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Mindlin's theory buckling building codes compressive strength dynamic analysis dynamic response earthquake finite element analysis finite element method mechanical properties nonlinear analysis optimization prefabrication pushover analysis reinforced concrete seismic analysis seismic design seismic isolation shallow foundations steel silo temperature effects thick plate wind

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Research Article

# An investigation on determining optimum wall ratio–cost relationship of shear walled reinforced concrete buildings

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## ABSTRACT

Reinforced concrete walls are very efficient structural elements in terms of carrying the lateral loads that are expected to affect the structures during the service of the buildings. These elements, which are not used for economic reasons in buildings designed in areas with low seismic hazard, can actually provide a significant increase in performance with a very small increase in construction cost. In this study, a total of 9 building models have been created and the relationship between optimum reinforced concrete wall ratio and cost on these buildings has been investigated. The design and analysis of the models were carried out according to the criteria specified in TSC 2018. Three different structural systems specified in TSC 2018 were used in the designed models. These structural systems used; RC frame structures, RC wall-frame structures and RC wall structures. These structures were analyzed by Response Spectrum Method which is linear analysis method and base shear forces were obtained. Then, push-over analysis, which is a nonlinear analysis method, was applied to obtain the base shear forces that the structure can actually carry. After the analysis, the quantities of materials to be used for the construction of the structural systems of the models were calculated and current manufacturing prices and rough costs were calculated. In order to compare the obtained costs with the structural performances, nonlinear shear forces and linear shear forces ratios were calculated and the over strength factors were calculated for each model. In the light of the data obtained from the studies in the literature, when the over strength factors and cost values are examined together, it is concluded that the optimum design for the conditions specified in TSC 2018 will be provided with the RC wall ratio between 0.001 - 0.0016. It is concluded that lateral load carrying capacity of construction increases up to 650% by increasing the construction cost by 17% for the designed models.

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## 1. Introduction

With the development of building manufacturing technologies, the costs of the buildings constructed have increased in line with the demand and purposes of use. The use of reinforced concrete walls has become an important issue in the design of the structural systems since the lateral forces that affect the buildings will increase with the increase of the building heights. In areas where the potential earthquake hazard is high, not only the presence of reinforced concrete wall elements in the floor plan, but also the positioning of these elements in the floor plan, the cross-sectional areas of the RC walls, the preferred RC wall ratio, etc. values are great importance in the design of the optimum structural system. In this study, it is aimed to compare the seismic strength and behavior of the building models designed by using different RC wall areas and to observe the effect of the change in the building cost on the building performance. It should be noted that it is possible to design different structural systems by keeping the construction cost constant. However, keeping the cost - performance relationship at an optimum level is also important in order to prevent loss of life and property under the effects of earthquakes.

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There are different studies on this subject in the literature. Andinç (2005), in his thesis study; concrete structures designed as high ductility level are considered as structural systems with walls and frames and RC walled structural system and determined the RC wall heights of these buildings up to 20 floors. Then, these RC walls were compared for different heights, different floor numbers, different site classes and seismic zones. In their study, Uçar et al. (2009), designed reinforced concrete walls symmetrically placed in the floor plan according to TDY 2007. Then they placed these elements on the internal and external axes of the floor plan and examined the effects of this positioning on the seismic behavior. Tekelli et al. (2008), have developed different analytical relationships using differential equations developed to calculate the displacement of reinforced concrete structures with framed and framed-framed structural systems. Thus, they have developed a very simple method for calculating structural shifts. Erken (2013), a concrete building designed as a residential building was used in his study. He designed 5 different structural systems in accordance with the architectural plan of this building. He investigated the effect of the changes in the ratio and distribution of reinforced concrete walls in structural systems on the parameters considered in the structure design. Aktan et al. (2010), examined the reinforced concrete walls, which are the most effective elements in carrying the lateral loads affecting on the structures. They emphasized the concepts of stiffness, strength and ductility required in building design and highlighted the importance of these titles in design of the seismic-resistant structures. They investigated behavioral changes obtained by changing the distribution of reinforced concrete walls on 8 different floor plans. Pakoğlu (2009), in his thesis, designed a 100 meter high building with reinforced concrete tube walled structural system and subjected this building to a performance analysis. Madddela et al. (2017) in their studies; they examined the effects of RC wall elements carrying lateral loads on performance by applying static push over analysis on two models with 10 and 15 storey symmetrical floor plans consisting of 5 equal bays in X and Y directions. In his study, Madenci (2019) presented an alternative solution procedure by using variational methods. The mixed-finite element method (FEM) is employed to obtain a beam element. The software (STA4Cad) used in this study uses the same method. Erkan et al. (2019) designed 3 different 5-storey building models with structural system with only frames, structural system with walls and frames and structural system with only RC walls. Then they applied linear static analysis (Equivalent Seismic Load Method) and static push over analysis on these structures. As a result of the study, they examined the effects of different RC wall ratios on the over-strength factors. Doğan (2019) determined 3 different building heights in his thesis. For each building height, 6 different RC wall ratios were determined and a total of 18 types of building models were created. Response Spectrum analysis and static push over analysis were applied on these models. After the analysis, he examined the over-strength factors for each building.

This study was carried out in order to determine the optimum wall ratio and cost relation in concrete reinforced concrete buildings. A total of 9 models were created within the scope of the study. One of them is the reference model without RC wall in its structural system, the remaining 8 models with structural systems that contain RC walls in the floor plan. The designed models were subjected to dynamic analysis with the Response Spectrum method. The analyses were performed in accordance the conditions specified in TSC 2018. The RC wall ratios in the analysis models range from ‰0.96 to ‰3.2, except for the reference model.

## 2. Analysis Study

### *2.1. Analysis models*

In this study, 9 different building models were created in accordance with the analysis and design conditions specified in TSC 2018. One of these models is designed as a model with a structural system consisting of reinforced concrete frame elements (model with a wall ratio of 0). This model was used as a reference model to compare the results of other analyzes. Afterwards, 4 different models consisting of structural system with RC walled frames and 4 more different models consisting of structural system with only reinforced concrete walls were created. In TSC 2018 of the reference model, the response modification coefficient is given as R=8, the response modification coefficients of models consisting of structural systems with RC walled frames are given as R=7 and the response modification coefficients of models consisting of structural systems with only RC walls are given as R=6.

In the study, the $h/b$ ratio of the smallest of the reinforced concrete wall dimensions used was determined as 6 and this ratio increased up to 20 ($h$: long face; and $b$: short face of shear wall).

RC wall ratio increased with the increase of preferred RC wall cross-sectional areas in the design of structural systems. In TSC 2018, the definition of RC wall ratio is given as; “the ratio of the total cross-sectional area of the reinforced concrete shear elements in any selected earthquake direction to the total gross area of all floors in the building”. In addition to this ratio being equal to or greater than 0.002, it is accepted that if the ratio of design base shear force to total wall area is less than half of the $f_{ctd}$ value of the concrete used in the structure design, the structural systems of the buildings consist of only RC walls (TSC 2018). The structural systems of buildings which cannot fulfill any of these conditions but still have RC walls in their structural systems are considered as RC walled frames.

Common geometric properties of models; all columns are 40x40 cm and all beams are 25x50 cm. Slab thickness was chosen as 15 cm and slab type was preferred as beamed plaque. The floor plans consist of 5 bays in each X and Y direction, 5 meters each. Total building height is 15 meters for all models with story heights of 3 meters. Concrete used for these models is C25 (25 MPa) and reinforcing rebar is S420 (420 MPa). All of the elements in structural systems are designed as frame elements so meshing process is not necessary for analyzes. Nomenclature of models in this study; The reference model was selected as Model 1 and continued until Model 9 for other RC walled models. The designed floor plans of these models are given in Fig. 1. Table 1 shows the selected RC wall sizes and RC wall ratios for each model. Fig. 2 shows the 3D views of the models.

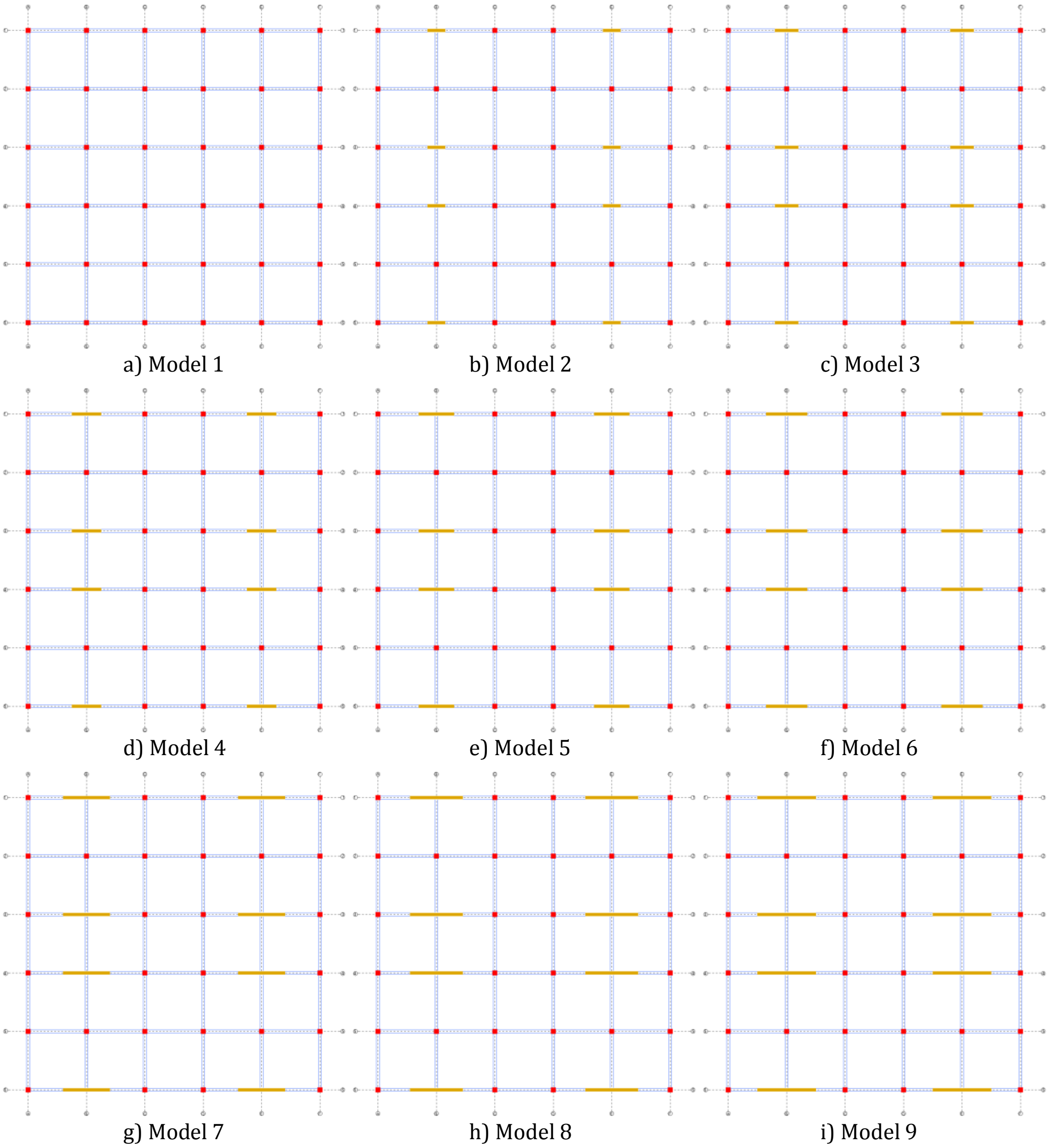
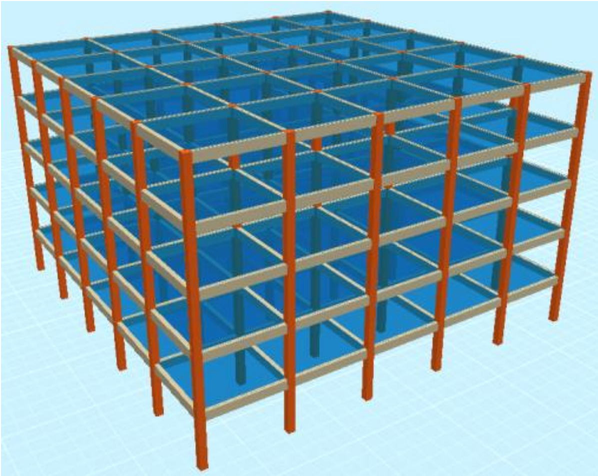


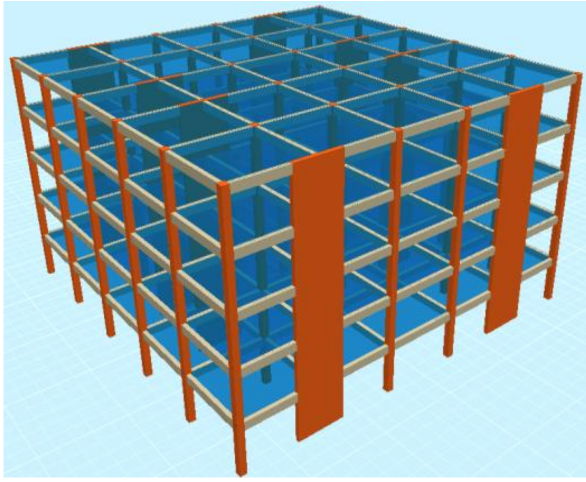

**Fig. 1.** Floor plans of the model.

**Table 1.** RC wall properties of the models.

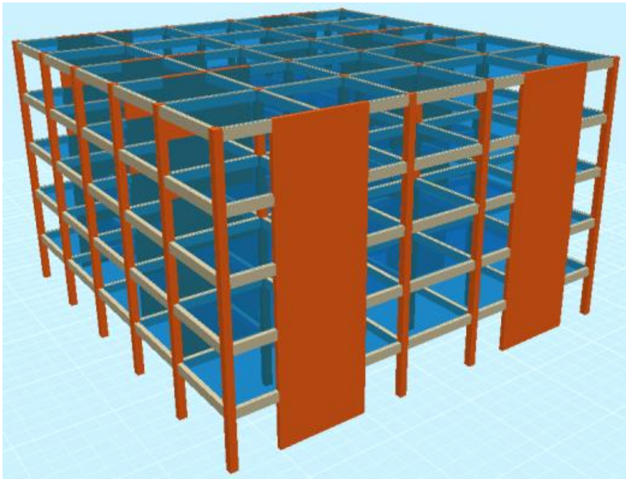
| Models | Number of RC Walls | Wall Dimensions (cm x cm) | Wall Ratio |
|---|---|---|---|
| Model 1 | 0 | 0 | 0 |
| Model 2 | 8 | 25 x 150 | 0.00096 |
| Model 3 | 8 | 25 x 200 | 0.00128 |
| Model 4 | 8 | 25 x 250 | 0.00160 |
| Model 5 | 8 | 25 x 300 | 0.00192 |
| Model 6 | 8 | 25 x 350 | 0.00224 |
| Model 7 | 8 | 25 x 400 | 0.00256 |
| Model 8 | 8 | 25 x 450 | 0.00288 |
| Model 9 | 8 | 25 x 500 | 0.00320 |



a) Model 1



b) Model 5



c) Model 9

**Fig. 2.** 3D views of Model 1, Model 5 and Model 9.

### *2.2. Analysis methods*

Following the design of the building models with floor plans and RC wall features, the analysis operations using STA4CAD v14 software were started (STA4Cad). The structures were first analyzed by the Equivalent Seismic Load Method and the base shear forces were obtained by linear calculation. The conditions given in the TSC 2018 for the application of this analysis are given in Table 2. In this table; *SDC* stands for seismic design class, *I* coefficient of significance, *BHC* stands for building height class, *BUC* stands for building use class and *n* is the live load multiplier.

**Table 2.** Conditions placed in TSC 2018.

| | | | |
|---|---|---|---|
| *SDC* | 1 | *BUC* | 3 |
| *I* | 1 | *n* | 0.3 |
| *BHC* | 5 | Site Class | ZC |

Response Spectrum analyzes were performed in accordance with the conditions given in TSC 2018. In similar studies in the literature, they used Equivalent Seismic Load Method as linear analysis method. However, Doğan (2019) compared Equivalent Seismic Load and Response Spectrum methods in his thesis and stated that more statistical and satisfactory results were obtained with Response Spectrum Method. Therefore, Response Spectrum method was chosen as the linear analysis method in this study. Using the interactive web application prepared for the Earthquake Hazard Maps in Turkey (AFAD, 2018), seismic parameters for the selected coordinates in Ankara - Çankaya were obtained. The obtained values are shown in Table 3. Selected coordinates was chosen because Ankara is the capital city of Turkey, and carries moderate risk in terms of seismic hazard.

**Table 3.** Seismic properties for selected coordinates.

| | | | |
|---|---|---|---|
| $S_S$ | 0.340 | $S_1$ | 0.118 |
| $S_{DS}$ | 0.442 | $S_{D1}$ | 0.177 |
| *PGA* | 0.148 | *PGV* | 9.981 |

Loads predicted to affect structures; $G$=2 kN/m$^2$ as dead loads, $Q$=2 kN/m$^2$ as live loads and $G_W$=5 kN/m as brick wall loads. Since the purpose of the buildings is residential, the live load multiplier in the load combination used for mass calculation is taken as $n$=0.3 from TSC 2018.

After the linear analysis, static push over analysis, which is a nonlinear analysis method, was started. The parameters of plastic hinges, which are one of the most important requirements of static push over analysis, have been determined in accordance with TSC 2018. A displacement value of 4% of the building height was loaded to the rigid diaphragm on the top floor of the building in a horizontal direction to obtain the base shear capacity of the structures. The term 'plastic hinge' should not be confused with the 'hinge' term commonly used in structural engineering. Because the hinge means a moment-free and freely rotating element. However, a plastic hinge means a cross-section of a member with a certain moment capacity, which carries moment until this capacity is reached and which can rotate under constant momentum when the capacity is reached. If we define the plastic hinge acceptance more theoretically; ductility coefficient, known as the ratio of the maximum deformations of the building element or model, with the deformations at the stage of yielding, where this coefficient value is high, the nonlinear deformations are restricted in a narrow area and the nonlinear bending deformations accumulate in certain regions known as plastic hinges; it can be assumed that the system or element sections other than those regions exhibit linear-elastic behavior. This acceptance is called as lumped plastic hinge. In line with these assumptions, static push over analyzes were performed on 9 different building models. Base shear forces obtained as a result of static push over analysis and base shear forces obtained from linear analyzes performed as the first step of the analysis study were compared and over-strength factors (D) were determined. The increase in the over-strength factors (D) is also examined in the results section due to the increase in the RC wall ratio.

### *2.3. Calculations of construction costs*

After the analyses were performed on the models, the manufacturing quantities and costs of these structures were compared. Manufacturing quantities are only calculated for concrete (as $m^3$), reinforcing rebar (as tons) and mold (as $m^2$). The comparisons were made according to the over-strength factors. While calculating these costs, current pricing values are used by using the quantities calculated for the models. The obtained values are examined in relation to the increase in RC wall ratios in the results section. One of the main objectives of this study is to correlate the increase in building cost and the change in over-strength factors.

## 3. Analysis Results

### *3.1. Linear analysis*

In the section of the analysis methods, the necessary data of the building models and the conditions specified in TSC 2018 are explained. Table 4 shows the base shear forces, total building weights and period values for the first 3 modes of the structures obtained from the linear analysis using these data. When these values in Table 4 are considered, it is seen that the linear base shear forces, which are predicted to affect the structure, increase with the increase of the ratio of reinforced concrete walls used in the buildings. This increase is valid as long as the decreasing period value is equal to or greater than the TA value in the acceleration spectrum with increasing stiffness. Building weights and base shear forces obtained as a result of linear analysis of the structures are compared in Fig. 3. It is seen that with the increase of RC wall ratio, the structural system weight of the buildings were increased by 5% and the shear forces were increased by 335% due to the increase in structural rigidity.

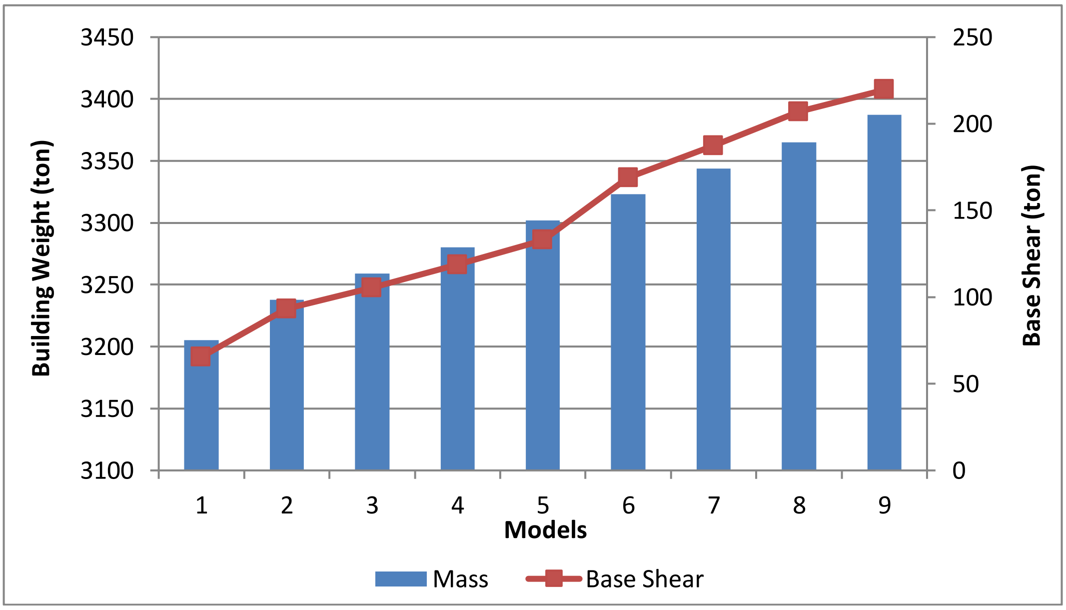



**Fig. 3.** Comparison of building masses and base shear forces.

**Table 4.** The results of linear analysis.

| Models | Model Weight (ton) | Base Shear $V_{tx}$ (ton) | Mod Periods (sec.) | Modal Mass Participation Ratios (%) |
|---|---|---|---|---|
| Model 1 | 3205 | 65.48 | 1.0042 (Y) | 83.32 |
| | | | 1.0042 (X) | 83.32 |
| | | | 0.8575 (T) | 83.271 |
| Model 2 | 3238 | 93.19 | 1.0480 (Y) | 83.486 |
| | | | 0.8053 (X) | 78.437 |
| | | | 0.7546 (T) | 79.729 |
| Model 3 | 3259 | 105.39 | 1.0467 (Y) | 83.391 |
| | | | 0.7152 (X) | 75.944 |
| | | | 0.6887 (T) | 77.471 |
| Model 4 | 3280 | 118.62 | 1.0453 (Y) | 83.284 |
| | | | 0.6346 (X) | 74.194 |
| | | | 0.6261 (B) | 75.652 |
| Model 5 | 3302 | 132.8 | 1.0443 (Y) | 83.173 |
| | | | 0.5682 (B) | 74.372 |
| | | | 0.5632 (X) | 73.116 |
| Model 6 | 3323 | 168.98 | 1.0434 (Y) | 83.058 |
| | | | 0.5154 (T) | 73.563 |
| | | | 0.5008 (X) | 72.521 |
| Model 7 | 3344 | 187.41 | 1.0429 (Y) | 82.946 |
| | | | 0.4679 (T) | 73.103 |
| | | | 0.447 (X) | 72.247 |
| Model 8 | 3365 | 206.89 | 1.0424 (Y) | 82.833 |
| | | | 0.4257 (T) | 72.893 |
| | | | 0.4011 (X) | 72.187 |
| Model 9 | 3387 | 219.6 | 1.0420 (Y) | 82.721 |
| | | | 0.3887 (T) | 72.857 |
| | | | 0.3621 (X) | 72.269 |

X: represents the mode in X direction, Y: represents the mode in Y direction, T: represents the mode in torsion

### *3.2. Nonlinear analysis*

After the linear analysis, static push over analysis step which is the second step of the analysis was started. In this analysis method; a horizontal displacement value determined by the height of the building is applied to a selected joint at the top level of the building. As a result of the analysis, the shear forces and plastic hinges that will occur on the structure are determined. Static push over analysis is based on obtaining the seismic performance of the structure by observing the base shear force that the structure can bear and the status of the plastic hinges formed on the structural system elements. The pushover curves obtained as a result of static pushover analysis of the structure models created on STA4CAD v14 program are given in Fig. 4. When these curves are examined, it is seen that the increase in shear rate and the increase in base shear forces become more pronounced compared to linear analyzes. RC wall elements, increase the stiffness of the structures against horizontal displacements and at the same time increase the base shear forces that the structure can bear. According to these data in Fig. 4, it is seen that the linear parts of the curves given for each structure, i.e. the stiffness of the structures, increase with the increase of the RC wall ratio.

Fig. 5 shows the ratio of the moment values that the RC walls meet to the total moments that affect the structure. These ratios increased with the increase in the amount of RC wall as shown in the Fig. 5. However, according to Fig. 5 it can be said that the increase in the slope of the curve is higher in the first four models, that are, the models with response modification coefficients R=7 compared to the models with R=6.

