SEISMIC PERFORMANCE ASSESSMENT OF A TALL BUILDING BASED ON REAL-TIME MONITORING

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ABSTRACT: To establish resilient communities, it is crucial to know the damage states of structures after catastrophic events such as earthquakes. For this purpose, probability density functions, which demonstrate the probability of failure for a prescribed damage parameter, have been adopted. The construction of such curves is dependent on results acquired through Finite Element Models (FEMs). However, due to the assumptions made in such models, seismic responses of buildings differ considerably from the simulated ones. Therefore, Structural Health Monitoring techniques have been adopted to update the FEMs based on modal parameters obtained from vibration-based identifications.
In this study, in order to examine feasible solutions to problems mentioned above, a twenty-six story, core-wall tall building in Istanbul was instrumented with data acquisition system. A real-time monitoring section, which displays vibration records from the building, is developed on the webpage of the research group. FEM of the structure was updated to represent the actual dynamic characteristics of the building. The seismic performance assessments of updated and non-updated FEMs are carried out with the ground motions which are selected according to the characteristics of the expected earthquake in İstanbul. Because of the fact that that the structure remains linear under the design-level earthquakes, probability density functions with identified damping and observed damping by different researches for inter-story drift ratios are established in order to find the probability of failure under those ground motions.

Introduction

In recent decades, due to a lack of adequate construction sites, tall buildings have been the dominant means of accommodation and places of business in metropolises where economy and population grows fast. Compared to ordinary buildings, tall buildings are more densely populated, resulting in a bigger impact on the economy. In seismically prone areas, such as San Francisco, Tokyo and Istanbul, the safety of such buildings should be known prior to an earthquake and any damage due to the earthquake should be detected. To meet such necessities, tall building initiatives have been active especially in California to establish a framework for the selection of input motions, modelling approaches and performance criteria (Moehle, 2007).
Structural Health Monitoring (SHM) systems allow us to understand the dynamic characteristics of the buildings. Based on the ambient vibration data records, the dynamic characteristics of a building such as modal periods, shapes and damping ratios can be determined. Taking those characteristics into account, the existence of any damage and verification of design assumptions can be determined. In addition, the finite element model (FEM) of the building can be updated to represent the true behavior of the building.
From the early 1980's, system identification with SHM systems has been applied in civil engineering. A comprehensive literature review on the vibration-based SHM was first presented by Doebling et al. (1996), summarizing hundreds of publications up to 1995. Recently, an updated review of the literature was presented by Sohn et al. (2003), summarizing publications from 1996 to 2001. In studies such as Beck and Jennings (1980); Yun and Shinozuka (1980); Safak (1991); Ghanem and Shinozuka (1995); Lus et al. (1999), structures were examined as a linear system and the modal parameters of this system were obtained in time or frequency domain. Moreover, earthquake responses of tall buildings have been

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investigated by using system identification techniques by Celebi and Safak (1991); Celebi et al., (2004). SHM was also employed for seismic performance evaluation in terms of inter-story drift ratios (Naeim, 1998; Celebi et al., 2004). Additionally, a FEM can be updated to acquire the value of structural parameters by minimizing the difference between the modal parameters obtained from FEM and from experimental data (Skolnik, 2006; Moaveni, 2009).

While vibration data can be utilized for damage detection, it can also be utilized to determine the real dynamic characteristics before any damage occurs. It is well-established that dynamic characteristics generated from FEM and vibration data, even for an intact building, show remarkable differences. The assumptions made for the FEM due the uncertainties in buildings are one of the main reasons of these differences as shown by Brownjohn et al. (2000).

