rea et al. 1968, Stephen et al. 1973, Celebi et al. 1977a, 1977b, Celik 2002,

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Yu et al. 2008), ambient vibration (Carder 1936, Crawford and Ward 1964, Stephen et al. 1973, Ivanovic et al. 2000, Skolnik et al. 2006), and seismic monitoring (Foutch et al. 1975, Celebi and Safak 1991, Trifunac et al. 2001, Celebi 2011), depending on the source of vibration. Forced vibration testing of existing structures dates back to 1934 when Blume and Jacobsen built a vibration generator at Stanford University for the United States Coast and Geodetic Survey (Blume 1935). Recent vibration generators consist of two identical counter-rotating eccentric masses attached to a common vertical shaft. The components of the inertia forces of the rotating masses cancel out in the x direction and add up in the y direction to produce a sinusoidal force expressed as

$$p(t) = m_e e \omega^2 \sin \omega t \tag{1}$$

where m_e is the total eccentric mass, e is the eccentricity, ω is the excitation frequency, and t is the time (cf. Figure 1). This low-amplitude horizontal harmonic force excites the building from an upper floor where the vibration generator is mounted, and the steady-state structural response of the building is recorded at each operated frequency. Upon digital signal processing of the records, frequency-response curves are determined, from which the dynamic structural properties are identified (Celik 2002). Forced vibration tests provide the most direct means of determining the dynamic properties. Ambient vibration tests; however, requires system identification (Ljung 1999) to extract the dynamic properties, and amplitudes of typically the wind-induced vibrations are lower by two orders of magnitude than the forced vibrations. Records of structural responses of instrumented structures during strong earthquake ground motions are ideal for understanding the actual behavior of structures during seismic events. Such records can be found from the websites of the Center for Engineering Strong Motion Data (CESMD 2013) and U.S. Geological Survey (USGS 2013).

This paper presents the forced vibration testing and finite element modeling of a nine-story reinforced concrete flat plate building with shear walls. The building is typical of growing condominium construction in Turkey in recent years. It was cast in by tunnel forms, a formwork system (NERU 2012) which allows the casting of shear walls and flat plates

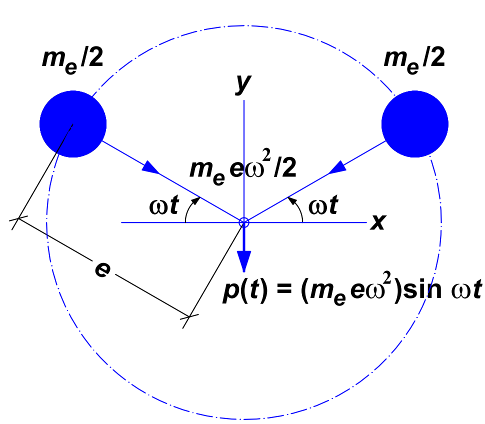


Figure 1. Harmonic force produced by the vibration generator.

together and hence reduces the construction time for such multi-story buildings. One floor is typically cast in a 24-hour cycle. Tunnel form construction emerged after World War II to rapidly supply the demand for residential units in Europe (Nasvik 2003). This structural system has been used in Turkey since late 1970s and demonstrated superior earthquake performance during the 1999 M_w 7.4 Kocaeli, 1999 M_w 7.2 Duzce, and 2003 M_w 6.4 Bingol earthquakes in Turkey (Yakut and Gulkan 2003, Balkaya and Kalkan 2004). The Housing Development Administration of Turkey (2012) alone has constructed more than 500,000 residential units by mostly tunnel form system if not all since 2003, as part of mass housing, urban renewal, and post-earthquake reconstruction projects. Following M_w 7.1 Van, Turkey earthquake of 23 October 2011, approximately 15,000 units of housing were constructed within 12 months to respond to safe housing needs after so many of the population lost their homes. Although the widespread use of tunnel form buildings, there is only one reference to our knowledge that reports on dynamic tests performed on such existing buildings in Turkey. Celebi et al. (1977b) performed forced vibration tests on two 14-story buildings. Elsewhere, Lee et al. (2000) performed ambient vibration tests on 50 buildings with 15–25 stories in Korea, and Kwon and Kim (2012) reported on seismic monitoring of four buildings in the United States. Current study adds to the very limited test data in the literature on dynamic properties of existing tunnel form buildings and examines the related structural modeling provisions of current building codes. Forced vibration testing of the nine-story tunnel form building had been performed before any nonstructural components such as partition walls and floor coverings were in place, and hence the recorded structural responses are free of their uncertain contribution. Exterior window panels which are added for closing the tunnel openings (see Figure 2) increase both the mass and stiffness of the system as well as its damping. Usually they contribute in shortening the vibration period (Kuroiwa 1967, Celebi et al. 1977b). The identified dynamic properties are compared with the eigenvalue analysis results of the three-dimensional (3-D) finite element structural model of the building developed subsequently. The comparisons are used in assessing the