Table 5 shows the over-strength factors (D=$V_{tx}$ / $V_{tx}$) obtained by the ratio of the base shear forces ($V_{rx}$) determined by the static pushover analysis, which is the nonlinear calculation method, to the base shear forces ($V_{tx}$) obtained by linear analysis. The aim of this study is to determine the most efficient structure design by comparing the over-strength factors and building costs together. The desired design of the structure was selected for the RC wall ratio and the distribution of the RC walls in the floor plan.

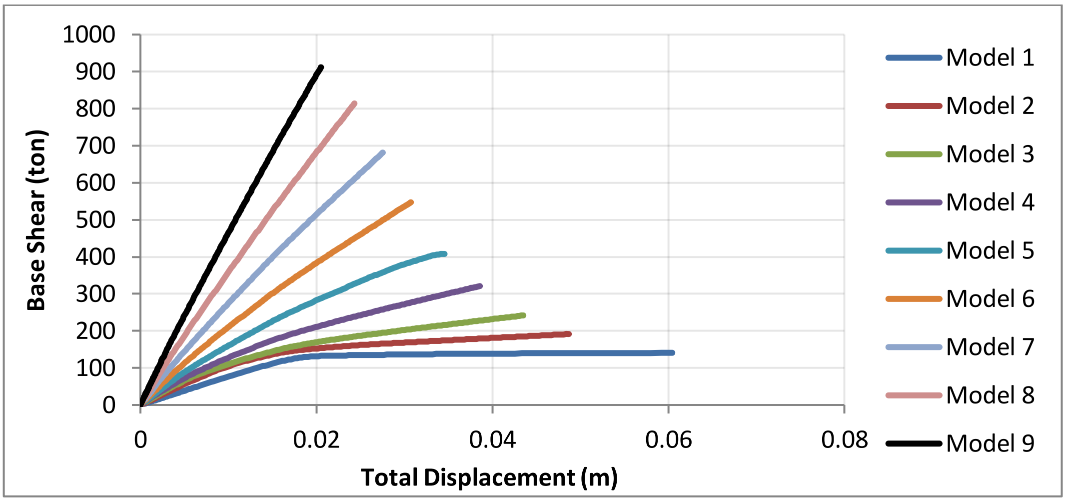



**Fig. 4.** Push-over curves for all models.

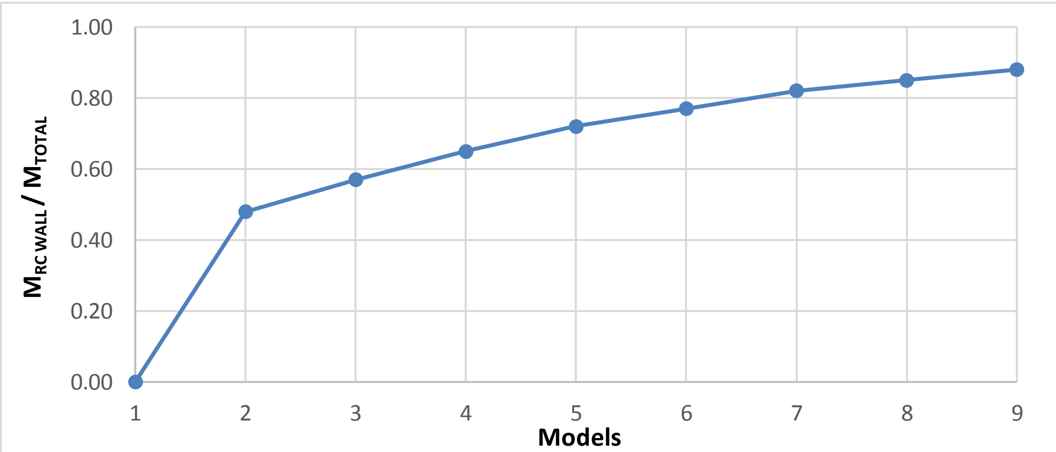



**Fig. 5.** The ratio of RC wall base moment to total turning moment.

**Table 5.** Nonlinear analysis results and over-strength factors.

| Models | $V_{rx}$ (ton) | $V_{tx}$ (ton) | Over-strength Factors ($D = V_{rx} / V_{tx}$) |
|---|---|---|---|
| Model 1 | 140.891 | 65.48 | 2.15 |
| Model 2 | 191.545 | 93.19 | 2.06 |
| Model 3 | 241.836 | 105.39 | 2.29 |
| Model 4 | 320.989 | 118.62 | 2.71 |
| Model 5 | 407.73 | 132.8 | 3.07 |
| Model 6 | 547.178 | 168.98 | 3.24 |
| Model 7 | 681.362 | 187.4 | 3.64 |
| Model 8 | 814.14 | 206.9 | 3.93 |
| Model 9 | 911.743 | 219.6 | 4.15 |

### *3.3. Calculation of construction costs*

The strength coefficients of the analysis models designed within the scope of the study and subjected to certain analysis operations were made by considering the base shear forces. The ratio of the over-strength factors to the rough cost values of the structures were determined and comparisons were made between the structures. Mentioned rough costs of building models are calculated as; multiplying the quantities of rebar, concrete and the mold elements of the structural systems were determined and the current unit price values. Fig. 6 shows the comparison of the rough costs of concrete, mold and rebar between models.

In order to observe the most accurate situation, the over-strength factor values and the roughly calculated costs of the structures were compared in a common diagram. These values are given in Table 6 for each building model. Then, the ratio of the over-strength factors to the rough costs of each structure was calculated. The values obtained with this ratio are compared in Fig. 7. D/$ values also increased with the increase in RC wall ratio, since the over-strength factors and building costs showed a consistent increase.

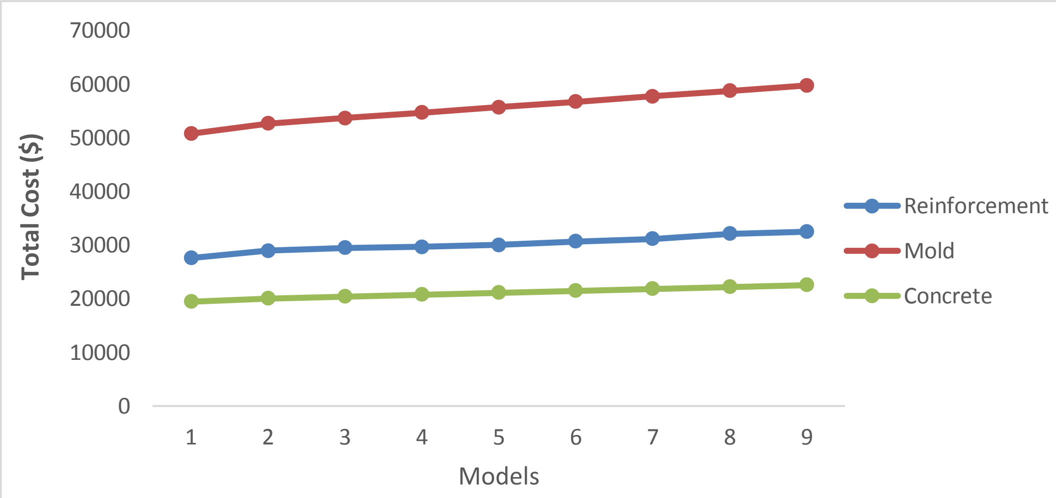



**Fig. 6.** Comparison of total costs of the models.

**Table 6.** Nonlinear analysis results and over-strength factors.

| Models | Roughly Calculated Total Costs ($) | Over-strength Factors (D) | ( D / $ ) |
|---|---|---|---|
| Model 1 | 97764.8 | 2.15 | $2.19916 \times 10^{-5}$ |
| Model 2 | 101639.9 | 2.06 | $2.02676 \times 10^{-5}$ |
| Model 3 | 103531.8 | 2.29 | $2.21188 \times 10^{-5}$ |
| Model 4 | 105098.3 | 2.71 | $2.57854 \times 10^{-5}$ |
| Model 5 | 106817.2 | 3.07 | $2.87407 \times 10^{-5}$ |
| Model 6 | 108851.1 | 3.24 | $2.97654 \times 10^{-5}$ |
| Model 7 | 110690 | 3.64 | $3.28846 \times 10^{-5}$ |
| Model 8 | 112987.9 | 3.93 | $3.47825 \times 10^{-5}$ |
| Model 9 | 114750.7 | 4.15 | $3.61654 \times 10^{-5}$ |

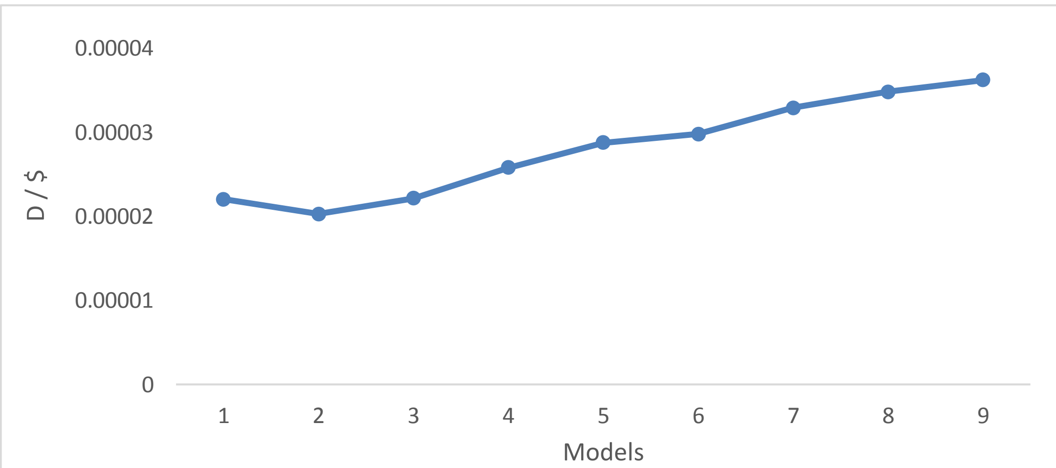



**Fig. 7.** 'Over-strength factor / total cost' values for each model.

## 4. Conclusions

Within the scope of the study, the models subjected to linear and nonlinear analyzes; base shear forces, over-strength factors, structural system quantities and rough costs were obtained. The base shear forces obtained in linear analyzes were obtained by using the Response Spectrum Method and the base shear forces obtained in the nonlinear analyzes were obtained by static pushover analysis. As shown in Table 5, the over-strength factors of the structures were obtained by the ratio of base shear forces obtained from nonlinear analysis to base shear forces obtained from linear analysis. According to TSC 2018, for structures with response modification coefficient R=8, the over-strength factor is D=3, for structures with response modification coefficient R=7, the over-strength factor is D=2.5 and for structures with response modification coefficient R=6, the over-strength factor is D=2.5. Models examined within the scope of this study; Model 1 with R=8 and Model 2 and Model 3 with R=7 remained below the over-strength factor values given in TSC 2018 and over-strength factors obtained for all other models were above the values placed in TSC 2018. It should not be understood that the structures do not provide the desired strength if the over-strength factor value is below the values specified by the regulations. The over-strength factor is a value obtained by ratio of yield strength to design strength of a building or structural element. It is understood that in all cases where this value is greater than 1, the structure can carry more loads than the design loads.

After the design and analysis procedures, in order to compare the costs of the structures, quantity calculations were made for the structural elements (concrete, rebar, and formwork). Rough costs of the structural systems of the models have been calculated by taking into consideration the current unit price values. As can be seen from Fig. 6, the cost for all buildings increased with an increase similar to the linearity with the increase in the ratio of the RC wall used. In particular, it is seen that the cost of formwork is higher than concrete and rebar. This is due to the fact that although the mold material is reusable up to a certain level of wear, the quantity of the mold is calculated for each floor without using this in consideration of the quantity calculations.

In the analysis, the rigidity of the structure increased as the RC wall ratio increased. As a result, the period of the models along the earthquake direction was reduced as expected and the seismic load affecting the structure was increased. However, as the stiffness of the building increased with the increasing RC wall ratio compared to the seismic force affecting the structure, the over-strength factor (D) also increased. Although the minimum D=2.5 given in TSC 2018, after Model 4, this value increases above 5. The model providing the results closest to the D value given in TSC 2018 was found to be Model 4 with a RC wall ratio of ‰1.4. Doğan (2019) found this value to be quite consistent with the optimum RC wall ratio of ‰1.

According to the results of the analysis, the increase in the weight of the structural systems of the models is due to the increase of the RC wall dimensions. With the increase of RC wall ratio, the shear strength of the structures increased approximately 6.5 times. Likewise, the increase in the structural system weight of the buildings by 5.6% increased the rough cost of construction by 17%. Accordingly, the weight of the structural system of the Model 1 (reference model with RC framed structural system) is 3205 tons, while the weight of the structural system of the model with the RC wall ratio of ‰3.2 (Model 9) has reached 3387 tons. As a result of this, it is seen that small increases in the dimensions of the vertical structural elements of the structure, significantly increase the strength of the structure against seismic effects.

Over-strength factors and rough construction costs of the structural systems are calculated by the analyses applied on the buildings. For the general purpose of the study, these two situations were handled together and an examination was made on the optimum design of the structures. If Table 6 and Fig. 7 are examined together, among the models designed and analyzed in accordance with the design principles specified in TSC 2018, the values obtained by the ratio of the over-strength factor value to the building cost of the Model 4 with the wall ratio ‰1.6, it was found that Model 4 provides the most efficient results for this study.

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Challenge Journal of Structural Mechanics

**Research Article**

# Evaluation of performance-based earthquake engineering in Yemen

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**ABSTRACT**

Building codes follow a common concept in designing buildings to achieve an acceptable seismic performance. The objective underlying the concept is to ensure that the buildings should be able to resist minor earthquake without damage, resist moderate earthquake with some non-structural damage, and resist major earthquakes without collapse, but some structural as well as non-structural damage. This study aims to evaluate the performance-based seismic to come up with necessary recommendations for both future practices, essential review, and restoration of existing structures in Yemen. To do this real case studies incorporated, and nonlinear pushover analysis is carried out. The analysis results presented and then assessed to find out the conformity with the required performance. The structural sections assumed at the beginning of the design, then the design repeated many times to achieve the selected performance criteria (the plastic hinge properties and the maximum displacement).

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## 1. Introduction

Earthquakes have long been feared as one of nature's most terrifying phenomena. Early in human history, the sudden shaking of the earth, the death, and destructions that resulted were mysterious and uncontrollable (FEMA 454, 2006).

It may be said that the occurrence of earthquakes is well understood and must be accepted as a natural environmental event. They represent one of the periodic adjustments that the earth makes in its evolution. Arriving without warning, the earthquake can, in a few seconds, create a level of death and destruction that can only be equaled by the most extreme weapons of war. This uncertainty, combined with the terrifying sensation of earth movement, creates a fundamental fear of earthquakes (Ishihara, 2003; FEMA, 2006).

Yemen is generally considered along with its history among countries which subjected from time to time to different natural disasters that comprised earthquakes, flowage, landslides, etc. Besides, many of its areas are located within those recognized with high volcanic activities, and; moreover, several activities are expected to hit back (Almunifi, 1995; Alyafei, 2007; Kulaib et al., 2008; Almunifi and Alameri, 2019).

The location of Yemen in the south of the Arabian plate makes it exposed to seismic attacks. That hazards due to the tectonic Situation of Arab Plate, which make Yemen close to seismic activities in the Red Sea and the Gulf of Aden. Fig. 1 shows the tectonic situation of Arab Plate which gives an illustration of seismic hazards in Yemen (Almunifi, 1995).

Seismic hazards of Yemen are not in the same level in all specialists' considerations (Almunifi, 1995; Alyafei, 2007; Kulaib et al., 2008; Almunifi and Alameri, 2019). Also, there is no formal standard data to specify the hazard magnitude in the different zones in the country. The Uniform Building Code UBC (1997) categorized the capital "Sana'a" in the seismic zone (Z=3) whereas some of the other searches conclude to that hazard in Yemen is characterized by low to moderate seismic activities (Alyafei, 2007; Kulaib et al., 2008).

Despite that seismic hazard in Yemen might not in highly intensive magnitude, the seismic risk is still present. However, Seismic risk is not a function of hazard only, it is based on the vulnerability of structures as well.

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So, although that some researchers conclude that Yemen considered in low to moderate earthquake hazards, and the majority of Yemeni regions might not expect to have a high intensive seismic attack. However, due to wrong construction practices and ignorance for earthquake resistant design of buildings; many of the existing buildings in Yemen might be vulnerable to future earthquakes. That means if the seismic hazard is probable, seismic vulnerability of structures should be considered (Almunifi, 1995; Aldafiry 2005; Alyafei, 2007; Kulaib et al., 2008).

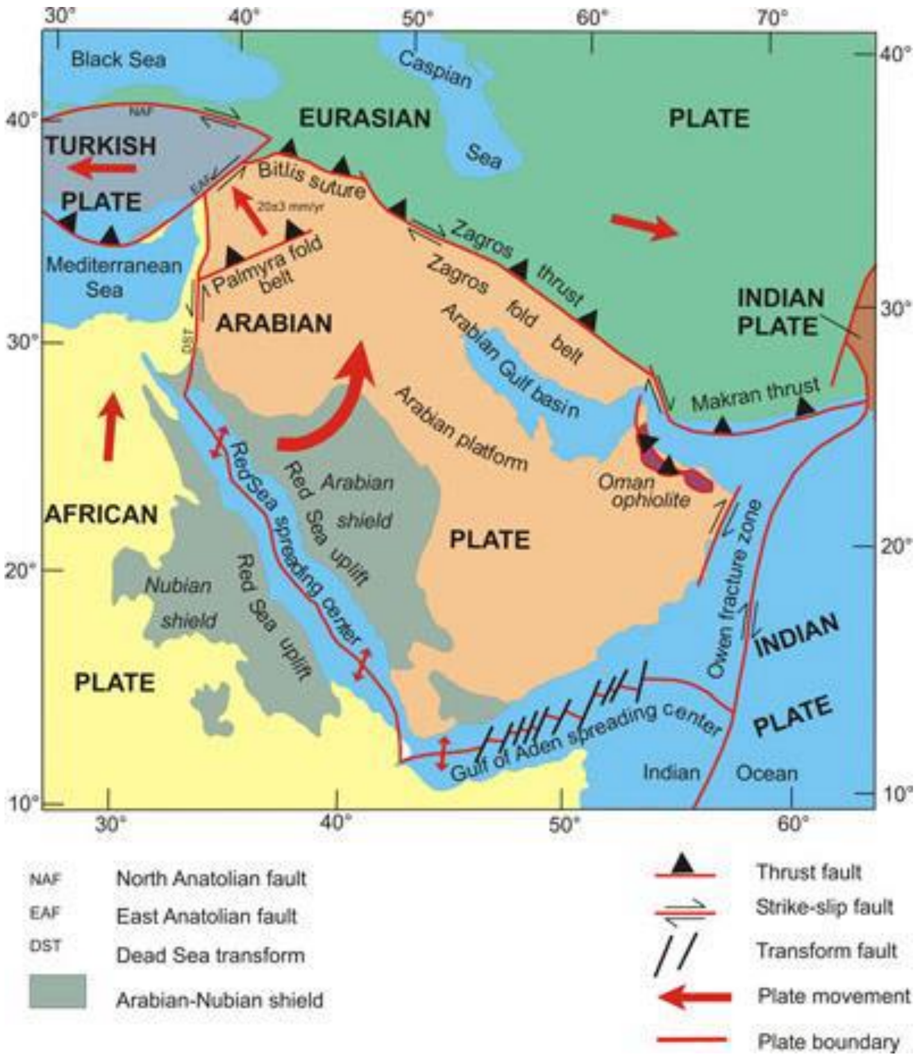


**Fig. 1.** Tectonic situation of Arab Plate.

Performance-based seismic design (PBSD) is a comparatively new concept that was at first developed to be used in predicting the upgrade strategy required for existing buildings. However, it's equally applicable to checking the design of new buildings and may be a paradigm shift in seismic engineering design moving away from the prescriptive code approach. It's presently an evolving methodology with the key difficulties being to reach globally acceptable precise definitions of performance objectives and to quantify performance levels (Manohar and Madhekar, 2015). Generally, in displacement-based or force-based methods of seismic design codes, it is presumed that the structure enters the inelastic phase to dissipate the seismic energy to bear the lateral seismic loads or to attain the performance objectives. In this case, the residual deformations due to inelastic behavior, which are considered as "damages", would depend on the number and layout of the seismic load resisting members and the magnitude of seismic load. These damages would remain in structure in the forms of story drifts or members' deformations (Shoeibi et al., 2017). A lot of studies recommended the performance-based seismic design method to evaluate the damage state of the building. Zeris and Repapis (2018) studied the seismic performance of existing RC buildings designed to different codes and concluded that buildings of the 90s, designed to modern codes exhibit an exceptionally good performance. Ashkezari (2018) proposed a performance based strategy for design of steel moment resisting frames under far range blast loads. For this purpose, he presented an algorithm to calculate the capacity modification factors of frame members in order to simplify design of structures subjected to blast loading. The method provides a simplified design procedure in which the linear dynamic analysis is preformed, instead of the time-consuming nonlinear dynamic analysis. Turker and

Gungor (2018) studied the Seismic performance of low and medium-rise RC buildings with wide-beam and ribbed-slab, The results indicated that the predicted seismic performances were achieved for the low-rise (4-story) building with the high ductility requirements and addition of sufficient amount of shear-walls to the system proved to be efficient way of providing the target performance of structure. Inel and Meral (2016) evaluated seismic performance of existing low and mid-rise reinforced concrete buildings by comparing their displacement capacities and displacement demands under selected ground motions experienced in Turkey as well as demand spectrum provided in 2007 Turkish Earthquake. The results show that the significant number of pre-modern code 4- and 7-story buildings exceeds LS performance level while the modern code 4- and 7-story buildings have better performances. The findings obviously indicate the existence of destructive earthquakes especially for 4- and 7-story buildings. Significant improvements in the performance of the buildings per modern code are also obvious in the study. Almost one third of pre-modern code buildings is exceeding LS level during records in the past earthquakes. Jiang et al. (2017) studied the Seismic performance of high-rise buildings with energy-dissipation outriggers. Two high-rise structures, one with conventional outriggers, the other with energy-dissipation outriggers, were designed. The results show that compared to the ordinary structure, the seismic performance of the new structure is improved significantly. Gorji and Cheng (2017) investigated the plastic behavior and mechanisms of steel plate shear walls with outriggers (SPSW-O), it was shown that such systems are considerably effective in improving the flexural stiffness of conventional SPSWs. Shoeibi et al. (2017) studied the performance for structures with structural fuse system Analyses results showed that in moderate earthquake hazard level, only fuse members yielded and other structural members remained elastic.

Considering these facts, it is imperative to seismically evaluate the existing building and with the present-day knowledge to avoid the major destruction in future earthquakes. This paper aims to evaluate the performance-based seismic in to come up with necessary recommendations for both future practice, essential review, and restoration of existing structures for any underestimated of such obligations.

## 2. Case Studies

In order to study the local practice of earthquake engineering in Sana'a, the selected case studies are real reinforcement concrete buildings situated in Sana'a city. Buildings of case studies are 10, 16, 20 story. The first case study building (10 stories) has a frame structure system, but the others have a shear walls system.

### *2.1. First case study building (10 story building)*

Figs. 2 and 3 show the elevation and some plans of the first case study building (10 story) used as real case study for this investigation. However, the building utilizes frame structural system in both directions to resist the gravity forces. The cubic strength of concrete is *Fc*'= 300 kg/cm$^2$, and *Fy*=4140 kg/cm$^2$ for concrete reinforcement steel. Columns-beam joints analyzed as rigid joints with full transformation of the moments and shear, structural drawings confirmed that in there details. This arrangement may be taken to decrease any undersigned lateral forces, but this manner lead to decrease the designed moments in the middle of beams and increase the moments in joints, and if the joints are not executed well, high moment will developed in the middle which not include in design. Such cases may lead to design sections less than the proper sections, which make serious dangerous to safety under gravity loads (Elghazouli, 2009).

