Uludağ University Journal of The Faculty of Engineering, Vol. 25, No. 1, 2020 RESEARCH

DOI: 10.17482/uumfd.670347

SEISMIC PERFORMANCE ASSESSMENT OF PREFABRICATED INDUSTRIAL BUILDINGS WITH SEMI-RIGID CONNECTIONS

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Received: 04.01.2020; revised: 25.02.2020; accepted: 28.03.2020

Abstract: In this study, the performances of 120 precast factory buildings consisting of five different types in an industrial area in Istanbul, Turkey are examined before and after the applications of retrofit methods through nonlinear time history analyses. The factory buildings were constructed and became operational before the 1999 Kocaeli earthquake. Following the earthquake, they were operationally safe, however, after the evaluation according to the 1998 Turkish Earthquake Code (TEC), most of the connections were found to be inadequate. The buildings were retrofitted by using bolted steel plates in the connection regions and diagonal steel braces in 2005. A detailed finite element model of the connection was developed before and after the retrofit methods and the behavior of the region was implemented in the three-dimensional models of the structures. The buildings were analyzed according to the Turkish Earthquake Code 2007. The number of damaged beams by the TEC-2007 was higher than the rate of damaged beams per the FEMA-356. In three-dimensional performance analysis of buildings, the importance of examination of connection regions and implementing the results to the full building model were emphasized. The performance of the building were increased by reducing the distance between the plane frames in the longitudinal direction and the beam length in the transverse direction.

Keywords: Precast factory building, Beam-column connection, Finite element model, Nonlinear behavior, Performance assessment

Yarı-Rijit Bağlı Prefabrik Endüstriyel Binaların Deprem Performansının Değerlendirilmesi

Öz: Bu çalışmada, İstanbul'da bir sanayi bölgesinde beş farklı tipten oluşan 120 adet prefabrik fabrika binasının performansları güçlendirme yöntemlerinin uygulanmasından önce ve sonra, zaman tanım alanında doğrusal olmayan analizler ile incelenmiştir. Fabrika binaları 1999 Kocaeli depreminden önce inşa edilmiş ve çeşit endüstrilerde faaliyete geçmiştir. Kocaeli depreminin ardından binalar işletme açısından güvenli kabul edilmiş, ancak 1998 Türk Deprem Yönetmeliği (TDY) koşulları açısından değerlendirildikten sonra kiriş-kolon bağlantılarının çoğunun yetersiz kaldığı tespit edilmiştir. Binalar 2005 yılında birleşim bölgelerinde bulonlu çelik levhalar ve yapı genelinde diyagonal çelik çubuklar kullanılarak güçlendirilmiştir. Bu çalışmada farklı güçlendirme yöntemlerinin uygulanmasından önce ve sonra kiriş-kolon bağlantı bölgesinin detaylı bir sonlu eleman modeli geliştirilmiş ve bölgenin davranışı yapıların üç boyutlu modellerinde dikkate alınmıştır. Türk Deprem Yönetmeliği 2007 kurallarına uygun olarak fabrika binaları analiz edilmiştir. Güçlendirme tekniklerinin ve genel plan boyutlarının performans seviyeleri üzerindeki etkileri tespit edilmiştir. Analizler sonucunda, TDY 2007 kriterlerine göre hasar gören kiriş elemanları sayısı FEMA-356 koşullarına göre hesaplanan elemanlardan daha fazla bulunmuştur. Yapıların üç boyutlu performans analizlerinde, birleşim bölgelerinin detaylı incelenmesi ve tüm yapı modeline davranışın aktarılması gerektiğinin önemi vurgulanmıştır. Yapıların boyuna doğrultuda düzlem çerçeveler

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arası uzunluğunun ve kısa doğrultudaki kiriş uzunluklarının azaltılması ile yapı performansının arttığı görülmüştür.

Anahtar Kelimeler: Prefabrik endüstriyel bina, Kiriş-kolon birleşimi, Sonlu elemanlar modeli, Doğrusal olmayan davranış, Performans değerlendirme

1. INTRODUCTION

Precast concrete structures have fundamental advantages over cast-in-place reinforced concrete structures: increased speed of construction, improved quality control, efficient use of materials, and reduced site formwork and labor (Yee et al., 2011). After establishing the Prestressed Concrete Institute in 1954, many buildings and structures such as parking garages, office buildings, and bridges have been designed and built using precast/prestressed concrete in a vast number of countries (Xue and Yang, 2010). After the 1994 Northridge and 1995 Kobe earthquakes, however, the failure of the precast concrete beam to column connections caused significant concerns for these systems in regions of high seismicity. Field observations in the aftermath of the 1998 Adana-Ceyhan (Adalier and Aydingun, 2001) and 1999 Kocaeli and Duzce earthquakes (Saatcioglu et al., 2001; Arslan et al., 2006, Sezen and Whittaker, 2006) also showed that inadequate stiffness and strength and/or problems caused by insufficient connection detailing raised the major structural deficiencies in RC precast industrial facilities. Posada and Wood (2002) reported that the damage level of the industrial buildings are mainly influenced by the drift demand and the drift capacity of the buildings. The damage of precast structures in Sakarya was more pronounced due to the drift demand invoked by the soft soil conditions, as compared to the no damage in Gebze where the soil is basically stiff clay to rock.

Several different types of connections have been used for the last two decades to connect the beams to the columns, including welded plate and billet, high-performance fiber-reinforced cement composite, hybrid post-tensioning frame, precast reinforced concrete, and specially developed bolted connection systems (Elliot et al., 1998; Vasconez et al., 1998; Priestley et al., 1999; Vidjeapriya and Jaya, 2013). The steps of the proper design and detailing of connection system are well defined in the literature (ACI 318R-95; NEHRP 2001; Fib 2003; Englekirk, 2003; Bournas et al., 2013; Bournas et al., 2014). Also, according to the TEC (2007) requirement, all moment-resisting connections in precast concrete structures should maintain similar rigidity, strength, energy dissipation and ductility with the monolithic system behavior under cyclic loadings occurred during earthquakes.

Considerable amount of experimental and analytical studies have been carried out to investigate the seismic performance of precast beam-column joints subjected to reversed cyclic loading simulating severe earthquake action and develop retrofitting techniques for vulnerable existing precast joints (Baysal, 1991; Ersoy and Tankut, 1993; Oztug, 1994).

Two types of structural damage were frequently observed: flexural hinges at the base of the columns and pounding of the precast elements at the roof level after 1999 Kocaeli and Duzce earthquakes (EERI, 2000). In contrast to the precast frames used in Turkey, concrete tilt-up wall panels are commonly used to construct one-story industrial structures in the US. The seismic performance of low-rise industrial buildings in Turkey and the US were compared. Although the structural systems were very different, both are vulnerable to earthquake damage. Rehabilitation schemes used in southern California to improve the performance of tilt-up structures were proposed as an option for Turkish construction (Wood, 2003).

