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EFFECT OF X-BRACING CONFIGURATION ON EARTHQUAKE DAMAGE COST OF STEEL BUILDING

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Abstract. Seismic structural design of X-braced steel buildings using life cycle cost analysis aims to reveal the most appropriate structural solution for both satisfying economic aspects and earthquake resistant design code requirements among a number of variant solutions accounting architectural concerns. In this study, five storey X-braced steel building with three different X-bracing configurations is designed using various base shear values and the total cost of each design of three configurations is calculated for different earthquake intensities. Initial costs and the cost of the expected damages caused by future earthquakes are determined for each X-bracing configuration. The maximum interstorey drift ratio is selected as seismic performance parameter for satisfying earthquake code demands and evaluated through nonlinear static analysis. The optimum X-bracing configuration is determined by using the balance between the initial cost and the lifetime earthquake damage cost.

Keywords: X-brace, steel building, nonlinear static analysis, pushover, earthquake damage cost, optimum total cost, optimum system force coefficient.

1. Introduction

In the current earthquake resistant design practice, steel buildings are designed in such a manner that structural members have adequate capacity of extensive yielding and plastic deformation, without exhibiting loss of strength under strong earthquakes. Traditionally, steel moment-resisting frames are popular structural systems that are commonly used in seismic regions. However, these buildings did not meet anticipated structural behavior in some cases and significant economic losses occurred under ground motions even less than the design earthquake. Based on the previous experience and experimental research, it is observed that beam-to-column welded connections can develop catastrophic failures due to their brittle response (Mahin 1998; Mahin *et al.* 2002). Also, excessive lateral deformability of unbraced frames can result intolerable damage at non-structural elements even under moderate earthquakes. These damages increase investigation, repair and long-term costs.

Another popular way of providing lateral resistance to seismic forces and minimizing lateral drifts is to employ concentrically X-braced steel frames. Steel braced frame systems are efficient since frame members work together like a truss and resist primarily axial loads with little or no bending in the members until the compression braces buckle. The parameters controlling the behavior of a brace are effective slenderness, compactness of the cross-section and end connection details. Energy dissipation in X-braced steel frames almost entirely relies on the cyclic behavior of diagonal braces, which may exhibit significant stiffness and strength degradation (Maison and Popov 1980; Gugerli and Goel 1982). In order to ensure ductile response under severe earthquakes, structures are designed against higher design forces than those assumed for moment frames by accounting for lower behavior coefficients, relatively low slenderness ratios for diagonal members, neglecting the resistance of the compressed diagonal and strict over- strength conditions proposed for the design of the diagonal end connections in Eurocode-3 (1992). Nevertheless, it must be noted that, especially for industrial facilities, X-braced steel frames still represent the preferred design solution for lateral loads.

Earthquake resistant design codes aim to protect human life and reduce the damage. However, costs that could reveal in future earthquakes and the difficulties in repairing the post earthquake damages put forward the need for consideration of damage control in the design rather than accounting only for life loss prevention. Life safety is obviously essential and the main concern of seismic design. Nevertheless, cost issue has not been explicitly included in the design requirements yet. Both direct and indirect economic losses under earthquakes can be enormous and comparable to the initial costs of the buildings. It appears that, even if possible, designing for absolute safety with no damage under all likely earthquake intensities would be extremely costly. Therefore, the issue of optimizing earthquake induced damage cost needs to be taken into account adequately, which may lead to acceptation of some risks in earthquake resistant design philosophies. In this respect, uncertainties in structural behavior and performance under a given earthquake loading should be considered when determining adequate design criteria (Jankovski and Atkočiūnas 2010). Ang and Lee (2001) recommended a systematic approach for cost effective optimum design of RC buildings in Mexico using cost functions based on the Park-Ang damage index. Wen (2001) adopted a design methodology based on minimum expected life cycle cost criterion and examined the concept on numerical examples for steel structures. Seçer and Bozdağ (2008) investigated the effect of earthquake induced damage cost in the structural design of moment-resisting steel frames. Sarma and Adeli (2002) applied life cycle cost optimization of steel structures using fuzzy logic. Furthermore, Lee *et al.* (2004) performed a life cycle cost analysis on steel bridges.

In this study, the effect of X-bracing configuration on earthquake-resistant and cost-effective steel building design is investigated by considering displacement based structural design procedure. The maximum inter-storey drift ratio is selected as the seismic performance parameter in nonlinear static analyses. Three X-bracing configurations are taken into consideration in numerical examples and each case is designed for various base shear values. The optimum cost effective structural design of each configuration is determined with respect to base shear and total cost values.

2. Nonlinear Static Analysis

Seismic design codes usually define a single design earthquake for evaluating structural performance against earthquake hazard. Recent catastrophic earthquakes such as Northridge in 1992, Kobe in 1996, Kocaeli in 1999 caused serious damages in buildings alarming the structural engineering community to improve seismic design codes. Most of the current seismic design codes are regulatory design procedures where the design criteria are expressed in terms of forces. Modern seismic design procedures are based on performance based design criteria in that the building should be designed to consume the input energy released during the earthquake, and should be able to absorb this energy through inelastic deformation.