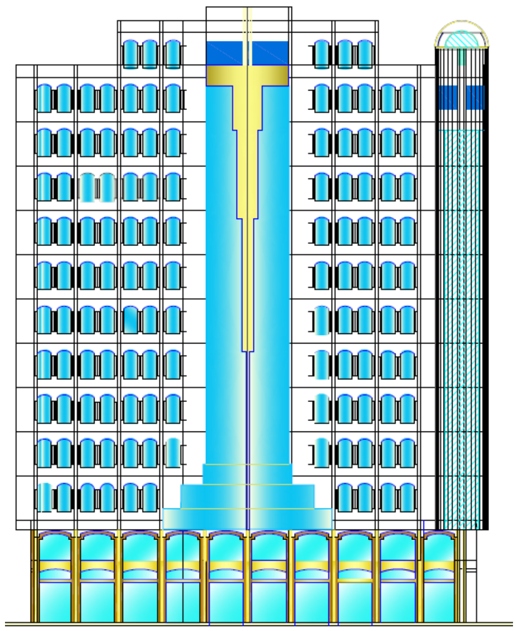

**Fig. 2.** Elevation of the first case study building.

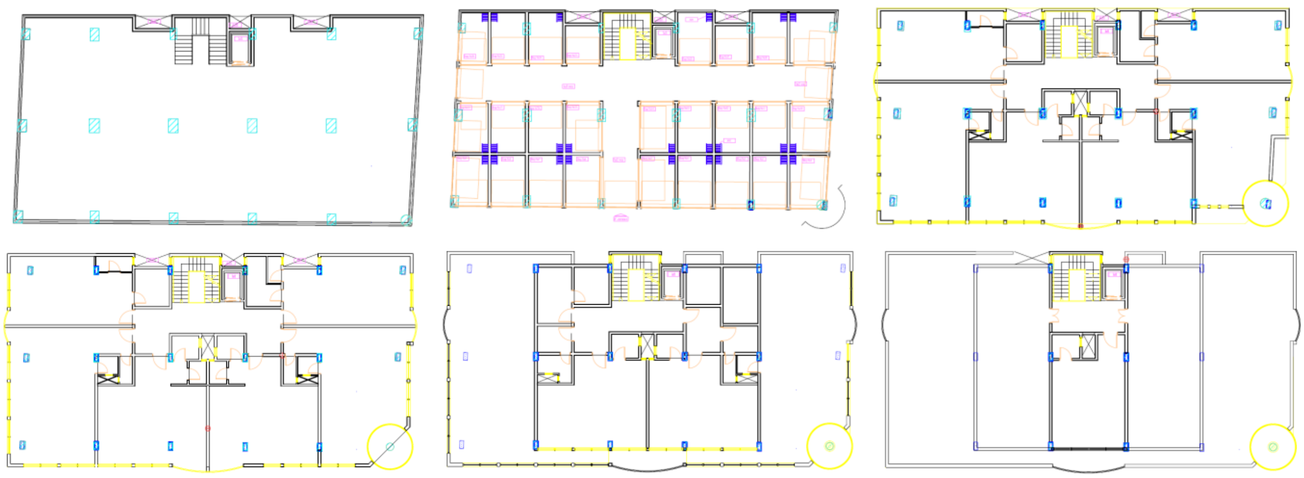

**Fig. 3.** Some plans of the first case study building.

### *2.2. Second case study building (16 story building)*

Figs. 4 and 5 show the plan and elevation of the second case study building (16 story) used; it is situated in Sana'a and it is under construction. The building utilizes a structural system with a dual structural system to resist the lateral forces. Systems are consisting of shear walls and moment-resisting frames in both directions. Lateral forces designed according to the basis of the 1997 UBC Zone 2 (regions of middle seismicity) requirements. The cubic strength of concrete is $Fc'$= 400 kg/cm$^2$, and $Fy$=4200 kg/cm$^2$ for concrete reinforcement steel.

**Fig. 4.** Some elevations of the second case study building.

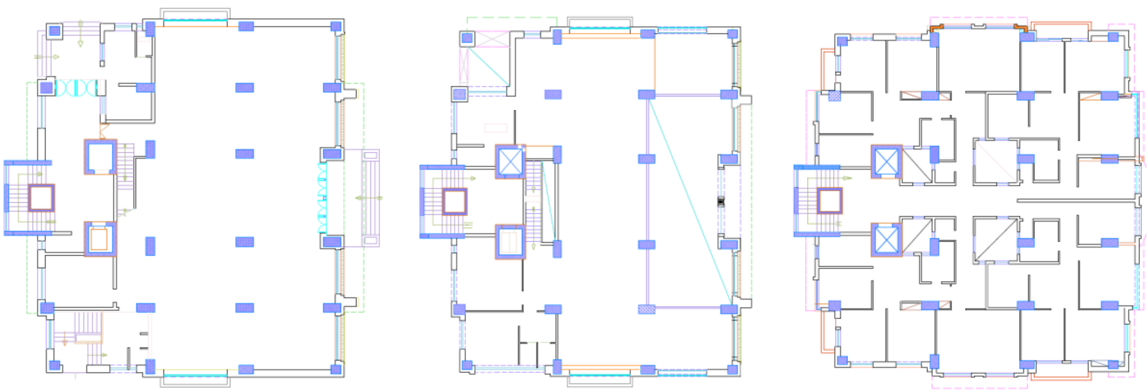

**Fig. 5.** (continued).

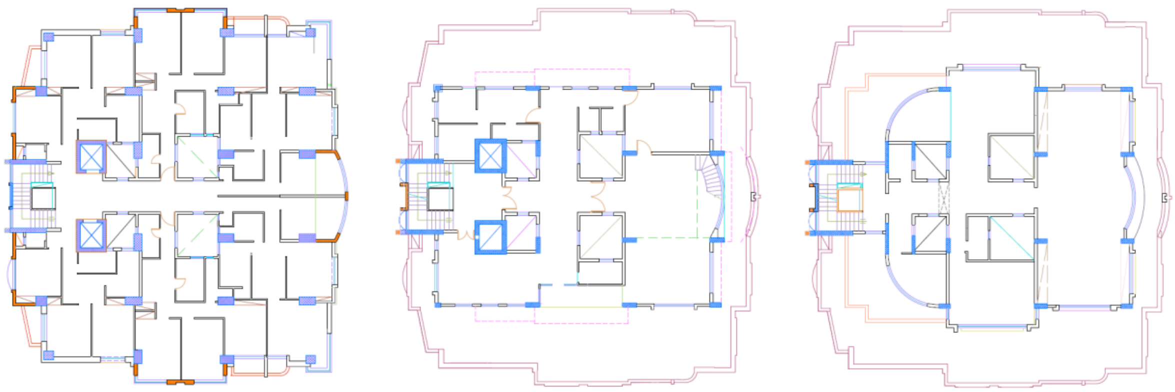

**Fig. 5.** Some plans of the second case study building.

### *2.3. Third case study building (20 story building)*

Figs. 6 and 7 show the plan and elevation of the third case study building (20 story) used. It is situated in Sana'a and it is in design stage. The building utilizes a dual structural system to resist the lateral forces (shear walls and moment-resisting frames in both directions). Lateral forces designed according to the basis of the UBC (1997) Zone 2 requirements.

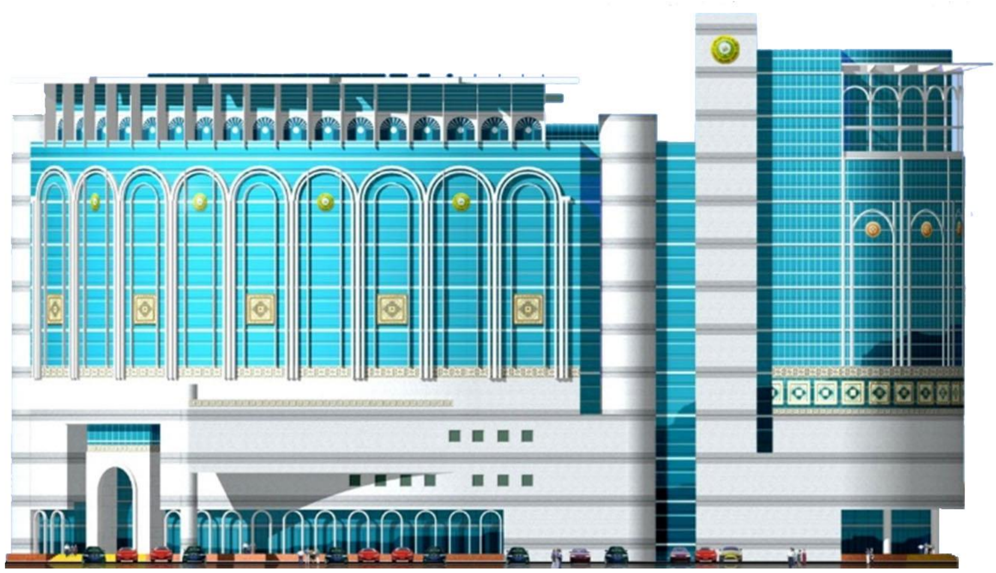

**Fig. 6.** Elevation of the third case study building.

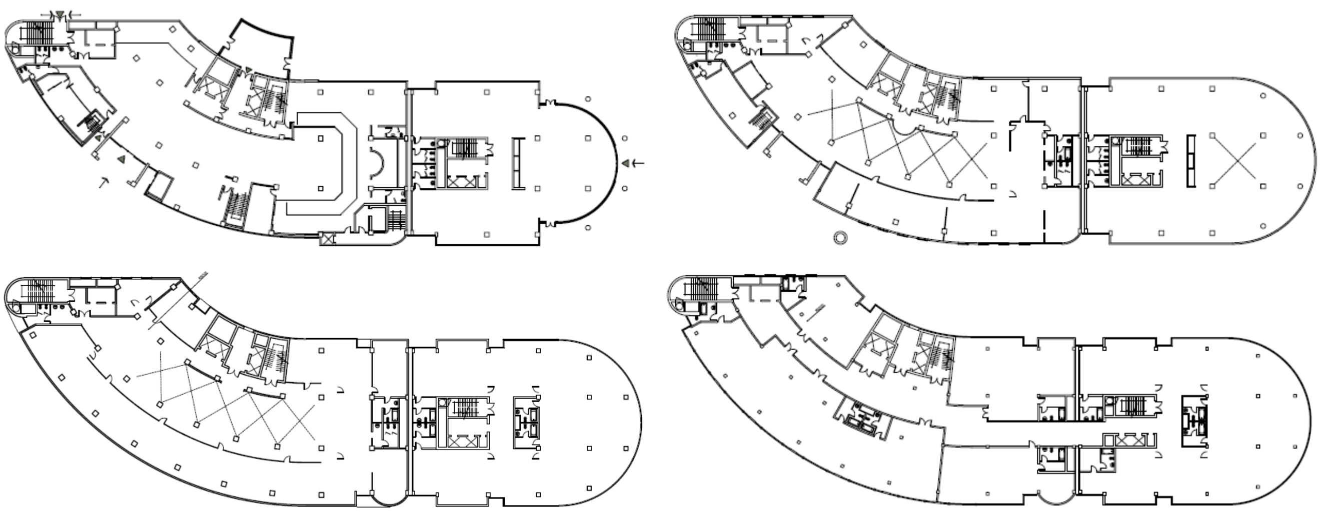

**Fig. 7.** Some plans of the third case study building.

## 3. Non-Linear Static Pushover Analysis

Buildings which used as case studies in this paper are investigated by nonlinear static pushover analysis. The analysis carried by default ETABS nonlinear frame hinge properties. Models used the analysis based to their existing design models with their related specifications, geometric, loads, etc.

### *3.1. First case study building (10 story building)*

Fig. 8 shows the model of the building which developed by ETABS. Earthquake loads has applied in all sides as UBC97 requirements (Z=0.2, Soil Type=SC, R=8.5, I=1). From the static linear analysis results, base shear is $V$=1378.07 kN. Fig. 9 shows pushover analysis results, and relation between the base shear and monitored displacement was drawn.

Results show that pushover accelerations leads to generate hinges in structure which work towards to lose the stability of the structure (Fig. 10).

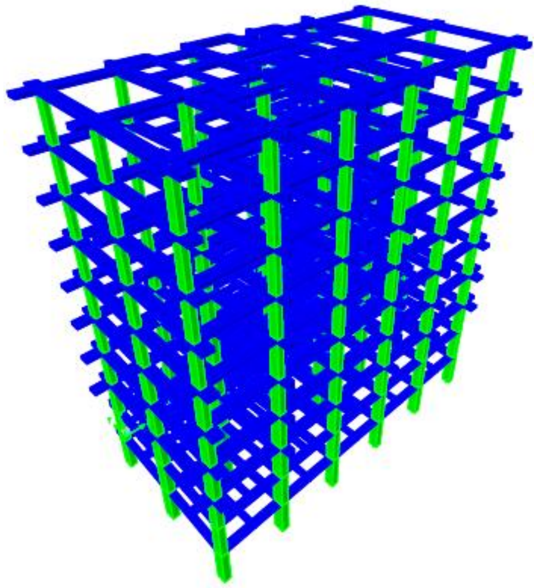

**Fig. 8.** ETABS model of first case study building.

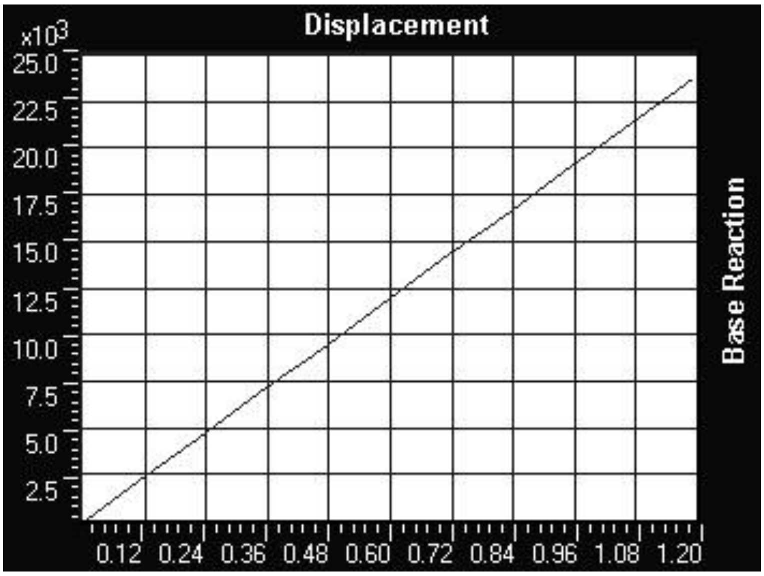


**Fig. 9.** ETABS output of pushover curve.

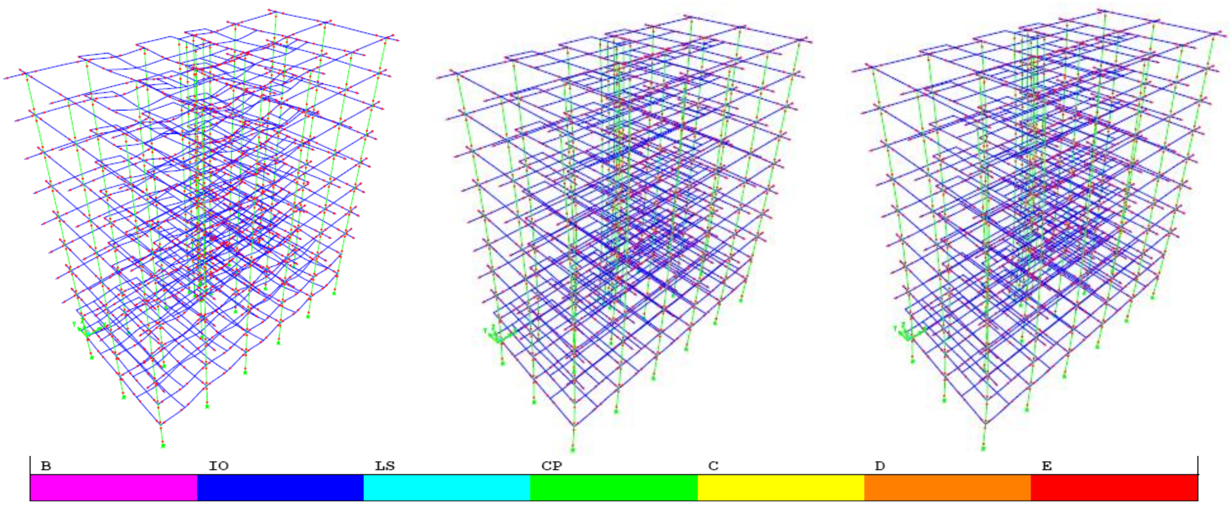


**Fig. 10.** Some steps of pushover analysis and hinges formation (ETABS output).

From intersection of response spectrum and capacity spectrum (Fig. 11), it is deem that the demand curve tends to intersect the capacity curve in elastic response. ETABS identify the performance point indicated to base shear of $V$=6113.784 kN, and target displacement value $D$=0.303 m.

The ratio between base shear of performance point and related shear in elastic is 3 times which suggest acceptable structural behavior. But ETABS identify that the maximum inter-story drift ratio is 2.6%, which suggests "Collapse" performance level according to SEAOC 2000, and "Collapse prevention (S-5)" according to FEMA273 limits.

That result may understandable, because that earthquake requirements not taken in consideration in the design stage. Fig. 12 shows the maximum inter-story drift for case study 1.

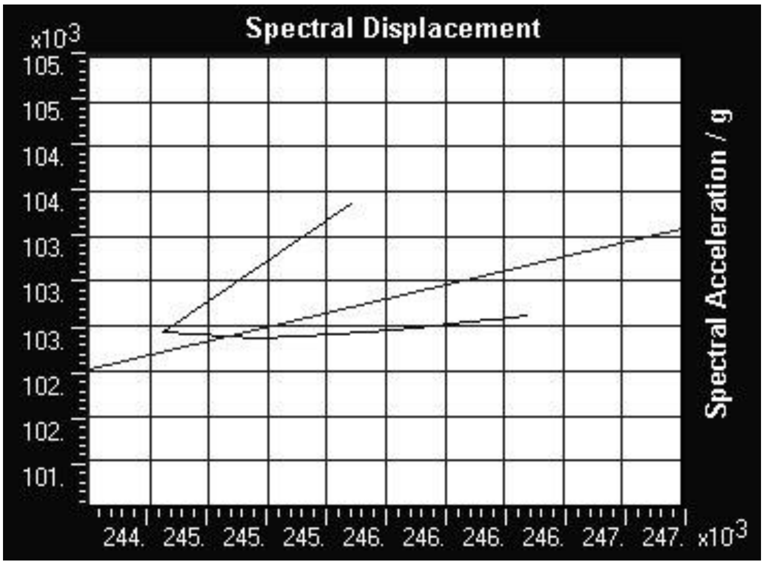


**Fig. 11.** Capacity spectrum and demand spectrum curves (ETABS output).

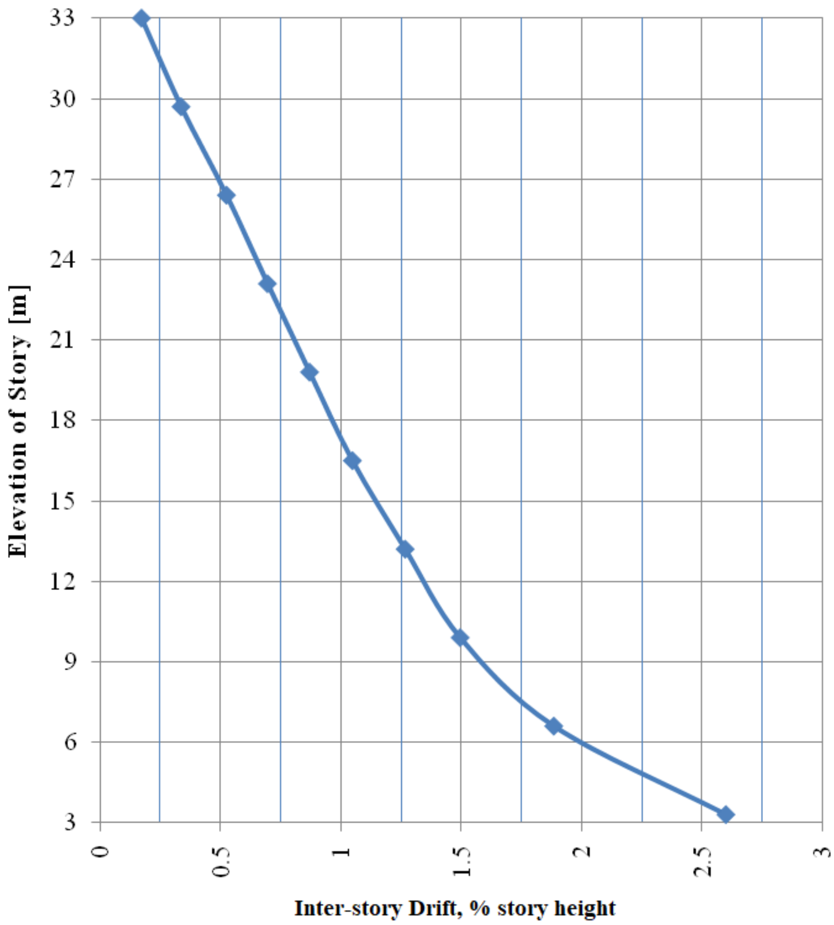


**Fig. 12.** Maximum inter-story drift.

### 3.2. Second case study building (16 story building)

Fig. 13 shows the model of the building which developed by ETABS. Earthquake loads has applied in all sides as UBC97 requirements (Z=0.2, Soil Type=SC, R=8.5, I=1). From the static linear analysis results, base shear is $V$=4607.26 kN. Fig. 14 shows pushover analysis results, and relation between the base shear and monitored displacement was drawn.

Results show that pushover accelerations lead to generate hinges in structure which work towards to lose the stability of the structure (Fig. 15).

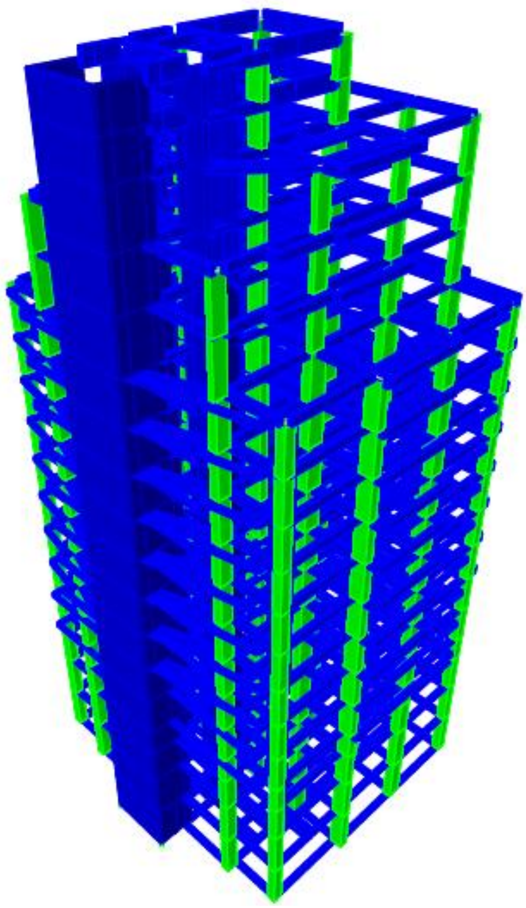

**Fig. 13.** ETABS model of second case study building.

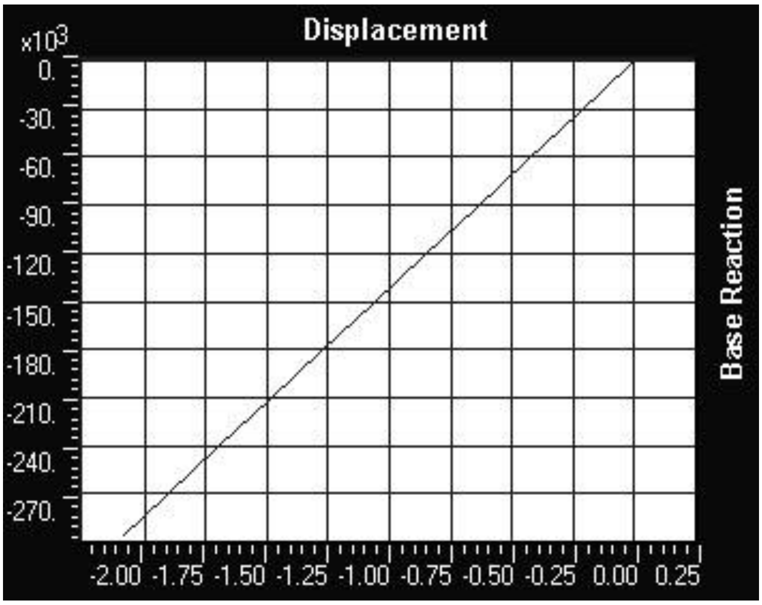


**Fig. 14.** ETABS output of pushover curve.

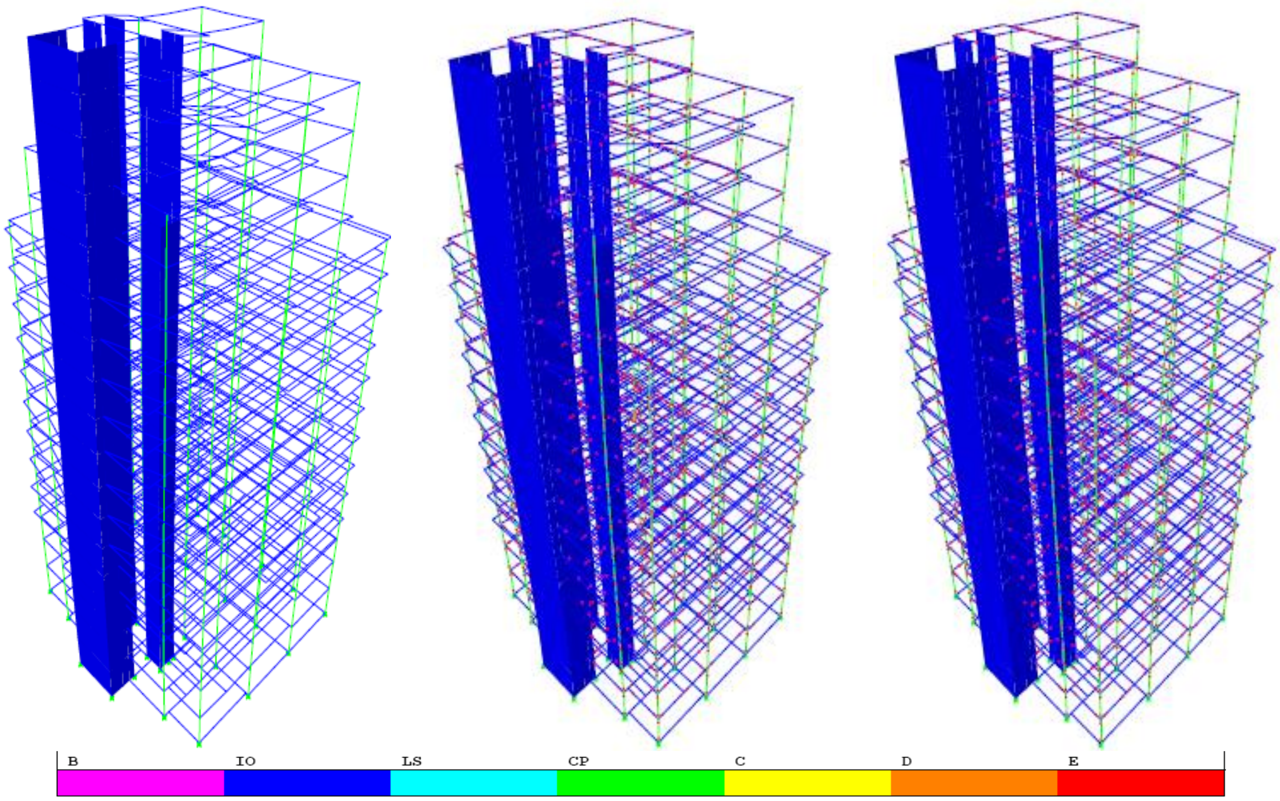


**Fig. 15.** Some steps of pushover analysis and hinges formation (ETABS output).

From intersection of response spectrum and capacity spectrum, it is deem that the demand curve tends to intersect the capacity curve in elastic response (Fig. 16). ETABS identify the performance point indicated to base shear of $V$=23,295.01 kN, and target displacement value $D$=0.165 m.