The full-scale test results of three-story precast building studied in the framework of the SAFECAST project were presented (Negro et al., 2013; Bournas et al., 2013). The effect of two types of beam–column connections on the seismic behavior was evaluated. They were the pinned beam–column joints and a new connection system with dry connections. It was concluded that the new beam-to-column connection system is a viable solution toward enhancing the response of precast RC frames subjected to seismic loads, in particular when the system is applied to all joints and quality measures are enforced in the execution of the joints.

The main vulnerabilities observed in precast industrial buildings during Emilia 2012 earthquakes (Belleri et al., 2015) were related to inadequate connection strength and ductility or to the absence of mechanical devices. The most severe type of damage was the loss of support and consequent fall of structural and non-structural members. Seismic performance of industrial facilities in Italian territory was investigated according to European Building Code and retrofit solutions were proposed to improve the local and global response of precast industrial buildings.

Two distinct exterior beam–column connections made of normal-strength concrete were subjected cyclic displacement reversals in order to obtain information on strength, stiffness and ductility characteristics of the connection details as a part of SAFECAST at Structural and Earthquake Engineering Laboratory of Istanbul Technical University (ITU). The preliminary design of the joints has been updated during the tests based on the damages observed, thus a set of improved specimens have also been built and tested, and a relatively better performance is obtained expectedly. The industrial and residential types of connections showed stable load–displacement cycles with high energy dissipation up to structural drift of 2%, though a significant level of pinching and deterioration of the critical section have occurred at around 3% drift level. The tested specimens have been numerically modeled to calibrate the analytical tools, and a satisfactory approximation has been obtained between experimental and numerical results (Yuksel et al., 2015).

A new dry mechanical beam-column joint for fully restrained moment connections of precast concrete frames were proposed and tested under cyclic loadings (Nzabonimpa et al., 2017). The new joint consisted of extended steel plates with bolts designed to transfer tension and compression forces, providing fully restrained moment connection at the joint. Experimental and analytical investigations were performed to verify the structural behavior of the connections for concrete components. The connections were expected to be utilized in modular offsite constructions. In the later study, detailed nonlinear finite element analyses of the mechanical joint were performed to reproduce the experimental response of the joints (Nzabonimpa et al., 2017). Nonlinear finite element model of the precast beam column joints was based on concrete damaged plasticity. The results of the analyses uncovered the failure modes including the deformations of beam end plates and the concrete damage of the novel connections. Experimental investigations of three full-scale test specimens and analytical results were in good agreement despite its complex geometry and complexity of the mechanical joint.

The main objective of the study is to investigate the performances of 120 precast structures consisting of five different types in an industrial area in Istanbul, Turkey before and after the applications of retrofit methods. Detailed FE models of the welded beam-column connection region of the structures are developed before and after retrofit and the results are incorporated as an input to represent the joint region in the three-dimensional (3-D) models of the precast concrete structures. Two different strengthening methods applied to the precast structures after the 1999 Kocaeli earthquakes are taken into account to decrease the effect of the primary and secondary forces acting on the connection regions and consequently to reduce the maximum deformation of the precast structures. Nonlinear time-history analyses are conducted to observe the behavior and consequently the performance of the structures before and after the retrofit according to deformation limits defined in the TEC-2007. The effects of the retrofitting techniques and the overall plan dimensions on the performance levels are quantified. The performance levels before and after retrofitting are also compared and discussed. Based on the findings of this study, recommendations to the designer for the selection of plan dimensions and span lengths are provided based on the differences between the structures.

2. EVALUATION OF THE NON-RETROFITTED PRECAST STRUCTURES

2.1. Geometry and Reinforcement Detailing

Five different types of bare/non-retrofitted precast structures are investigated in this part of the study (Fig. 1). The structure types are denoted as A, B, and C depending on the number of span along the transverse directions of the buildings. Details related to these structures are given in Table 1. The structures generally consisted of one mezzanine floor in the production part of the building and two mezzanine floors in the office areas. The columns of all investigated structures lay on individual spread footings which are connected using cast-in-place tie beams both in the longitudinal and transverse directions. The structures are located in the first seismic zone having an effective strong motion coefficient of 0.40, and the soil class is Z2 according to TEC-2007, which is similar to class B in FEMA 356 (2000) guideline.

Figure 1:
Google image of the studied area (Type A1, A2, B1, B2, and C)

Table 1. Structural properties of the precast buildings

	Type A1	Type A2	Type B1	Type B2	Type C
Number of span in X direction	3	3	2	2	4
Number of span in Y direction	7	6	4	5	7
Span length in X direction (m)	6.67	6.67	5.00	7.00	7.50
Span length in Y direction (m)	7.5	7.5	7.5	6.6	7.5
Story Height (m)	11.0	10.2	7.0	10.4	10.5

A typical plane frame in the investigated precast structures consists of a single beam and two columns as shown in Fig.2. All columns and beams have the same cross-sectional dimensions of

0.30×0.55 m and 0.30×0.70 m in all the structures, respectively. Only at the corbel region, the beams' cross-sectional dimensions are reduced to 0.30×0.40 m. Each column has 16 longitudinal reinforcements which consist of 4Ø20, 4Ø22, 4Ø24, and 4Ø26 mm bars that all result in 4% reinforcement ratio. At both ends of the beams and the upper face of the corbels, steel plates were anchored. Through welding these plates, the precast beams and columns were joined during the construction. In the structures studied, precast columns with corbels were connected to prefabricated beams by welding these steel plates between the corbels and the beams. Roof trusses are composed of beams with cross-sectional dimensions of 0.30×0.50 m. Along the Y directions of the structures, the roof trusses are connected using ridge beams. The roof trusses are connected to the columns by using 2Ø22 mm bolts in the connection regions as shown in Fig. 3. The flooring systems consist of ready-made 0.15 m thick hollow-core precast slabs which are standing freely on the upper faces of the beams. Additionally, 0.05 m concrete topping was poured during the construction.