The damping ratios of the structures depend on numerous factors and it used to not be possible to know the exact damping ratio of a structure unless a full-scale experiment of the structure was performed. However, it has been recently possible to acquire the damping ratio of a particular building by the virtue of SHM techniques. In many researches, it was examined that the damping ratio diminishes as the height of a structure increases, making tall buildings vulnerable due to low amount of dissipated energy. The damping measurements from buildings with different heights are presented and best fit trend-lines are obtained according to these measurements in several studies (Goel at al. ,1997), (Smith at al. , 2003), (Smith et al., 2010). However, the findings from those studies differ from each other due to the amplitude of vibrations and characteristic of structures such as type of foundation, amount of partition walls, construction materials and design type. Therefore, there are still many discussions about modelling the damping in FEM's during conducting linear and non-linear time history analysis. In non-linear seismic assessments, large force imbalances can occur in the softening structural elements in the non-linear phase when Rayleigh Damping is utilized (Bernal, 1994), (Charney, 2006), (Hall, 2005). In the linear analyses, accurate determination of modal damping has crucial effect on the seismic response of buildings.

In this research, a twenty-six story, core-wall tall building in Istanbul was instrumented with sixteen accelerometers. Following the installation, system identification was performed by the Frequency Domain Decomposition (FDD) method (Brincker et al., 2001). These results were compared with the modal values obtained from the FEM of the building which is created according to design drawings. The effect of FEM updating with identified mode shapes and frequencies on the seismic demand of the structure is discussed in an article which is about the structure of this study by Kırkpınar at al. (2013). However, in this paper, the effect of damping is taken into consideration and real-time data monitoring section is added. It is observed that tall buildings have lower damping than ordinary buildings and overestimation of damping engenders higher accelerations and forces on structural elements whereas underestimation of damping can lead to conservative design. So, an updated FEM with the identified mode shapes and frequencies is generated. In addition to modal parameters obtained in the previous study, the damping ratio is also identified. Because of the fact that the structure in interest responds in linear range under the design-level earthquakes, linear time history analyses are performed in order to observe the performance of the building in a possible earthquake caused by the North Anatolian Fault. Probability density functions in terms of inter-story drift ratios are established in order to quantify the probability of failure under different ground motions. Due to linear response of the building, the identified damping ratio under ambient vibrations is utilized. The cracking of concrete gives rise to an increase in damping. However, since it is an instant increase that occurs once in cyclic loadings, this increase can be ignored. Linear seismic demand analyses are also carried out with the damping values obtained from the trend-lines in the studies of Goel and Chopra, and Smith at. al., and the results are compared. The result shows that accurate determination of damping is a crucial factor in seismic demand of tall buildings.

Building Information and Data Acquisition System

The building in the study is one of the four high-rise buildings of an outstanding project which has been recently constructed in one of the financial centers of Istanbul's metropolitan region. Those buildings are connected on the first stories by a large shopping mall. The structural system of the high-rise building is a cast-in-situ reinforced concerete structure, consisting of two cores of structural walls in the center, and columns on the outer region. The structural plan is the same for all of residential floors in the tower and for almost all of the mall and basement floors. The height of each story, 3.5 meters, is also the same for the residential floors but they vary for the mall and the basement floors.
The instrumentation of the structure is carried out with thirteen accelerometers to measure the vibrational responses of the building. For each story, accelerations in x and y directions are recorded at a position close to the centre. On the uppermost floor, an additional acceleromater is placed on the lateral y-direction, approximately 6 meters away from the centre, in order to observe the torsional modes of the structure.. Figure 1 and 2 show the building instrumented and the sensors layout respectively.

