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A methodological approach for seismic performance of existing single-storey industrial RC precast facilities

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Abstract: In critical earthquake-prone regions, many kinds of old-dated industrial facilities having structural deficiencies exist. Evaluation of seismic performance of these buildings to reach a sufficient level is quite vital. This paper scopes seismic performance assessment of an existing industrial structure. In the study, a comprehensive methodology is presented to carry out the seismic evaluation process of the buildings. The entire process is illustrated using a case study from an existing industrial precast facility. In this scope, initially, construction system, geometry, layout and material properties of the examined structure were determined through lab studies and site surveys to assess the performance level. Secondly, the current status of the structure was modeled using Midas Gen finite element software and a series of analyses were performed to reveal the seismic performance. In the analyses of seismic performance, the non-linear pushover analyses method was employed in seismic code. In the model the fiber and lumped hinges were assigned to the columns and beams, respectively. The strains occurring in the column cross-sections were calculated using the curvature values obtained from the corresponding members assigned hinges. These values were compared with the limit values which were specified in the code. It is concluded that this case study presents a practical approach for engineering applications regarding the seismic evaluation of industrial structures from the perspective of update codes.

Keywords: RC precast structures, field observations, seismic performance analysis, seismic provision.

1. Introduction

Industrial structures are defined as buildings and structures for processing, assembly, mixing, cleaning, washing, packaging, storage, distribution and repair, as well as factories where many products can be manufactured. The structures have wide usage areas as a result of their functions. Industrial structures can be classified as; i. Prefabricate, ii. Steel construction, iii. Conventional, and iv. Composite structures. Building materials and construction techniques of these structures differ from other structures. Furthermore, the construction cost of the prefabricated structures, the parts of which are pre-prepared for assembly, is lower than the other industrial building types and the construction duration is shorter. Thanks to the prefabricated carrier systems, wider openings are provided, thereby enabling more efficient and more economical use of space. As a result of this advantage, a significant proportion of industrial facilities are constructed as prefabricated reinforced concrete. Main advantages of incorporating precast concrete in construction are the possible increased speed of construction, high quality of precast units, improved durability, reduction in site labour and formwork, and more importantly, social and environmental benefits (Khare et al. 2011). The precast concrete structures were approached with scepticism in seismic zones since the basic nature of seismic behaviour were not fully understood (Englekirk, 2003). However, this effectiveness could be compromised for construction in seismic regions, especially if the design follows the capacity design rule (Belleri and Riva, 2012). One of the major problem in the design and construction of structures including precast concrete members to provide seismic strength is to find out the practical and economical methods for connection of these members (Restepo et al., 1995).

In the construction including precast concrete, the structural elements such as columns, beams, floor units, wall panels and architectural members (i.e. cladding) are manufactured in a production facility, followingly transported to the construction site, and fitted to a pre-defined place (Kurama et al., 2018). In addition to dividing the spaces, the walls in the prefabricated system also serve as stiffener members in some cases. The joints of the prefabricated elements do not resemble the behavior of a monolithic building in terms of strength, ductility or displacement. For this reason, it is appropriate to define prefabricated structures as masonry systems. In this respect, the joints of the prefabricated structures have a critical importance in structural behavior. In precast RC structures, lateral forces are confronted by cantilever action of precast concrete columns. The pinned support generally included one or two anchorage dowels, which were used to allow the rotation about an axis perpendicular to the frame plane and to restrict the lateral movement (Sezen and Whittaker, 2006). In hinged joints, the columns are rigidly connected to a foundation. Since the joints are hinged joints, they do not transfer moment, but moment transfer occurs only at the points where the columns are connected to the foundations. As a result of this, plastic hinging occurs at the points of the columns close to the foundation under a seismic effect.

Design of structure using traditional design approaches and philosophies has been generally considered as inapplicable. The structures are typically designed by engineers with the intention of reducing lateral forces and allowing an accepted level of damage in potential hinge regions, which are particularly detailed for ductility. A series of economic parameters, such as damage cost of stored goods and equipments and the cost related to operation loss after a moderate or powerful earthquake, are still not accounted satisfactorily for design process (Holden et al., 2003). In major earthquake events of high seismicity, a performance objective for buildings and structures is to provide the safety of life and continuous operations following to powerful ground shakes. Structural members of buildings are also expected to satisfy the requirements of strength and serviceability limit states. Post-earthquake operational issues and extensive damages have been observed following to recent earthquake (Khare et al., 2011). Toniolo and Colombo (2012) examined the seismic design problems regarding the precast concrete structures after an earthquake. Following to the earthquake, a number of buildings were elaborated through the detailed surveys to analyze the behaviour of such structures under these effects. By means of this analysis, which reveals the inadequacy of the cladding panel connections, the paper provides a systematic approach to this problem analyzing the behaviour of the whole system including panels. Belleri et al. (2015) examined the industrial facilities to highlight the primary vulnerabilities of one-story precast concrete structures which are not designed and detailed for seismic loads. Fleischman et al. (2014) conducted a survey study focusing on the shaking levels exposed to the buildings. In this study, the properties of different solutions for precast seismic systems were explained, damages on some of the selected precast buildings were evaluated. The construction details and monitored structural damages were presented, as well. The observed performance was also evaluated taking into account level of shaking, design intent and anticipated performance.

A significant amount of literature including the seismic behaviour of RC precast structures. The researches contribute in understanding the improvement in seismic performance of the structures. Magliulo et al. (2008) worked on seismic vulnerability evaluation of industrial precast buildings. A series of analyses such as elastic, non-linear static and dynamic cases were performed on representative buildings which were chosen based on a significant investigation on materials, structural typologies and details. It was shown that, through the elastic analyses, the high frequency modes are employed to deal with the effects originated from the seismic vertical component. By means of non-linear dynamic analyses, it was shown that the capacity is always higher than that demanded with regard to column chord rotation. however, the variations in axial force, which is significant at corner columns exposed to relatively low static vertical load, can bring about the beam sliding on columns and, therefore, the building failure might occur due to the loss of support. Ercolino et al. (2018) assessed the seismic performance for a series of RC single-story precast buildings, which are designed based on modern building codes, performing static and non-linear dynamic analyses. Evaluating the results, it was found that most of the structures are under the influence of seismic details related to the code and the seismic action does not represent the most prominent need of the structural elements for many cases. The structure capacity remains almost the same with the geometry and it is slightly affected by the site seismicity due to seismic details and overstrength of the design.