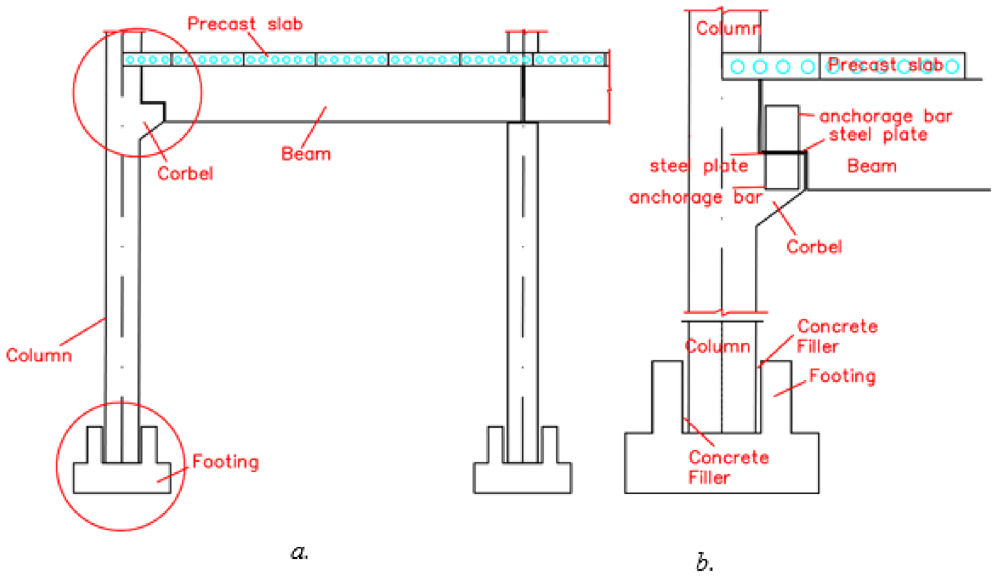

Figure 2:
A typical plane frame in precast structures ***a.*** *Whole view* ***b.*** *Details of the connections*

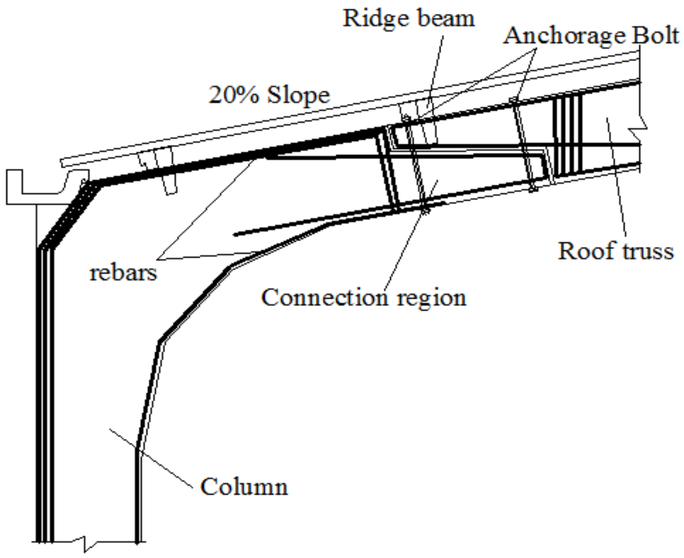

Figure 3:
Connection region of the roof truss

2.2. Materials

In the projects of the precast concrete structures, the characteristic compressive strength, elasticity modulus and density of the concrete are given as 30 MPa, 32 GPa and 2400 kg/m^3, respectively. The yield stress, elasticity modulus and density of the reinforcing steel are provided as 420 MPa, 200 GPa and 7850 kg/m^3, respectively. Yield strength and elasticity modulus of the steel plates used for anchorage and the bolts are 360 MPa and 200 GPa, respectively. The characteristic material properties are taken into account in the analyses.

2.3. Loadings

Dead and live loads were considered in the design of the structures. These loads were calculated as per the requirements of the Turkish Standard (TS-498, 1997). Live loads on the office floors and production levels and the roof and roof cover loads were taken as 350, 500, 80 and 8 kg/m^2, respectively.

2.4. Modeling of the Connection Region

A proper behavioral representation of the connection region is needed since the actual behavior of the region considerably affects the overall behavior of the precast concrete structures under lateral loading. In this context, the beam-column joint region of the considered structures is defined in detail to obtain the moment-rotation relation for the beam in the connection area using ABAQUS FE (2015) program. The model consists of a column, a beam, two steel anchorage plates and the weld around the edges of the plates. The longitudinal reinforcement and transverse reinforcement in the beam are also modeled. In the finite element model, the anchorage steel plates overlapped each other are connected to the upper face of the corbel and the lower face of the beam end by welding the edges of the plates as shown in Fig. 4. The concrete material in the column and beam members is modeled using the Damage-Plasticity Model implemented in ABAQUS program as this model can represent the inelastic behavior of concrete both in tension and compression as well as damage characteristics. In this material model, concrete tension stiffening and concrete compression hardening properties are used. Also, the concrete tension and compression damage options are defined to specify tensile and compressive stiffness degradation damage according to the program manual. A biaxial steel model is defined to simulate the behavior of both the longitudinal reinforcing and structural steel. In this model, the isotropic hardening rule is followed under cyclic loading.

The column, beam, steel plates and welding are modeled using 3D continuum 8-node brick elements, whereas longitudinal reinforcement and transverse steel hoops in the beam element are defined using 1D 2-node linear truss elements. Approximately 9000 elements are utilized in the model. Both ends of the column are defined as fixed. The free end of the beam is unconstrained, whereas the other end where the beam meets the column is constrained. Initially, there is a 5 mm gap between the column corbel and the beam. Hard contact interaction is defined between the corbel and beam surface after the 5 mm gap is closed due to bending. Uniform pressure surface load on the top of the beam and uniform shear load on the cross-section of the beam are obtained under the static loading of the 3-D FE model of the structures in SAP2000 and applied in the ABAQUS FE model. The deformed shape of the connection region is given in Fig. 5 after the loadings. First, the vertical displacement at the free end of the beam is obtained from the static analysis of the connection region. Then, the rotation values are calculated as a ratio of the beam free end displacement to the distance between the free end of the beam and the axis of the column. The moment values for the constrained end of the beam are computed using the pressure surface load and uniform shear load on the beam at the center point where the beam is located on the corbel. Consequently, the moment-rotation relationship for the beam in the connection region is acquired. The moment-rotation values are incorporated to define the semi-rigid connection properties of the beam cross-sections where the beam set on the corbel part of the column in the full 3-D model of each structure to be analyzed.

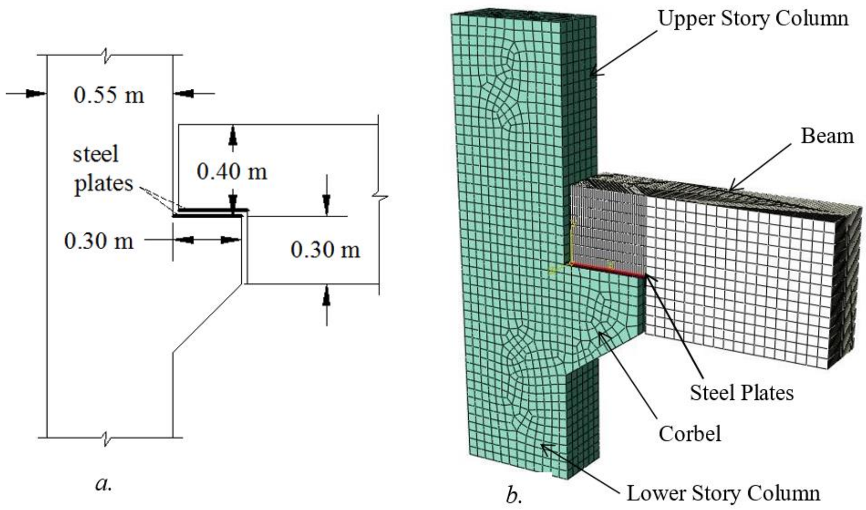

Figure 4:
The beam-column connection region; ***a.*** *Dimensions of the region* ***b.*** *Detailed model*