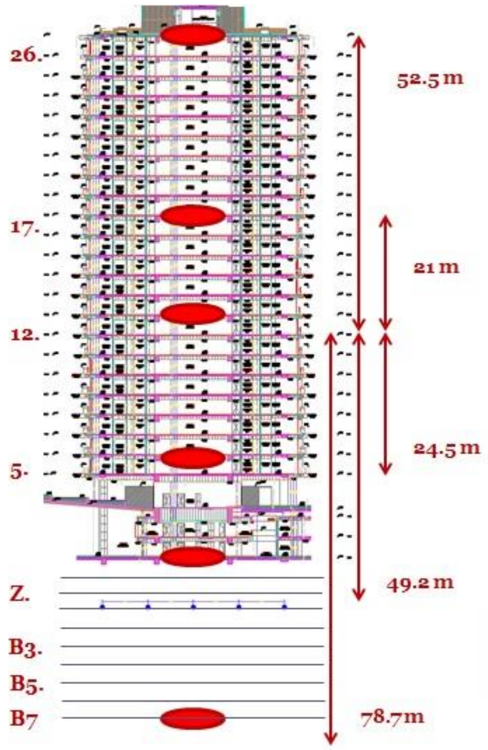

Figure 1. The structure monitored in the study.

Figure 2. Sensor layout.

Real-time Monitoring Section

To evaluate the damage condition of structures after earthquakes or the vibrations under ambient conditions or during the strong winds, to monitor the structures in real time is very crucial. Therefore, a real-time monitoring section, which displays vibration records from the building, is developed on the webpage of the research group (www.shm.ce.boun.edu.tr). Furthermore, a network between data acquisition system and the office computer is generated with the aim of analyzing the records automatically and permanently. Figure 3 shows the real-time monitoring section during a windy day so it can be clearly seen that the structure vibrates in its fundamental mode.

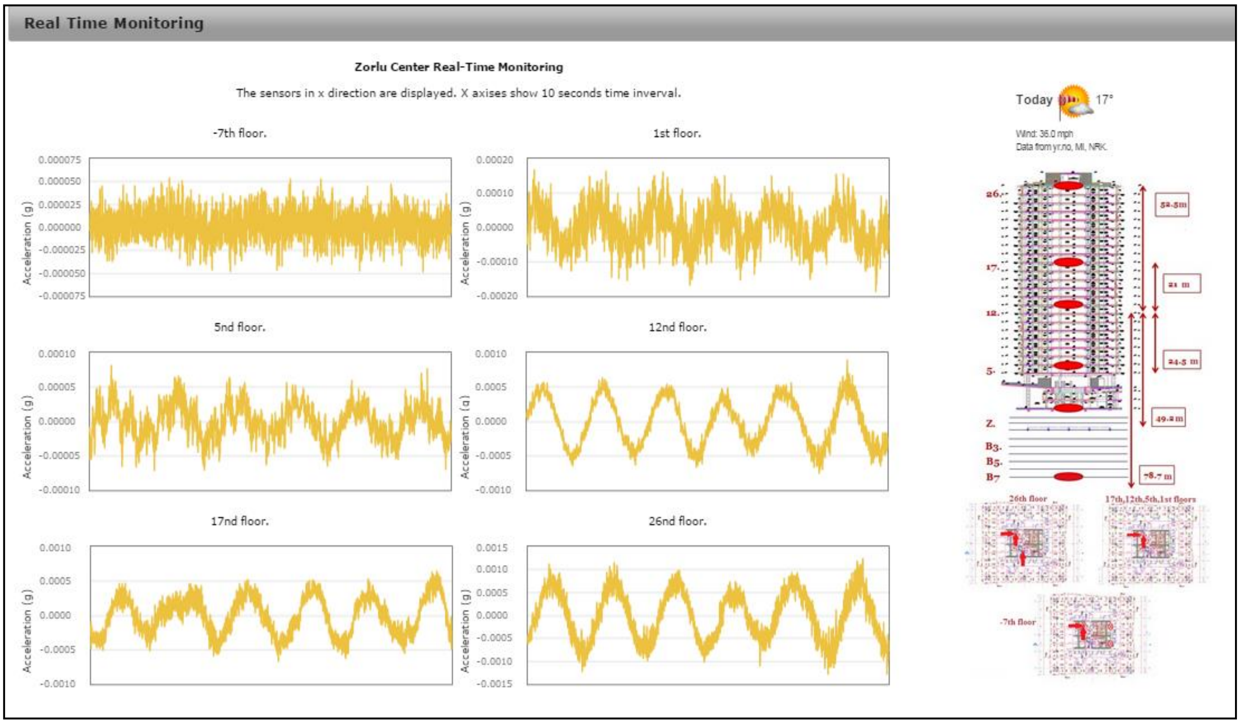

Figure 3. Real-time monitoring section on the webpage of the research group.