(a) (b) (c)

Figure 2. Views of the building (a) at the time of test and (b, c) at present.

structural modeling provisions of ACI Standard 318 (ACI 2008) and its Turkish counterpart TS 500 (Turkish Standards Institute 2000) and Turkish Earthquake Code (TEC; Ministry of Public Works and Settlement 2007). The impact of modeling considerations—for example, door and window openings in the shear walls—on structural dynamic properties is also investigated.

BUILDING DESCRIPTION

MESA Yonca Houses comprise five identical condominium blocks in Ankara, Turkey. Figure 2 shows the views of the fourth block at the time of the test and at present. Cast in by tunnel forms, it is a nine-story reinforced concrete flat plate building with shear walls.

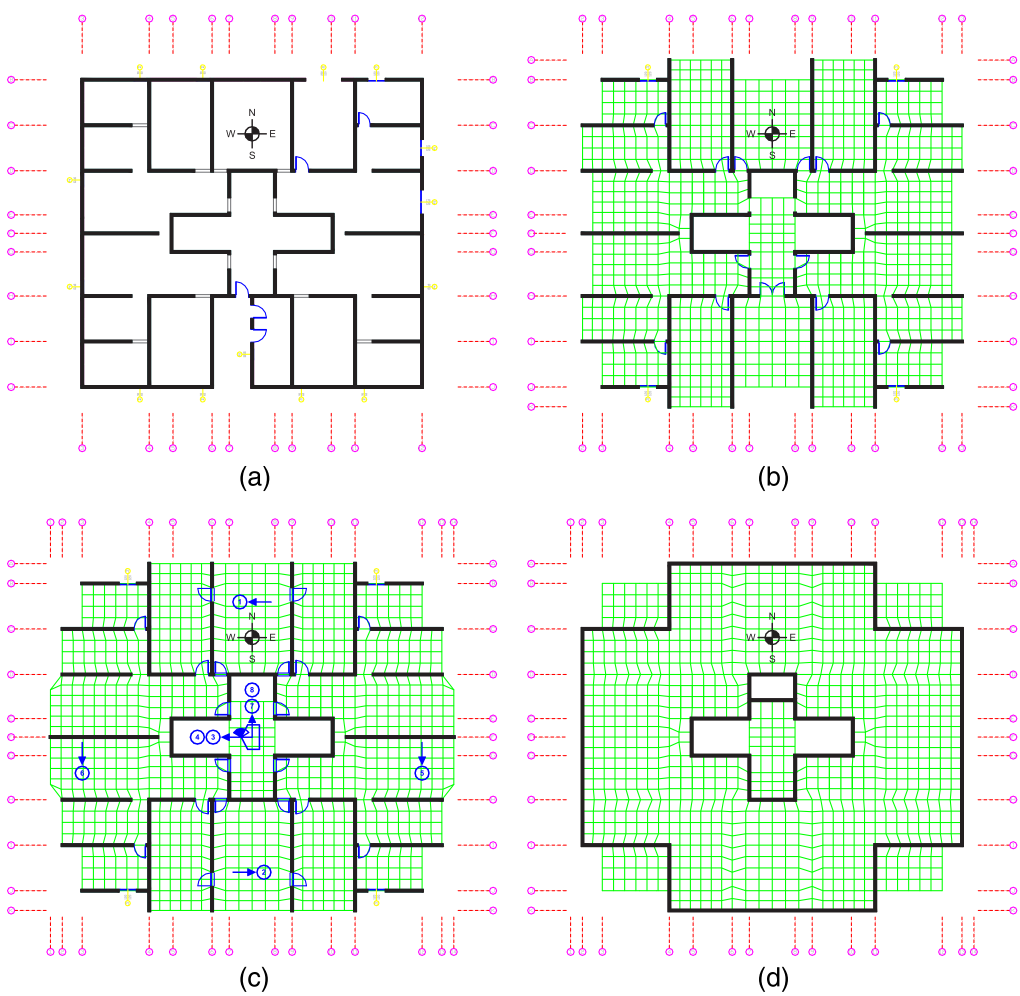


Figure 3. Floor plans: (a) basement, (b) ground, (c) typical, (d) roof; instrumentation scheme; finite element meshes.

The total height of the building is 32.0 m including the stair, elevator and duct shaft that extends into the roof by 4.0 m and the basement. The basement is isolated from the underground parking lot on the south and east faces by expansion joints, and is above the ground level on the other faces. The story height is 2.8 m, whereas the roof parapets are 1.8 m tall. It has an approximately square floor plan; 24.0 m by 21.6 m at the basement, 28.4 m by 24.6 m at a typical floor. Figure 3 shows the basement, ground, roof, and typical floor plans. Flat plates are 15 cm, whereas shear walls are 20 cm thick. Ready-mix