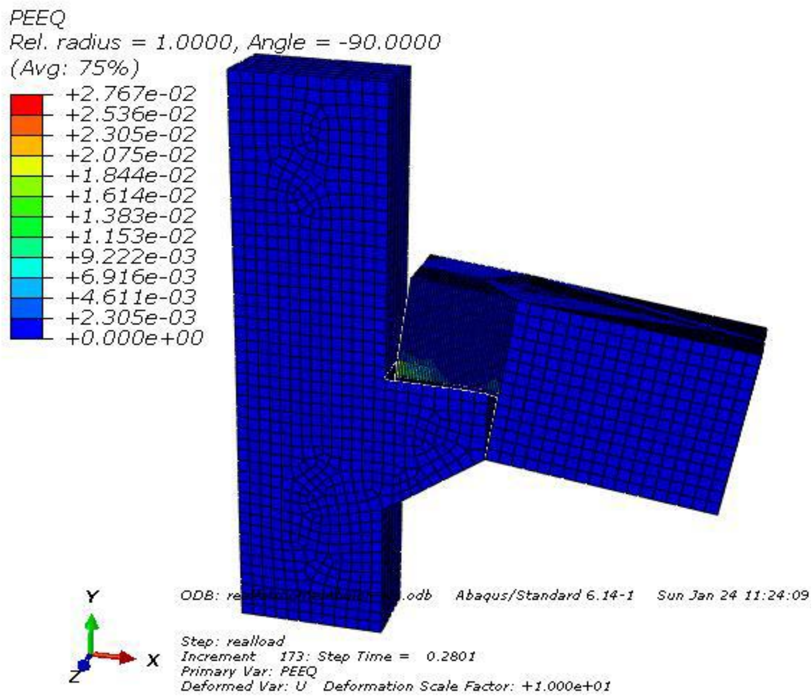

Figure 5:
Deformed shape of the connection region

2.5. Nonlinear Dynamic Analysis of the Precast Concrete Structures

Nonlinear time-history analyses of A, B and C type of precast structures are conducted through 3-D FE modeling using the SAP2000 FE Software (2018) as per the requirements of TEC-2007. The models of the structures consist of precast columns, beams, floor systems, roof trusses and ridge beams. As a reference, the FE models of type A1 and C structures are shown in Fig. 6. In the construction of the models, all dimensions, material properties and details are taken from the design projects of the structures. Mander concrete model (Mander et al., 1988) with Takeda hysteresis is defined for the behavior of concrete materials while kinematic hardening properties with cyclic strain softening is defined for the steel material. At the connection region, the beam ends on the corbel regions are partially constrained due to semi-rigid connection, and the beam ends connected to the columns directly are defined as fixed. The column ends on the connections and as well as at the foundation level are modeled as fixed to describe the moment-resisting connections. The partial fixities (semi-rigid connection) for beams are achieved by providing nonlinear rotational springs whose stiffness values are calculated from the described moment-rotation relationship above. Lumped plastic hinge properties of the beams and columns are determined by using XTRACT sectional analysis program (2004) with confined and unconfined concrete material properties defined by Mander Model and the stress-strain behavior of the steel material defined with a strain hardening branch. The damage mechanism of the cross-sections of the beam and column members in the FE model is specified by utilizing the plastic hinge characteristics according to TEC-2007. For all the columns, axial force-moment interaction surfaces for different axial forces are also defined in the models. The connections between the roof beams and columns are defined as rigid. The ridge beam ends are defined as pins. All other sections of the structural members are modeled with linear elastic properties both for the concrete and reinforcing steel.

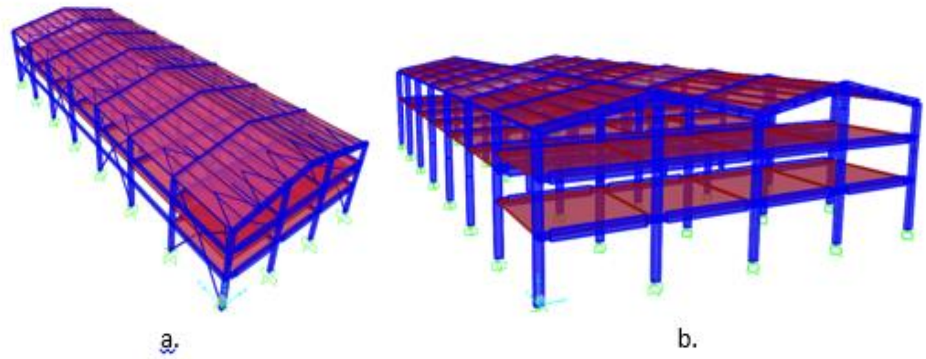

Figure 6:
3-D models; ***a.*** *Type A* ***b.*** *Type C structures*

In the nonlinear time history analyses of the 3-D models of the structures, the same dead and live loads as those taken in the design of the structures are used. Instead of applying the earthquake loading statically, the recorded earthquake acceleration data are utilized to impose the effect of ground motion to the structures dynamically. The ground motions are selected from the recent destructive earthquakes in Turkey. At least three recorded motions should be chosen to perform the nonlinear dynamic analysis according to the criteria defined in TEC-2007. Therefore; the 1999 Kocaeli and Duzce (recording station Bolu-Directorate of Public Works and Settlement called as Bolu in the study) earthquake records (AFAD, 2015; CESMD, 2015) are utilized since the earthquakes occurred are close to the industrial area where the structures are located. The details of the ground motions are presented in Table 2. The North-South and East-West acceleration components of the ground motions are applied in the transverse and longitudinal directions of the structures. Acceleration time histories of the earthquakes are shown in Figs. 7-9.