Many researches have been performed regarding seismic behaviour of precast structures. Bellotti et al. (2009) addressed that the main properties of the seismic response RC precast buildings to examine design methods and to suggest details of opportune strengthening techniques. In particular, two 3:4 scaled two-bay three-storey frames were imposed to a cyclic displacement history with a quasistatic cyclic characteristics at increasing drift levels with a lateral load distribution calculated as a function of the storey heights and masses. In addition, two full-scale columns - precast pocket foundation systems were exposed to a displacement history, which is a quasi-static loading-unloading, at the columns top. Holden et al. (2003) designed and examined two half-scale precast concrete cantilever wall units, which are geometrically identical, under the exposure of under the reversed cyclic and quasi-static lateral loading. In order to emulate the characteristics of a ductile cast-in-place concrete wall, one of the units was reinforced conventionally according to the code. The other unit remained as a part of precast system which is partially prestressed and includes steel fiber reinforced concrete and, unbonded and post-tensioned carbon fiber tendons. The conventionally reinforced precast wall showed a quite good performance with regard to energy absorption capability, which reaches 2.5% drift rate before occurring of meaningful strength degradation, and ductility capacity. The other wall unit, prestressed partially one, delivered a satisfactory performance of drift levels which excessed the rate of 3% without any observable damage on wall panel before the damage occurs.

In this study, the seismic performance of industrial RC precast facility with a single-storey was assessed employing lab tests, numerical simulations and field studies. 3D finite element model of the structure is prepared and then nonlinear static analyses were performed using Midas Gen (2018) software based on the data obtained from the material tests and field studies. It is crucial to evaluate the behavior of single-storey industrial buildings and improve their seismic performances. Therefore, this study can be regarded as the assessment task procedure to this purpose.

2. Methodology

Determining the seismic performance of existing structures located in areas exposed to earthquake effects is an obligatory step in terms of minimizing loss of life and property by predicting the structure behavior under the affected loads in a possible earthquake. As the studies presented to this aim vary, an approach involving the purpose of determining the seismic behavior of existing structures is required. To do this, steps to determine the performance of the examined facility in the study presented are given in this section. This approach is aimed to apply to industrial buildings as well as low-rise RC structures.

The structural strength level should be determined for the structures whose strength is decreased or insufficient due to the expiry of the material life, creep, shrinkage and fatigue as well as construction defects. Also, it should be determined whether the structural strength of an existing building is sufficient to meet the current regulation requirements depending updating the earthquake regulations. As a result of determining the current strength level of a building, there are three ways related to the building. The first of these is that the structure, which has a sufficient strength level, can be used with its current state. If the structure does not have enough strength, the structure may need to be strengthened. If the first two cases are not possible, the structure must be demolished due to the insufficient strength that cannot be strengthened.

The limits of the three conditions are given in the regulations by supporting them with numerical values. Although there is no strengthening stage regarding the structure examined in this study, the strengthening phase is also included in the scope of the methodology, since the approach presented is not limited to the structure examined in this study. There are conventional or modern methods and approaches in strengthening. Nevertheless, the selection of the method or technique to be used in terms of structure is considered as the most important criteria. Another issue is the economic parameter of the process. When it is determined that a examined structure is at a strength level, it is necessary to compare the strengthening and the construction costs, and choose the appropriate one. In addition to the strengthening cost, it should also be considered the restriction of the areas of use, and the renewal cost of existing electrical and sanitary installations in next period. If the structure is defined as historical value and cultural asset, the cost ratio of the strengthening to reconstruction should not be considered. In this case, the improvement of the structure can be chosen with minimum intervention and by preserving the original state of the building.

This methodology is presented in order to be an effective tool in determining the appropriate option by revealing the strength level of the structures within the scope of the analyzes, provided that they are limited to existing RC structures. The methodology includes six stages and each of them provides data to the following stages. The first stage involves providing and examining the design projects of the building. If the building has no project, its geometric properties are determined for the existing building. In this context, dimensions of structural element and plan features are determined. In the second stage, the material properties of the current state of the building are determined. In this step, the material properties of RC members (beam, column, shear wall, slab, foundation) are determined using undamaged and undamaged techniques.

In the undamaged method, concrete strength values are obtained by using test hammer and ultrasound device. In the damaged method, compressive strength is determined in the laboratory environment considered the number of core samples specified in the local regulation. Core samples are obtained from the location where the moment effect is minimum in the column element and from the part without the longitudinal and stirrup. The final concrete strength is determined by correlating the data obtained from the undamaged and damaged method. Rebar intervals are determined by using an x-ray device. In the other method, the plaster layer on the concrete and the cover are removed and the rebar is exposed. Thus, the corrosion level and rebar diameters in the reinforcement are determined. At the third stage, the data in the first two stages are compared, and the compatibility of the design project and the actual situation is determined in terms of geometry and material properties. The fourth stage involves the determination of soil properties.

In case the building has a project, the ground parameters in the project can be considered. In the absence of a project, drilling is done from four corners of the building and samples are taken and soil parameters such as ground safety stress and bearing coefficient are determined in the laboratory environment. However, in case it is impossible to take samples due to the location of the building, the soil parameters of the adjacent plots are considered. In the fifth stage, the numerical model of the building is prepared by using all the data obtained in the previous stages and taking into account the regulations. Subsequently, the structural model is analyzed and it is determined whether it meets the seismic conditions specified in the current regulation. As a result of the analysis, it is stated that if the seismic performance level of the current state of the building is sufficient, it can be used in its current form. In the event that the strength is insufficient, a proposal for strengthening the structure or reconstruction of the building is offered.