grade C20 concrete was used; TS 500 defines its characteristic compressive strength as 20 MPa. The building has a 0.6 m thick mat foundation. The soil medium consists of hard clay (standard penetration resistance, $N = 23–49$; unit weight, $\gamma = 20\,\mathrm{kN/m^3}$) down to five meters depth and marl ($N > 50$) at higher depths (Ergun 2000). Shear wave velocity is at least $700\,\mathrm{m/s}$ according to the site classification in TEC.

FORCED VIBRATION TESTING

INSTRUMENTATION SCHEME

The building was excited by a vibration generator (Model VG-1; Kinemetrics 1975) bolted to the eighth floor (one floor below the roof) slab at the geometric center (Figure 4a). It has 0–9.7 Hz operating frequency range and produces a horizontal unidirectional sinusoidal force up to 22 kN amplitude. Eight uniaxial accelerometers of force balance type (EpiSensor ES-U; Kinemetrics 2000) connected to a 12-channel digital recorder (K2; Kinemetrics 1997) were used in recording the structural vibrations. Four accelerometers (# 1, 2, 5, 6) were mounted on the sides of the eighth floor (see Figure 4b for # 1), two (# 3, 7) on the geometric center of the sixth floor, and another two (# 4, 8) similarly on the fourth floor, as shown in Figure 3c with the location of the vibration generator. The basement level was not instrumented, as preliminary analysis for possible soil-structure interaction according to ASCE

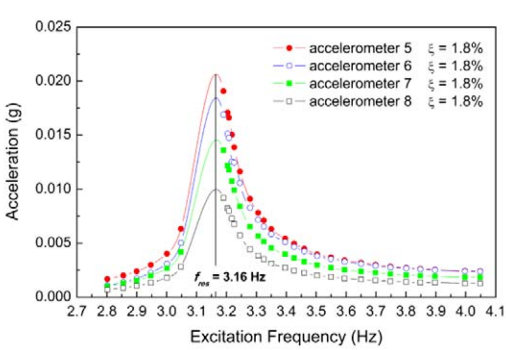
(a) (b)

Figure 4. (a) Vibration generator and (b) accelerometer #1.