The ratio between base shear of performance point and related shear in elastic is 5.1 times which suggest acceptable structural behavior. ETABS identify that the maximum inter-story drift ratio is 1.13%, which suggests "Life Safe" performance level according to SEAOC 2000, and "Life Safety (S-3)"according to FEMA273 limits. That result may reasoned that earthquake loads are taken in consideration in the design stage. Fig. 17 shows the maximum inter-story drift of case study 2.

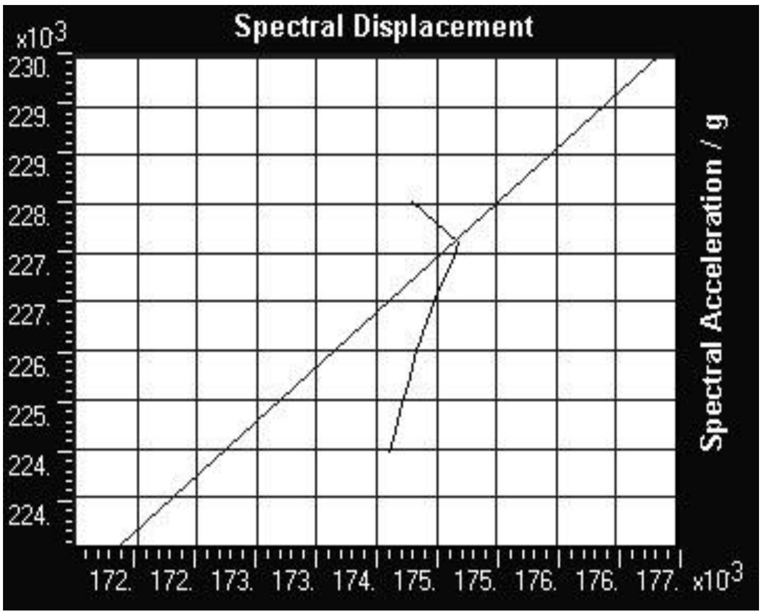


**Fig. 16.** Capacity spectrum and demand spectrum curves (ETABS output).

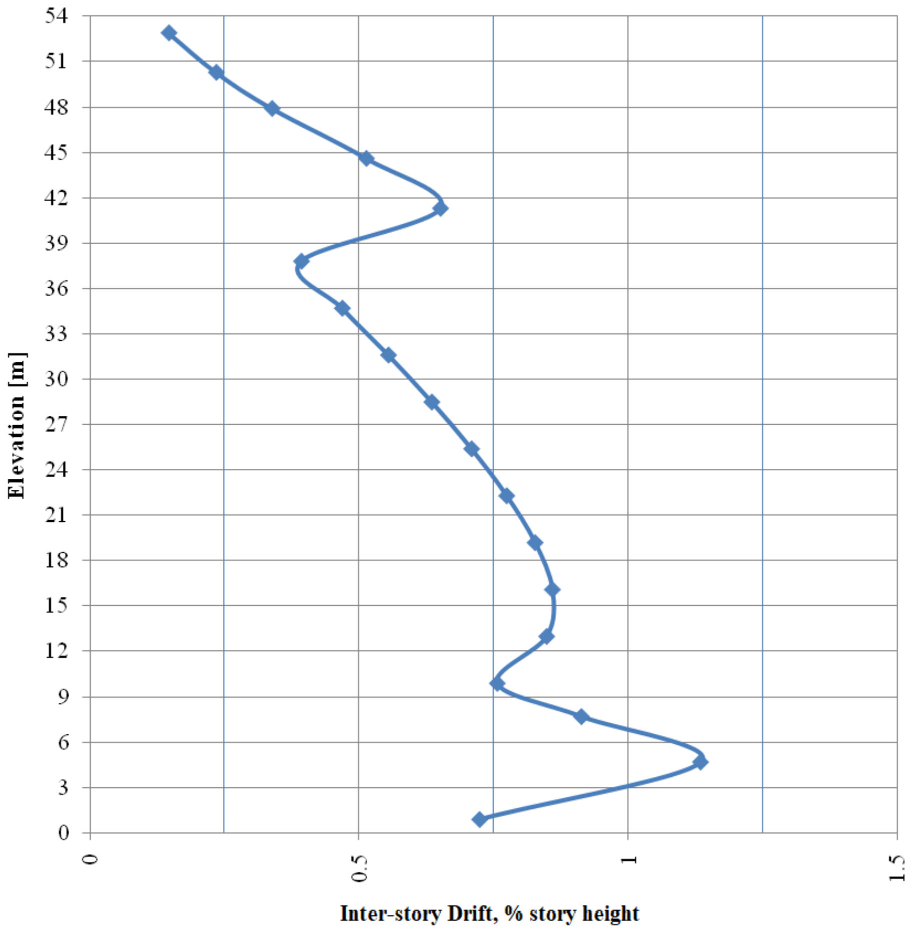


**Fig. 17.** Maximum inter-story drift.

### *3.3. Third case study building (20 story building)*

Fig. 18 shows the model of the building which developed by ETABS. Earthquake loads has applied in all sides as UBC97 requirements (Z=0.2, Soil Type=SC, R=8.5, I=1). From the static linear analysis results, base shear is *V*=7594.28 kN. ETABS results develop pushover curves show the resultant base shear vs. monitored displacement as Fig. 19.

Results show that pushover accelerations leads to generate hinges in structure which work towards to lose the stability of the structure (Fig. 20).

From intersection of response spectrum and capacity spectrum, it is deem that the demand curve tends to intersect the capacity curve in elastic response (Fig. 21). ETABS identify the performance point indicated to base shear of *V*=25,737.34 kN, and target displacement value *D*=0.31 m.

The ratio between base shear of performance point and related shear in elastic is 3.4 times which suggest acceptable structural behavior. The maximum inter-story drift ratio is 0.61%, which suggests "Life Safe" performance level according to SEAOC 2000, and "Immediate Occupancy (S-1)" according to FEMA273 limits. That result may reason that earthquake requirement in regularity and loads are taken in consideration in the design stage. Fig. 22 shows the maximum inter-story drift of case study 2.

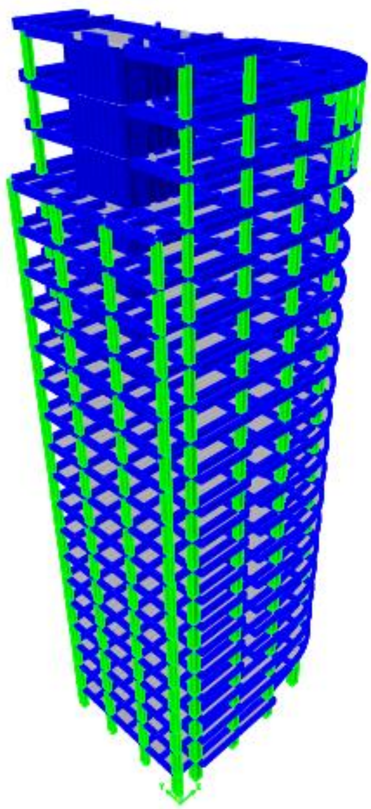

**Fig. 18.** ETABS model of third case study building.

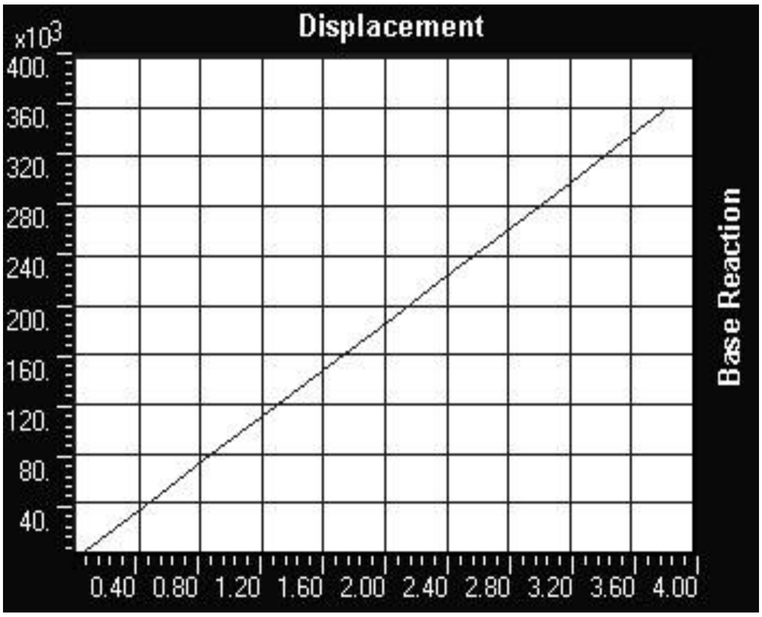


**Fig. 19.** ETABS output of pushover curve.

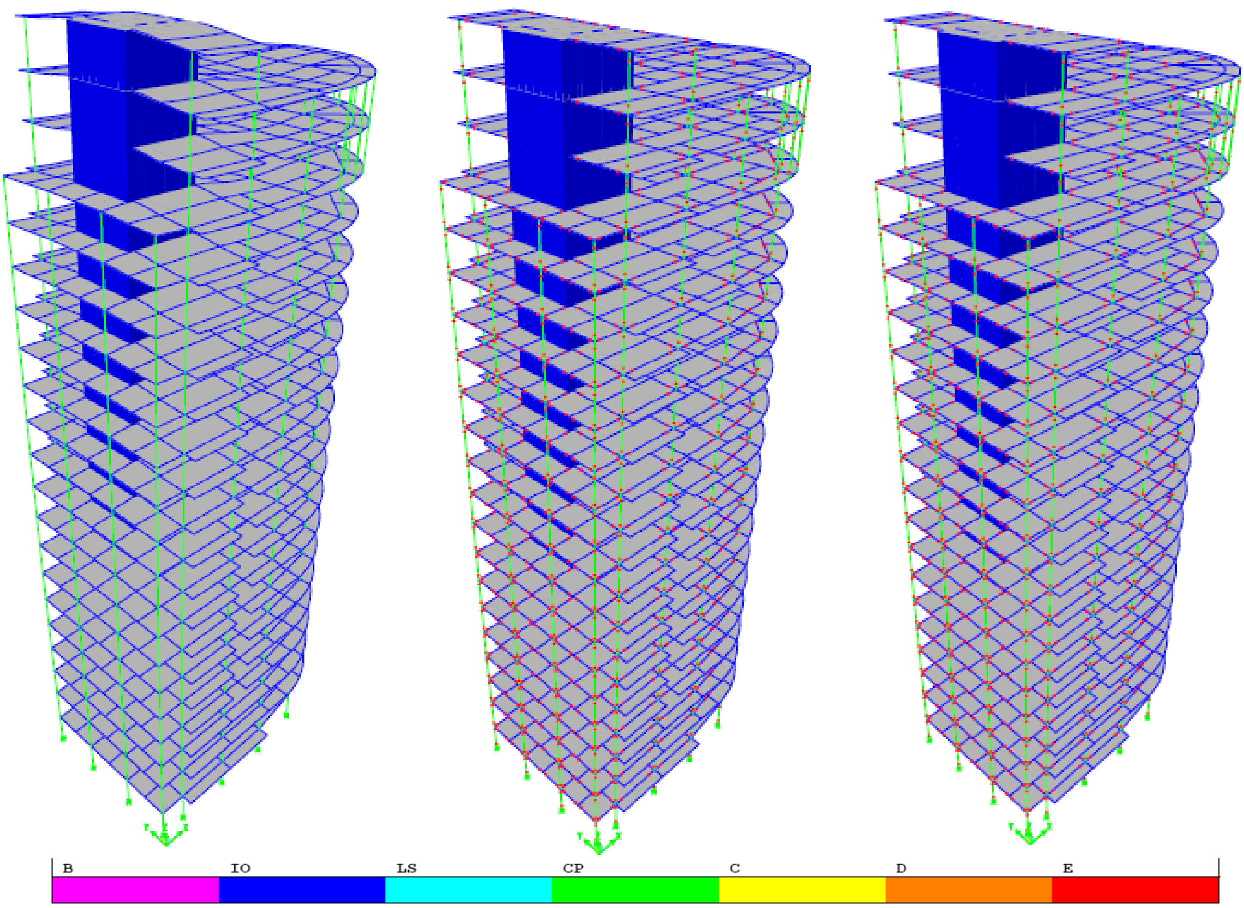


**Fig. 20.** Some steps of pushover analysis and hinges formation (ETABS output).

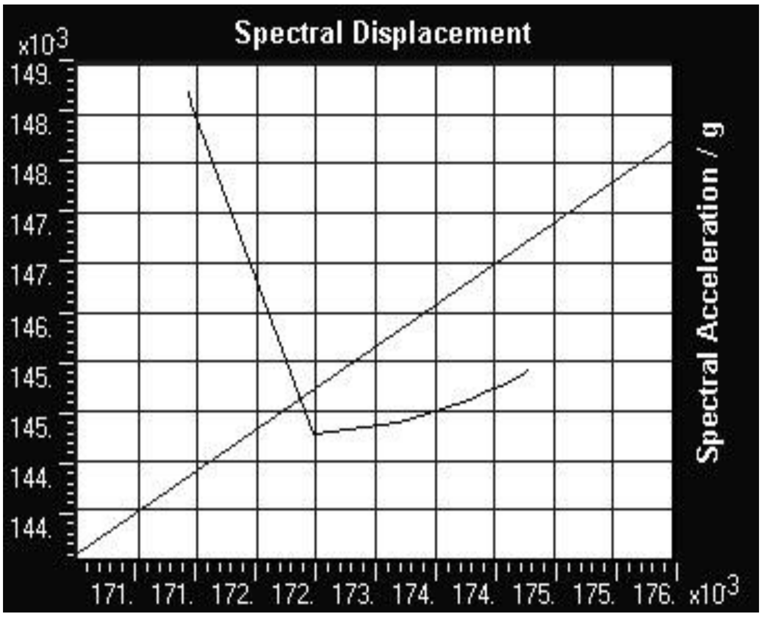


**Fig. 21.** Capacity spectrum and demand spectrum curves (ETABS output).

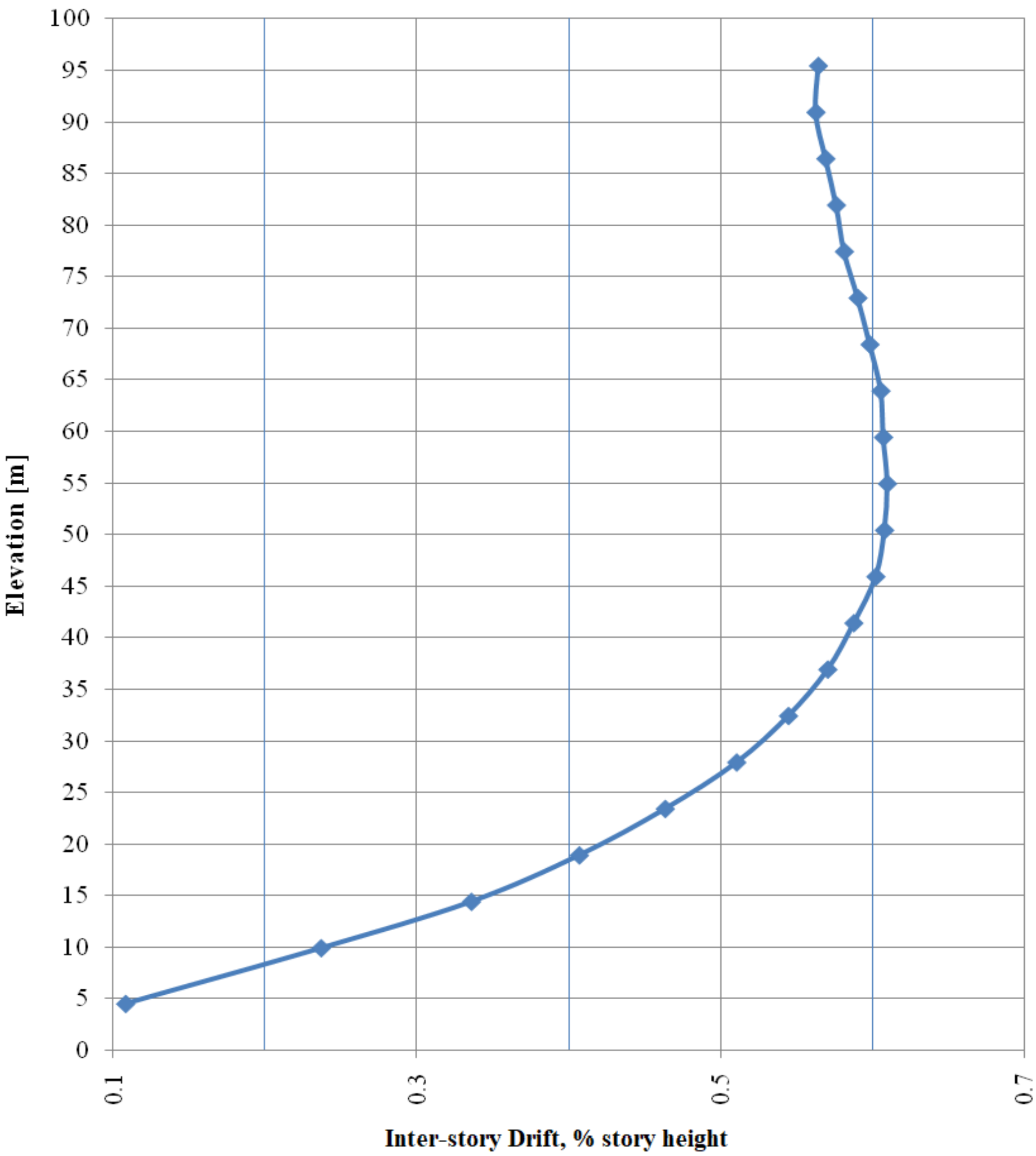


**Fig. 22.** Maximum inter-story drift.

## 4. Conclusions

Since Yemen is not far from earthquakes hazards, evaluation of the vulnerability of existing buildings is among essential priorities. Lack of presence (or availability) of local building code or such obligation may lead to un-implementation of the requirements of seismic buildings resistant which, in turn, may lead to high damage caused by any anticipated earthquake. Therefore, evaluation of performance-based seismic design in Yemen carried out. Evaluation of performance-based seismic design incorporated some case studies in Sana'a, Yemen. The study concerned with the conceptual and analytical three-dimensional modeling of buildings. The evaluation carried utilizing pushover analysis in compliance with the structural requirements for earthquake design. These are the conclusion drawn from this investigation:

- Seismic evaluation by nonlinear static pushover analysis is a useful common procedure in the world for evaluations, rehabilitation, and design stages;
- Buildings which not consider seismic requirements in codes at the design stage, have low performance and have more vulnerability against earthquakes, it may run between "near collapse" and "collapse prevention" to "collapse" performance level category.
- Buildings that consider seismic requirements in codes at the design stage have high performance, have less vulnerability against earthquakes, and may run between "operational" to "life safe" performance level category.
- There is a gap between architectural design and the requirement of earthquake engineering design in buildings in Yemen. However, structural decisions are -commonly- considered the architectural purposes against the seismic requirement.

For future work, the performance-based seismic design can be modeled for other systems such as steel structures. Also, the technics that increase the performance of the structures can be studied.

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Challenge Journal of Structural Mechanics

Research Article

# Investigating the synergy between lean construction practices and post disaster management processes

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**ABSTRACT**

Lean aims to maximize value while minimizing waste. Lean practices are likely to reduce the number potential hazards and errors. The use of Lean practices in construction is essential to experience less hazards. Benefitting from Lean practices has gained much attention in the last decade. Especially, the destroying effect of hazards and accidents is of utmost importance in terms of seeking for better strategies. Within this context, Lean practices offer a wide variety of advantages and provide means for achieving greater success in projects. This study investigates the use of Lean practices in post disaster management. Since post disaster management includes the activities to help community in rebuilding, Lean tools and techniques might be employed to better handle post disaster management processes. The study also scrutinizes the integration of Lean practices with the post disaster processes and encourages the community to compete against the destroying effect of disasters thanks to using Lean tools and techniques. The main contribution of this study is that it introduces Lean practices to be used in the post disaster management processes, which might potentially remove safety concerns in construction sites up to a great extent.

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## 1. Introduction

Lean thinking is a term, which defines the conscious use of resources in an effective manner. One of the most important objectives of lean thinking is to minimize waste while maximizing value to the costumer. Lean has first been appeared in the manufacturing industry. Lean Production System first emerged with the Toyota Company to offer high quality, wide variety and low-cost production to best address customer desires (Lean Enterprise Institute, 2019). After the considerable success of lean production in the automotive industry, the system was introduced to other industries.

Construction industry produces waste more than the other industries in the entire world (Meadows, 2011; Hu, 2011; Ajayi et al., 2016). Research studies implied different waste types in construction. For example, Bossink and Brouwers (1996) identified six sources of waste in construction namely the design, procurement, material, handling, operation, and residual. In another study, Garas et al. (2001) introduced two main waste types as; (1) time wastes such as waiting periods, stoppages, clarifications, variations in information, rework, ineffective work, interaction between various specialists, delays in plan activities, and abnormal wear of equipment, and (2) material wastes such as over ordering, overproduction, wrong storage, wrong handling, manufacturing defects, theft or vandalism. Lean construction is therefore an effective means of production management for the project delivery, which allows designing and building capital facilities. Lean Construction is even beyond the lean production system in that it adopts the principle of minimizing waste and maximizing value while improving the total project performance in accordance with the customer expectations (Lean Construction Institute, 2019). The need behind the lean construction comes from the failure of mass production in the construction industry. Due to the changing needs of the customer, who desire variety, lean construction is essential to provide this variety.

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To minimize waste and maximize value, researchers have previously focused on several different lean construction methods. For example, Alshayeb (2011) clearly implied that modular construction is very effective reducing the waste and bringing resource efficiency. In his study, it was also demonstrated that modular is reusable, which evidences the essential function of modular construction. In Khanh and Kim's (2014) study, it was indicated that there are several waste factors in high-rise building projects and the determination of those waste factors is essential. Therefore, lean construction was proposed an opportunity for estimating the impacts of waste on overall project performance (Khadem et al., 2008). Unfortunately, the functional role of lean concepts in the construction industry has not been well understood yet by the construction professionals. Hence, a clarified roadmap, generic approach or introduction of a new concept is very important in order for the construction professionals to conceive the essential role of lean construction. On the other hand, application of lean technology into the construction industry provides a tremendous opportunity for the reduction of waste and increase in production (Marhani et al., 2013). The reduction of non-value adding activities has a significant contribution to the construction productivity improvement (Zhao and Chua, 2003; Han et al., 2011).

Disasters have devastating effects and result in serious defects. Hence, managing processes after disasters is of utmost importance in terms of facilitating the rebuilding activities. Especially, structural deformations and structurally damaged buildings after disasters put a high risk for the public health. To prevent, more innovative solutions and new techniques must be adopted. Within this respect, Lean practices shall be utilized to provide innovative solutions, prevent hazards or reduce the likelihood of occurrence of hazards. Several studies have already proven that there is a strong link between Lean and safer practices revealing that going leaner leads to experiencing less hazards (Mitropoulos et al., 2007; Bashir et al., 2011; Dehdasht et al., 2018). This study aims to put Lean practices forefront of disaster management strategy.