Performance based design implies that the buildings should be able to resist earthquakes and fulfill the target performance levels. The main advantage of performance based seismic design is that the procedure allows the building owner and the structural engineer to choose both the seismic hazard level and the corresponding performance levels of the building. Performance based design concepts have been proposed in various guidelines such as SEAOC Vision 2000 (1995), ATC–40 (1996), FEMA–356 (2000), FEMA–440 (2005) and ASCE/SEI 41–06 (2007).

In order to assess the building performance, the design codes recommend use of various types of analysis methods (Seifi *et al.* 2008). Generally, four analysis methods based on linear and nonlinear structural response are suggested for the structural analysis of buildings under earthquake loading. Linear analyses methods consist of two different procedures, which are generally called as the response spectrum and the time history methods. In a similar manner, the nonlinear analysis methods are grouped as the pushover and the nonlinear time history analysis procedures. In linear analysis methods, some simplifying assumptions are employed and calculated structural responses may be over conservative causing uneconomic designs. On the other hand, design procedures based on nonlinear analysis are more complex compared to the linear analysis. Nonlinear methods increase the computational cost and demand highly trained engineers as well.

Pushover analysis is a very useful nonlinear structural analysis tool for evaluating the seismic performance of buildings in terms of strength and deformation capacity of the whole structure. This method is based on the assumption that the response of the building is proportional to the response of an equivalent single degree of freedom system with properties linked to the fundamental mode of the building. The sequence of member yielding, inelastic deformation of critical members, maximum interstorey drifts and the possible collapse mechanisms of the buildings can be determined based on pushover analysis results.

In pushover analysis, a mathematical model of the building is established and incremental lateral loads are applied following the application of the initial gravity loads. The lateral load distribution throughout the building height is generally chosen as inverted triangular, uniform or proportional to the fundamental mode shape along the analysis direction (Barros and Almeida 2005). The building model is loaded to monotonically increasing lateral forces using the predefined invariant lateral load pattern. The lateral load is applied incrementally until the lateral displacement of the control node reaches to the displacement demand of the selected earthquake level or when local collapse or storey mechanism takes place. The displacement demand of the earthquake, which is also known as the target displacement, is calculated according to ASCE/SEI 41–06 (2007) as given in Eq. (1):

$$\delta_t = C_0\, C_1\, C_2\; S_a\, \frac{T_e^2}{4\,\pi^2}\, g\,, \tag{1}$$

where T_e is the effective fundamental period of the building in the direction under consideration; S_a is the response spectrum acceleration coefficient corresponding to the effective fundamental period; C_0 accounts for the difference between the roof displacement of an MDOF building and the displacement of the equivalent SDOF system; C_1 is the modification factor to account for the difference between the maximum elastic and inelastic displacement amplitudes in structures; C_2 adjusts design values based on component hysteresis characteristics, stiffness degradation, and strength deterioration. The coefficients for estimating the target displacement in ASCE/SEI 41–06 (2007) are adopted from FEMA–440 (2005). The capacity curve obtained from pushover analysis is converted to an idealized bilinear curve that balances the area below and above the capacity curve, and the yield base shear of the building is determined.

In the present study, lateral load pattern is chosen proportional to the fundamental mode shape of the analyzed direction of the building. This approximation provides satisfactory results for the maximum inter-storey drift and plastic rotation of the members of regular mid-rise buildings.

3. Assessment of Earthquake Damage Cost in Seismic Design of Buildings

Life cycle cost analysis is a suitable tool for assessing the structural performance when the structure is expected to fulfill the aspects of lifetime structural engineering (Bragança and Koukkari 2007). The life cycle cost of a building includes various cost components. Initial cost functions depend on the design intensity. Basic initial costs originate from structural planning and design, building materials, fabrication, transportation, handling, storage of building materials in the construction site, erection, tool operations and machinery on the construction site, excavation work in the project site including the foundations. Nonstructural component costs, such as partitioning walls, are neglected in initial cost calculations. Life cycle cost includes several components such as maintenance, inspection, repair, operating, damage and demolition costs in addition to initial costs (Bozdağ and Seçer 2007).

In recent years, the limit state cost analysis, an important part of the life cycle cost, has become more significant for minimizing the cost of the seismic hazards. The limit state cost is the potential damage cost from earthquakes that may occur during the lifespan of the building. Limit state cost is only related with earthquake damages and disregards other expenses such as maintenance or any type of nonstructural costs.