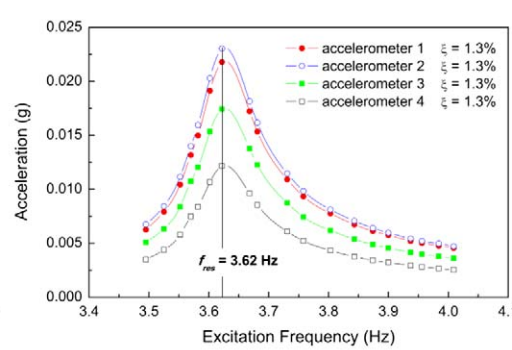
Standard 7-10 (2010) Chapter 19 had revealed that the increase in the fixed-base period would not be greater than 4%. The digital recorder was set to record accelerations at 200 samples per second, more than required to satisfy the Nyquist frequency criterion. Detailed information about the instrumentation can be found in Celik (2002).

FREQUENCY-RESPONSE CURVES

Forced vibration testing of the building was performed in the North-South (N-S) and East-West (E-W) directions, respectively, and the first translational modes were excited in both directions. Forced vibration tests are small-amplitude non-destructive tests; however, occupants especially at upper floors can feel the structural vibrations near resonance frequencies. Being at the eighth floor next to the vibration generator during the test, a frequency sweep without taking any records was performed first to determine approximately the resonant frequency in the direction of excitation. When the structural vibrations became noticeable, the excitation frequency was set to a lower value. Sweeping the frequency of the vibration generator from there on with increments of typically 0.05 Hz, the steady-state structural response was recorded for 15 s at each operated frequency. We waited typically 45 s for the transient response to damp out before recording. Low-pass filters were used to filter out the high frequency noise present in the records taken at frequencies away from the resonant frequency (Celik 2002). Plotting the response amplitudes at each frequency resulted in frequency-response curves in the form of acceleration amplitude versus excitation frequency, $f = \omega/2\pi$ (Rea et al. 1968) as shown in Figure 5 for the N-S and E-W directions. Note that the acceleration amplitudes at resonance were as high as 0.02 g and the ranges of frequencies swept were wide enough to capture the rise and fall regions of frequency-response curves. These frequency-response curves were determined for a force with amplitude proportional to the square of the excitation frequency ω (cf. Equation 1). Measured responses can be divided by ω^2 to determine acceleration-frequency response curves, and can further be divided by ω^2 to determine displacement-frequency response curves, for a constant-amplitude harmonic force. Any of these frequency-response curves can be used to determine the natural frequency and damping capacity. For the practical range of damping in structures, the natural frequency is essentially equal to any of the three resonant frequencies, f_{res} (Chopra 1995). The natural



(a) $T = 0.32$ s



(b) $T = 0.28$ s

Figure 5. Acceleration-frequency response curves for (a) N-S and (b) E-W directions.

frequencies for the first translational modes were determined as 3.16 Hz for the N-S and 3.62 Hz for the E-W directions. As structural engineers are accustomed to thinking in terms of periods rather than frequencies, the natural periods, T, are also reported: 0.32 s for the N-S and 0.28 s for the E-W directions. Associated damping capacities, ξ, on the other hand, were determined as 1.8% and 1.3%, respectively, using the half-power bandwidth method (Rea et al. 1968, Chopra 1995). These low damping values represent low-amplitude response where internal frictional mechanisms in concrete members do not fully develop as in the higher amplitude response, which usually leads to 5% damping on average. Mode shapes determined from the measured responses at resonant frequencies are presented subsequently. For comparison, forced vibration test results of two identical 14-story tunnel form buildings in Turkey (Celebi et al. 1977b) are reported in here. One was tested before its precast front panels mounted and the other after. The fundamental vibration periods were 0.66 s and 0.61 s, and the associated damping capacities were 1.5% and 2.7%, respectively.