In this context, conventional and / or contemporary techniques can be proposed by taking into account the conceptual and functional requirements that do not cause architectural constraints in the strengthening phase that constitutes the sixth stage. While conventional techniques are mostly cross-section increase in the element, adding new elements, adding a new steel frame system, seismic isolators and fiber polymers can be preferred within the scope of modern techniques. The most important criteria in the selection of the technique are strengthening cost, usability of the structure during construction, ease of application and construction period. Thus, strengthening application of the structure is carried out with all necessary data obtained. Fig. 1 presents the seismic performance of existing structures, and next processes.

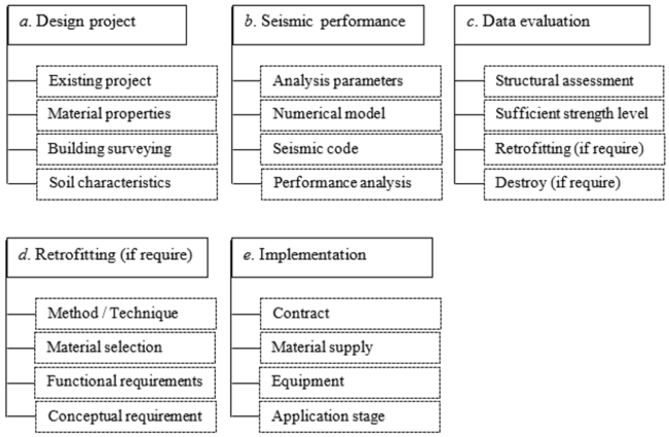

Figure 1. Entire flow-chart of a proposed methodology for a reconstruction procedure.

3. Investigation site

The prefabricated RC frame system is an industrial structure with a single storey (Fig. 2). The building, with a 3240 m^2 construction area and a 5805 m^2 total area, is 66 m and 54 m long in the x and y directions, respectively (Fig. 3). Building height varies between 8 m and 10 m.

Figure 2. Aerial view of the examined structure.

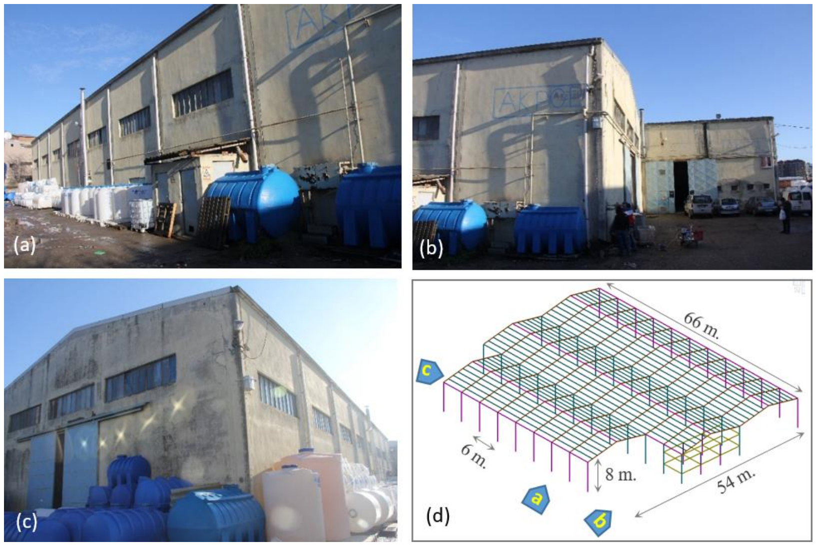

Figure 3. The examined structure: (a-c) Facade views, (d) Geometry.

In the design project, although the structure is specified as one-floor, the two-storey administrative section partially is included in the on-site examination (Figure 3d, Figure 4b).

The storey height of the administrative section, which is located between the 11-12 and C-B axes at the size of 6×18 m, is 2.8 m. There are columns and beams with 30/60 cm and 20/50 cm on the first and second storeys of the administrative section, respectively. The first floor has a RC slab system with 15 cm thick. The foundation plan and cross-section view of the examined structure are given in Figures 5 and 6. It was determined that the structure was composed of side/middle columns, V-type member, roof beam, purlins and middle/edge gutters elements (Figure 7). Element connections were examined in situ. No situation that reduces the structural security level was found in the connections.

Figure 4. (a) inner view, (b) the view of administrative section.

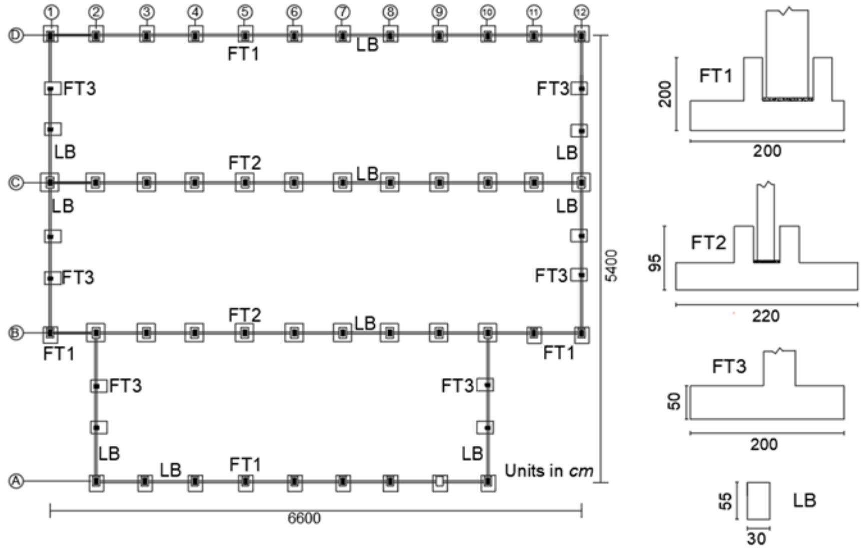

Figure 5. Foundation plan and cross sections (footing types, FT1:180/200/40, FT2:220/220/40, FT3:200/150/50, Link beam, LB:30/55).