In addition to waste identification and waste reduction, this study aims to indicate the number of losses that the construction industry experiences every year. The studies have not yet proven an effective solution to overcome this barrier and successfully manage post disaster processes. Hence, the motivation of this study arose from this need to take the attraction towards the warning number of hazards and accidents in the industry and enlighten the need towards more innovative solutions. In this respect, the significance of this study is to motivate construction practitioners adopt Lean practices in their operations and benefit from them in post disaster management processes so as to limit or prevent losses.

## 2. Lean Construction

Lean construction is defined as "a clear set of objectives for the delivery process, aimed at maximizing performance for the customer at the project level, concurrent design of product and process, and the application of production control throughout the life of the product from design to delivery" (Howell, 1999). Lean approach aims to maximize value to customer while minimizing waste. Hence, the main purpose of going Lean is to come up with more value for customer by using fewer resources. Going Lean is needed for the defective processes in mass production and craft production. Lean production was first introduced by Taiichi Ohno -a pioneer of Lean production- in the manufacturing industry in 1950s. Then, Lean was firstly used as a term by Krafcik in 1988. Toyota was the first manufacturing company that applied Lean in production and Lean production helped Toyota increase its profits with an upstream trend. Lean has started to gain attention in other major industries in time. Ballard and Howell (1998) first used the term "Lean Construction" to eliminate waste and increase productivity in construction projects. Lean construction has been implemented in several major projects to increase productivity and efficiency.

The efficiency of Lean practices for higher customer satisfaction and enhanced project performance was previously implied by several studies (Horman and Kenley, 1996; Khadem et al., 2008). Several studies have investigated lean and lean in construction. For example, Sacks and Goldin (2007) developed a lean management model for high-rise apartment buildings by adopting lean principles (i.e., pull scheduling, reduced batch sizes, and a degree of multiskilling). The model intended to provide customized apartments, improved cash flow, and reduced apartment delivery cycle times. Aziz and Hafez (2013) concluded that lean projects are safer, easier to manage, completed sooner, cost effective, and are of better quality by referring to the impact of lean in minimizing waste in construction. Boyce et al. (2012) investigated the aspects of lean thinking and concluded that it helps to improve design phase of complex projects by emphasizing the essential function of collaborative planning process in highway design. Similarly, El-Reifi (2013) implied the positive impact of lean thinking adopted by design team in achieving higher customer satisfaction. Emuze and Smallwood (2013) conducted research on the interaction of lean, health and safety, and sustainability. Khanh and Kim (2014) emphasized the importance of identifying waste types in mid/high-rise building construction.

A major portion of studies in Lean has focused on identification of factors leading to a successful Lean implementation process. For example, the role of communication and management support regarding a successful Lean implementation was highlighted by Worley and Doolen's (2006) study. It was further studied that management commitment, employee autonomy, and transparent information flow are main Lean targets for a successful Lean management scheme (Scherrer-Rathje et al., 2009). Lean implementation was also mentioned to result in reduced lead time, work-in-process inventory, and manpower requirement (Singh et al., 2010). As previously implied in several studies, Lean approach is essential to experience higher productivity and efficiency in processes.

## 3. Post Disaster Management

Post disaster recovery is critical in terms of managing the rebuilding process. To facilitate, sound techniques and rescue technologies have to be put in place. This requires the use of innovative methods to help rebuild community leaders and smoothen the devastating effects in terms of fast recovery.

Even though pre disaster planning is important in not experiencing the losses, post disaster management is even more critical for the civil engineering. Especially, post-earthquake disaster management and structural assessment constitutes an important place in Turkish construction industry considering the earthquake prone zones in the country. Therefore, a well set action plan and preparedness for disasters are essential. Prieto and Whitaker (2011) indicated that post disaster environment brings the need for changing construction and engineering requirements as well as economic and political framework. This requires a deep understanding of how a framework might be designed to handle the scenarios for post disaster environments. Gandage and Ranadive (2008) highlights the role of civil engineers in post disaster management. In their study, they identify that planning, setting up and maintenance of emergency relief camps are among the main responsibilities of civil engineers in disaster management.

Fig. 1 presents the traditional disaster management cycle. It shows the phases in both pre and post disaster. According to Fig. 1, it is observed that preparedness, mitigation, and prevention are the phases of pre disaster, whereas response, recovery, and reconstruction are the phases of post disaster.

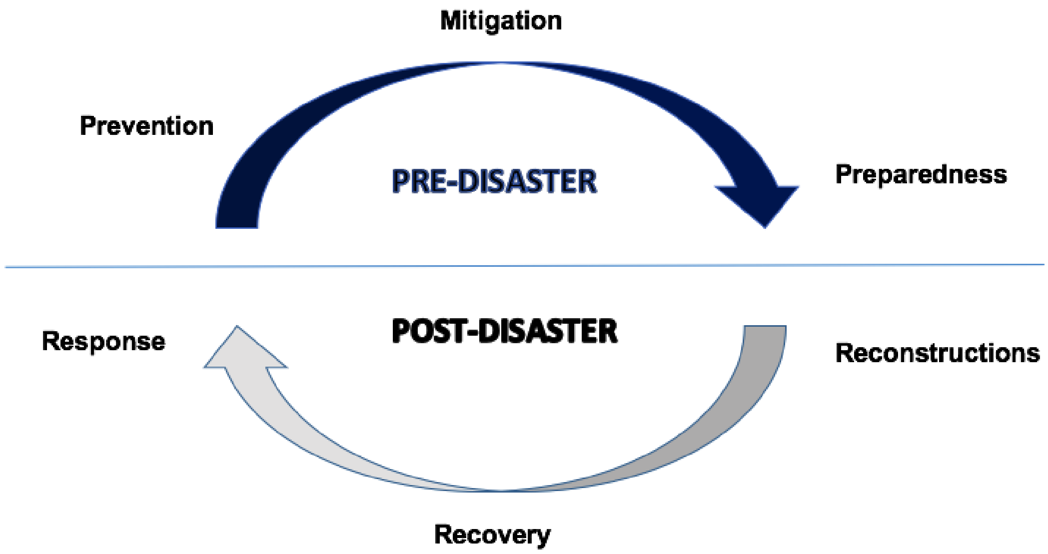


**Fig. 1.** Traditional disaster management cycle.

In this study, post disaster phases are investigated and Lean tools and techniques which might be used in those phases are identified accordingly.

### *3.1. Response*

A well set emergency plan is developed at this phase of disaster management. Response actions are developed to overcome the impacts of economic losses and suffering. This phase encompasses the mobilization of emergency services and help first responders get in the disaster area as quickly as possible. The actions taken in the response phase include the activation of emergency operations centers, evacuation of populations at risk, opening of shelters, and provision of medical care and emergency rescue.

Response either starts immediately when an emergency case is on or right after an event occurs. Hence, response phase rather includes the short term plans regarding the execution of emergency operations plan and incident mitigation activities aiming to limit life losses, property damage, and unfavorable consequences. Some main response phase actions are listed but not limited to the following:

- Following well-set emergency procedures for making the alarm active, evacuating people, and having the disaster site safe
- Conducting observations regarding the source of the threat
- Reaching the disaster response team leader and get the trained salvage personnel
- Conducting a preliminary assessment of the disaster site to determine the extent of damage, supplies, services, and the equipment needed.
- Stabilizing the environment for surveillance and testing processes, isolation, or quarantine.
- Ensuring access to critical services such as law enforcement and public works.

The very first response task includes the assessment of the situation. The government is mainly the responsible party for the continuity of situation assessment for protecting the citizens and the property in the community. Therefore, government officials and responder teams should work collaboratively for the rapid assessment of the local case (IFLA, 2019).

### *3.2. Recovery*

The activities in the recovery phases cover actions for getting the community back to normal conditions. These activities mainly consist of providing the basic services and repairing physical, social and economic damages. In the recovery phase, it is essential to provide financial assistance to those who were affected by the disaster as well as rebuilding the facilities damaged such as roads and bridges. It is also critical to sustain the mass care for human and animal populations. Recovery phase differs from the response phase for various reasons. Recovery is more intended to provide services for rebuilding of destroyed properties, re-employment, and repair of main infrastructure, whereas response involves actions for addressing immediate needs (Laemmermann, 2012).

Recovery phase starts following the emergency and some recovery activities might be concurrent with the response actions. Recovery involves coordination, development, and provision of service and site restoration plans for the impacted community in addition to reconstitution of government operations in terms of individual, nongovernmental and public and private sector programs. Some main recovery phase actions are listed but not limited to the following:

- Identification of needs and resources.
- Provision of housing and restoration.
- Provision of long term care for affected people.
- Developing mitigation procedures and techniques.
- Developing programs for preventing and controlling the effects of future incidents.
- Developing disaster preparedness measures and early warning systems for all sectors to minimize the number of losses (GFDRR, 2017).

The recovery phase sometimes takes longer than expected due to stabilization of all systems. Hence, it is important that the community and the government work in close coordination to overcome barriers regarding rebuilding of public health and safety.

### *3.3. Reconstruction*

Reconstruction is the last phase of post disaster management and it involves two main steps namely the building housing units and restoring or building infrastructure. Housing is the first priority of disaster victims in several countries (Hidayat and Egbu, 2010). It was indicated that housing constitutes the majority of expenditure with 30-50% financial allocation in terms post disaster financing (Freeman, 2004). Hence, housing must be immediately addressed in post disaster environment to meet the needs of the victims. Sustainability is a critical parameter in reconstruction projects to reduce vulnerability for future disasters. Moe and Pathranarakul (2006) stated that reconstruction projects target to provide a unique product in a given duration for elevating the living conditions of persons rather than focusing on the profit.

Moe and Pathranarakul (2006) identified post disaster reconstruction phase as public project management and introduced ten critical success factors, which must be addressed in disaster management. The critical factors are presented herein.

- Effective institutional arrangement: Decision making is speed up by the responsible government unit and authority line.
- Coordination and collaboration: Stakeholders' coordination and collaboration is a key factor for managing disasters.
- Supportive laws and regulations: The existence of supportive laws and regulations is a driver for the successful management of reconstruction phase in post disasters.
- Effective information management system: Existence of such system among key stakeholders results in successful outcomes after disasters.
- Competencies of managers and team members: Disasters are successfully managed when administrative, conceptual and technical skills are in place.
- Effective consultation with key stakeholders and target beneficiaries: Stakeholder consultation and communication with target beneficiaries are key processes in managing post disaster environment.
- Effective communication mechanism: This helps creating an environment, where stakeholder trust and cohesion is built and this eventually leads to higher project success in disaster management.
- Clearly defined goals and commitments by key stakeholders: Predetermined goals and responsive commitment helps reconstruction activities be completed with higher success.
- Effective logistic management: Technology, people, and expertise are part of the effective logistic management in disasters. This helps enhancing the capacity for coordination in organizations.
- Sufficient mobilization and disbursement of resource: Resource and mobilization is key for managing successful processes in disaster. This may be in the form of providing material, equipment, and workforce (Moe and Pathranarakul, 2006).

As the final phase of post disaster management, reconstruction is essential to help community recover and provide a safer environment to people.

## 4. Lean Practices in Post Disaster Management

This study investigates potential Lean construction practices that might be utilized in phases of post disaster management. This helps processes be conducted error free and provide a quicker rehabilitation after disaster. In a study conducted by Al Hattab and Hamzeh (2015), it was indicated that Lean practices reduce design errors. Similarly, Ko and Chung (2014) mentioned that design errors might be reduced with the Lean practices. In this respect, Table 1 presents relevant Lean construction practices along with their definition and presents relevance to post disaster management phases by summarizing how these tools and techniques be used in most effective manner.

**Table 1.** Lean construction practices in post disaster management.

| Lean Construction Practices | Description | Relevance to Post Disaster Management Phases |
|---|---|---|
| Safety-by-Design (SbD) | SbD, also called as Prevention through Design (PtD) is a concept to apply methods for minimizing occupational hazards at the early phases of design process and aims to optimize employee health and safety through processes. The method encourages construction practitioners to design out risks in health and safety through design development (CDC, 2019). | Designing health and safety risks helps producing a less error prone environment and facilitates the post disaster management processes. SbD also involves the design of safer systems so as to minimize or prevent the effect of accidents. Especially, design of safer practices and processes results in less losses and speeds up reconstruction phase. |
| Last Planner System | Last Planner System is a management tool for implementing Lean in design and construction projects. The system is a collaborative planning technique for making planning processes more effective, reducing variability, and making work flow more reliable (Davidson, 2013). Last Planner System is also a tool to experience less safety breaches and schedule errors motivating collaborative work. | Collaborative planning is essential in post disaster management. Therefore, Last Planner System might be used effectively in response phase of post disaster management, where immediate care is needed and collaborative work is a must. This helps a quicker assessment of the disaster site and manage processes in coordination with the teams. Moreover, Last Planner System might be also practiced in the reconstruction phase bringing up effective communication and collaboration for rebuilding activities. |
| Mistake proofing | "Mistake proofing, or its Japanese equivalent poka-yoke is the use of any automatic device or method that either makes it impossible for an error to occur or makes the error immediately obvious once it has occurred." Mistake proofing might be used to fix processes, where human error leads to defective or error prone processes (ASQ, 2019). | Mistake proofing practices might be utilized in every post disaster management phases. In response phase, the mistake proofing tools (i.e. color coded tags) might guide people for the damaged zones and prevent the likelihood of unexpected consequences. In recovery phase, mistake proofing devices (i.e. warning sensory alarms) might be utilized to develop mitigation procedures and prevent the effects of future incidents. In reconstruction, mistakeproofing tools might play a vital role in providing quicker assessment of losses and helps rebuilding in terms of making defects clear. |
| Visual Management | Visual management is a strategy for benefitting from sensory systems through visual ability and controlling information flow and work through visual cues (Tezel et al., 2017). The use of visual tools are catalysts for warning systems. | Visual cues are of utmost importance in terms of facilitating work control. Hence, visual management is an effective Lean technique to prevent defective processes. The cues of visual management might be used in all phases in post disaster management. In response phase, the cues might be used for better explaining the emergency procedures to help for evacuation. In recovery phase, the cues might be used in developing the mitigation procedures for future incidents. In reconstruction phase, the cues might be utilized in tagging the restoration and disaster area for effective institutional arrangement. |
| Value Stream Mapping (VSM) | Value stream mapping is a material and information flow technique for comparing the current and future state of production (Rahman et al., 2012). It helps visualize all the processes for getting a product, service, and value adding project. | VSM might be used in response phase of post disaster management to visualize current and future state of emergency procedures. This might provide a quicker assessment of public needs and develop actions accordingly. VSM might be further utilized in reconstruction phase to speed up rebuilding processes by visualizing the current and future state. |
| 5S | It is a workplace organization system to create a more efficient, safe, and clean working environment (Hiwale, 2018). It comes from the Japanese words translated as sort, straighten, shine, standardize, and sustain in English. | 5S might be used in all phases of post disaster management since it is about providing an organized work environment. Hence, the disaster site might be organized properly so that the negative effects of disasters are minimized. Especially, in the reconstruction phase, it is essential to sort and standardize items for easy rebuilding. |
| Time and Motion Studies | A time and motion study refers to recording movements of an employee when accomplishing a task. The technique aims to increase productivity and efficiency (Lopetegui et al., 2014). These studies are of paramount importance to identify and eliminate non value adding activities. | In reconstruction phase of disaster management, time and motion studies might be used to detect non value adding activities to provide a better scheme of rebuilding processes. This speeds up the rebuilding activities and provide community a better environment in short term. |

| | | |
|---|---|---|
| Kaizen | It is a Japanese management philosophy aiming to sustain continuous improvement. Kaizen helps enhancing efficiency and productivity at workplace resulting in higher quality and reduced costs (Shang and Pheng, 2013). | Kaizen might be used in all three phases of post disaster management since it aims to increase efficiency and productivity. The technique is applicable to response, recovery, and reconstruction phases during which efficient operations are in need. |
| Kanban | Kanban means "signboard or billboard" in Japanese. In Kanban technique, the flow of work is controlled through a set of cards, signals, and tokens (Tezel et al., 2017). It also helps teams work together effectively. | Kanban might be used in all processes of post disaster management. The use of informative cards, alarming signals and tokens would help save numerous lives in addition to quick rehabilitation of community. Therefore, Kanban systems are essential to inform community during response, recovery, and reconstruction phases. |
| Modularization and Prefabrication | Modularization and prefabrication is important in terms of reducing the risk of health and safety hazards. It also helps provide a faster delivery process (Gosling et al., 2016). | Modularization and prefabrication might be used in the reconstruction phase of post disaster allowing to a faster rebuilding process. It may be also utilized to reduce health and safety risks and protect community from a number of potential hazards. |
| Last Planner System | Last Planner System is a management tool for implementing Lean in design and construction projects. The system is a collaborative planning technique for making planning processes more effective, reducing variability, and making work flow more reliable (Davidson, 2013). Last Planner System is also a tool to experience less safety breaches and schedule errors motivating collaborative work. | Collaborative planning is essential in post disaster management. Therefore, Last Planner System might be used effectively in response phase of post disaster management, where immediate care is needed and collaborative work is a must. This helps a quicker assessment of the disaster site and manage processes in coordination with the teams. Moreover, Last Planner System might be also practiced in the reconstruction phase bringing up effective communication and collaboration for rebuilding activities. |

## 5. Discussion

The construction is a risky business. Hence, the use of innovative systems and materials are in greater demand to sustain community health and safety. Lean construction is therefore an efficient strategy to provide best practices in construction projects. Since Lean aims to minimize waste, it offers various advantages for construction practitioners to deliver cleaner and safer environments to the clients.

Considering the destroying impacts of disasters, post disaster management constitutes and important part to remove those effects. Hence, it is essential that community leaders work in close coordination with innovators and team leaders to smoothen the negative impacts of disasters on public health and safety. In this respect, Lean tools and techniques are unique solutions in terms of providing a faster rehabilitation process for the community. Especially, the use of some Lean tools such as Mistake proofing devices and Kanban cards could save millions of lives and potentially reduce the negative consequences of disasters. Mojtahedi and Oo (2012) discussed the applicability of Lean in post disaster reconstruction and concluded that Lean elements help improve performance of post disaster reconstruction leading to higher quality in built environment and smoothened work flow. Hence, Lean tools are effective means of managing post disaster processes in terms of smoothening negative consequences of disasters. As a result, one might think that use of these tools shall be successfully integrated in the processes, where public health and safety is the highest priority.

In response phase of post disasters, following emergency procedures on time and stabilizing the environment for surveillance is critical for the timely provision of supplies, services, and the equipment. Traditional procedures are sometimes in lack of completing these tasks. Previous research implies that information and communication based technology must be put in place to better handle post disaster management processes (Ofori, 2002; Nishigaki et al., 2011).Therefore, Lean practices might be utilized in those processes to evacuate people more quickly from the disaster area and deliver services in a more organized manner.

In recovery of post disaster management, needs and resources are identified along with providing housing and restoration. Moreover, programs for preventing and controlling the effects of future incidents are developed. Therefore, several Lean tools and techniques would be efficient to use in those steps for a better recovery process. For example, use of Last Planner system and Kaizen technique could be effective in developing the programs and inform the community about future incidents.

In reconstruction phase, setting up an effective communication mechanism is essential. Moreover, faster rebuilding process might result in enhanced community satisfaction and faster rehabilitation. Therefore, Lean tools and techniques might be effective to rebuild the disaster environment and increase the efficiency of the processes. For example, color coded tags might be placed on the damaged structural areas to keep people away from the dangerous zones. Moreover, modularized and prefabricated construction elements might be used to have a quicker building process.

The use of Lean practices in processes help reduce the number of hazards. Considering the high number of hazards in construction, it is apparent that the industry needs more innovative and early warning tools for overcoming this challenge. This requires an in depth analysis of current tools and techniques and motivate practitioners to think of more beneficiary and effective systems to be set up at workplaces. Lean is not the best approach yet but it is clear that it has certain impact on reducing hazards. In this respect, several research studies pointed out to the interaction between Lean and safety (Nahmens and Ikuma, 2009; Bashir et al., 2011; Howell et al., 2017). Hence, it is essential practitioners beware of availability of Lean tools and techniques and utilize them when needed.

This study intends to reveal the strong tie between Lean construction practices and post disaster management phases. The link between Lean and post disaster was assessed in a few studies (Mojtahedi and Oo, 2012; Mojtahedi and Oo, 2017) but there has not yet been a broad research to reveal the interaction. Hence, this study aims to fill this gap by revealing the clear link between Lean practices and post disaster phases. In this respect, it provides a systematic interaction between those to encourage the use of Lean tools and techniques in response, recovery, and reconstruction phases.

## 6. Conclusions

This study investigates the use of Lean construction practices in the phases of post disaster management. In this respect, an in-depth literature review was conducted to reveal the practices applicable to use in response, recovery, and reconstruction phases. In the first part of the study, Lean construction studies were researched and best practices were highlighted. Then, post disaster management processes were considered to indicate main activities conducted during those processes. Several Lean construction practices such as use of Kanban systems, Last Planner system, 5S, VSM, and Kaizen were identified and the use of those was explained in relevant post disaster management phases. The study is a preliminary research for reflecting the interaction between Lean construction and post disaster management. Moreover, it also puts safety forefront of Lean construction practices in the post disaster management context. This might potentially impact the community for taking a step forward towards developing safety measures with the help of Lean tools. Furthermore, it is expected to guide construction practitioners to become familiar with Lean construction practices and benefit from them for a quicker rehabilitation after disasters.

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Challenge Journal of Structural Mechanics

Research Article

# Effect of configuration of shear walls at story plan to seismic behavior of high-rise reinforced concrete buildings

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**ABSTRACT**

In developing countries, the need for shelter, working area, shopping and entertainment centers is increasing due to the increasing population effect. In order to meet this need, it is necessary to turn to high-rise buildings. Significant damages have been observed as a result of insufficient horizontal displacement stiffness of high-rise buildings in major earthquakes in previous years. It is known that as the height of the structure increases, the displacement demand of the structure also increases. Since it is accepted that the structure will make inelastic deformation in the design of the structure, these displacements increase to very high levels as the number of stories increases. For this reason, damages can be much higher than expected. In order to limit the level of damage that may occur in high-rise buildings, the horizontal displacement of buildings is limited in many regulations in our age. This limitation is possible by increasing the rigidity of the structures against horizontal displacement. In recent years, the use of shear wall has increased due to the horizontal displacement limitation in the regulations. The use of shear walls in buildings limits the horizontal displacement. However, the choice of where the shear walls will be placed on the plan is very important. Failure to place the shear walls correctly may result in additional loads in the structure. It can also lead to torsional irregularity. In this study, a 10-storey reinforced concrete building model was created. Shear wall at the rate of 1% of the plan area of the building was used in the building. The shear walls are arranged in different geometric shapes and different layouts. The earthquake analysis of 5 different models were performed. Equivalent Earthquake Load, Mode Superposition and Time History Analysis methods were used for earthquake analysis. The results were compared and a proposal was made for the geometry and configuration of the shear wall.

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## 1. Introduction

Earthquake is an oscillatory motion caused by the release of strain energy under or in the earth's crust (Chen and Qian, 2002; Mo and Kuo, 1998). Earthquake oscillations cause large horizontal displacements in buildings. As a result of this, significant damages occur in the buildings. 95% of Turkey's population is at risk of earthquakes (Yaman et al., 2019). It is an important issue that the structures show the necessary performance during an earthquake. Adequate strength, rigidity and durability are expected from the structures during the earthquake (Chandiwala, 2012). The use of curtain wall elements that increase the horizontal displacement stiffness of buildings during earthquakes prevents story drift of buildings. (Aktan and Kiraç, 2010). The shear walls are vertical elements that resist the horizontal load in the buildings (Nainan and Alice, 2012). The shear walls are a preferred element type in the load-bearing system due to their high rigidity and strength (Firoozabad et al.,

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2012). When the earthquakes in the previous years are examined, it is seen that shear wall in buildings perform better than frame system.  et al., 2013). Especially the increase in the height of the building makes the horizontal loads more determinant than the vertical loads in the buildings (Ucar and Merter, 2009). Therefore, in order to prevent story drift, many regulations contain rules limitation of story drift. The inelastic behavior is also limited by limiting the story drift.

The lack of clear information in the regulations regarding the use, placement and amount of shear walls to be used in reinforced concrete frame systems has led to the tendency of these studies. Young-Hun Oh examined the deformation capacity of the shear walls using a displacement-based approach (Oh et al., 2006). In Sakcalı et al.'s (2017) study, the earthquake performance of 8-storey reinforced concrete structures with different shear wall ratio examined. Halkude et al. in their study, the effect of the use of different lengths of shear walls in different places on the plan examined the earthquake performance (Halkude et al., 2015). In Gent Franch et al.'s (2008) study, they have tried to establish an acceptable connection with the ratio of shear wall to story area by modifying the vulnerability index value for masonry structures. In Rokanuzzaman et al.'s (2017) study, the effects of the configuration of the shear walls at story plan on the 16-storey building are examined under horizontal load.

In this study, the change of shear wall location in a 10-storey reinforced concrete building was investigated. Shear wall at the rate of 1% of the plan area of the building was used in the building. 5 different models have been created in terms of shear wall location. Models are analyzed according to Equivalent Earthquake Load (EEL), Mode Superposition (MS) and Time History Analysis (TH) methods on ETABS.