FINITE ELEMENT MODELING

The 3-D linear elastic finite element structural model of the building was developed using SAP2000 (Computers and Structures 2010; Figure 6). Shell elements, which combine independent membrane and thin-plate behavior, were used to model the shear walls and flat plates

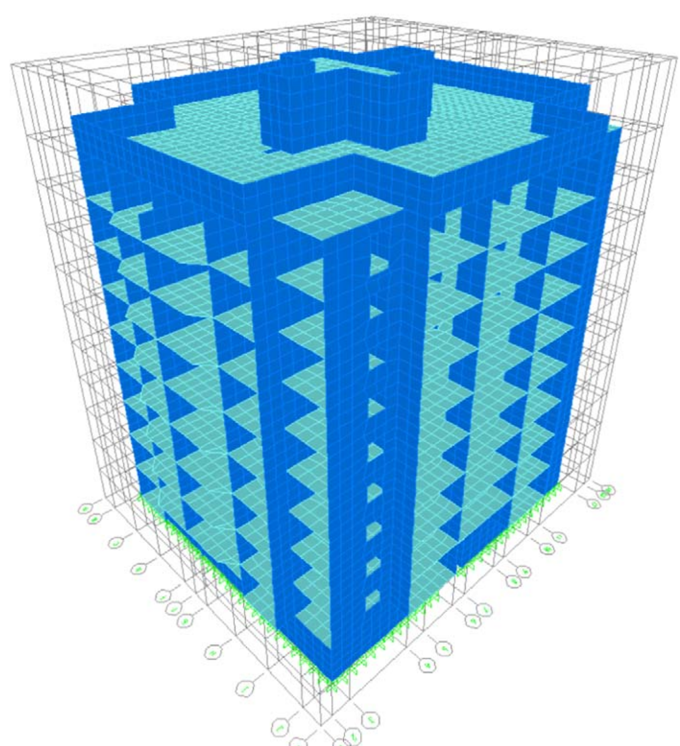


Figure 6. 3-D finite element structural model.

in the structural model. Door and window openings in shear walls and elevator and stair openings in flat plates were considered. The basement and the roof were also modeled. However, soil-structure interaction effects were ignored as the preliminary analysis suggested. Fixed support conditions were employed in the structural model. The finite element meshes for the ground, roof and typical floor slabs are shown in Figures 3b–d. The unit weight of concrete was taken as 24 kN/m^3 in the structural model. There were not any nonstructural components in place during the test, which could have contributed to the mass and stiffness of the building. The Poisson's ratio for concrete was taken as 0.2 as in TS 500.

ACI 318 VERSUS TS 500 AND TEC

The modulus of elasticity (E) for grade C20 concrete is defined as 28,000 MPa in TS 500, whereas it is 21,200 MPa in ACI 318. TEC Sec. 3.2.3 requires using uncracked (gross) section stiffnesses for calculating earthquake loads (and hence the fundamental vibration period of a structure in the earthquake direction). No distinction is made between service and design loads in TEC. ACI 318 Sec. 8.8, on the other hand, requires using member stiffnesses for service lateral loads as 1.4 times the flexural stiffnesses defined in Sec. 10.10.4.1 for design (factored) lateral loads. This, in effect, corresponds to using gross moment of inertia, $1.0I_g$, for uncracked shear walls and $0.35I_g$ for flat plates. There are four possible combinations for the values of modulus of elasticity for concrete and stiffness coefficients for shell elements per these building codes. Table 1 presents the eigenvalue analysis results of the 3-D finite element models developed for all four combinations, and the natural vibration periods identified from the forced vibration testing of the building in the N-S and E-W directions.

Typical stresses in flat plates under the self-weight of the building as obtained from the third model (cf. Table 1) are shown in Figure 7. Tensile stresses at the bottom faces of slab spans along the N-S (Figure 7a) and E-W (Figure 7c) directions do not exceed the characteristic tensile strength (1.6 MPa) defined in TS 500 for grade C20 concrete. It can be concluded that the natural vibration periods from the third model, in which the uncracked (gross) moments of inertia for both shear walls and flat plates as stated in TEC and the modulus of elasticity for concrete in ACI 318 were used, fit best to the forced vibration test results. The fourth model is actually as good as the third model; cracks in flat plates do not significantly affect the natural vibration periods (cf. Table 1). The modulus of elasticity for concrete

Table 1. Natural vibration periods from forced vibration testing of building and eigenvalue analysis results of finite element models