The limit state cost mainly includes damage, loss of contents, relocation, economic loss as a sum of rental and income loss, injury, human fatality costs, and other direct or indirect economic losses related to earthquake hazard. Calculation procedures of limit state costs are outlined in ATC–13 (1985), FEMA–227 (1994) and FEMA–228 (1994) documents. Earthquake induced damage cost is the focus of this study and other cost components are neglected in order to monitor the damage cost effect on the total cost. The expected life cycle cost function under a single hazard can be calculated by the formula given in Eq. (2) (Wen and Kang 2001):

$$E[C(t,x)] = C_0 + (C_1 P_1 + \ldots + C_k P_k)\frac{\upsilon}{\lambda}\left(1 - e^{-\lambda t}\right), \quad (2)$$

where: P_k is the probability of the k^{th} damage state violation given the occurrence of earthquake and C_k is the corresponding cost; C_0 is the initial cost; λ is the annual momentary discount rate and assumed as constant; υ the annual occurrence rate of significant earthquakes; and t is the service life of a building.

Earthquake induced damage states of a building are simply classified according to the maximum interstorey drift ratio. Performance levels corresponding to each damage state are listed in Table 1 (ATC–13 1985).

Table 1. Performance levels and damage costs of a building in terms of interstorey drift ratio (ATC–13 1985)

Performance level	Damage state	Interstorey drift ratio (%)
I	None	$\Delta<0.2$
II	Slight	$0.2<\Delta<0.5$
III	Light	$0.5<\Delta<0.7$
IV	Moderate	$0.7<\Delta<1.5$
V	Heavy	$1.5<\Delta<2.5$
VI	Major	$2.5<\Delta<5$
VII	Destroyed	$\Delta>5$

The probability of each damage state is calculated with following Eq. (3):

$$P_i = P_i(\Delta > \Delta_i) - P_{i+1}(\Delta > \Delta_{i+1}). \quad (3)$$

According to Poisson's law, the annual probability of exceedance of an earthquake is given by Eq. (4) (Wen 2001):

$$P_i(\Delta > \Delta_i) = (-1/t)\ \ln\left(1 - \overline{P}_i(\Delta > \Delta_i)\right), \quad (4)$$

where $\overline{P}_i(\Delta > \Delta_i)$ is the annual exceedance probability of the maximum interstorey drift value Δ_i. The annual exceedance probability of the i^{th} damage state is obtained using Eq. (5):

$$\overline{P}_i(\Delta - \Delta_i) = a\, e^{-b\Delta_i}. \quad (5)$$

The parameters a and b are obtained via regression analysis. These pairs correspond to the earthquakes with probability of exceedance 2%, 10%, and 50% in 50 years.

4. Numerical Example

The effect of X-bracing configuration is investigated on a five storey concentrically X-braced steel office building. The base area dimensions of the steel building are 30.00 m × 24.00 m and the storey height for each floor is 3.00 m. Since, there are many X-bracing configurations, considering structural and architectural concerns for a steel building, three different X-bracing configurations are investigated in order to keep the content of the text concise. The structural steel building models with three different X-bracing configurations and the typical floor plans are shown in Figs 1 and 2, respectively. In the first model X-braces are placed in between the 3rd and 4th axes (Fig. 1a), whereas they are located between 1–2 and 5–6 axes in the second case (Fig. 1b) along East–West (E–W) direction. Finally, braces are between 1–2, 3–4 and 5–6 axes in the third model as shown in Fig. 1c. X-braces are placed between A–B and D–E axes along North–South (N–S) direction for all cases. One should note that X-braces are located only on the outermost frames for both directions (i.e. E–W and N–S). In order to avoid torsional irregularity, X-braces are positioned symmetrically in plan. The beam-to-column connections are modeled perfectly rigid as recommended in TERDC (2007) for satisfying high ductile behavior. Foundation is assumed as rigid foundation and modeled with fixed supports.

a)

b)

c)