Table 2. Details of selected strong ground motions

Ground motion name	Date	Magnitude (Md)	Recording station	Earthquake depth (km)	PGA (g)	Distance to the structures (km)
Kocaeli	17.08.1999	7.4	Arcelik Research-Development Lab.- Yarımca	19.6	0.22	9.5
Düzce	12.11.1999	5.2	Bolu-Directorate of Public Works and Settlement	10.0	0.18	180.0
Duzce	12.11.1999	7.1	Duzce-IRIGM-France	10.0	0.17	160.0

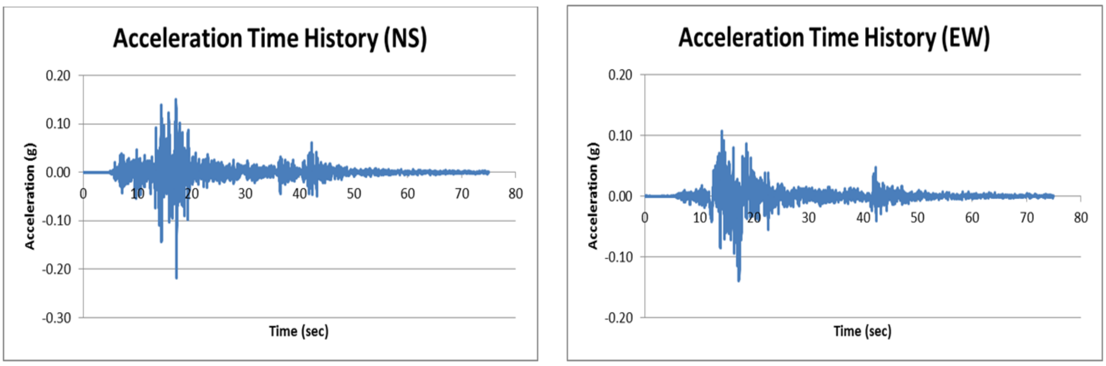

Figure 7:
Acceleration time histories of Kocaeli earthquake

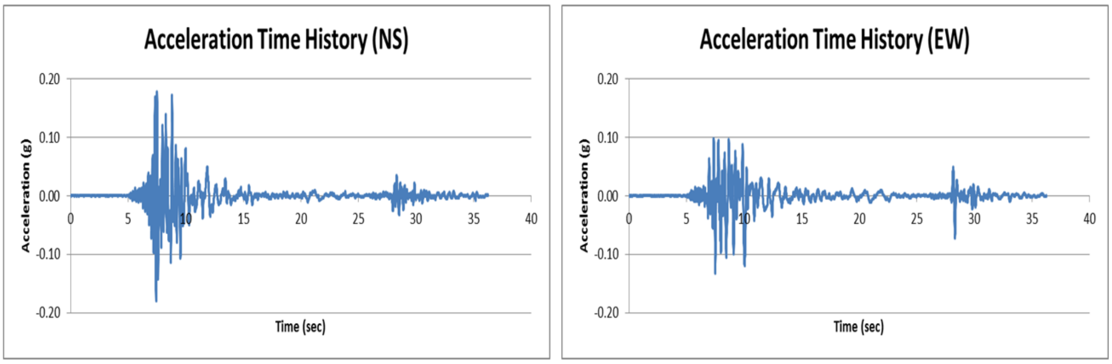

Figure 8:
Acceleration time histories of Düzce (Bolu) earthquake

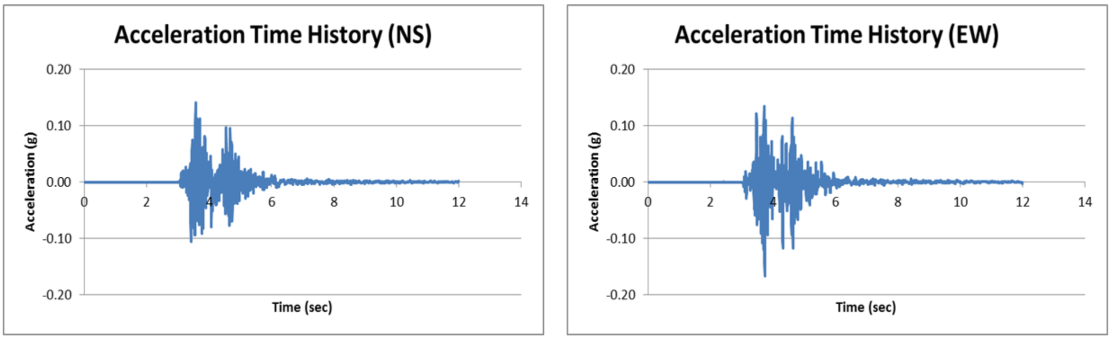

Figure 9:
Acceleration time histories of Düzce (IRIGM) earthquake

2.6. Performance assessment of the precast concrete structures

The performance evaluations of the types A, B, and C precast structures are determined as per the criteria of the TEC-2007. From the results of the time history analyses, the plastic hinge locations along the X (transverse) and Y (longitudinal) directions of the buildings under each earthquake record are obtained. Plastic hinges occurred in the type A1 structures after Kocaeli and Bolu earthquake analyses are shown in Fig. 10. The plastic hinges are generally concentrated on the beam elements on the first floor. There are no plastic deformations observed in the columns due to the strong column-weak beam design and soil type in the industrial area. The strain values in the concrete and reinforcing steel corresponding to total maximum curvature of the cross-section during the earthquake motions are calculated and compared with criteria specified in TEC-2007 to assess the performance levels of each damaged beam member.

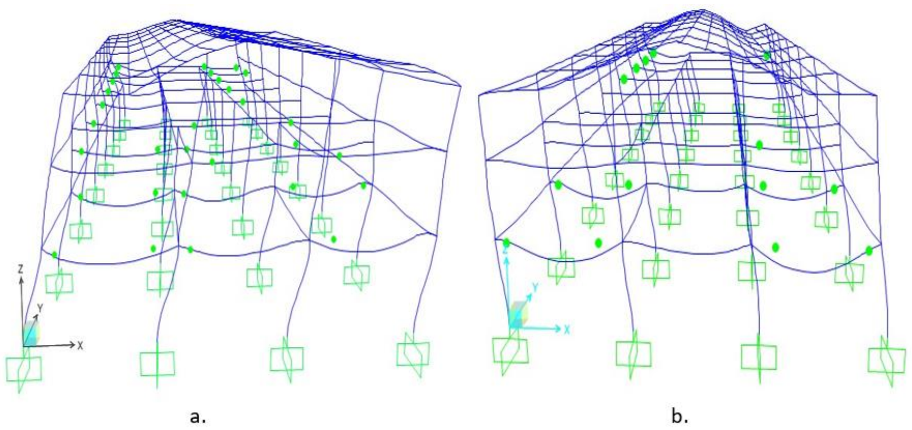

Figure 10:

Plastic hinges in the type A1 structure after; ***a.*** *Kocaeli* ***b.*** *Bolu earthquake*

For ductile reinforced concrete members, three limit conditions are defined in the TEC-2007 similar to Applied Technology Council-40 (1996), FEMA-356, and FEMA-440 (2005) guidelines. The limits are immediate occupancy (IO), life safety (LS), and collapse prevention (CP) conditions. In type A1 and C structures, more than 10% of the beams on any floor during Kocaeli and Bolu earthquake motions got damaged in the advanced damage zone defined in TEC-2007 while there is no beam damaged under Duzce ground motion. The performances of the structures are determined as LS level for Kocaeli and Bolu earthquakes, however; the performance level is accepted as IO level for Duzce motion according to the requirements in TEC-2007. The final performance level should be selected by taking the worst performance of a structure after the time history analyses with the three different ground motions as specified in TEC-2007. Therefore, the performances of type A1 and C structures are accepted as LS level.