	T(s)	
	N-S	E-W
Forced vibration testing	**0.32**	**0.28**
Model 1/$1.0I_g$ (shear walls, flat plates) (TEC); E (TS 500)	0.27	0.25
Model 2/$1.0I_g$ (shear walls), $0.35I_g$ (flat plates) (ACI 318); E (TS 500)	0.28	0.26
Model 3/$1.0I_g$ (shear walls, flat plates) (TEC); E (ACI 318)	0.31	0.29
Model 4/$1.0I_g$ (shear walls), $0.35I_g$ (flat plates) (ACI 318); E (ACI 318)	0.32	0.30

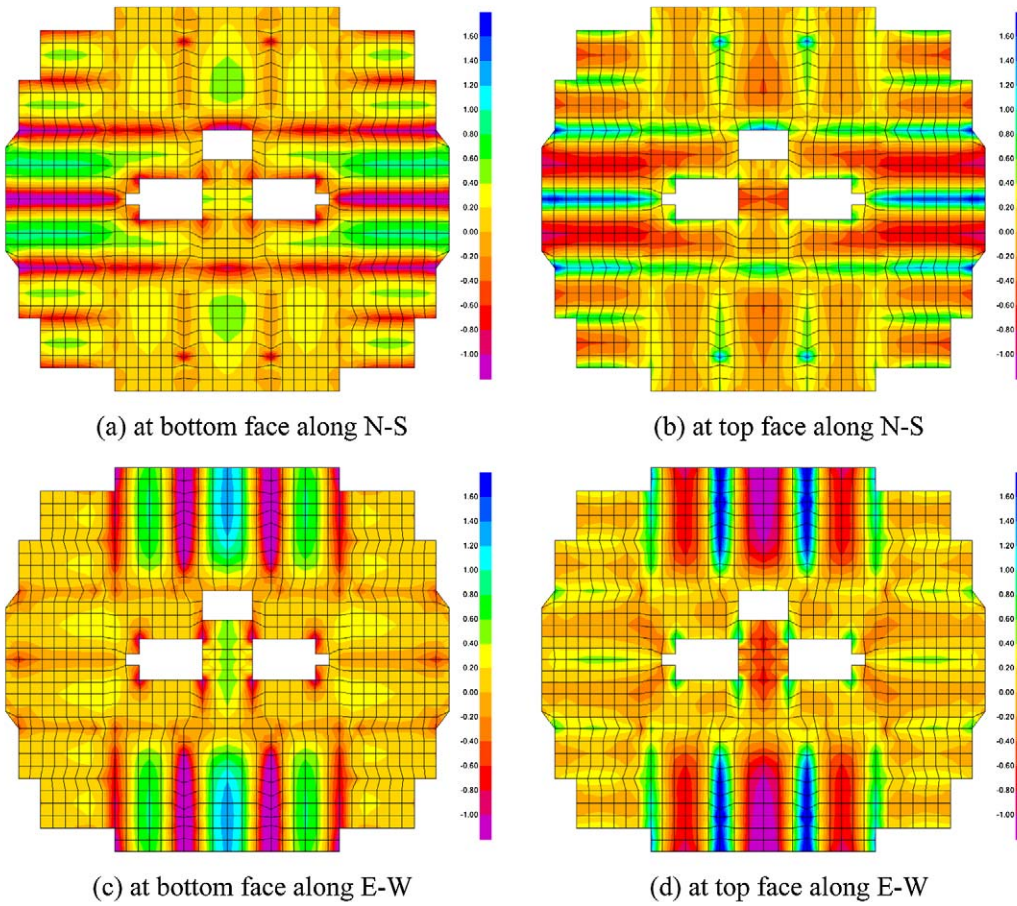


Figure 7. Typical stresses in flat plates under self-weight of the building (MPa; + tension, − compression).

in TS 500 results in shorter period values when used in the finite element structural models. In the remainder of the manuscript, the third model is used for all structural analyses.

Mode shapes of the building in the N-S and E-W directions from the finite element model when compared with the forced vibration test results show good correlation as illustrated in Figure 8.