## 2. Material and Method

In this study, 5 different shear wall location have been created for a 10 storey reinforced concrete building. The three-dimensional view for the created models is given in Fig. 1. The story plans of the models according to the shear wall locations are given in Fig. 2.

For each model, 1.5 kN/$m^2$ dead load and 5 kN/$m^2$ live load were loaded on the story in addition to the self-weight of the building. The total length of the shear walls is the same for all samples. Additional infilled wall load is not added to the building. It was assumed that the structure was fixed in to the ground. The shear wall elements are modeled as shell elements. Column and beam dimensions were taken as 50x50 cm and 25x60 cm respectively. Slab thickness was taken as 15 cm. The short edge of the shear wall was accepted as 20 cm.

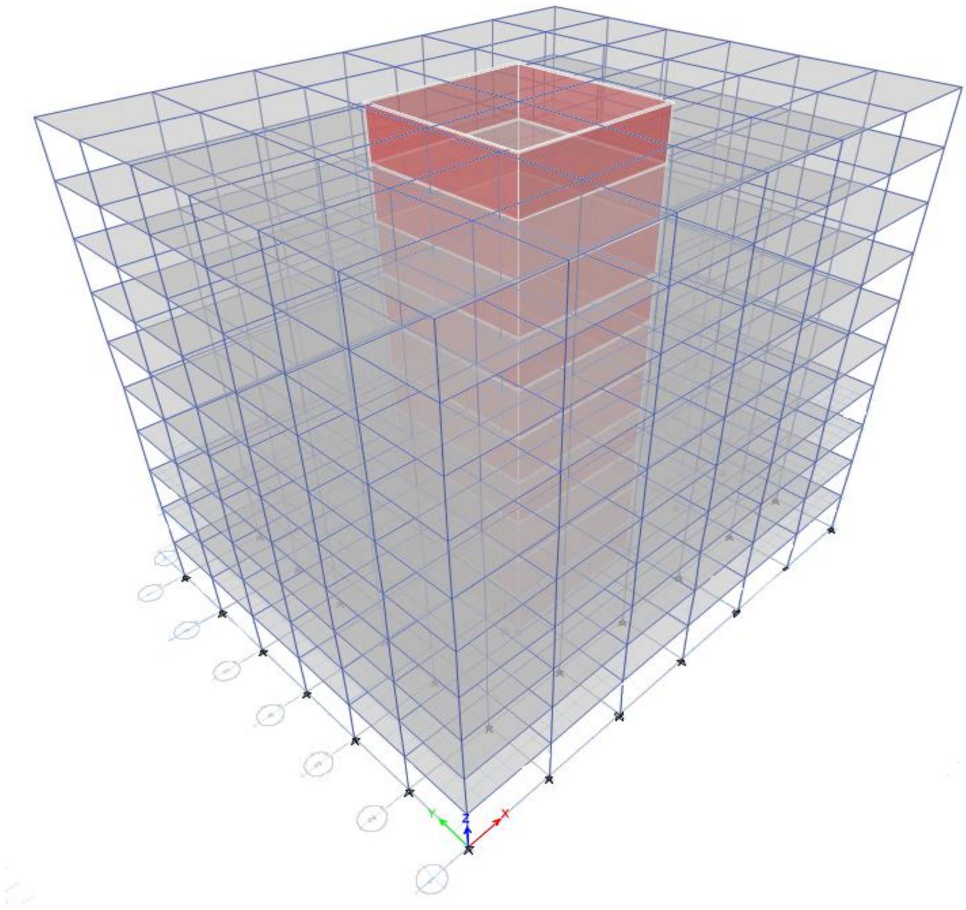


**Fig. 1.** Sample 5 (S5) 3D modelling.

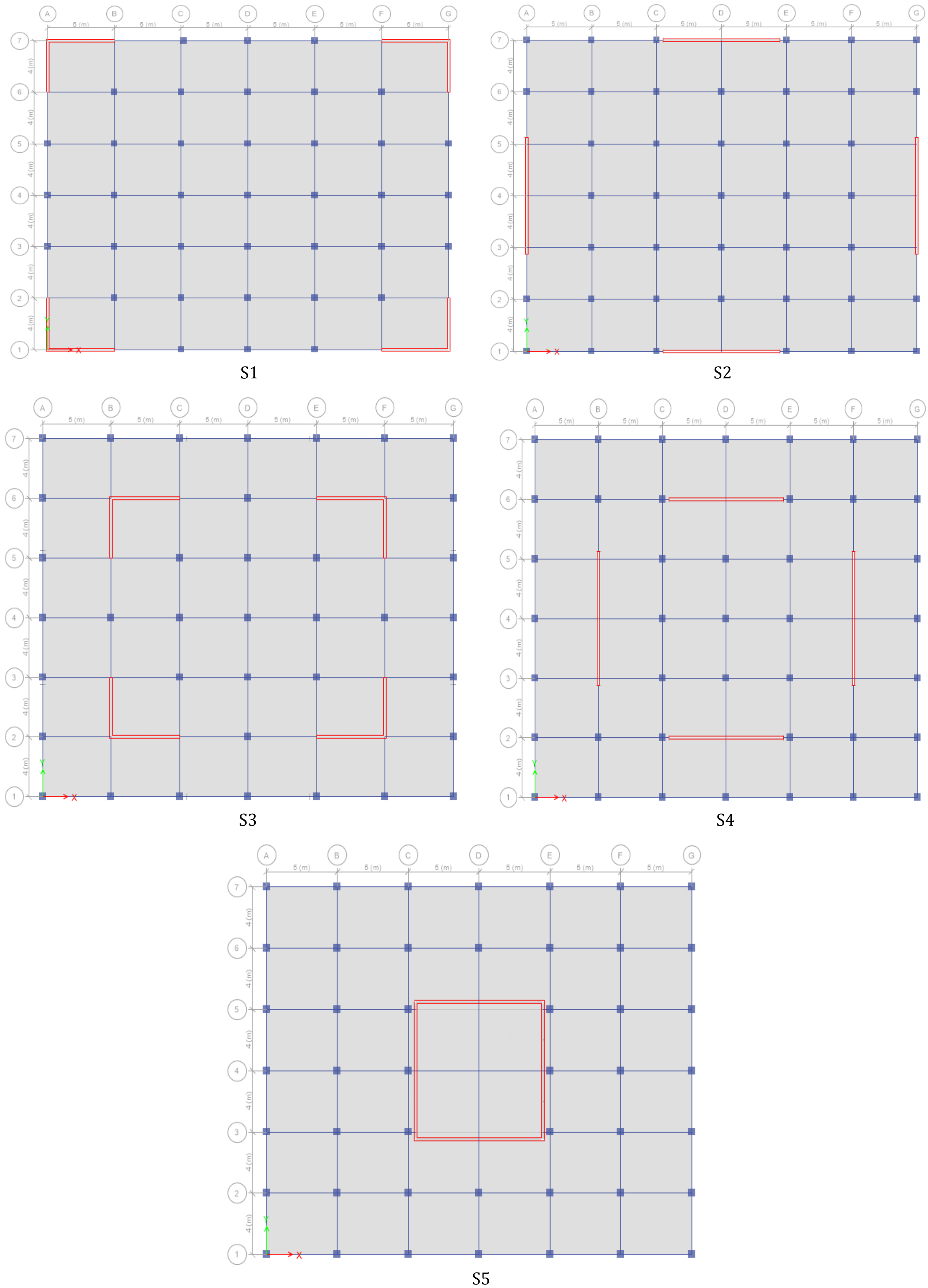



**Fig. 2.** Story plans of samples.

The earthquake parameters that will be used to calculate the earthquake load affecting the building are given in Table 1. Earthquake parameters given in Table 1 are obtained from Turkish Seismic Code – 2018 (TSC-2018).

**Table 1.** Seismic parameters.

| Parameters | Value |
|---|---|
| Ground Seismic Motion Level | DD-2 |
| Soil Classification | ZE |
| Latitude | 40.667344° |
| Longitude | 30.408702° |
| $S_S$ | 1.723 |
| $S_1$ | 0.466 |
| $S_{DS}$ | 1.3784 |
| $S_{D1}$ | 1.0569 |
| PGA | 0.710 |
| PGV | 58.826 |

The earthquake acceleration record to be used in TH is given in Fig. 3. The earthquake acceleration record is scaled to TSC-2018 response spectrum created with parameters given in Table 1. In this way, two different result obtained from EEL and TH can be compared with each other.

## 3. Results and Discussions

As a result of the analysis, natural vibration periods of the samples are given in Fig. 4. Natural vibration periods of the buildings are obtained from modal analysis on ETABS program. Natural vibration period for x and y direction is obtained from mode 1 and mode 2 respectively. As can be seen from Fig. 4, the natural vibration period of the buildings decrease as the shear wall is taken inwardly from the external axis. As a result, the fact that the shear wall is on the external axis increases the natural vibration period of the building and causes it to be exposed to less earthquake force. The fact that the shear wall is in the center of the building makes the building more rigid.

The base shear forces obtained from EEL, MS and TH methods are given in Fig. 5.

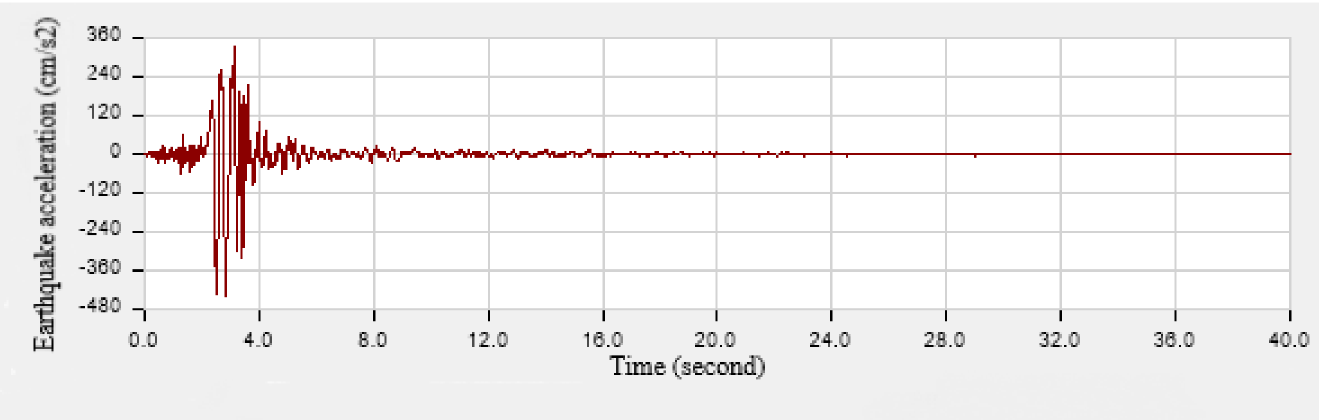



**Fig. 3.** ALTADENA - Earthquake acceleration record.

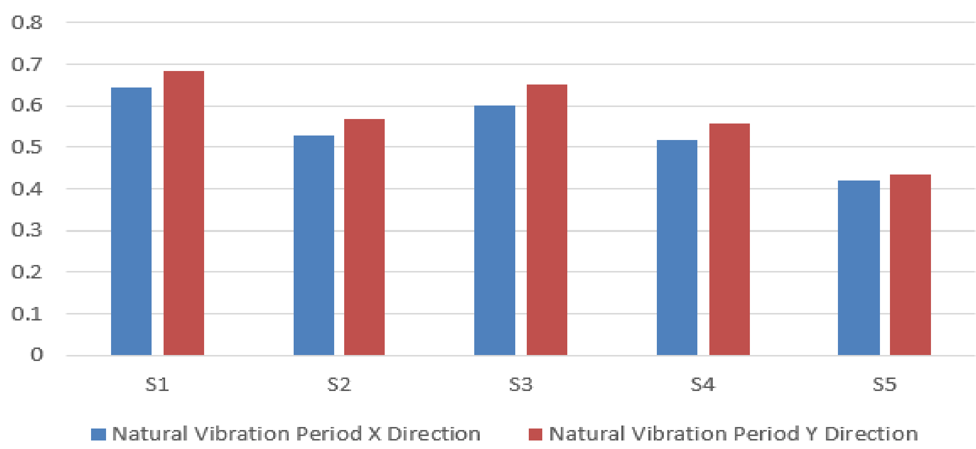



**Fig. 4.** Natural vibration period of models.

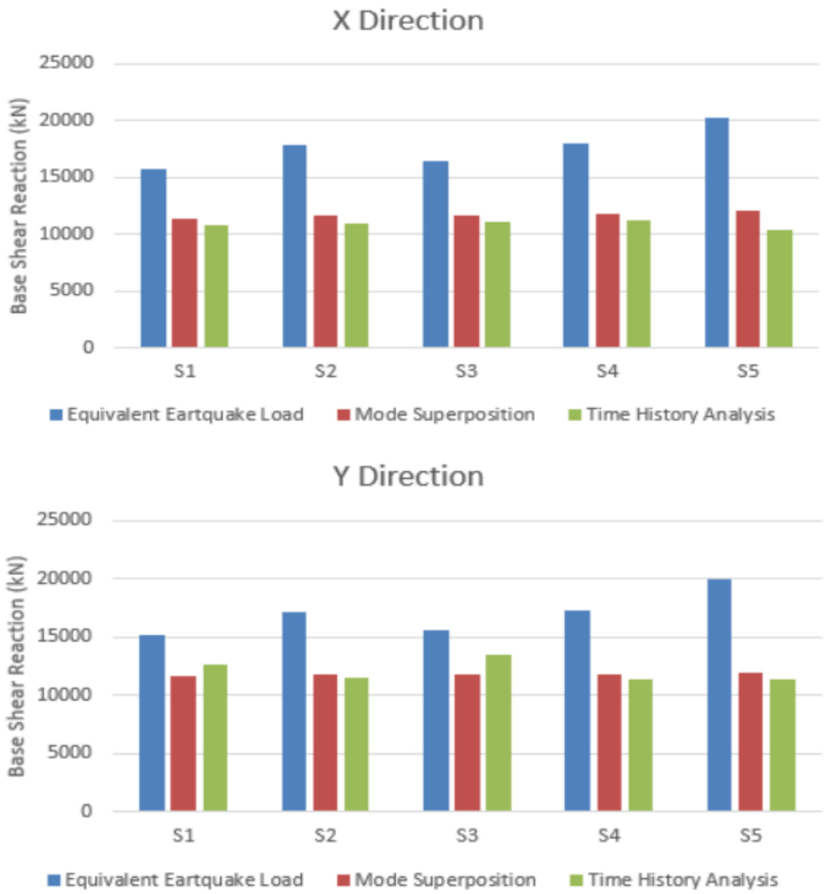



**Fig. 5.** Base shear force of models.

As shown in Fig. 5, the shear forces obtained in the EEL method were higher than the other two methods. It can be understood from this that the EEL method does not provide accurate results in multi-storey buildings. As seen in the analysis methods in MS and TH, the results were very close to each other. Placing the shear wall from the external axis to the internal axis increased the earthquake load affecting the building. Earthquake load increased in all three methods. In addition, it was observed that the earthquake load affecting the building decreased when the shear wall were placed at the corner of the building. When the natural vibration periods are examined, the earthquake load that affects the building increases as the natural vibration period of the building decreases. This occurs because the spectral acceleration increases as the natural vibration period decreases. Due to the relationship between the spectral acceleration-natural vibration period curves, the earthquake load affecting the building increases for a while. Fig. 6 shows that the shear forces affected to the stories are more variable than the other methods in the EEL method. This shows that the EEL method does not give correct results in multi-storey buildings. When the increase rates for earthquake load in the stories are examined, it can be said that it is approximately same for each method. The ratio of shear forces affecting to shear walls to the base shear force is given in Table 2 and Table 3.

Relative story drifts according to earthquake calculation methods are given in Fig. 7. For each method and direction (*X* and *Y*) are presented in separate graphs.

## 4. Conclusions

In this study, earthquake analysis was carried out for 5 different shear wall configurations. Results of the analyses are given below:

- The natural vibration period of the building decreases as a result of the shear wall being taken in from the external axis.
- The fact that the shear wall is on the external axis increases the natural vibration period of the building so that it is exposed to less earthquake load.
- The shear forces obtained by the EEL method were higher than the other two methods. It can be understood from this that the EEL method does not provide accurate results in multi-storey buildings.
- Placing the shear wall from the external axis to the internal axis increased the earthquake load affecting the building.
- It was observed that when the shear walls were placed at the corner of the building, the earthquake load affecting the building decreased.
- As the shear walls are placed from the external axis to the internal axis, the shear force ratio of the shear walls increases. In S5, this ratio increased to 0.95.
- Relative story drifts were maximized on the 6th story in all samples. In all samples, the limit value defined in TBDY-2018 has not been exceeded.
- When the relative story drift are examined, it is seen that the building behaves more rigid as a result of moving the shear walls from the external axis to the internal axis.

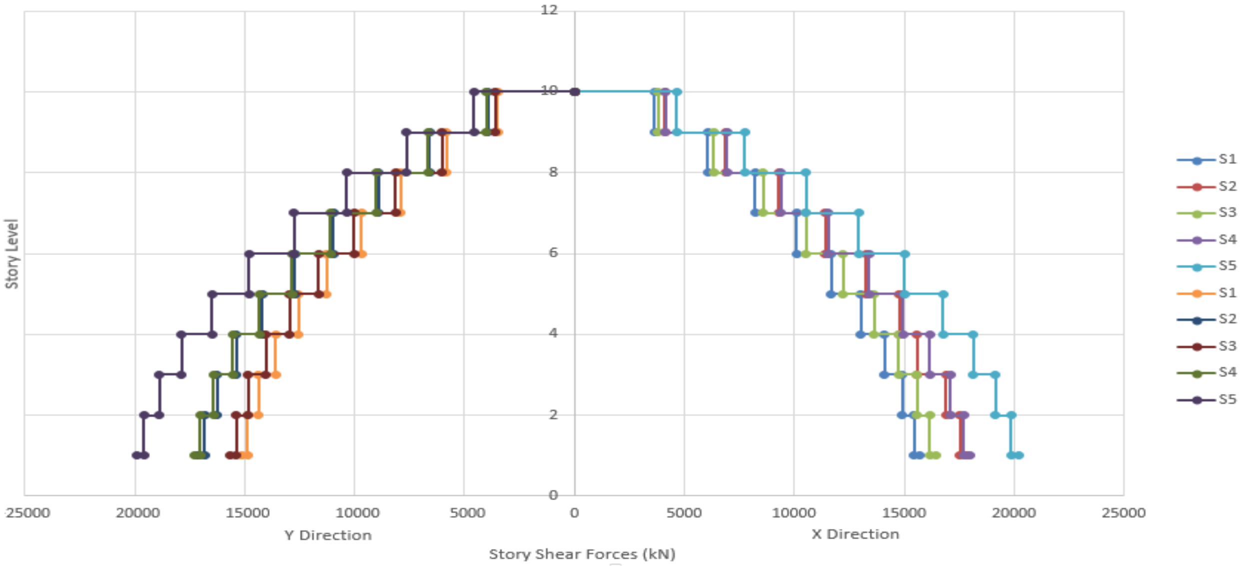
- If there is a problem with the relative story drift in the building, it would be more accurate to place the shear walls on the internal axis.
- If the earthquake load affected the building is high value while relative story drift is under the limitation of the relative story drift, it would be more accurate to move the shear wall to the external axis.

**Table 2.** Ratio of shear force acting on shear wall to base shear force for *x* direction.

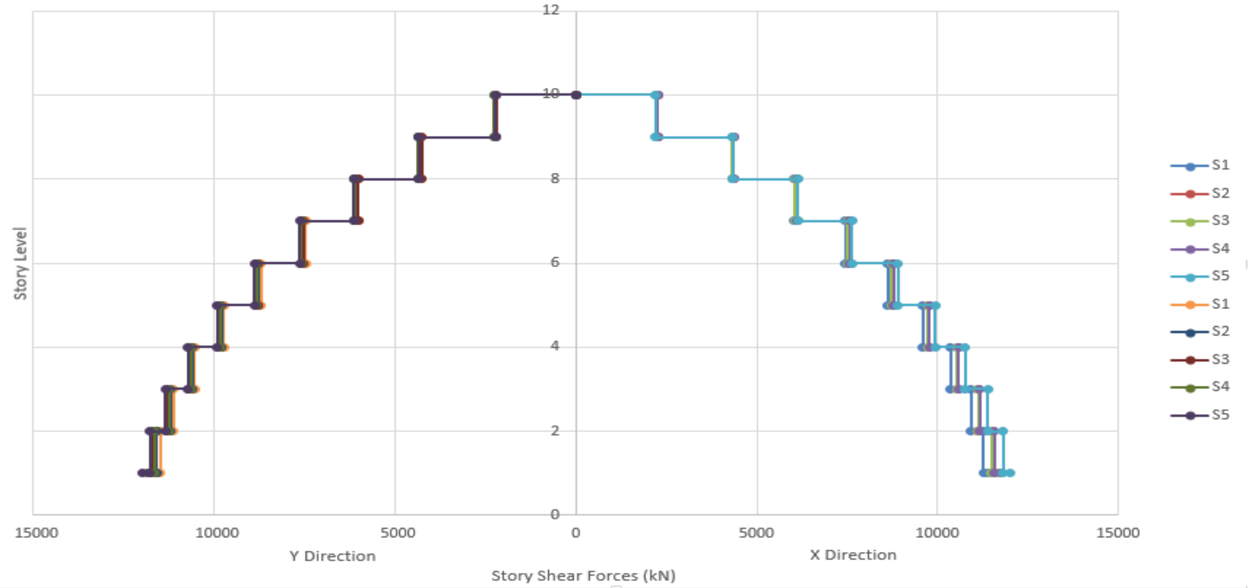
| Number of Model | Type of Seismic Analysis | Shear Force Acting on Shear Walls (kN) | Base Shear Force (kN) | Ratio |
|---|---|---|---|---|
| S1 | EEL | 14193 | 15719 | 0.90 |
| | MS | 9819 | 11420 | 0.86 |
| | TH | 9405 | 10743 | 0.88 |
| S2 | EEL | 16532 | 17828 | 0.93 |
| | MS | 10383 | 11685 | 0.89 |
| | TH | 9714 | 10953 | 0.89 |
| S3 | EEL | 16143 | 16440 | 0.98 |
| | MS | 10804 | 11653 | 0.93 |
| | TH | 10228 | 11035 | 0.93 |
| S4 | EEL | 17994 | 18024 | 0.99 |
| | MS | 11046 | 11755 | 0.94 |
| | TH | 10532 | 11215 | 0.94 |
| S5 | EEL | 19257 | 20208 | 0.95 |
| | MS | 11457 | 12021 | 0.95 |
| | TH | 9897 | 10382 | 0.95 |

**Table 3.** Ratio of shear force acting on shear wall to base shear force for *y* direction.

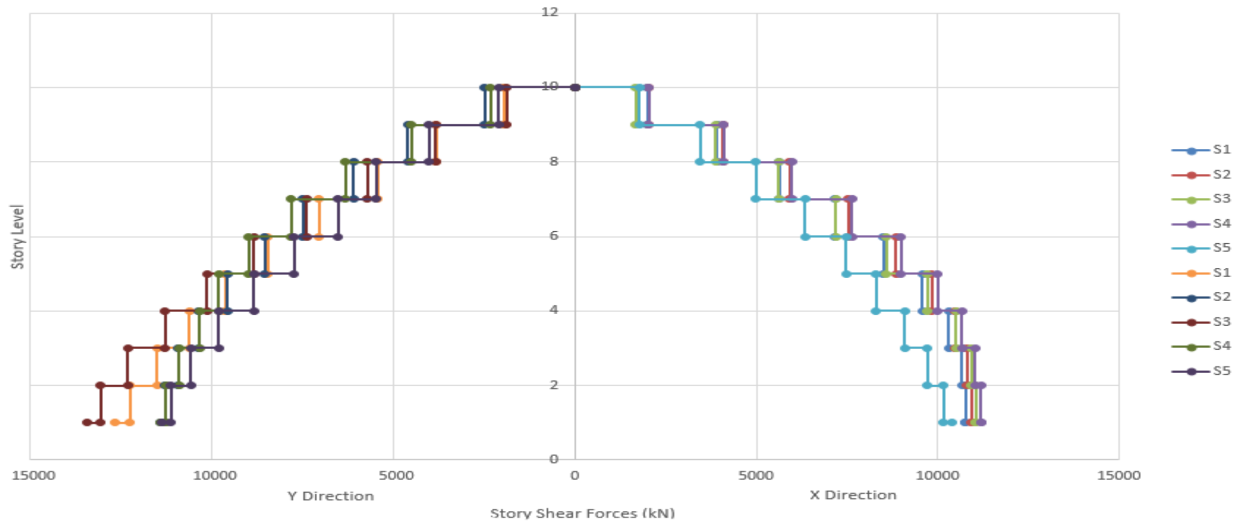
| Number of Model | Type of Seismic Analysis | Shear Force Acting on Shear Walls (kN) | Base Shear Force (kN) | Ratio |
|---|---|---|---|---|
| S1 | EEL | 12974 | 15137 | 0.86 |
| | MS | 9403 | 11636 | 0.81 |
| | TH | 10267 | 12695 | 0.81 |
| S2 | EEL | 15978 | 17127 | 0.93 |
| | MS | 10383 | 11685 | 0.89 |
| | TH | 10315 | 11445 | 0.90 |
| S3 | EEL | 15244 | 15644 | 0.97 |
| | MS | 10642 | 11840 | 0.90 |
| | TH | 12086 | 13464 | 0.90 |
| S4 | EEL | 16148 | 17318 | 0.93 |
| | MS | 11025 | 11819 | 0.93 |
| | TH | 10643 | 11418 | 0.93 |
| S5 | EEL | 18976 | 19919 | 0.95 |
| | MS | 11403 | 11819 | 0.97 |
| | TH | 10861 | 11419 | 0.95 |