System Identification

In this study, the frequency domain decomposition (FDD) method which transforms the data in time domain to the frequency domain is used for system identification of modal frequencies and mode shapes. This method is useful because it is not in need of any input and it is capable of identifying close modes. In this method, square matrices whose indices are the cross power spectral density of the data from two different sensors are generated for each frequency output. The singular value decomposition of those matrices gives spectral values and mode shapes for each matrix, therefore, for each frequency output discretized in the frequency domain.
Figure 4 demonstrates the cross power spectral density matrix where the peaks are the natural frequencies of the structure. From the figure, it is observed that the frequencies of the first, second, third, and fourth translational modes are 0.59, 2.15, 3.81, and 7.23 Hz, respectively in the x-direction as the result of modal identification. The other small peaks represent the torsional modes identified, which are not considered in this study for FEM updating.

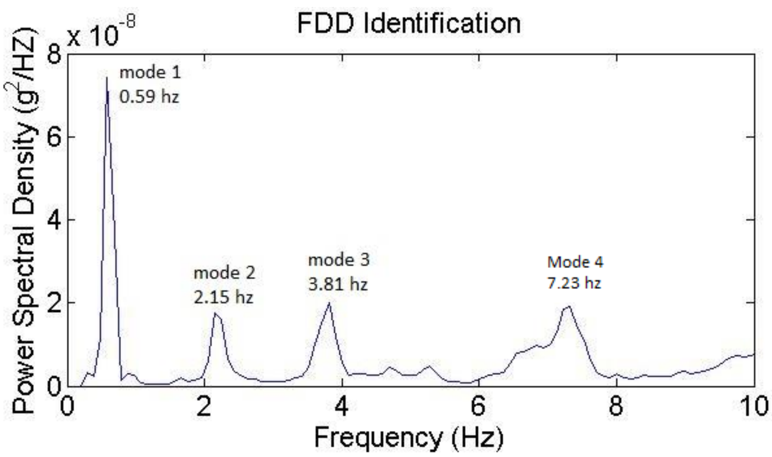

Figure 4. Identified natural frequencies.

After obtaining the cross power spectral density, the damping identification of the structure is performed. For this purpose, the impulse response function is generated by transforming the response of first mode in frequency domain to time domain. The level of decrease in the impulse response function gives the damping ratio and this decrease is an exponential decay unless there is any damage in the structure. Therefore, an exponential line is fitted to the peaks of each cycle in the impulse response and the damping ratio of the structure is identified as 2% from Figure 5.

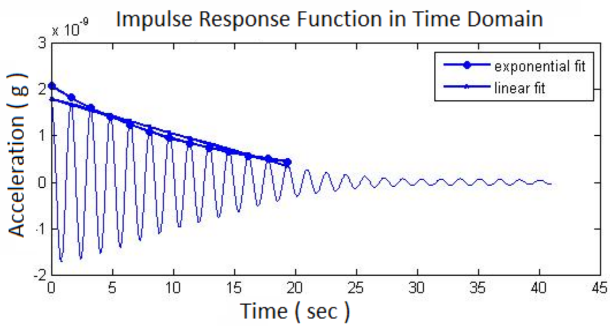

Figure 5. Impulse response function.

Finite Element Model

Based on the design drawings and site investigations, a FEM is created in the SAP 2000 v16 software platform. The shear walls, columns and beams are modelled as stick elements. The slabs are explicitly modelled and diaphragm constraint at each floor level is avoided in order not to affect the shear wall-shear frame interaction.