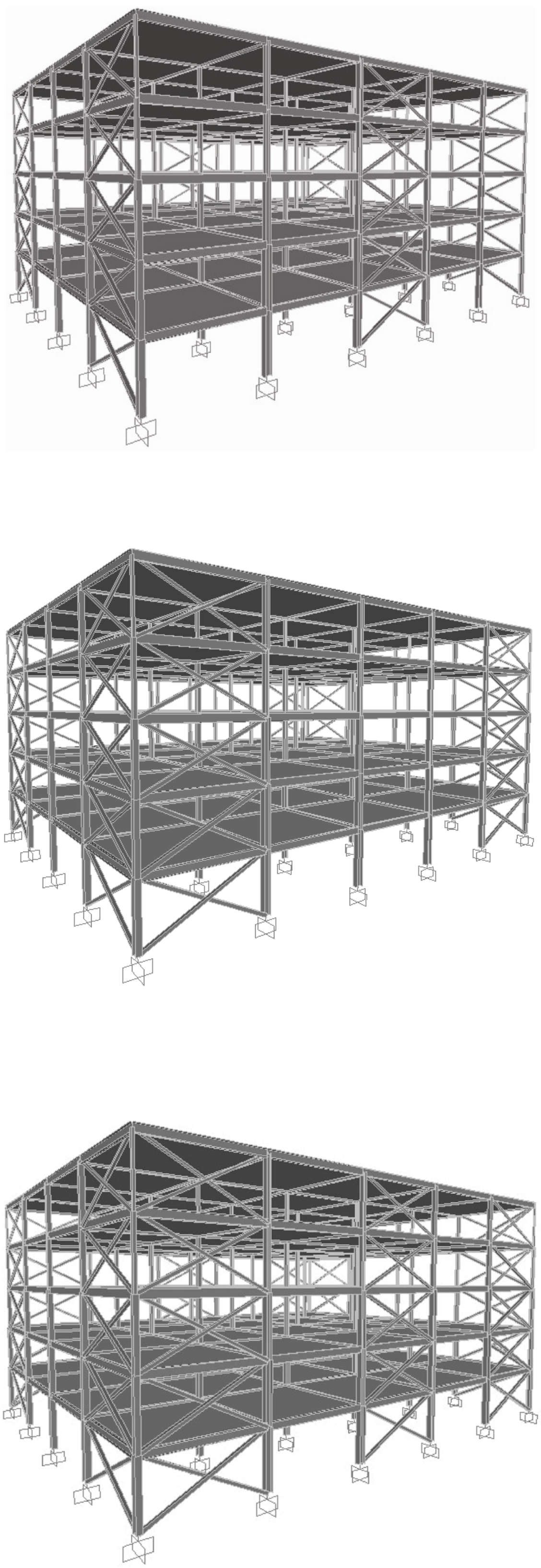

Fig. 1. 3D view of concentrically X-braced five storey steel building: a) configuration–I; b) configuration–II; c) configuration–III

a)

b)

c)

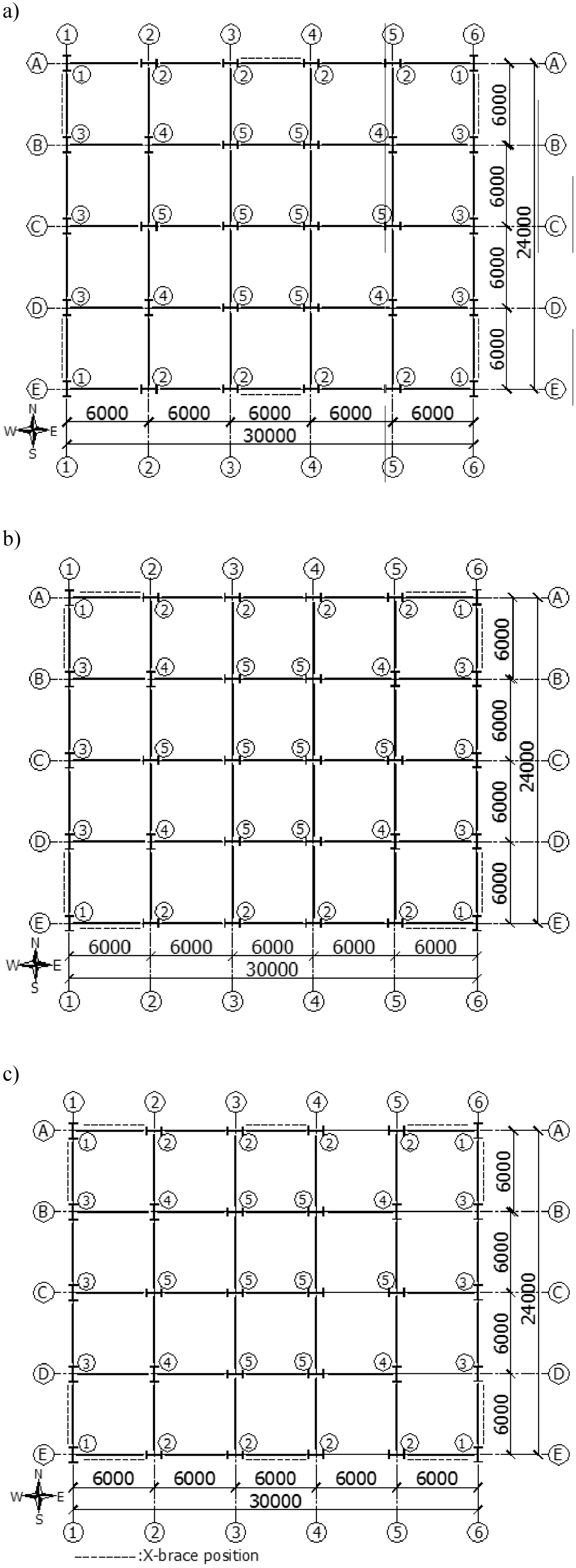

Fig. 2. Storey plan of five storey steel moment resisting frame building: a) configuration–I; b) configuration–II; c) configuration–III

The frame sections used for the designs of the steel buildings consist of IPE and HEB profiles. Floor beams are assigned as IPE profiles and all the beams of one floor are assumed to have the same section type. On the other hand, there are five groups of HEB profile columns corresponding to each design case which are given for a particular X-bracing configuration in Tables 2, 3 and 4. All members of the steel buildings are designed according to provisions of TS-648 (1980) and TERDC (2007). The modulus of elasticity for steel is 210 GPa and the yield stress is 235 MPa. In all cases, X-braces are designed to sustain large inelastic deformations without experiencing premature buckling failures. Reinforced concrete is used in floor slabs and live loads are considered as 2.0 kN/m^2.