There is no damage observation in type B1 building under Kocaeli, Bolu and Duzce ground motions. Therefore the performance of the building B1 is considered as IO level. The performance levels of type A2 and B2 structures are within the IO limit under Kocaeli motion since there are no beams in the advanced damage zone, whereas there are more than 10% of the beams in the advanced damage zone under Bolu motion the performances of the structures are in the LS level.

The final performance levels are summarized in Table 3. All types of structures except the type B1 are found to be within the LS range.

Although the peak ground acceleration of Kocaeli ground motion is the highest, and the Kocaeli station is the closest to the investigated structures as presented in Table 2, in general, Bolu ground motion resulted in a higher percentage of damaged beams. This result could be the reason for the spectral acceleration of the Bolu ground motion being the highest among the others and the closest to the peak value of the spectrum defined in TEC-2007.

Table 3. Performance levels of the precast structures for different earthquake loads

Structure type	Ground motion name	Story name	Percentage of damaged beams (%)	Element performances	Performance of structure	Final Performance
A1	Kocaeli	1st	33	IO	LS	LS
			67	LS		
		2nd	33	IO		
	Bolu	1st	67	LS	LS	
		2nd	29	IO		
A2	Kocaeli	1st	33	IO	IO	LS
		2nd	11	IO		
	Bolu	1st	33	IO	LS	
		2nd	28	IO		
			11	LS		
B2	Kocaeli	1st	75	IO	IO	LS
		2nd	20	IO		
	Bolu	1st	50	LS	LS	
		2nd	60	IO		
C	Kocaeli	1st	63	IO	LS	LS
			12.5	LS		
		2nd	14	IO		
		3rd	88	IO		
	Bolu	1st	50	IO	LS	
			25	LS		
		2nd	-			
		3rd	88	IO		

3. EVALUATION OF THE RETROFITTED PRECAST STRUCTURES

The considered precast structures are suffered LS damage under the recorded earthquake loadings and the relative story drift values (between 0.02-0.03 for all the structures) are exceeded the displacement criteria in the TEC-2007 and thus the structures are retrofitted according to the code. In this second part of the study, two retrofitting methods are applied to the bare structures. In the first method, bolted steel plates are utilized to improve the seismic performance of the beam-column joint regions which can be considered as a member level retrofit. The second method consists of applying diagonal steel braces between the plane frames in the longitudinal direction of the structures and along roof trusses to decrease the lateral deflections and to enhance the strength of the structures. Since the braces provide load paths to transfer the lateral loading, the second retrofitting method is considered as a system-level retrofit. Type A and C structures are strengthened by using both methods, and type B structures are retrofitted by using the second method only.

The additions to upgrade the bare structures are the steel braces, steel plates and bolts to retrofit the connection regions and the whole structures. Steel braces are hollow circular cross-sections having 102, 114, 140, 168, 178, and 218 mm diameter. The thicknesses of the braces are 5 and 6 mm. Steel plates on the beam-column connection regions are 550x725x12 mm. Steel plates are connected to the precast beams and columns by using 4 grade 8.8 M16 bolts.

3.1. Modeling of the retrofitted connection region

The FE model of the retrofitted connection region is constructed using ABAQUS FE program by adding steel plates and bolts to the FE model of the bare connection region as shown in Fig. 11(a). The same boundary conditions, concrete and steel material constitutive relationships as those utilized in the earlier model are implemented.

Steel plates and bolts are constrained to the precast column and beam in the new FE model. The column, beam, steel plates, welding and bolts are modeled using 3D continuum 8-node brick elements, whereas longitudinal reinforcement and transverse steel hoops in the beam element are defined using 1D 2-node linear truss elements. After the static loadings, the deformed shape of the connection region after retrofit is given in Fig. 11(b). The moment-rotation relation for the beam in the retrofitted connection region is obtained under the static loading using the same procedure as described above. The comparison between the moment-rotation relations of the beam before and after retrofitting is shown in Fig. 12. The moment capacity of the beam in the retrofitted model is almost doubled up after applying the steel plates and the bolts. The moment-rotation values are used to represent the semi-rigid connection properties of the beam cross-sections as defined in the analysis of the bare structures. The flexural rigidity (rotational spring stiffness) of the beam is then assigned to the beams in the full 3-D model of each retrofitted structure.

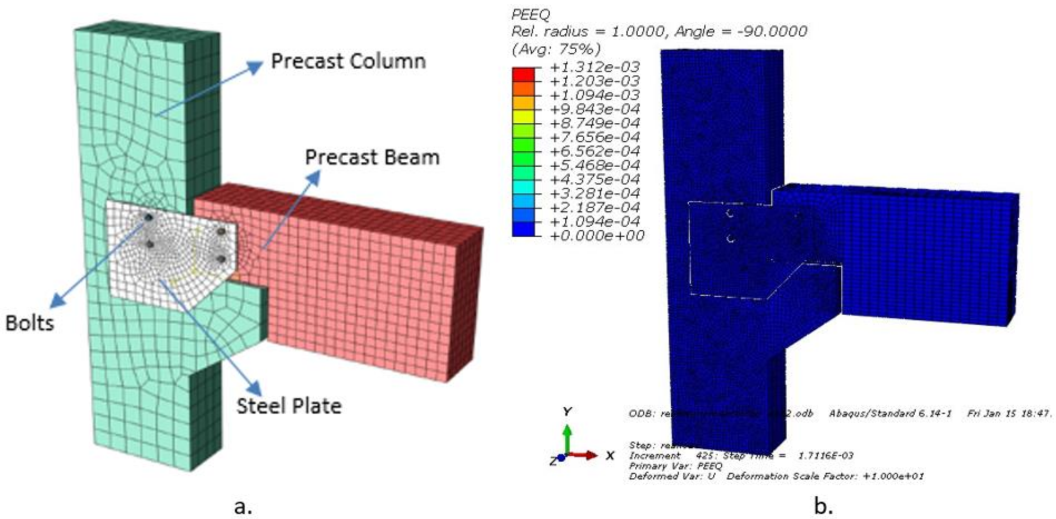

Figure 11:
The beam-column connection region after strengthening; ***a.*** *FE model* ***b.*** *Deformed shape*