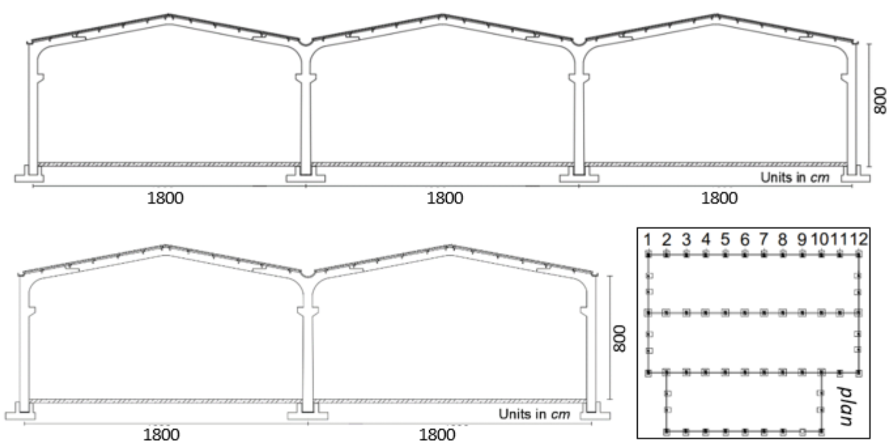

Figure 6. Elevation views: axes 2 to 10 (top), axes 1, 11 and 12 (bottom).

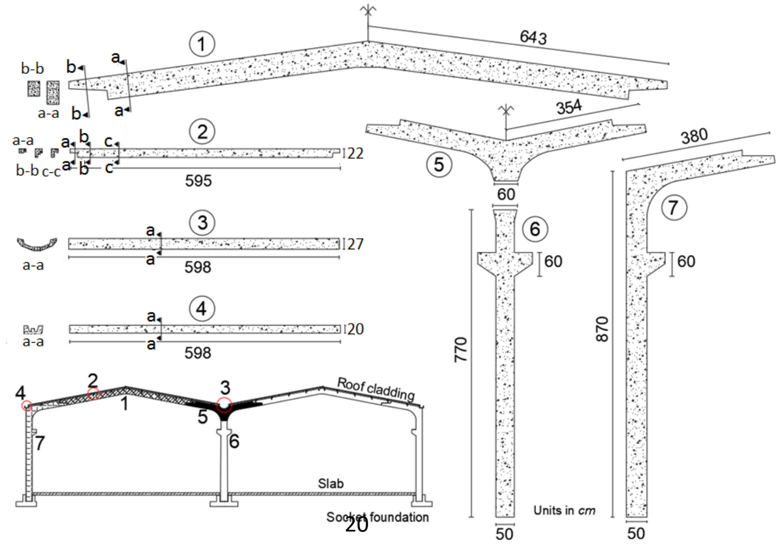

Figure 7. Structural and nonstructural members: (1) roof beam, (2) purlin, (3) middle gutter, (4) side gutter, (5) v-type member, (6) interior column, (7) exterior column.

On-site examinations, it was determined that there were nonstructural cracks on some brick wall surfaces and on the edges of the doors/windows (Figure 8a). In addition, several RC elements have been exposed to corrosion damage (Figure 8b). The geometrical and material properties of the load-bearing elements obtained from the design project are summarized in Table 1.

Figure 8. (a) wall cracks, (b) rebar corrosion.

Table 1. Geometrical and material properties of the structure.

Member	Location	Dimension (cm)	Rebar (L: longitudinal, S: stirrup)
Columns	Middle	25×50	L: ϕ20 and ϕ25, S: ϕ8/10~20
	Side	25×53	L: ϕ16-ϕ18, S: ϕ8/10~20, ϕ10/20
Member V [1]		25×20 and 25×50	L: ϕ14, ϕ20, ϕ22 and ϕ25 S: ϕ10/10~15
Roof beam		25×15 and 25×50	L: ϕ14, ϕ20 and ϕ22 S: ϕ8/15 and ϕ10/10
Purlins		8~15/10~22 Length:598	L: ϕ8 and ϕ12 S: ϕ8/10-15
Gutter	Middle	Length:598	L: ϕ8 and ϕ16 S: ϕ8/10-15
	Side		L: ϕ8 and ϕ16, S: ϕ8/12.5~15
Foundation [2]		40×200×180 40×220×220 50×200×150	L:ϕ12 and ϕ14, S: ϕ10

[1] The element that provides connection of top beams and middle columns, [2] C16-S220 for the foundation, and C25-S220 for ϕ8-ϕ10, S420 for the others

3.1. Laboratory study

The mechanical properties of concrete, and rebar arrangement have been obtained from the original design documentation. In order to assess the average concrete strength, a sampling study for concrete core were conducted on vertical structural components (TS EN 12504, TS EN 2010). Two different techniques exist with explained details in TBEC (2018) to locate and determine the rebars in concrete based structural components. The methods consist of rebar scanning and the observational rebar measurement with stripping of cover concrete. Both approaches were utilized for the building under the consideration to find out the required arrangements of reinforcement of the structural components.