Table 2. Column sections for each design of X-bracing configuration–I

Storey	Group	Design A	Design B	Design C	Design D	Design E	Design F
1	Column 1	HEB180	HEB200	HEB200	HEB260	HEB320	HEB450
	Column 2	HEB240	HEB240	HEB260	HEB360	HEB650	HEB800
	Column 3	HEB240	HEB200	HEB240	HEB300	HEB400	HEB650
	Column 4	HEB260	HEB240	HEB280	HEB280	HEB320	HEB320
	Column 5	HEB260	HEB240	HEB260	HEB280	HEB300	HEB300
2	Column 1	HEB180	HEB200	HEB200	HEB240	HEB260	HEB300
	Column 2	HEB240	HEB240	HEB260	HEB300	HEB450	HEB500
	Column 3	HEB220	HEB200	HEB240	HEB280	HEB320	HEB360
	Column 4	HEB240	HEB240	HEB260	HEB280	HEB200	HEB300
	Column 5	HEB220	HEB240	HEB240	HEB240	HEB260	HEB280
3	Column 1	HEB160	HEB 180	HEB180	HEB200	HEB220	HEB240
	Column 2	HEB240	HEB240	HEB260	HEB280	HEB300	HEB400
	Column 3	HEB220	HEB180	HEB240	HEB260	HEB280	HEB300
	Column 4	HEB220	HEB240	HEB220	HEB240	HEB260	HEB280
	Column 5	HEB200	HEB240	HEB200	HEB220	HEB240	HEB260
4	Column 1	HEB140	HEB140	HEB160	HEB160	HEB160	HEB180
	Column 2	HEB220	HEB240	HEB240	HEB260	HEB260	HEB300
	Column 3	HEB220	HEB140	HEB220	HEB240	HEB260	HEB280
	Column 4	HEB180	HEB240	HEB180	HEB200	HEB220	HEB220
	Column 5	HEB180	HEB200	HEB180	HEB200	HEB240	HEB220
5	Column 1	HEB120	HEB120	HEB120	HEB160	HEB160	HEB140
	Column 2	HEB180	HEB200	HEB200	HEB220	HEB240	HEB240
	Column 3	HEB180	HEB120	HEB180	HEB200	HEB220	HEB220
	Column 4	HEB140	HEB200	HEB140	HEB140	HEB160	HEB160
	Column 5	HEB140	HEB200	HEB140	HEB140	HEB140	HEB160

Table 3. Column sections for each design of X-bracing configuration–II

Storey	Group	Design A	Design B	Design C	Design D	Design E	Design F
1	Column 1	HEB180	HEB200	HEB200	HEB240	HEB280	HEB340
	Column 2	HEB240	HEB240	HEB240	HEB280	HEB360	HEB700
	Column 3	HEB240	HEB240	HEB260	HEB300	HEB400	HEB800
	Column 4	HEB260	HEB260	HEB260	HEB280	HEB300	HEB300
	Column 5	HEB260	HEB260	HEB260	HEB260	HEB280	HEB300
2	Column 1	HEB180	HEB180	HEB200	HEB220	HEB240	HEB280
	Column 2	HEB240	HEB240	HEB240	HEB260	HEB320	HEB400
	Column 3	HEB240	HEB240	HEB260	HEB280	HEB320	HEB400
	Column 4	HEB240	HEB240	HEB240	HEB240	HEB260	HEB280
	Column 5	HEB240	HEB240	HEB240	HEB240	HEB260	HEB280
3	Column 1	HEB160	HEB180	HEB180	HEB200	HEB220	HEB220
	Column 2	HEB220	HEB240	HEB240	HEB240	HEB280	HEB300
	Column 3	HEB220	HEB240	HEB240	HEB260	HEB280	HEB300
	Column 4	HEB200	HEB200	HEB200	HEB220	HEB240	HEB240
	Column 5	HEB200	HEB200	HEB200	HEB220	HEB220	HEB240
4	Column 1	HEB160	HEB140	HEB140	HEB160	HEB160	HEB160
	Column 2	HEB220	HEB220	HEB240	HEB220	HEB240	HEB260
	Column 3	HEB220	HEB240	HEB240	HEB240	HEB240	HEB280
	Column 4	HEB180	HEB180	HEB180	HEB180	HEB200	HEB200
	Column 5	HEB160	HEB160	HEB180	HEB180	HEB200	HEB200
5	Column 1	HEB120	HEB120	HEB120	HEB120	HEB120	HEB120
	Column 2	HEB200	HEB200	HEB200	HEB220	HEB220	HEB240
	Column 3	HEB180	HEB180	HEB220	HEB220	HEB220	HEB240
	Column 4	HEB140	HEB140	HEB140	HEB140	HEB140	HEB160
	Column 5	HEB140	HEB140	HEB140	HEB140	HEB140	HEB160