Equivalent Earthquake Load

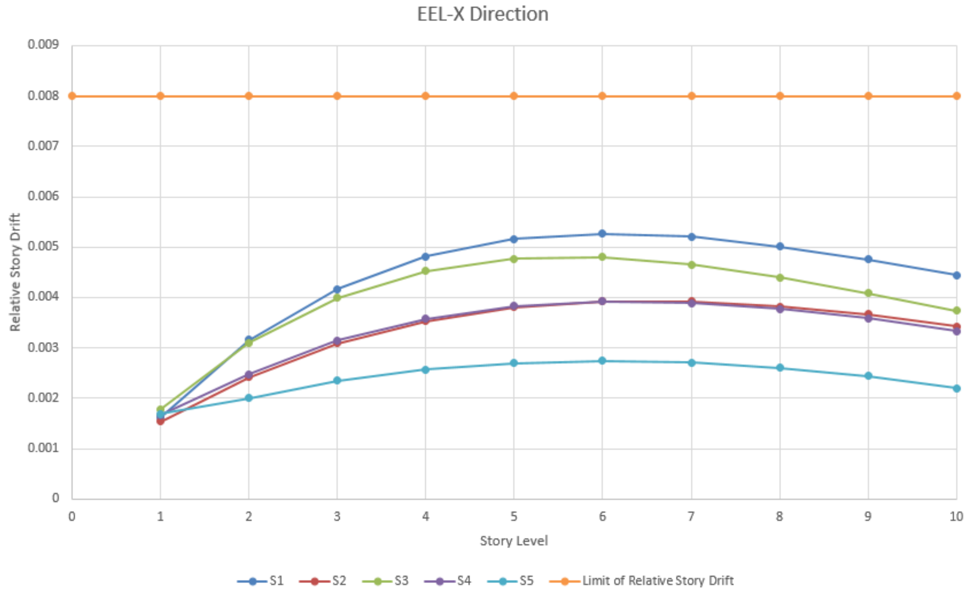


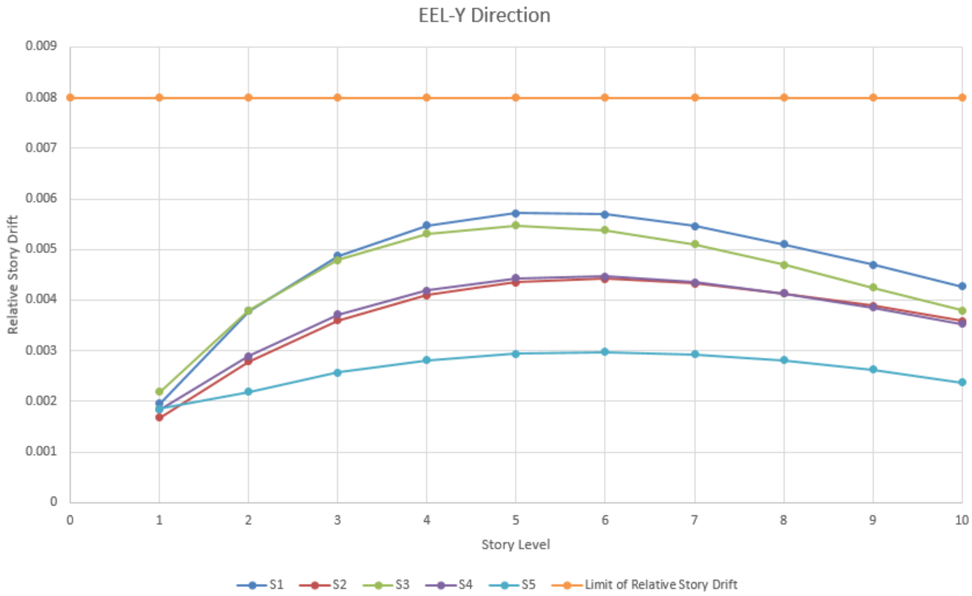

Mode Superposition




Time History Analysis

**Fig. 6.** Shear forces on each story of models.



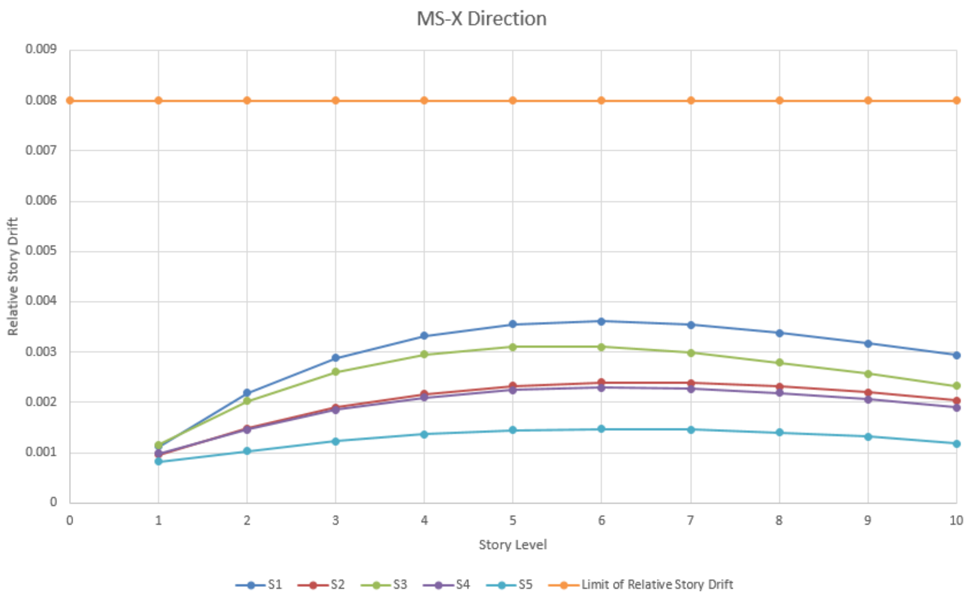




**Fig. 7.** (continued).

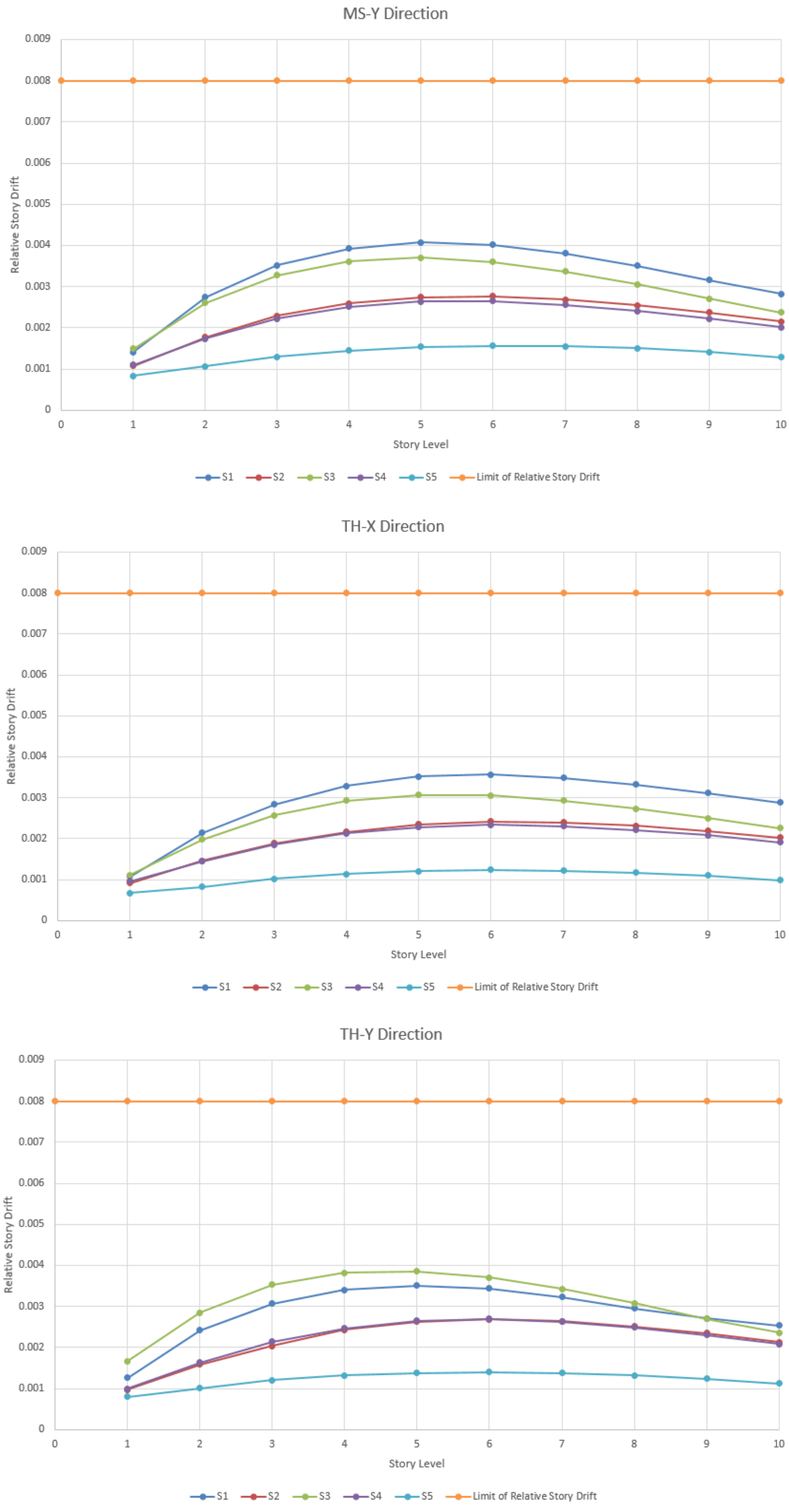



**Fig. 7.** Relative story drift.

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Challenge Journal of Structural Mechanics

**Research Article**

# Model updating of a reduced-scaled masonry bridge by using response surface method

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**ABSTRACT**

Historical structures reflect the historical and cultural properties of countries and also contributes to the economy in terms of cultural tourism. Therefore, it is important to understand the structural behavior of these kinds of structures under dynamics loads such as earthquakes, etc. to protect and transfer them safely to future generations. For this reason, this study aims to investigate the dynamic behavior of a reduced-scale one-span masonry arch bridge constructed in laboratory conditions by performing experimental and numerical analysis. Operational Modal Analysis (OMA) Technique was performed under ambient vibrations for experimental study to determine modal parameters of the reduced-scaled bridge model. Sensitive three-axial accelerometers were located on critical points on the bridge span and signals originated by accelerometers were collected to quantify the vibratory response of the scale bridge model. The experimental natural frequencies, mode shapes and damping ratios resulting from these measurements were figured out by using Enhanced Frequency Domain Decomposition (EFDD) technique. ANSYS software was utilized to carry out 3D finite element (FE) modeling of the reduced-scale masonry bridge and determine the natural frequencies and mode shapes of the bridge numerically. Experimental results were compared with FE analysis results of the bridge. Significant differences appeared when comparing the results of the experimental and numerical with the initial conditions. Therefore, the finite element model is calibrated by using the response surface (RS) method according to the experimental results to minimize the uncertain finite element modeling parameters of the reduced-scale bridge model such as material properties.

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## 1. Introduction

Historical masonry structures have significant value for countries due to reflecting their heritage and culture. These kinds of structures also contribute to the countries economically in terms of cultural tourism. Historical structures have been damaged or failed partially or completely since they have exposed to loads such as earthquakes, wind and explosion effects for many years. Also, deterioration of the building materials, time-dependent deformations, excessive and irregular loading caused by misuse, ground settlements, flood disasters, fires, etc. are other factors that play a role to cause serious damages to the structures. To protect the historical structures, the structural behaviors of these structures should be investigated.

**Nomenclature**

| | |
|---|---|
| $G_{xx}(jw)$ | power spectral density (PSD) matrix of the input signal |
| $G_{yy}(j\omega)$ | PSD matrix of the output signal |
| $H(j\omega)$ | frequency response function (FRF) matrix |
| $*$ | complex conjugate |
| $T$ | transpose |
| $y_{RS}$ | function value of the obtained RS model |
| $y$ | FE calculation results |
| $\bar{y}$ | mean of $y$ |
| $N$ | number of confirmation sample points in design space |

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Due to the importance of masonry structures for a historical point of view, many researchers have focused to understand the dynamic characteristics of such structures using experimental and numerical methods. Experimental modal analysis (EMA) and OMA methods are used to determine the dynamic characteristics of structures such as mode shapes, natural frequencies, and modal damping ratios. In the EMA method, the specific input force (impulse hammer, drop weight and electrodynamics shaker) is applied to the structure to obtain its modal parameters. In the OMA method, the dynamic characteristics of the structure can be obtained by using vibrations in the structure under the environmental effects (wind, traffic, etc.) without applying any external force to the structure. The use of this method is more appropriate and common, especially in historical buildings since it is both fast and practical and does not have any risk of damaging the structure. In the literature, historical structures such as historical bridges, historical minarets, and historical towers are studied by using the OMA method (Brencich and Sabia, 2008; Gentile and Gallio, 2008; Diaferio et al., 2011; Foti et al., 2012; Oliveira et al., 2012; Gentile et al., 2015).

In civil engineering, FE models are one of the most common methods used to obtain the responses of structures under dynamic loads such as earthquakes, wind, traffic, and explosion. While creating a numerical model of any structure, assumptions about many unknown and uncertain system parameters such as geometrical and material properties, loading and boundary conditions must be made. For these reasons, the accuracy and validity of the established numerical models should be investigated. Model update techniques have been developed and named as a model calibration or, more clearly, parameter estimation or determination. In general terms, the model calibration technique aims to update or reevaluate the unknown structural system properties used as a parameter in numerical modeling. The data (acceleration-time values, frequency response functions, natural frequencies, and natural mode shapes, modal stresses, and curvature grades, modal elasticity) obtained from the vibration experiments give detailed information about the general and local behaviors of the structure. Therefore, this vibration data has an important role in the FE model calibration process.

Model updating methods are divided into two groups as non-iterative and iterative methods. Non-iterative methods directly update the stiffness and mass matrices of the numerical model through a closed-form direct solution. However, such methods cause the loss of structural connectivity, and the proposed corrections are not always physically meaningful. On the other hand, iterative methods require sensitivity matrices to conduct iteration in minimizing the objective function. However, the sensitivity-based method seems not practical in case of high degrees of freedom structural system, as it results in the problem of slow convergence and a time-consuming process due to the increase in the degree of freedom. Therefore, the RS method has appeared as an alternative tool in the FE model calibration process due to simplicity and provides fast optimization because of smooth gradients, thus decreasing the convergence problem (Umar, 2018). RS methodology is a combination of statistical and mathematical techniques to represent the relationship between the inputs and outputs of a physical system by explicit functions. This methodology has been widely employed in many applications such as design optimization, response prediction, and model validation and damage detection.

Ren and Chen (2010) presented the RS-based FE model updating procedure for civil engineering structures in structural dynamics. Residuals between analytical and measured natural frequencies were chosen as the objective function. The proposed procedure was demonstrated by a simulated simply supported beam and a full-size precast continuous box girder bridge tested under operational vibration conditions. The results were compared with those obtained from the traditional sensitivity-based FE model updating method. The real application to a full-size bridge appeared that the FE model updating process was efficient and converges fast with the RS to replace the original FE model. Casciati (2010) used RS method based on detecting a change in the statistical distribution of the error associated to the function that approximates the relationship among measurements obtained on different sensors locations across the structure. The results from ambient vibration tests demonstrated that the correct damage scenarios can be achieved by using the method. Deng and Cai (2010) applied the RS method in a simply supported beam and model updating of an existing bridge. The structural parameters were updated using the genetic algorithm by minimizing an objective function. The results of the study demonstrated that this method works well and achieves reasonable physical explanations for the updated parameters. The D-optimal design was performed to generate RS models by updating uncertain parameters by Fang and Perera (2011). The accuracy of the proposed method was demonstrated experimentally by testing on a reinforced concrete frame and a full-scale bridge and numerically on a beam model. The modulus of elasticity and the moment of inertia as input parameters and modal frequencies as output parameters were chosen. In addition to the numerical study, the method provides sufficient accuracy for damage estimation in real structures with single and multiple damage scenarios.

Deshan et al. (2015) proposed a new FE model updating method of bridge structure by combining the substructure FE model updating method with the response surface model updating method for updating the FE model of a certain combined cable-stayed suspension model bridge. Samples of updating parameters were obtained by the homogeneous design method. The experimental results illustrate that the updated parameters obtained from the proposed method has a reasonable physical meaning, and the proposed finite element model updating method can be effectively used for the finite element model of the bridge structure. Zong et al. (2015) presented an application of the RS method for the FE model updating of bridge structures. A third-order polynomial response surface was created and then utilized to improve the computation efficiency in the model updating. The proposed procedure is demonstrated on a full-size long-span prestressed continuous rigid-frame bridge. The real application to a full-size bridge has illustrated that the FE model updating process is efficient and converges fast compared to the traditional sensitivity-based model updating method. The updated FE model can relatively reflect the actual condition of the bridge in the design space of parameters and can be further applied to FE model validation and damage identification. The study has also emphasized that a high-order RS function would be necessary to consider more input parameters in the case of more complex civil engineering structures. Hacıefendioğlu et al. (2017) investigated the influences of uncertainty in material parameters on the stochastic response of a historic masonry bridge subjected to random ground motion. For this purpose, probabilistic analysis of the bridge was carried out with Monte Carlo simulation obtained through the RS method. The probabilistic responses of the bridge at specific node points were compared with the same response obtained by using deterministic material properties. The study concluded that increasing the coefficient of variation values of material parameters, elastic modulus, Poisson's ratio, and mass density, produced a greater effect on the stochastic response of the bridge.

Worden and Cross (2018) have investigated the changes in environmental conditions on the civil infrastructures which are usually openly exposed to the weather and may be subject to strongly varying operational conditions. The approach is based on constructing a data-based RS model that can represent measurement variations as a function of environmental and operational variables. The models can then be used to take out environmental and operational variations so that change detection algorithms signal the event of damage alone. The study has proposed a Treed Gaussian Process model as developing RS on the Z24 Bridge in Switzerland and Tamar Bridge in the US. The proposed model has provided an effective approach to RS modeling and that in the Tamar case, a linear model is, in fact, sufficient to solve the problem. Fang (2020) has proposed a new method based on the fourth-order polynomial RS model in order to solve the problem of high risk and low precision of existing damage detection methods for long-span Bridges. The parameters of the FE model of the bridge were modified according to the RS model. Based on the FE model, the modal strain energy before and after the damage of the element was calculated, and the damage index of the element was calculated, so as to understand the damage detection of the long-span bridge structure. Experimental results have demonstrated that the proposed method can accurately detect the damage location of long-span Bridges under different damage conditions.

Most previous studies investigated the RS method updating approach of different civil engineering structures. Moreover, there is a limited number of studies relating to FE model updating of historic masonry structures by using the RS method, especially masonry bridge-type structures. Therefore, this paper aims to reveal that the RS method is a suitable and accurate method for model updating in masonry bridge structures. For this purpose, a reduced-scale masonry bridge model constructed in the laboratory environment by considering the similarity requirements of the single-span Historic Sarpdere Bridge based on a part of the doctoral dissertation by Alpaslan (2019) was used for both experimental and numerical study to obtain a more realistic FE model that can reflect the modal behavior of the masonry bridge structure by using RS based FE model updating approach. OMA method was performed to identify the dynamic behavior of the reduced-scale bridge under environmental vibrations. FE model of the reduced scale bridge was developed in ANSYS (2013) software program and natural frequencies and mode shapes of the bridge obtained numerically. The results of the modal parameters obtained from the field measurements are compared with those identified by the FE model. Significant differences were observed in natural frequency values between experimental and numerical analysis. Therefore, calibration of the FE model was performed based on the experimental results of the reduced-scale bridge by using the RS method. Correlation studies were conducted between the experimental and numerical natural frequencies of the reduced scale bridge to minimize the uncertain finite element modeling parameters.

## 2. Materials and Method

### *2.1. Reduced scale masonry bridge model*

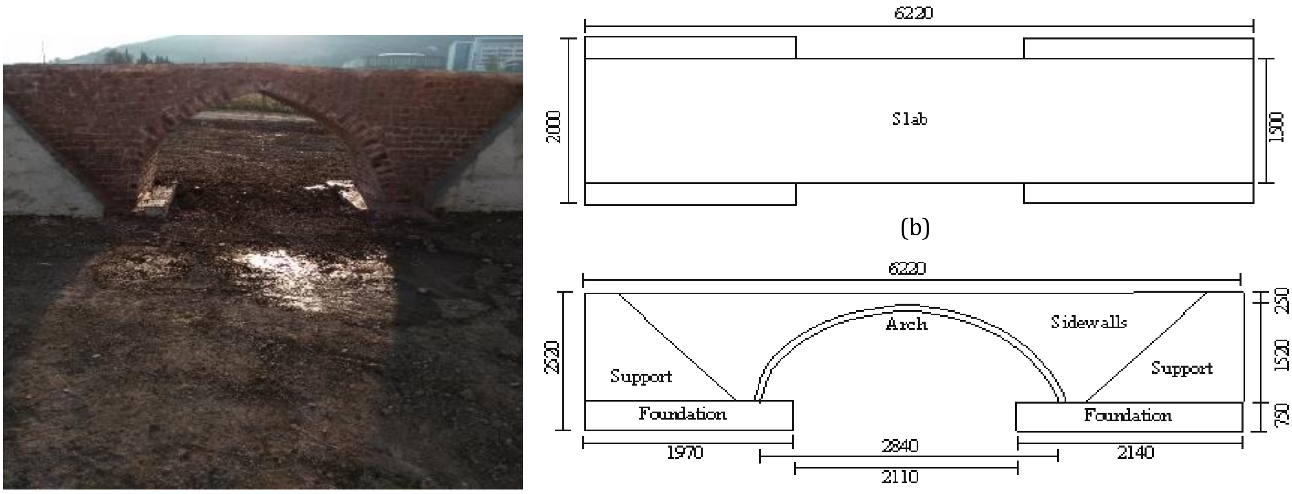
In the construction of the bridge model, andesite stones were used for the arches, sidewalls, and slab elements. Arch stones were cut to be 5x10 cm, 10x10 cm, 15x10 m and 20x10 cm. Crushed andesite stones were used for the side walls. The slab stones were supplied as 30x80x4 cm and 30x100x4 cm plates. Straw soil was used as filling material. The right and left abutment of the masonry bridge model was designed as concrete. Khorasan mortar was used as the binding material for the arch, sidewall and slab elements on the bridge model. Khorasan mortar consisted of 40% building tile powder, 40% stone powder and 20% hydrated lime powder. The workability of the mixture was obtained by adding water.

The construction steps of the masonry bridge model are represented in Fig. 1. Abutments were completed, the necessary framework process was performed and the arrangement of arch stones was started from both sides simultaneously. Some of the arch stones were placed perpendicularly in order to strengthen the joints between arch and the sidewalls filling material. The side walls are then arranged on both sides. After the filling process, the slabs were placed and the masonry bridge model was completed. The arches, sidewalls, and slab stones are arranged in a staggered manner. The side walls consisted of two rows of crushed stones with a total thickness of 15 cm. The reduced-scale masonry bridge model bridge and its geometry are represented in Fig. 2. The mechanical properties of materials used in the construction of the reduced-scale bridge model were investigated experimentally. Unit weight and compressive strength of assemblages were determined within the scope of experimental study. The assemblages with Khorasan mortar were prepared in two different ways as single and staggered form as demonstrated in Fig. 3. The experimental results are presented in Table 1. The modulus of elasticity of the samples was calculated by using the empirical equation, Eq. (1), obtained from a study in the literature by Ocak (2009) which is based on calculating the modulus of elasticity by iteration method. The UCS values in the equation show the average compressive strength of the test specimens.

$$E = 6.195 \cdot e^{0.0074UCS} \text{ (GPa)} \tag{1}$$

(a) (b) (c) (d)

**Fig. 1.** Construction steps of masonry bridge model: (a) Arch; (b) Side walls; (c) Filling; (d) Slab.




(a) (c)

**Fig. 2.** (a) Reduced-scale masonry bridge model; (b) Top view; (c) Front view (mm).

**Fig. 3.** Experimental assemblage samples with Khorasan mortar.

**Table 1.** Physical and mechanical properties of the assemblages.

| Width x Length x Height (cm x cm x cm) | Unit Weight (gr/cm$^3$) | Axial Compression $P_{max}$ (kN) | Compressive Strength (MPa) | Mean (MPa) | Modulus of Elasticity (MPa) |
|---|---|---|---|---|---|
| 9.9x19.5x32 | 2.25 | 432 | 22.4 | | |
| 10.0x14.8x30.8 | 2.22 | 349 | 23.6 | | |
| 10.0x19.9x32.3 | 2.15 | 594 | 30.0 | 24.7 | 7438 |
| 10.0x20x32.1 | 2.16 | 559 | 28.0 | | |
| 10.0x14.7x31.5 | 2.13 | 341 | 23.2 | | |
| 10.0x19.7x31.9 | 2.20 | 420 | 21.3 | | |

The flowchart of the study is summarized in Fig. 4. The first step is that the initial finite element model of the bridge model is generated by using the material and geometrical properties and modal analysis are performed to investigate the dynamic behavior of the bridge model. Then, experimental studies are conducted on the bridge model considering the natural frequency values obtained from the numerical analysis. Natural frequencies, mode shapes and damping ratios of the bridge are identified by using the vibration records. Finally, the initial FE model was calibrated according to the obtained natural frequency values experimentally. In this process, it was tried to represent the dynamic behavior of the bridge model more accurately.