According to the results of the lab tests performed and the concrete strength values included in the design project of the building, the strength value was determined as 20.6 MPa. There are two different methods regarding rebar detection. In the first method, undamaged rebar detection method, rebar spacings were determined in eleven-vertical load bearing elements. Accordingly, there are 5ϕ18 longitudinal rebars on the long side and 3ϕ18 on the short side of a column under examination. In the column, ϕ8 stirrups are used with 16 cm spacing. The other rebar detection method is the visual rebar features such as dimension and corrosion effect in the load bearing elements by stripping the cover. To this end, stripping the cover concrete is performed in 4 of structural members on all storeys. At the end of the stripping process, ϕ16, ϕ18 and ϕ20 longitudinal rebars and ϕ8 stirrups were detected in the columns. It was determined that the stirrups were used in the element with an average 14~23 cm spacings. The steel classification of the rebars used in the structural members is identified as S220 (TS 708).

3.2. Soil properties

In the area where the building is located, a study was conducted to determine the soil properties. In this scope, soil properties and seismic parameters employed in analyses were determined taking into account soil analysis report of in-situ tests and the criteria listed in TBEC 2018. In soil research, allowable bearing stress of soil and modulus of subgrade reaction values were determined as 170 kPa and 30000 kN/m^3, respectively. The local soil class is determined using the average shear wave velocity map (IBB 2009) of Vs (0 to 30 m). According to these values; It was determined that local soil class in the location of the building is determined as a ZC group corresponding to the series of less weathered claystone, limestone and sandstone known as the *Kartal* formation. Four ground movement levels of earthquake are determined in in TBEC 2018 (Table 2). Due to the performance requirements for this kind of buildings as stated, Level 2 (DD-2) was utilized in the analysis. The ground movement level of earthquake will be used for the performance target.

Table 2. Earthquake ground motion levels (TBEC 2018).

Earthquake ground motion level	Probability of exceedance (50 years)	Return period	Definition
DD-1	2%	2475 years	Maximum
DD-2	10%	475 years	Standard Design
DD-3	50%	72 years	–
DD-4	68% *	43 years	Service

* Probability of exceedance in 30 years = 50%.

The characteristic values of the elastic acceleration spectrum are selected according to the ground class (Table 3). According to these values, the spectrum graphic will be determined depending on the local ground impact coefficients given in TBEC 2018.

Table 3. Local soil impact factors (TBEC 2018).

Local soil class	Local soil impact factors for short period zone, F_S					
	$S_S \leq 0.25$	$S_S=0.50$	$S_S=0.75$	$S_S=1.00$	$S_S=1.25$	$S_S \geq 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8
ZF	In-situ soil behavior analysis should be made					
Local soil class	Local soil impact factors for period=1.0 sec., F_1					
	$S_1 \leq 0.10$	$S_1=0.20$	$S_1=0.30$	$S_1=0.40$	$S_1=0.50$	$S_1 \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0
ZF	In-situ soil behavior analysis should be made					

Spectral acceleration coefficients obtained from the risk map of earthquake (S_s and S_1) are determined using interactive web application for Turkey Earthquake Hazard Maps presented by AFAD (2019). The location of the building is marked on the map and then S_s=1.024g and S_1=0.286g are determined for an earthquake which has a repeat period of 475 years. Local ground impact coefficients were calculated as F_s=1.20 and F_1=1.50 based on the determined spectral acceleration coefficients and local soil class (ZC). Then, acceleration coefficients were obtained using map spectral acceleration coefficients and local ground impact coefficients (Eq. 1 and 2).

$$S_{DS} = S_S \times F_S = 1.024 \times 1.20 = 1.230 \quad (1)$$

$$S_{D1} = S_1 \times F_1 = 0.286 \times 1.50 = 0.429 \quad (2)$$

Accordingly, horizontal elastic spectral acceleration values, $S_{ae}(T)$ and horizontal elastic acceleration spectrum corner periods (T_A and T_B) were calculated depending on the values calculated above and the criteria given in the regulation and a horizontal elastic acceleration spectrum was illustrated. Horizontal elastic spectral acceleration values of different regions in Figure 9 are obtained using Eqs. 3 to 6. In the equations, T_A=0.2 (S_{D1}/S_{DS}), T_B=S_{D1}/S_{DS} and T_L = 6 sec. was taken into account.

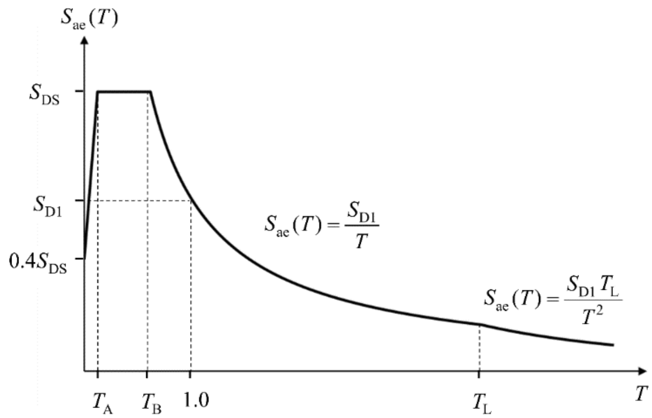

Figure 9. Criteria propose to calculate the spectral acceleration values and elastic spectrum in the code (TBEC 2018).

$$S_{ae}(T) = \left(0.4 + 0.6\frac{T}{T_A}\right)S_{DS} \qquad (0 \le T \le T_A) \qquad (3)$$
$$S_{ae}(T) = S_{DS} \qquad (T_A \le T \le T_B) \qquad (4)$$
$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad (T_B \le T \le T_L) \qquad (5)$$
$$S_{ae}(T) = \frac{S_{D1} \cdot T_L}{T^2} \qquad (T_L \le T) \qquad (6)$$

4. Numerical modeling

Depending the function of the examined structure, it was evaluated in the class specified as “Other buildings” in TBEC 2018 and the building usage class (BKS) 3 and building importance coefficient (I) were determined as 1 (Table 4). Based on the function of the building, the live load mass participation factor (n) was determined as 0.30 according to the regulation. 1.5 kN/m^2 coating load and 2 kN/m^2 live load were taken in the office part and 0.5 kN/m^2 coating load was taken on the roof part. Response spectrum loading in x and y directions was converted to nodal loads at each node using the Midas Gen software and applied as pushover initial loading to the structure. The building has a structural system consisting of precast elements. Precast structural elements (columns and beams) are connected to each other with steel pins in a way to transfer moment at the connection points. Therefore, the structure is modeled and analyzed as a system with moment-transferred connections. The vertical earthquake effect caused by the relative displacement was increased by 30% to the horizontal earthquake effect in the calculation. According to the criteria given in TBEC 2018, earthquake design class to the examined structure are determined as a 1 short period design spectral acceleration coefficient, S_{DS} (1.23 from *Section 3.2*) and building usage class, BKS (Table 5). Afterwards, building height class (BYS) was determined as 7 considering the earthquake design class and height of the existing building (Table 6).