MODELING CONSIDERATIONS

The basement, the roof, and openings in the vertical and horizontal force-resisting systems, were considered in developing the 3-D finite element structural model of the building. To which extent these affect the natural vibration periods from the eigenvalue analysis of the model is presented in Table 2. Door and window openings in the shear walls have the greatest impact; if not considered in the model, the natural vibration periods in the N-S and E-W

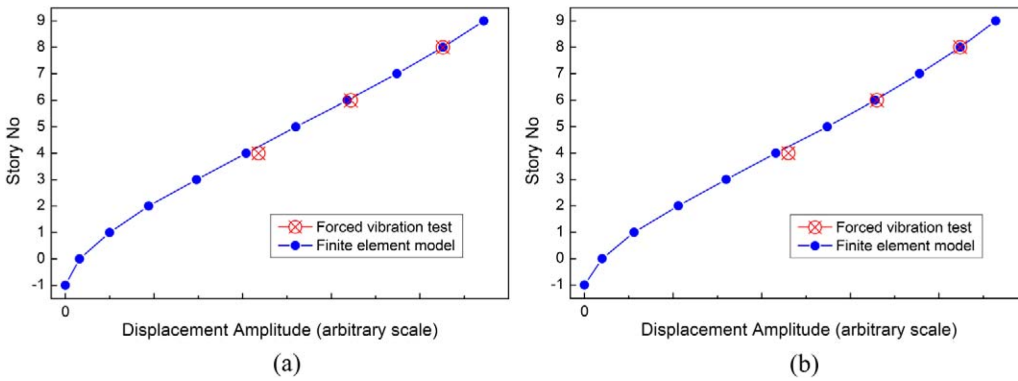


Figure 8. Mode shapes in (a) N-S and (b) E-W directions.

Table 2. Impact of modeling considerations on natural vibration periods

	T (s)		% Difference with respect to model 3		
	N-S	E-W	N-S	E-W	
Forced vibration testing	0.32	0.28			
Model 3	**0.31**	**0.29**			
No door and window openings*	0.24	0.20	24	29	Stiffer
No basement*	0.26	0.25	15	13	Stiffer
No roof*	0.29	0.27	5	5	Stiffer
No openings in flat plates*	0.31	0.29	<1	<1	Flexible

*Based on model 3.

directions are respectively shorter by 24% and 29%, and hence the building is stiffer. Openings in the flat plates, on the other hand, have virtually no impact on the period values. The basement has the second greatest impact; if not modeled, the natural vibration periods are shorter by 15% and 13% in the N-S and E-W directions, respectively. The decrease is 5% if the roof is not modeled.

CONCLUSIONS

In recent years, tunnel form construction has become the preferred method for mass housing, urban renewal, and post-earthquake reconstruction projects in Turkey and many developing countries in the Middle East, North Africa, and Central Asia. Although the widespread use of this structural system, there have been few dynamic tests performed on existing reinforced concrete flat plate buildings with shear walls. This study has identified the dynamic structural properties of a typical nine-story tunnel form building by forced vibration testing and developed its 3-D finite element structural model. The building had been tested before any nonstructural components were in place. The measured translational vibration periods, damping ratios and the first translational mode shapes added to the limited dynamic test database in the literature on existing tunnel form buildings.

The 3-D linear elastic finite element structural model of the building was developed by using shell elements for the shear walls and flat plates. The model that used the modulus of elasticity for concrete in ACI 318 and the uncracked (gross) moments of inertia as stated in TEC predicted well the natural vibration periods identified from the forced vibration testing. Mode shapes were also in good agreement with the test results. When the modulus of elasticity for concrete in TS 500 was used, the finite element model predicted shorter periods. Door and window openings in the shear walls, and the basement emerged as modeling considerations that have the greatest impact on structural dynamic properties.

Clearly, more dynamic tests on existing tunnel form buildings need to be performed for improving building codes, verifying the assumptions made in the analysis and design of these buildings, and validating and calibrating the structural models that are used to predict their dynamic structural responses to earthquakes. Ambient and forced vibration tests performed after major earthquakes and seismic monitoring during major earthquakes are of special importance as the identified dynamic properties from these tests represent the true structural behavior under design loads.

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