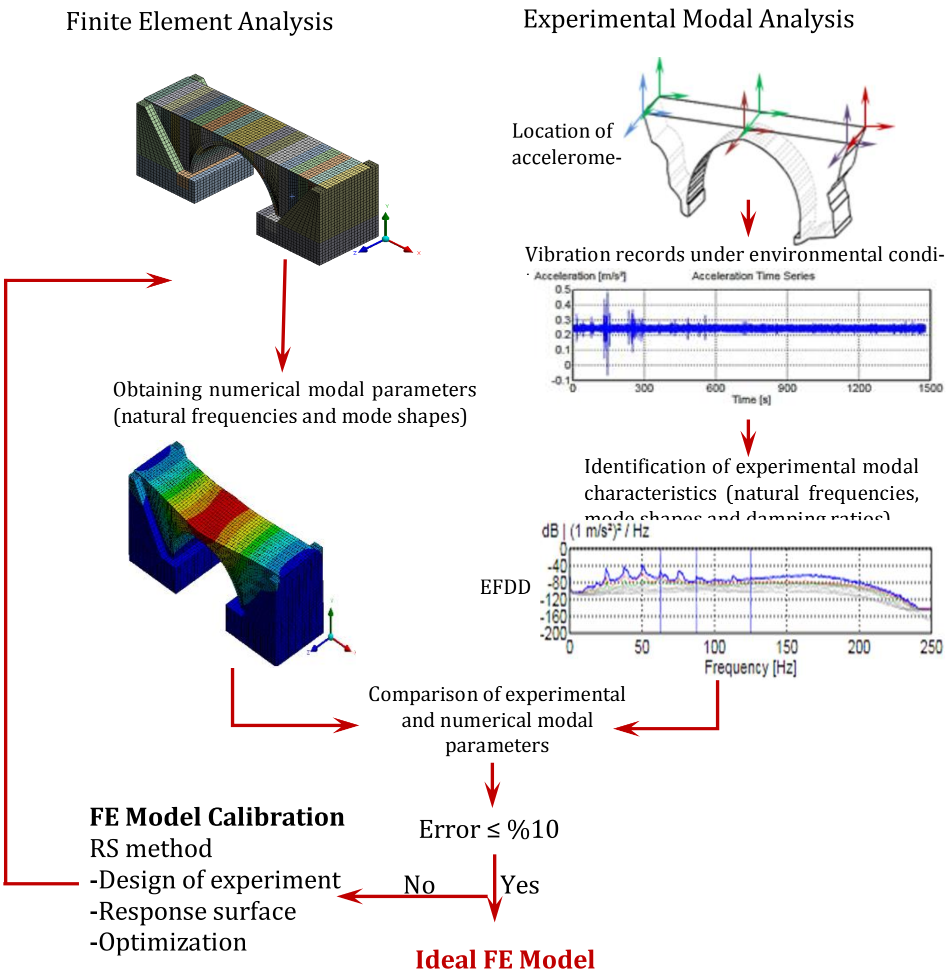



**Fig. 4.** Flowchart of the study.

### *2.2. Enhanced frequency domain decomposition*

In general, the EFDD technique is used for OMA in the civil engineering industries. In the EFFD technique, the spectral density matrix is approximately separated into a set of single degree of freedom (SDOF) systems utilizing the Singular Value Decomposition. It is possible to get exact results in the case where loading is white noise, the structure is lightly damped, and if the mode shapes of close modes are geometrically orthogonal. Even if these assumptions are not satisfied, the results are significantly reasonable. The relationship between unknown input x(t) and the measured responses y(t) is expressed as (Brincker and Zhang, 2009; Bendat and Piersol, 2010);

$$G_{yy}(j\omega)=H(j\omega)^{*}G_{xx}(jw)H(jw)^{T} \quad (2)$$

### *2.3. Response surface method*

RS method is an approximate optimization method that looks at various design variables and their responses, which seeks the best experimental design using the minimum number of design samples, to determine the combination of design variables. RSM can achieve a satisfactory accuracy between the measured data and the FE model-generated data (Cheng et al., 2007; Landman et al., 2007). The ANSYS DesignXplorer (2013) used in this study employs the approximation method in RS creation. RS method used in the calibration process of the numerical model consists of the three steps as the design of experiments, response surface and optimization. The accuracy of using RS method are depended on the number of variable input parameters. The method is not effective considering large number of input parameters. Therefore, there is a limitation of input parameters (fewer than 20 is ideal) for RS method in ANSYS DesignXplorer (2013) to be able to generate an appropriate and accurate DoE and RS.

#### *2.3.1. Design of experiments*

Design of experiments is a technique used to scientifically determine the location of sampling points. There are a variety of design of experiments algorithms or methods in engineering studies namely star, full factorial, central composite, and Box-Behnken designs. A common feature of all the techniques used is to locate sampling points in the space of random input parameters in the most efficient way or to try to position them with the least sampling point to obtain the necessary information. Sample points in effective locations not only reduce the required number of sampling points but also increase the accuracy of the response surface obtained from the results of the sampling points. In this study, the central composite design for three variables is selected as a sampling method.

#### *2.3.2. Generation of RS model*

In this study, genetic aggregation technique was used to generate RS models. The technique automates the selection, configuration, and creation of the RS. It automatically generates the most suitable approach for each output and provides more reliable results than other RS models due to multiple solutions of RS and cross-validation processes. The fitted RS models should be used to conduct the FE model updating after validated through their fitness. $R^2$ criterion and root mean squared error (RMSE) criterion are usually utilized for multi-RSM and complicated models (Box and Draper, 1987; Fang and Perera, 2009). R-square Statistic and Relative Mean Square Error are presented in Eq. (3) and Eq. (4) are regarded as the criteria to check the fitness of the fitted RS models.

$$R^2 = 1 - \frac{\sum_{j=1}^{N} \left[ y_{RS}(j) - y(j) \right]^2}{\sum_{j=1}^{N} \left[ y(j) - \bar{y} \right]^2} \tag{3}$$

$$RMSE = \frac{1}{N * \bar{y}} \sqrt{\sum \left[ y_{RS}(j) - y(j) \right]^2} \tag{4}$$

The larger the value of R-square is, the more accurate the RS model. The smaller the value of RSME is, the more accurate the RS model.

#### *2.3.3. Optimization*

In the structural dynamic, the natural frequencies and modal shapes are crucial parameters for the structural response features. In this study, the natural frequency was taken as a response feature. The primary model previously obtained is updated herein to seek the reference values of input parameters by minimizing the response discrepancies between the primary RS model and the experimental model. The measured frequencies from the minaret employing OMA are used as the objective responses. Optimization computation in this study is carried out with the screening approach. This approach is a non-iterative direct sampling method by a quasi-random number generator based on the Hammersley algorithm. In general, the screening method is the most convenient way to carry out a preliminary design study, since it is a low-resolution, fast and detailed study that can be useful to quickly find approximate solutions (ANSYS, 2013).

## 3. Results

### *3.1. Initial finite element model of reduced-scale bridge model*

3D dimensional FE model has been created by using ANSYS Workbench (2013) software program to obtain the natural frequency and mode shapes of the reduced-scale bridge model numerically. The FE model of the reduced-scale bridge is composed of one arch, filling and slap, and two sidewall parts. In addition, concrete abutments were placed on the right and left sides of the FE model. In the initial FE model, the contact type between the infill and the all other parts was chosen as frictional contact and friction coefficient was selected as 0.01. Body contact element was used for connection of all other parts and the SOLİD186 three-dimensional solid element model having 20 joints with the degree of freedom in each x, y and z-direction were chosen as an element type. The material properties represented in Table 2 were used in the initial FE. At the support points where the reduced-scale bridge model contacts to the ground, the bottom parts of the concrete abutments were assumed as fix support. Therefore, all the DOFs at these support points were restrained. It was assumed that the material would have elastic behavior and analyzes were performed by neglecting the decrease in stiffness. The FE model consists of 24772 elements and 135159 nodes. The FE model of the reduced-scale bridge is demonstrated in Fig. 5. The transverse, bending and torsional mode shapes obtained from the numerical analysis of reduced-scale bridge model are illustrated in Fig. 6 and also their corresponding natural frequency values are presented in Table 3.

**Table 2.** Material properties used in the FE model.

| | Modulus of elasticity (MPa) | Unit weight (kg/m$^3$) | Poisson's ratio |
|---|---|---|---|
| Arch | 7400 | 2200 | 0.2 |
| Side walls | 7400 | 2200 | 0.2 |
| Slab | 7400 | 2200 | 0.2 |
| Infill | 1500 | 1500 | 0.05 |
| Concrete | 28000 | 2400 | 0.2 |

**Table 3.** Initial FE model natural frequencies.

| Mode | Initial FE model |
|---|---|
| | Natural frequencies (Hz) |
| 1 | 71.37 |
| 2 | 114.35 |
| 3 | 146.56 |

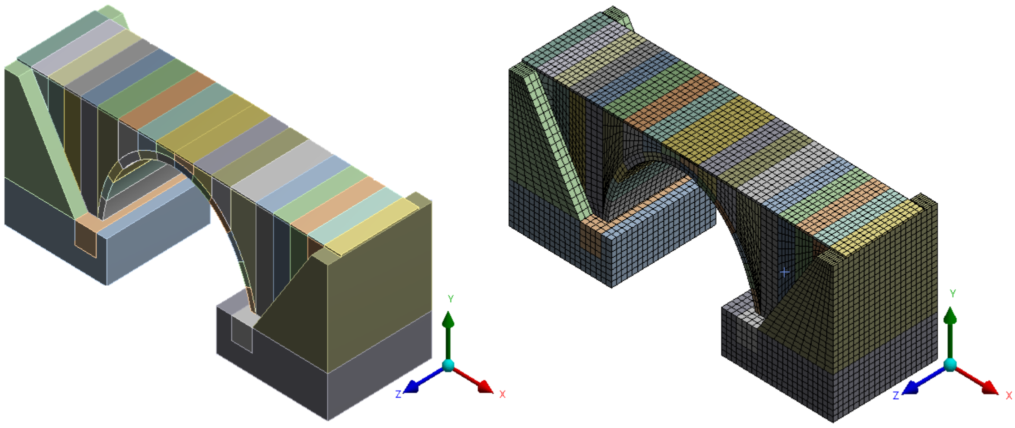
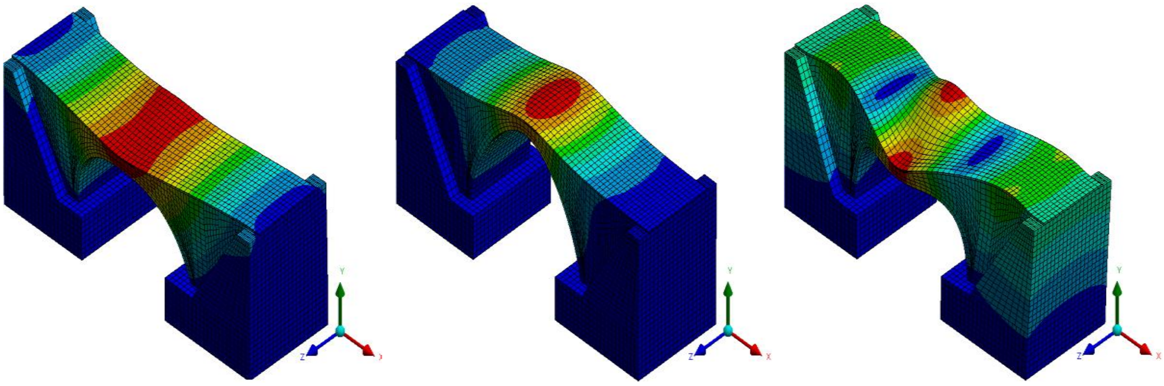


**Fig. 5.** FE model of the reduced-scale bridge.



Transverse mode Bending mode Torsional mode

**Fig. 6.** Mode shapes of the reduced-scale bridge FE model.

### *3.2. Experimental study*

The vibrational response of the reduced-scale masonry bridge model was taken under the environmental effects by using the OMA method. A total of six single-axis accelerators with a range of 0-400 Hz was used in the experimental study. The measurement time and frequency range were selected as 30 min and 0-500 Hz, respectively. Fig. 7 represents the experimental study applied for the reduced-scale masonry bridge model. Vibration recordings were taken as two measurements using reference accelerometers. The accelerometers were located to the projection of the center of the bridge and the arch span. The configuration of the accelerometers is shown in Fig. 8. Arrows in blue color represent reference accelerometers. Vibration signals from accelerometers are recorded using Testlab_V2 software via the data acquisition system. The experimental modal parameters of the masonry bridge were analyzed using ARTeMIS Modal 1.5 (2012) software program.

**Fig. 7.** Experimental application of reduced-scale masonry bridge.

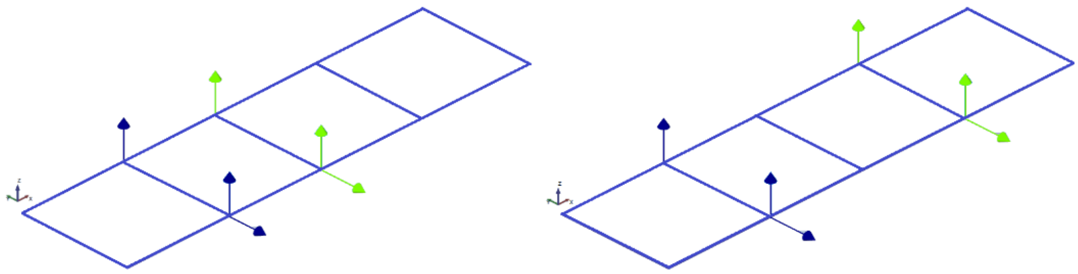


**Fig. 8.** Configuration of accelerations.

Natural frequencies, mode shapes and damping ratios of reduced-scale bridge model were obtained by using the EFDD method. The singular values identified by the EFDD method are represented in Fig. 9. Mode shapes were identified as the transverse, bending and torsional mode. The natural frequency and damping ratio values corresponding to these three modes of the structure are demonstrated in Table 4. In addition, the mode shapes of the experimentally obtained reduced scale bridge are shown in Fig. 10.

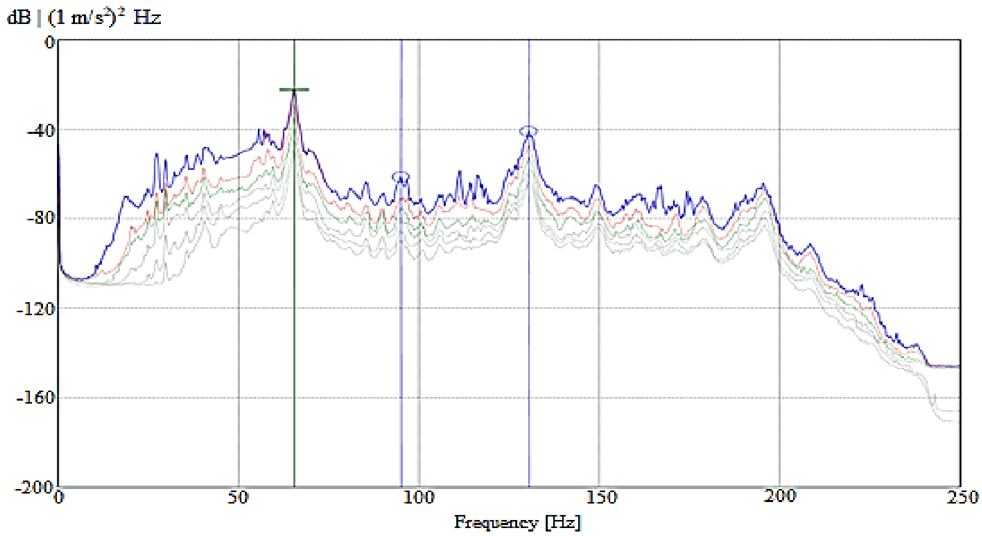



**Fig. 9.** Average of normalized singular values of spectral density matrices of all test setups.

**Table 4.** Experimentally identified natural frequencies and damping ratios.

| Modes | Natural frequencies, $f$ (Hz) | Damping ratios, $\xi$ (%) |
|---|---|---|
| Transverse | 65.46 | 0.526 |
| Bending | 95.09 | 0.561 |
| Torsional | 130.55 | 0.368 |

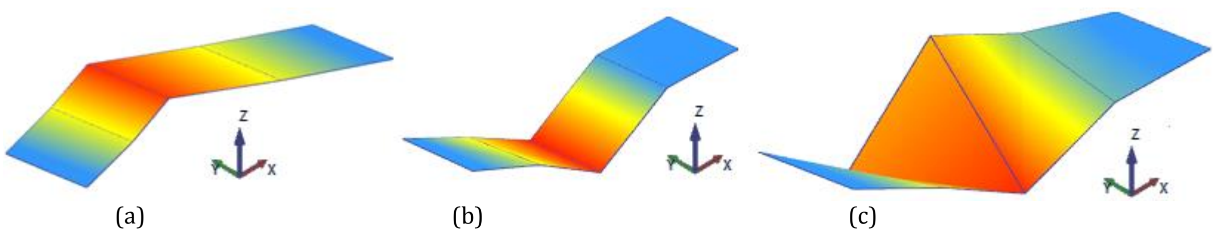



**Fig. 10.** Experimentally identified mode shapes of the masonry bridge model;
(a) Transverse mode; (b) Bending mode; (c) Torsional mode.

The comparison between natural frequency values obtained by experimental and numerical analysis are demonstrated in Table 5. It can be seen that the differences between natural frequency values are not satisfied, therefore, the initial FE model needs to be calibrated to obtain a more realistic FE model that represents the dynamic behavior of the reduced-scale bridge.

**Table 5.** Comparison of natural frequency values.

| Mode | Experimental analysis | Numerical analysis with initial conditions | Error (%) |
|---|---|---|---|
| | Natural frequencies, $f$ (Hz) | | |
| Transverse | 65.46 | 71.37 | 9.03 |
| Bending | 95.09 | 114.35 | 20.25 |
| Torsional | 130.55 | 146.56 | 12.26 |

### *3.3. Finite element model calibration of the reduced-scaled masonry bridge*

There are significant differences between natural frequency values in the numerical and experimental analysis of the reduced-scale bridge. It is thought that the reason for these differences may be due to the modulus of elasticity of Khorasan mortar that may cause labor during construction. Another reason may be due to the lack of continuity of the Khorasan mortar used in the connection of stones during construction. Also, the filling material is compacted and placed during construction and this might result in important changes for material properties of the filling material. These applications can lead to a difference in the material properties used in the initial FE model of the structural components. Therefore, in order to minimize these residuals and obtain a more reliable FE model, the modulus of elasticity and unit weight of the arch, sidewalls, slap and filling material was chosen as updated parameters in the FE calibration process by using RS method. The lower and upper limits of the updated modulus of elasticity and unit weight of the structural components for experimental design are shown in Table 6. In the experimental design, a total of 81 analyzes were performed using the face-centered central composite design approach at these upper and lower limits of the updated parameters. A genetic aggregation approach is performed to generate the RS models. The validity of the RS models is checked by using $R^2$ and root mean squared error (RMSE) criteria. $R^2$ and RMSE values obtained for each corresponding mode are illustrated in Table 7. It is demonstrated that all $R^2$ and RMSE values are close to 1 and 0, respectively, which represents that the obtained RS models have a high regression accuracy.

After the RS models are obtained, the natural frequency values corresponding to the third mode shapes is tried to be converged with the frequency values identified by the experimental analysis. For this purpose, experimentally obtained natural frequencies are used as objective values as demonstrated in Fig. 11.

**Table 6.** Lower and upper limits of updated parameters.

| | Limits | Modulus of elasticity (MPa) | Unit weight (kg/m$^3$) |
|---|---|---|---|
| Filling material | Lower | 1300 | 1500 |
| | Upper | 1700 | 1900 |
| Slab | Lower | 4000 | 1800 |
| | Upper | 7400 | 2200 |
| Arch | Lower | 6000 | 1900 |
| | Upper | 7400 | 2200 |
| Sidewalls | Lower | 4000 | 1500 |
| | Upper | 7500 | 2200 |

**Table 7.** Accuracy check for response surfaces.

| | Transverse Mode | Bending Mode | Torsional Mode |
|---|---|---|---|
| $R^2$ | 0.999 | 0.999 | 0.999 |
| RMSE | 0.0234 | 0.0633 | 0.0803 |

| Name | Parameter | Objective | | Constraint | | |
|---|---|---|---|---|---|---|
| | | Type | Target | Type | Lower Bound | Upper Bound |
| Seek P5 = 65,463 Hz; 65 Hz <= P5 <= 70 Hz | P5 - Transverse Mode | Seek Target | 65,463 | Lower Bound <= Values <= Upper Bound | 65 | 70 |
| Seek P6 = 95,092 Hz; 87 Hz <= P6 <= 1005 Hz | P6 - Bending Mode | Seek Target | 95,092 | Lower Bound <= Values <= Upper Bound | 87 | 1005 |
| Seek P7 = 130,55 Hz; 127 Hz <= P7 <= 143 Hz | P7 - Torsional Mode | Seek Target | 130,55 | Lower Bound <= Values <= Upper Bound | 127 | 143 |

**Fig. 11.** Optimization objective values.

As a result of the optimization process, the updated modulus of elasticity and unit weight of the structural components are represented in Table 8 and Table 9, respectively. As can be seen, significant differences occurred in the updated material properties in the calibrated FE model. The modulus of elastic values of the components decreased by about 4%-45%. Moreover, the unit weight of the arch, sidewalls, and slab components reduced between 3%-17%. On the other hand, the unit weight of the filling material increased by approximately 7%.

The comparison between the natural frequency values identified by experimentally and numerically is indicated in Table 10. Residuals between the natural frequency values obtained from the initial FE model and experimentally obtained natural frequency values were significantly reduced. Differences between numerical and experimental natural frequencies corresponding to the transverse, bending and torsional modes vary between 0.31% and 9.08%. Therefore, it is seen that the FE model obtained as a result of the calibration process reflects the dynamic behavior of the reduced-scale masonry bridge in a more realistic way.

**Table 8.** The updated modulus of elasticity of reduced-scale masonry bridge components.

| | Initial FE model | Calibrated FE model | Differences (%) |
|---|---|---|---|
| | Modulus of elasticity (MPa) | | |
| Ach | 7400 | 6355 | -14.12 |
| Side walls | 7400 | 4569 | -38.25 |
| Slab | 7400 | 4129 | -44.20 |
| Infill | 1500 | 1433 | -4.47 |

**Table 9.** The updated unit weight of reduced-scale masonry bridge components.

| | Initial FE model | Calibrated FE model | Differences (%) |
|---|---|---|---|
| | Unit weight (kg/m$^3$) | | |
| Ach | 2200 | 2131 | -3.14 |
| Side walls | 2200 | 2080 | -5.45 |
| Slab | 2200 | 1843 | -16.23 |
| Infill | 1500 | 1610 | 7.33 |

**Table 10.** Differences in natural frequencies between before and after calibration.

| Mode | Experimental | Initial FE model | Calibrated FE model | Initial FE model | Calibrated FE model |
|---|---|---|---|---|---|
| | Natural frequencies, $f$ (Hz) | | | Error (%) | |
| Transverse | 65.46 | 71.37 | 65.25 | 9.03 | 0.31 |
| Bending | 95.09 | 114.35 | 103.73 | 20.25 | 9.08 |
| Torsional | 130.55 | 146.56 | 135.30 | 12.26 | 3.63 |

## 4. Conclusions

In this study, modal parameters of a reduced-scale masonry bridge were investigated by using the OMA method performed under ambient vibrations. The EFDD method is implemented to identify the natural frequencies, mode shapes, and damping ratios experimentally. 3D finite element model of the reduced-scale Masonry Bridge is constructed and detected the natural frequencies and mode shapes analytically by using ANSYS software. Experimental and analytical results with the initial conditions were compared to each other and significant differences were identified. For this reason, the response surface-based FE model calibration technique was utilized to close the residuals between frequencies obtained from the analytical and experimental analysis. After the finite element model calibration, the differences between experimental and analytical natural frequencies reduced considerably.

According to the results obtained in this study, the initial FE model of the structures is not sufficient to represent their dynamic behavior. Therefore, the initial FE model should be updated by considering the changes in parameters such as material properties and boundary conditions. Thus, a more realistic FE model can be created which can exhibit the dynamic behavior of engineering structures. Furthermore, it can be seen that the RS method allows to further improve the correlation between experimental and analytical modal parameters, therefore, it is an effective approach in the FE model updating process.

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