Table 4. Use classes, purposes of use and importance factors for buildings (TBEC 2018).

Building use class	The purpose of use	Building importance factor (I)
BKS=1	Buildings planned to be used following to the earthquake, long-term and intensively utilized buildings, buildings conserving hazardous materials and valuable goods: schools, hospitals, museums, etc.	1.5
BKS=2	The buildings that individuals visit often and stay for a short period of time and sport facilities, malls, cinemas, concert halls, theatres, religious facilities, etc.	1.2
BKS=3	Other buildings: houses residences, hotels, offices, industrial buildings, etc.)	1.0

Table 5. Classes of seismic design (DTS) (TBEC 2018).

Design spectral acceleration coefficient of short period *	Building's use class (BKS)	
	BKS=1	**BKS=2,3**
$S_{DS} < 0.33$	DTS=4a	DTS=4
$0.33 \leq S_{DS} < 0.50$	DTS=3a	DTS=3
$0.50 \leq S_{DS} < 0.75$	DTS=2a	DTS=2
$\mathbf{0.75 \leq S_{DS}}$	DTS=1a	**DTS=1**

***** According to DD-2 earthquake ground motion level

Table 6. Building height margins and building classes determined through the classes of seismic design (TBEC 2018).

Building height class (BYS)	DTS=1, 1a, 2, 2a	DTS=3, 3a	DTS=4, 4a
1	$H_N > 70$ m.	$H_N > 91$ m.	$H_N > 105$ m.
2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
6	$10.5 < H_N \leq 17.5$	$17.5 < H_N \leq 28$	
7	$\mathbf{7 < H_N \leq 10.5}$	$\mathbf{10.5 < H_N \leq 17.5}$	
8	$H_N \leq 7$	$H_N \leq 10.5$	

The three-dimensional structural model was prepared based on the studied field and laboratory data (Figure 10). Afterwards, seismic performance analyzes of the building were carried out.

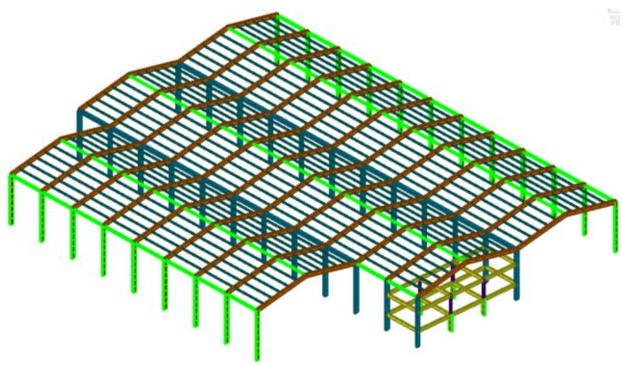

Figure 10. Finite element model of the industrial structure.

Performance analysis of the structure was carried out based on the criteria of TBEC 2018 regulation. Material strengths are not divided into material coefficients, and material coefficients are not taken into account in the calculation of the element capacities, so existing material strengths are used. Building knowledge level was accepted as limited level and so information level coefficient was taken as 0.75. Nonlinear static pushover analysis method was chosen to determine the level of performance. In accordance with the chosen method, the earthquake load reduction coefficient [$R_a(T)$] was not considered in the acceleration spectrum, and the load-bearing system behavior and strength excess coefficients (R and D) were not used depending the nonlinear analysis method. In the analysis, plastic hinges (fiber) are defined for the load-bearing elements. The structure is aimed to satisfy "Controlled Damage Performance Target" conditions for earthquake (DD-2 Earthquake Ground Motion Level) with a 10% probability of exceedance 50 years. In order to check the controlled damage performance level in the existing building, damage limits were compared to structural members with the criteria specified in the regulation.

Table 7. Minimum performance goals for existing structures regarding distinctive earthquake levels (TBEC 2018).

Earthquake ground motion level	DTS=1, 2, 3, 3a, 4, 4a		DTS=1a, 2a	
		Design/assessment approach	Advanced performance level	Design/ assessment approach
DD-3	–	–	Limited damage level	Strain based
DD-2	Controlled damage level	Strain based	–	–
DD-1	–	–	Controlled damage level	Strain based

Strain-based evaluation approach is used in the sectional damage zones defined in the regulation (Figure 11). For the ductile elements, three damage levels and damage limits are defined at cross section level. These are Limited Damage (LD), Controlled Damage (CD) and Pre-Collapse Damage (PCD) and their limit values. Limited damage describes a limited inelastic behavior in the cross section. Similarly, controlled damage defines the inelastic behavior in which the section strength can be safely provided, while the pre-collapse damage condition defines the advanced elastic behavior in the section. This classification does not apply to brittle damaged elements. If the damage of a member's critical section does not reach MNL, the member is assumed in the "Minimum Damage" region. The ones between MNL and SL are assumed in the "Significant Damage" region, and the ones between SL and CL are assumed in the "Advanced Damage" region. If the damage of a member's critical section exceeds CL, the member is assumed in the "Collapse" region.