The structure consists of three different levels including the basement section, shopping mall section, and the residential tower section. The basement section includes the stories under the ground level and it is connected with the foundation. The mall section of the building is comprised of several stories which are surrounded by the overall mall structure in an irregular manner. Even though there is a significant discontinuity and complexity of many structural elements of the basement and the mall floors, all the sections of the building is modelled in every detail whereas more coarse FEMs are utilized in many system identification studies. The basement floors are separated from the surrounding structures with seismic gaps at three sides. One side of the building in interest is separated with the soil strata with a concrete wall and the stiffness contribution of soil is modelled as horizontal springs in the FEM. In addition, vertical springs are assigned at the base to simulate the foundation-structure fixity. The coefficients of vertical and lateral springs are calculated according to Boussinesq Theorem. The modulus of elasticity of concrete is chosen with respect to Turkish Standards 500 (TS500).

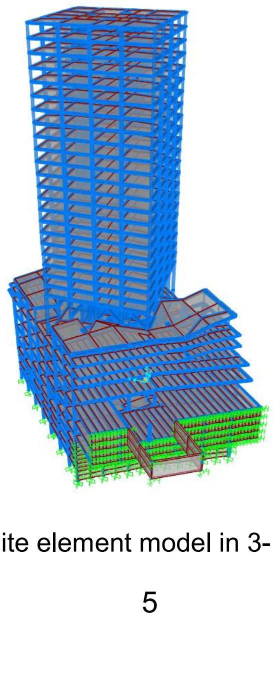

Figure 6. Finite element model in 3-D and top view.

FEM Updating

The FEM Updating Procedure was performed based on minimizing the difference between experimental and analytical modal values. For this purpose, a Matlab code which can automatically update the initial FEM by changing the structural parameters within determined values, and which gives the model values of each updated model was developed. The code compares modal outputs from each model with the identified modal parameters based on an error function to find the most optimal model. Following the FEM Updating Procedure, modal frequencies and mode shapes which represent the ones generated from model identification are obtained. Equation 1 exhibits the objective function which quantifies the error between the simulations and the identification results.

$$E = \sum \left(k_i \cdot \left[\frac{(f_i^* - f_i)}{f_i^*} \right]^2 + h_i \cdot [1 - MAC_i]^2 \right) \tag{1}$$

Where,

i is mode number;

k_i *is the weighing coefficient for* i^{th} *modal frequency;*

h_i *is the weighing coefficient for* i^{th} *modal assurance criteria;*

f_i^* *is the measured modal frequency of* i^{th} *mode;*

f_i *is the simulated modal frequency of* i^{th} *mode;*

MAC_i *is the modal assurance criteria for* i^{th} *mode shape*.

For this study, four different updating parameters and 1296 (6x6x6x6) different combinations of these parameters are utilized. One parameter is the modulus of elasticity of concrete along the tower section, which represents the rigidity of this section when it is multiplied by moment of inertia coefficients. The reason to consider this parameter is that it is not possible to known the overall stiffness of the structure due to uncertainties in rigid end zones, stiffness contribution of partition walls, amount of mass and modulus of elasticity of concrete. The other two parameters are the coefficient of horizontal soil springs between -7th and -1th floors and on the ground level. The reason that lateral soil springs are divided into two segments is to catch the identified mode shapes. The decision of the location where the horizontal springs are divided into segments is based on the comparison of identified mode shapes with the mode shapes of the non-updated FEM. The last parameter considered in the updating procedure is the coefficient of vertical springs at the base.

Because of the fact that the primary modes have higher effects on the dynamic behavior of the structures, the objective function involves weighing coefficients to define the contribution of different modes. However, it is a well-known fact that higher modes play a significant role in the dynamic characteristics of tall buildings. Consequently, the weighing of coefficients such as 0.60, 0.25, 0.15 are used respectively for comparison of the first, second and third modal parameters. These coefficients are chosen according to the modal participation factors of corresponding modes and to the fact that the first modes respectively have major impacts on dynamic characteristics of the structure. Therefore, according to FEM analysis, the first three modes are adequate to be considered in the updating procedure.