Table 4. Column sections for each design of X-bracing configuration–III

Storey	Group	Design A	Design B	Design C	Design D	Design E	Design F
1	Column 1	HEB180	HEB200	HEB200	HEB240	HEB280	HEB340
	Column 2	HEB240	HEB240	HEB240	HEB280	HEB320	HEB550
	Column 3	HEB240	HEB240	HEB260	HEB300	HEB400	HEB800
	Column 4	HEB260	HEB260	HEB260	HEB280	HEB300	HEB300
	Column 5	HEB260	HEB260	HEB260	HEB260	HEB280	HEB300
2	Column 1	HEB180	HEB180	HEB200	HEB220	HEB240	HEB280
	Column 2	HEB240	HEB240	HEB240	HEB260	HEB300	HEB320
	Column 3	HEB240	HEB240	HEB260	HEB280	HEB320	HEB400
	Column 4	HEB240	HEB240	HEB240	HEB240	HEB260	HEB260
	Column 5	HEB240	HEB240	HEB240	HEB240	HEB260	HEB280
3	Column 1	HEB160	HEB180	HEB180	HEB200	HEB220	HEB220
	Column 2	HEB220	HEB240	HEB240	HEB240	HEB280	HEB300
	Column 3	HEB220	HEB240	HEB240	HEB260	HEB280	HEB300
	Column 4	HEB200	HEB200	HEB200	HEB220	HEB240	HEB240
	Column 5	HEB200	HEB200	HEB200	HEB220	HEB220	HEB240
4	Column 1	HEB140	HEB140	HEB140	HEB160	HEB160	HEB160
	Column 2	HEB220	HEB220	HEB240	HEB220	HEB240	HEB260
	Column 3	HEB220	HEB240	HEB240	HEB240	HEB240	HEB280
	Column 4	HEB180	HEB180	HEB180	HEB180	HEB200	HEB200
	Column 5	HEB160	HEB160	HEB180	HEB180	HEB200	HEB200
5	Column 1	HEB120	HEB120	HEB120	HEB120	HEB120	HEB120
	Column 2	HEB200	HEB200	HEB200	HEB220	HEB220	HEB240
	Column 3	HEB180	HEB180	HEB200	HEB220	HEB220	HEB240
	Column 4	HEB140	HEB140	HEB140	HEB140	HEB140	HEB160
	Column 5	HEB140	HEB140	HEB140	HEB140	HEB140	HEB160

A commercially available finite element structural analysis software suit is used in pushover analysis (CSI 2007). Total lateral load is applied in increments until the lateral displacement reaches the demand of the selected earthquake level. Since mass participation factor in both directions are greater than 75%, the lateral load distribution is taken proportional to fundamental mode of the building throughout the building height. Inelastic deformations at structural members are modeled as plastic hinges at both ends of beam and column elements. Plastic hinges are represented with ideal elastic-perfectly plastic material model. The pushover curves of the six design cases for each X-bracing configuration are given in Fig. 3 to Fig. 5. The pushover curves are idealized as bilinear curves with a horizontal post-yield branch that balances the area below and above the capacity curves. The yield base shears of the buildings are calculated. The displacement demand of the earthquake, which is also called as the target displacement, is obtained using Eq. (1).