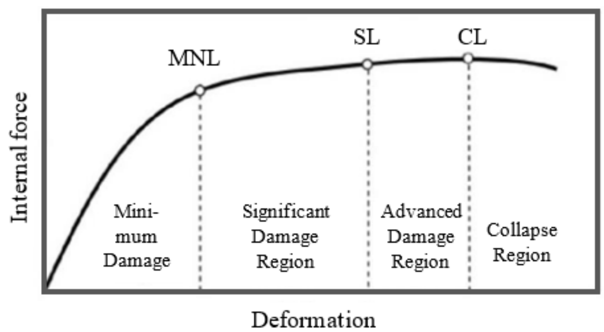

Figure 11. Section damage regions (TBEC 2018).

Performance level for prefabricated reinforced concrete buildings is determined based on the criteria specified in the regulation. The regulation does not allow member in the "Advanced Damage" region for prefabricated buildings. The performance targets of the structure are based on pushover analyses. The drift achieves the performance targets of "Limited Damage" to 0.3%, "Controlled Damage" to 0.7%, "Pre-Collapse" to 1% of the building height.

5. Results and discussion

The analyses are performed to the earthquake (DD-2 earthquake ground motion level) with a 10% probability of over 50 years. The mod shapes provided with the modal analysis are shown in Fig. 12. At the performance point, base shear (V) vs roof displacement (d) and demand spectrum vs capacity curve diagrams (spectral acc./spectral disp.) are shown in Fig. 13.

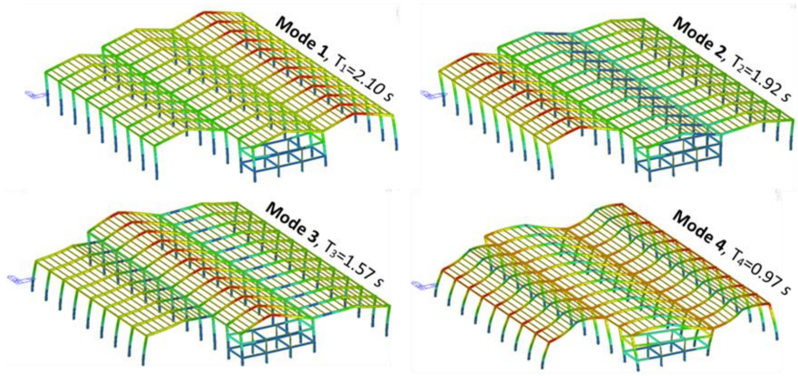

Figure 12. Mod shapes.

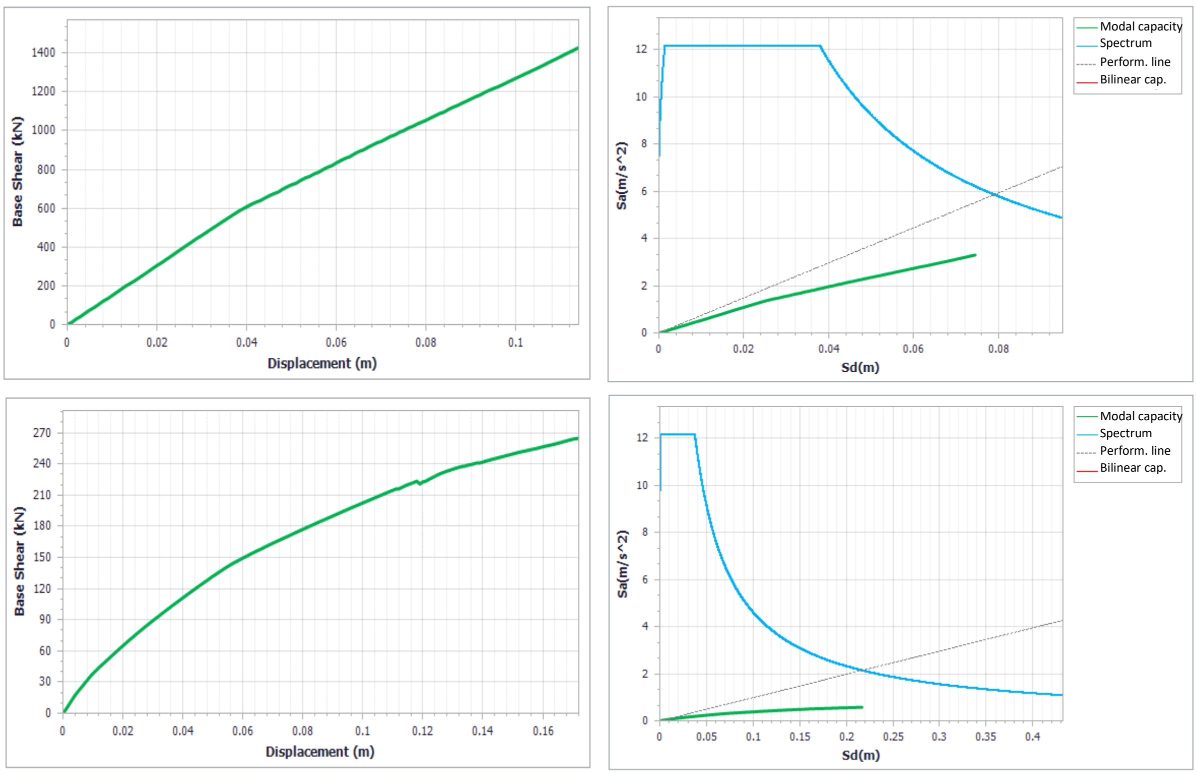

Figure 13. Curves of pushover and demand/capacity x direction (up) and y direction (down).

Lateral displacements provided at the performance points under earthquake effects are shown in Fig. 14. The deformations occurring at the performance point are given for the fibers of column sections 214 and 234 (Fig. 15). These members were chosen randomly to show the deformations in the fibers. The results obtained from the analysis for all storeys and the performance evaluations are given in Table 8 for vertical and lateral load bearing elements.