As a result of the minimization of the objective function for the first FEM, the modulus of elasticity along tower section, the spring coefficients between -7th and the -1th floor, the spring on the ground level, and the vertical springs in the base are identified as 35.3 GPA, 5.4, 23 and 4.9 GN/m, respectively. So, the final FEM with identified mode shapes and frequencies is produced by taking the uncertainties in the stiffness contribution of the surrounding soil, in overall stiffness of the structure and the fixity of base into account. Table 1 demonstrates the frequency of modes of non-updated and updated FEM as well as the ones determined from the system identification. In addition, Figure 7 displays the mode shapes before and after the FEM Updating Procedure. It is well illustrated that the real dynamic characteristics of the structure is well represented by the virtue of system identification.

Table 1. Identified, non-updated and updated modal frequencies.

Mode Number	Frequencies of Vibration (Hz)		
	Updated FEM	**Modal Identification**	**Non-updated FEM**
1	0.59	0.59	0.55
2	2.30	2.15	1.85
3	3.90	3.81	3.37

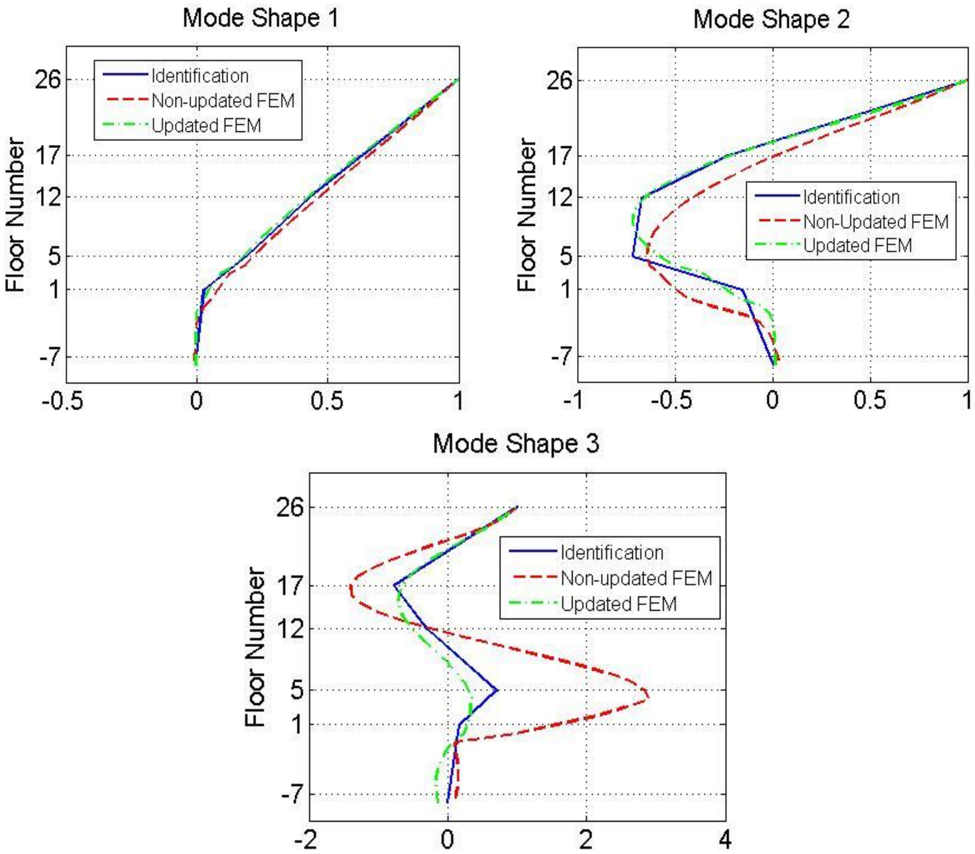

Figure 7. Experimental, updated and non-updated mode shapes.