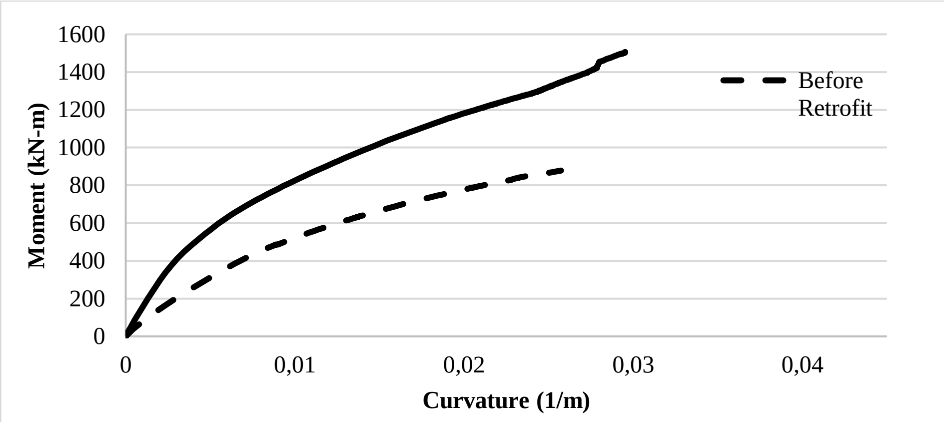

Figure 12:
Moment-rotation relations of the beam before and after strengthening

3.2. Nonlinear seismic analysis of the retrofitted precast structures

Three-dimensional FE models of the types A, B, and C retrofitted precast structures are developed using the SAP2000 FE program. The steel braces are added into the FE models of the bare structures to construct the retrofitted models of the buildings. The FE model of type A1 retrofitted structure is shown in Fig. 13(a) as a reference. Material properties, structural modeling, analysis procedures and loadings are taken the same as described in the FE model of the bare structures. Plastic hinges occurred in the type A1 structure after Kocaeli earthquake are shown in Fig. 13(b).

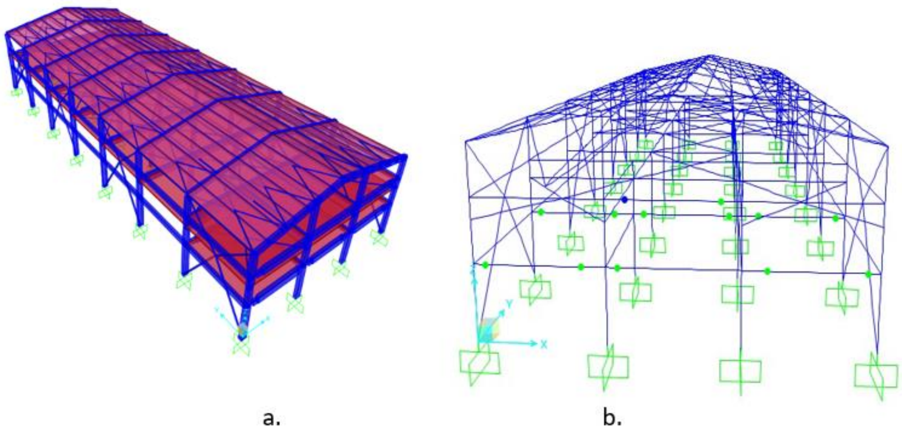

Figure 13:

Type A1 retrofitted structure; ***a.*** *FE model* ***b.*** *Plastic hinges after Kocaeli motion*

3.3. Performance evaluation of the retrofitted precast structures

The performance assessments of the retrofitted precast structures are made based on the results of the nonlinear time history analyses. Performance levels of the beams are specified according to the TEC-2007 and FEMA-356 requirements, whereas the performance levels of the buildings are evaluated based on the TEC-2007 only.

Under Kocaeli ground motion, all the beams on the first floor of type A1 structure are observed to be in life safety (LS) level of performance and consequently, the performance of the structure is accepted as LS level as per the TEC-2007 criteria. However, as per the FEMA-356 requirements, 33% and 14% of the beams are in LS level on the first and second floors, respectively and 67% of the beams on the first floor are in the IO performance level. The analyses results obtained under Bolu ground motion revealed 67% of the beams to be within collapse prevention (CP) performance limit and 33% of the beams to be in LS limit on the first floor according to the TEC-2007 and the performance of type A1 structure is determined as CP level. On the contrary, there is no CP damage limit obtained in beams according to the FEMA-356 guidelines. The performance level of the structure is calculated as IO level since there is no plastic deformation under Duzce ground motion. The final performance level of type A1 structure is considered as CP performance level since it is the lowest performance level obtained from the results of analyses under three different earthquake ground motions.

The same procedure as defined for type A1 structures is followed to identify the performance levels of the type A2, B1, B2 and C structures. A summary of the performance levels according to the TEC-2007 and FEMA-356, damage percentages of the structural members is presented in Table 4 and Table 5, respectively.

Table 4. Performance levels of the retrofitted precast structures based on TEC-2007

Structure type	Ground motion name	Story name	Percentage of damaged beams (%)	Element performances	Performance of structure	Final Performance after retrofit	Final Performance before retrofit
A1	Kocaeli	1st	100	LS	LS	CP	LS
		2nd	14	IO			
	Bolu	1st	67	CP	CP		
			33	LS			
		2nd	14	IO			
A2	Kocaeli	1st	33	IO	IO	IO	LS
	Bolu	1st	33	IO	IO		
B2	Kocaeli	2nd	20	IO	IO	IO	LS
	Bolu	1st	50	IO	IO		
		2nd	60	IO			
C	Bolu	1st	13	LS	LS	LS	LS
			63	IO			
		2nd	14	IO			
		3rd	13	LS			
			75	IO			

Table 5. Element performances of the retrofitted precast structures based on FEMA-356

Structure type	Earthquake station	Story name	Percentage of damaged beams (%)	Element performances
A1	Kocaeli	1st	33	LS
			67	IO
		2nd	14	LS
			67	LS

	Bolu	1st	33	IO
		2nd	14	IO
A2	Kocaeli	1st	33	IO
	Bolu	1st	33	IO
B2	Kocaeli	2nd	20	IO
	Bolu	1st	50	IO
		2nd	60	IO
C	Bolu	1st	76	IO
		2nd	14	IO
		3rd	88	IO

4. DISCUSSION ON THE PERFORMANCE LEVELS OF THE PRECAST STRUCTURES

Span length in the longitudinal (Y) axis, beam lengths along the transverse (X) direction and the ratio of plan dimensions are examined to identify their effects on the performance of the structural elements. In this study, the rate of plan dimensions is defined to be the ratio of the total length in the X direction to the total length in the Y direction of a structure.

Although type A1 and A2 structures have the same span lengths in both X and Y directions, damaged beams of type A1 and A2 bare structures exceeding the LS level are calculated as 67% and 11%, respectively. The differences between these structures are the ratio of the plan dimensions and total heights. A higher rate of plan dimension results in an increase in the stiffness of a structure and that could be the reason for the difference in the performance levels. The relationship between the ratio of the plan dimensions and the percentage of beams in the LS range are presented in Fig. 14. The height of type A1 structures is more than that of type A2 structures which implies that lateral loading effects, primary and secondary moments, are higher in type A1 structures and this could be the other reason for the difference in the performance levels.