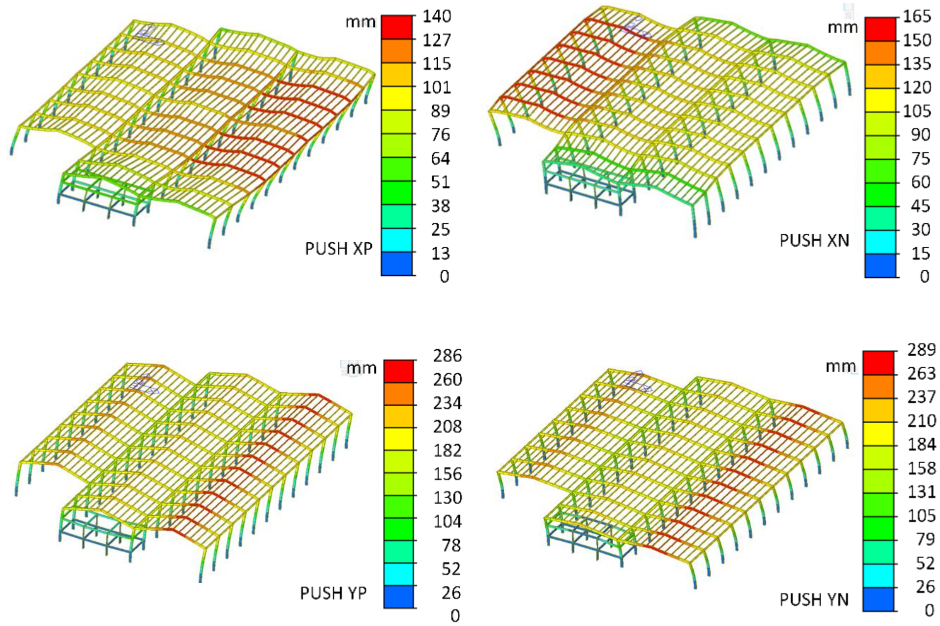

Figure 14. Displacement contour of the structure under seismic effects in positive (P) and negative (N) directions.

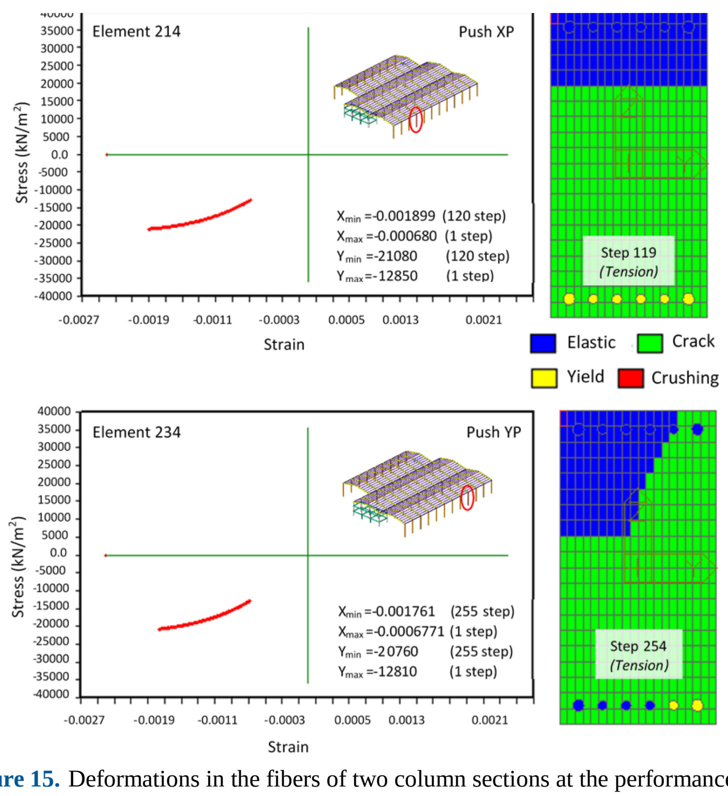

Figure 15. Deformations in the fibers of two column sections at the performance point.

Table 8. Performance assessment results.

Story	Strain check		Brittle failure check		Target performance level (Controlled Damage)
	Column	Beam	Column	Beam	
Ground	×	×	√	√	×

For the “controlled damage” performance level in the examined structure:

- The regulation does not allow member in the "advanced damage" region for prefabricated buildings. Although there is RC principal beam in the “collapse” region, there is a beam in “minimum damage”, “significant damage” or “advanced damage” region. Therefore, the regulation condition is not met.
- Reinforced concrete vertical bearing elements are corresponded in the "limited damage", "significant damage" or "advanced damage" region. In the regulation for the examined structure, it is not allowed to have member in the “advanced damage” region. For this reason, the regulation condition is not satisfied.
- There is no RC element damaged in “brittle” due to the absence of any elements in the collapse area of the building. Therefore, the brittleness condition specified in the regulation is satisfied.

Evaluating the obtained results, it was determined that the existing structure does not deliver the controlled damage performance level for the earthquake and it has only 10% probability of exceedance in 50 years highlighted in TBEC 2018.

6. Conclusions

This study consists of seismic performance of an existing single-storey industrial RC precast facility from the latest code’s perspective. In the numerical analyses, the structural safety of the structure is determined by checking the results with the requirements of the controlled damage. In this scope, the damage level of the structural elements is estimated for the seismic forces acting on x- and y-orthogonal directions, separately. Following results are achieved based on the damage assessment provided for the structural members where the structure is pushed at its performance point in agreement with the seismic code. Evaluating all these results, it was deemed that the structure does not deliver the “Controlled Damage” level to respond the earthquake with 10% (return period: 475 years) probability of exceedance in 50 years. Therefore, it is specified that the building does not provide sufficient structural safety and should be evacuated for required precautions. The structural safety can be ensured by retrofitting with a proper approach. It is expected that the presented methodology will assist the researchers and current practice as example guidance for the evaluation and analysis of this kind of RC facilities in determining the seismic performance and safety issues.

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