Reliability Estimation with the non-updated and the Updated FEM

With the advent of sensor and computational technologies, SHM has experienced significant advance in the last 30 years. However, reliability estimation is rarely discussed in the literature from health monitoring perspectives. For this reason, further contribution regarding this subject is essential for a better comprehension of health monitoring results.

While trying to anticipate the seismic demand of the building, the North Anatolian Fault, which is the most important fault line in Anatolia, is considered as the main risk source. This fault line crosses Anatolia from one end to another and it is a very active seismic area with several major and lots of minor fault lines. Numerous historical records show that very destructive earthquakes were produced by this fault line. The last two of them were devastating Kocaeli (M=7.4) and Duzce (M=7.2) earthquakes which occurred in 1999.

To simulate the seismic demand on the building, twenty one ground motions are selected according to design-level earthquake via *Peer Strong Motion Database NGA-West 2 Ground Motion Selection Tool* based on the ASCE41-13 design based uniform hazard spectrum with 2% identified damping ratio, which is generated based on PGA maps of Istanbul Metropolitan Municipality. The characteristics of the selected ground motions, whose mean is over the design spectrum in the range of 0.2T and 1.5T are chosen according to the characteristics of the expected earthquake in Istanbul These are an approximately 7.5 magnitude, strike-slip fault mechanism and, 20-30 km distance between the site and the fault segment as suggested by Erdik et al. (2004). Figure 8 shows the spectra of the selected inputs and Figure 9 shows the building site with a star on the map of the Marmara segment of the North Anatolian Fault.

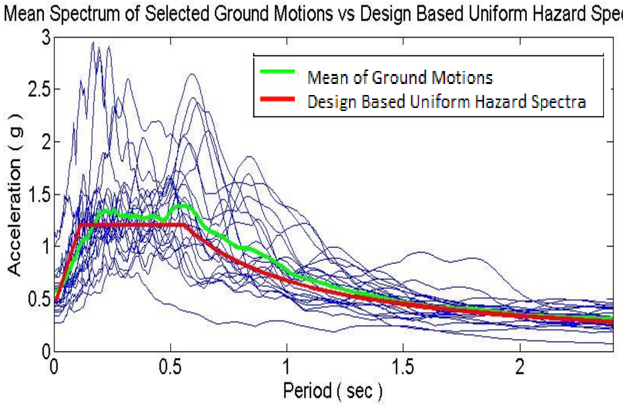

Figure 8.Spectra of selected ground motions.

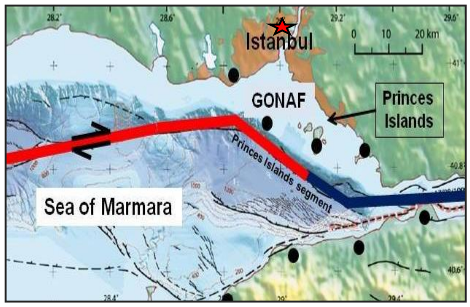

Figure 9. Location of building and the fault line.

In this study, the maximum inter-story drift ratio is determined as the damage indicator parameter. Twenty one earthquake records are used as input data to simulate earthquake demand on the building, and structural response is generated in terms of inter-story drift ratio time histories. Non-linear time histories demonstrated that the building remains elastic under the selected earthquake records. Consequently, linear time history analyses are performed and results are used to construct probability density functions with log-normal distributions where random variables are the maximum inter-story drift ratios. The main purpose of this paper is to exhibit the impact of the damping ratio, therefore probability density functions are established for 2%, 3.8% and, 1% damping ratios. The ground motions are not selected according to each ratio in order to observe the influence of different damping ratios on the seismic response of the building. The first ratio is the identified one and it is very close to the trend-line generated in the study Smith at al. (Smith at al., 2010). The other two ratios are chosen according to the similar trend-lines in the study of Goel and Chopra (Goel at al.,1997), and Stake at al. (Stake at al. ,2003) Figure 10 exhibits the probability densities as functions of maximum drift ratios with the considered damping ratios.