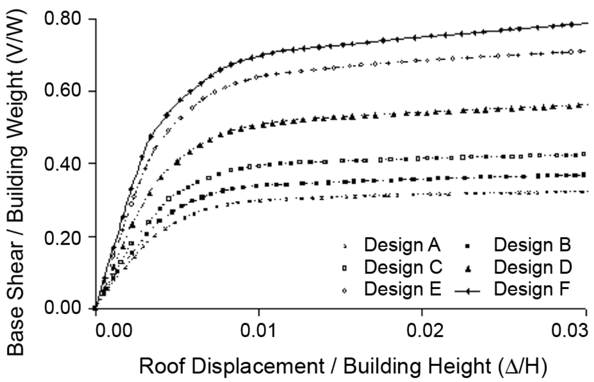

Fig. 3. Static pushover curves of configuration–I

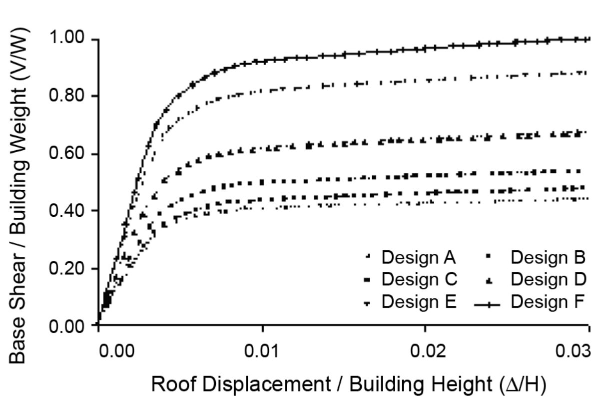

Fig. 4. Static pushover curves of configuration–II

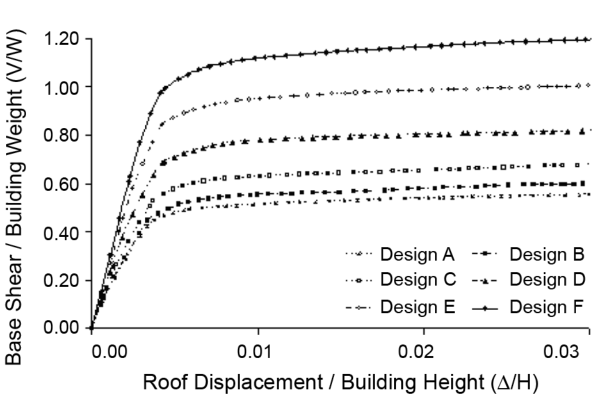

Fig. 5. Static pushover curves of configuration–III

The calculation of the total cost function for each design case is done separately. Initial costs are calculated using unit cost values of Turkish Ministry of Public Works and Settlement (MPWS 2009). The annual momentary discount rate which is used to calculate the value of benefits or costs that will occur in the future is generally accounted as 4–6% for private sector according to FEMA 227 (1994) and is accepted 9% for Turkish market as a reasonable value. The earthquake damage cost function calculation steps are given in detail for Design E of configuration–II in order to show the procedure in detail. Based on pushover analysis results, three pairs of maximum inter-storey drifts are obtained and the annual probability of exceedance of an earthquake with a probability of exceedance 2%, 10% and 50% in 50 years are calculated as $\overline{P}_{2\%}=0.000404$, $\overline{P}_{10\%}=0.0021$, $\overline{P}_{50\%}=0.0139$, respectively. Using the maximum interstorey drifts and annual probability of exceedance values, $(\Delta_i-\overline{P}_i)$ pairs corresponding to the three hazard levels with the given annual probabilities of exceedance are used to get the curve by means of an exponential function which is obtained by performing curve fitting. Once the function of the curve is plotted, annual probabilities of exceedance for seven damage states can be interpolated by using Fig. 6 for Design E of configuration–II. Besides, the results are substituted in Eq. (2) to calculate the values of total cost functions for each design cases separately.

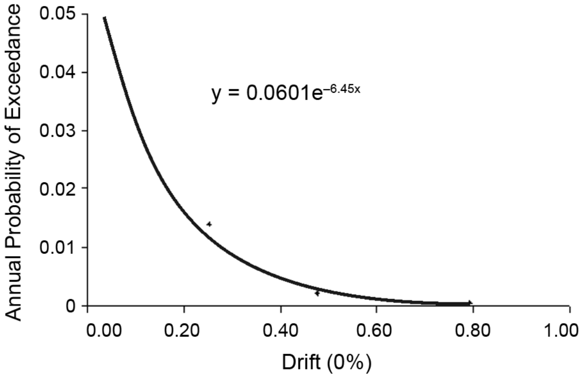

Fig. 6. Annual probability of exceedance for each damage state for Design E of configuration–II

Total expected damage cost is equal to the sum of cost functions multiplied by the corresponding limit state probabilities. The system force coefficient S_y is calculated as the ratio of total base shear corresponding to the target displacement of the building to the total weight of the building. Total expected earthquake damage costs are calculated for each design case of X-bracing configurations as shown in Fig. 7. In order to obtain the system force coefficient value, it is required to estimate the minimum total cost. For this purpose, total costs are determined for each design case of X-bracing configurations as in Fig. 8. Curve fitting is performed in order to achieve the optimum building design for X-bracing configurations. General form of the fitting curve and curve coefficients are given in Table 5. The optimum system force coefficient S_y and optimum total cost values for each X-bracing configuration are calculated by equaling the first derivatives of the fitted curves to zero and given in Table 6.

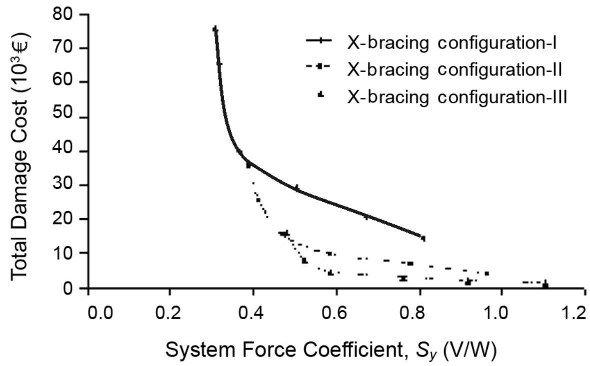

Fig. 7. Total expected damage cost as a function of system force coefficient for each X-bracing configuration €

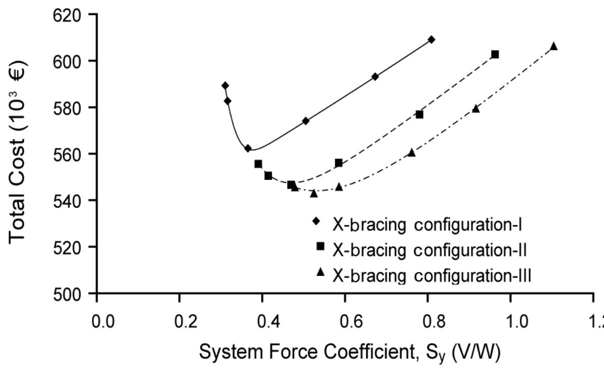

Fig. 8. Total expected cost as a function of system force coefficient for each X-bracing configuration