The ratio of the plan dimensions of the structures B2 and A1 are close to each other. However, under Bolu ground motion loading, type B2 bare structures had fewer beams in LS range than the type A1 bare structures as shown in Fig. 14. The difference in the span lengths in the Y direction of these structures could be the cause for the difference in the performance levels. Under Kocaeli ground motion, none of the damaged beams in type B2 structures exceeded LS performance limit, but the percentage of the damaged beams in type A1 structures remained the same as given in Table 3. No damage in type B1 bare structures is observed. The type B1 structures have the shortest beam length in the transverse direction. In this regard, it makes the type B1 structure stiffer than the other investigated structures.

Under the Kocaeli and Bolu ground motions, the type C bare and retrofitted structures have less damage than the type A1 bare structures even though the beam lengths in the type C structures are longer in the transverse direction as shown in Fig. 14. However, the ratio of plan dimensions in type C structure (57%) is higher than that in type A1 (38%) and this could be the reason for the minor damage in the beams of type C structures.

After retrofitting the precast structures, beams in type A1 retrofitted structure exceeding the LS limit is 100% whereas there is no damaged beam in LS level in type A2 retrofitted structure as illustrated in Fig.14. The main differences between A1, A2 and C structures are the ratio of plan dimensions and the height of the structures which may be the parameters causing the significant contrast between the performance levels.

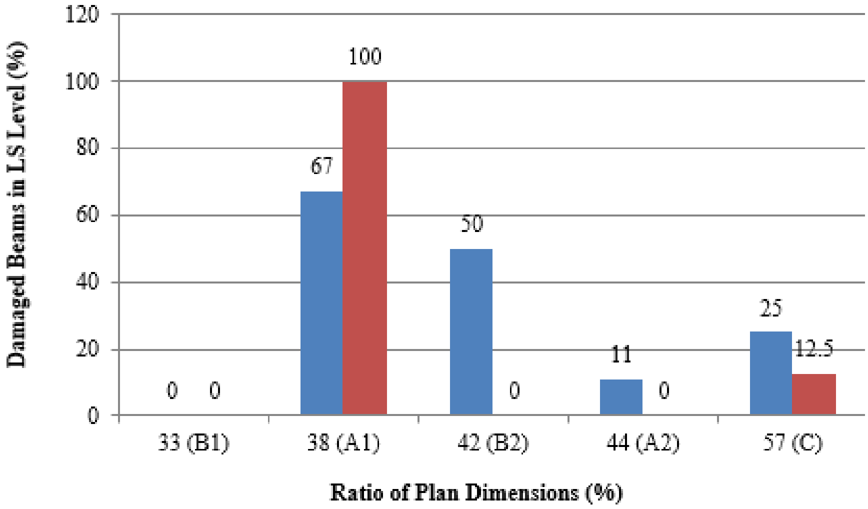

Figure 14:
The relation between the ratio of the plan dimensions and the percentage of beams in LS performance range

5. COMPARISON OF PERFORMANCE LEVELS BEFORE AND AFTER RETROFITTING

The performance levels before and after retrofitting of the structures are presented in Table 3 and Table 4. Improvement in the performances of type A2 and B2 structures is observed. However, a significant drop in the performance level is obtained when the A1 type structure is retrofitted. Moreover, the performance level of bare C type structures is not enhanced after retrofitting. Therefore, additional calculations are conducted and the results are given below to identify the discrepancy in the performance levels for A1 structures and the close performance levels of C structures.

The effective periods, mass participation ratios and the maximum base shear forces acting on the buildings under the earthquake loads are computed for A1 and C types of structures before and after retrofitting and presented in Table 6. It is found that the natural periods of type retrofitted A1 and C structures in X and Y directions are decreased and in the Z direction is increased. This implies that the global structure stiffness in X and Y directions are improved. However, forces about Z axis are obtained due to the increase in mass participation ratio and the distance between the center of rigidity and center of mass. These additional torsional forces move the locations of damage and amplify the plastic rotations of the beams. Moreover, the max. base shear force of type retrofitted A1 structure increased and this force results in observation of additional plastic rotations in the beams. These interpretations could be the explanation for the decrease in the performance level of type A1 structure.

Through comparing the effective periods in X direction before and after retrofitting of type C structure in Table 6, it is deduced that no significant improvement in retrofitting is achieved. Moreover, the maximum base shear is reduced after retrofitting and that decreased the percentage of the damaged beams. However, these observations are not adequate to increase the overall performance level of type C structure.

Table 6. Effective periods, mass participation ratios, and base shear forces

		Effective period (sec.)		Mass participation ratio		Max. base shear force (kN)	
Structure type	Direction	Before retrofit	After retrofit	Before retrofit	After retrofit	Before retrofit	After retrofit
A1	X	0.71	0.52	0.60	0.33	2025.43	2504.53
	Y	0.99	0.32	0.97	0.98		
	Z	-	0.29	-	0.18		
C	X	1.05	0.96	0.52	0.93	3333.18	2933.31
	Y	1.61	0.59	0.97	0.99		
	Z	-	-	-	-		

6. CONCLUSIONS

In this study, the performance assessments of five different types of industrial precast structures are conducted according to the current TEC-2007. The precast structures are retrofitted by utilizing member-based and system based strengthening techniques. The performance evaluations of the retrofitted precast structures are also examined according to the current TEC-2007 and FEMA-356 guidelines. Performance levels before and after retrofitting the structures are compared to quantify the efficiency of retrofitting methods. The effects of the dimensional characteristics of the structures on the performance of the precast structures are investigated. The following conclusions are extracted from the results of the numerical analyses of the precast structures and recommendations are provided.

- A higher ratio of plan dimensions and shorter beam lengths in transverse directions of the structures, in general, lead to a reduction in the damage observed and consequently an increase in the performance level of the structures.
- The performance level of the structures can also be increased by reducing the distance between the plane frames in the longitudinal direction.
- Strengthening techniques applied to type A2, B1 and B2 structures improved the lateral load-carrying capacities and enhanced the performance levels from Life Safety to Immediate Occupancy level.
- The performance of type A1 structure after retrofitting is reduced to the Collapse Prevention level although both strengthening techniques are employed. The reason could be the increase in the distance between the center of rigidity and center of mass after retrofitting the structure. It is recommended that when a strengthening method is applied to a structure, the designer should either maintain or reduce this distance.
- Analysis results of the detailed modeling of the connection region showed that the utilized member-based strengthening technique doubled up the moment capacity of the connection region. It is recommended that on the evaluation of the performance level of a retrofitted structure the detailed model of the connection region should be taken into account.
- In the considered structures, the percentage of beams defined as damaged by the TEC-2007 is higher than the rate of damaged beams identified as per the FEMA-356 criteria. The

performance levels determined by the TEC-2007 criteria satisfy the performance levels requirements given in FEMA-356 guidelines.

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