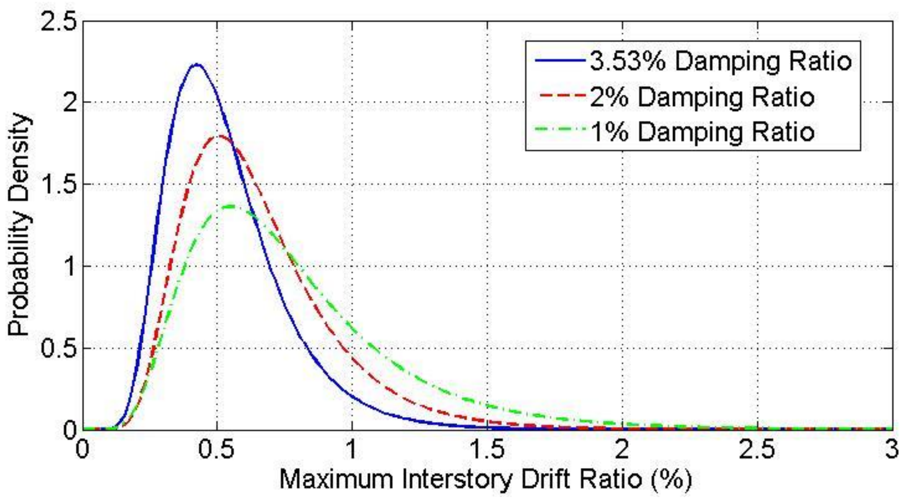

Figure 10. Probability densities as functions of maximum drift ratios.

Turkish Earthquake Code states that 1% interstorey drift ratio is the acceptance criteria for the immediate occupancy and that, life safety performance range lies between 1% and 3% interstorey drift ratios. Therefore, when a threshold is set to 1% in order to see the effect of the damping ratios on the performance of the structure, the probabilities of experiencing life safety damage state are determined as 6% for the 3.53% damping ratio, 11% for the identified 2% damping ratio and 22% for the 1% damping ratio. Based on the outcomes, the structure has two times more possibility to experience life safety damage state with 1% damping ratio than with the identified damping ratio. Such condition may lead to conservative design for this structure. When 3.53% damping ratio is compared to the identified one, it is seen that the life safety capability of the structure is underestimated as 5%.

Moreover, even if the probability density functions demonstrate that the level of damage is not destructive and the building in interest remains mostly in the immediate occupancy damage level, the cost of repairment may be too much or the building owner may want to sustain the full functionality of his or her investment right after the seismic event. Therefore, anticipating the real seismic demands on the structures plays a crucial role in the performance of structures. To emphasize this fact, a threshold value may be set as 0.5% interstory drift ratio. Figure 10 shows that that the possibilities of exceeding 0.5% drift ratios are 27%, 34% and 50% for the 1%, 2% and 3.53% damping ratios, respectively. It is clear that the possibilities of performing the prescribed threshold damage level differ from each other significantly.

Conclusion

In this study, a real-time monitoring section is developed on the webpage of research group. SHM techniques are employed in order to eliminate the uncertainties in FEM. Dynamic characteristics of a tall building is identified by the FDD method and the impulse response function. The differences between the model and actual dynamic parameters are minimized by FEM Updating Procedure. Linear time history analyses are carried out with the ground motions selected according to expected earthquake in Istanbul. Because it is impossible to accurately define the damping for a structure, the best fit trend-lines, which displays the damping ratio of structures according to their heights and, which are generated based on the vibration measurements, exhibit remarkable differences from each other. To underline the fact that precise determination of damping is crucial for the reliable seismic performance assessments, the identified damping ratio is compared with the ones obtained from the trend-lines of other studies. The results indicated that the possibilities of exceeding the prescribed damage levels are considerably dependent on the damping ratio, and that carrying out performance assessments with different damping ratios other than the actual one leads to wrong outcomes. Therefore, the damping ratios of tall buildings should be examined carefully by the virtue of SHM techniques.

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