Table 5. Curve fitting function and coefficients

$$\text{Total Cost}(€)=\left[a_1 e^{-\left(\frac{x-b_1}{c_1}\right)^2}+a_2 e^{-\left(\frac{x-b_2}{c_2}\right)^2}\right]\times 10^3$$

X-brace Config.	a_1	b_1	c_1	a_2	b_2	c_2
I	151	0.222	0.074	1.593 E+6	81.810	28.880
II	3850	17.620	12.230	3.976 E+16	–4.552	0.836
III	1601	9.300	8.311	2.234 E+16	–8.963	1.611

Table 6. Optimum system force coefficient and optimum total cost values

	X-brace configuration		
	I	II	III
Optimum S_y	0.376	0.470	0.530
Optimum Total Cost x 10^3 (€)	562	547	544

5. Conclusions

Structural engineers tend to design cost-effective seismic-resistant buildings that favorably establish balance between initial investments and future seismic risk. However, designers are often challenged by making a decision of either designing with the least initial expense limited by the maximum acceptable risk or finding a design solution with the lowest risk measure without exceeding a certain initial investment budget. When the total cost curve for a particular design case is plotted, the engineer will be able to judge the design in an economic point of view. The results of such an analysis can be quickly grasped by the public and the building owners since both the reliability and the performance of a structure can be demonstrated in economic terms.

Effect of X-bracing configuration on earthquake damage cost is investigated on a five storey steel building with three different X-bracing configurations. These configurations are arranged in an increasing X-bracing intensity in that X-braces are placed only in the middle span, first and last spans and finally first, middle and last spans of the E-W façade, respectively. Pushover analyses are performed for estimating the earthquake damage cost of each design case based on maximum inter-storey drift ratio for different earthquake intensities.

The earthquake damage cost analysis results reveal that X-bracing configuration–I is more sensitive to system force coefficient in the manner of damage cost than the other X-bracing cases. For the X-bracing configurations I, II and III; the damage cost differences between Design A and F are 62, 32 and 15 thousand euros respectively. Increasing X-bracing intensity consequences to reducing earthquake induced damage cost as well. For the 0.50 system force coefficient, the damage cost values are 29, 14 and 12 thousand euros for the X-bracing configurations I, II and III. On the other hand, optimum point of the total cost curve is obvious for X-bracing configuration–I and the total cost is highly affected from design system force coefficient. When the X-bracing intensity increases the optimum point becomes ambiguous and enhancing the system force coefficient does not have significant influence on the total cost. In conclusion, it is observed from the damage cost analysis that optimum total cost value corresponding to optimum design system force coefficient reduces with the increase in X-bracing intensity.

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KRYŽMINIŲ RYŠIŲ PAVIDALO POVEIKIS PLIENINIŲ KONSTRUKCIJŲ PASTATO APGADINIMO KAINAI DĖL ŽEMĖS DREBĖJIMO

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Santrauka

Pasitelkus gyvavimo ciklo kainos analizę, plieninių konstrukcijų pastatų su kryžminiais ryšiais seisminio konstrukcijų projektavimo tikslas – rasti tinkamiausią konstrukcinį sprendimą, kuris atitiktų ekonominę pusę, ir žemės drebėjimui atsparių statinių projektavimo kodekso reikalavimus, kai, atsižvelgiant į architektūrines sąsajas, yra daugybė sprendimų variantų. Šiame tyrime, naudojant įvairias pagrindų šlyties jėgų reikšmes, projektuojamas penkiaaukštis plieninių konstrukcijų pastatas su trimis skirtingais kryžminių ryšių pavidalais ir kiekvienam atvejui iš trijų pavidalų apskaičiuojama bendroji kaina, esant skirtingo stiprumo žemės drebėjimui. Pradinė kaina ir numatomos būsimų žemės drebėjimų padaryto apgadinimo kaina nustatoma kiekvienam kryžminių ryšių pavidalui. Siekiant laikytis žemės drebėjimų kodekso reikalavimų, kaip seisminių charakteristikų rodiklis pasirenkamas didžiausias tarpaukštinės slinkties santykis, kuris įvertinamas naudojant netiesinę statinę analizę. Optimalus kryžminių ryšių pavidalas nustatomas subalansuojant pradinę kainą ir per visą gyvavimo trukmę žemės drebėjimų padarytos žalos kainą.

Reikšminiai žodžiai: X pavidalo ryšiai, plieninis pastatas, netiesinė statinė analizė, šoninė slinktis, žemės drebėjimų padarytos žalos kaina, optimali bendra kaina, optimalios sistemos jėgos koeficientas.

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