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PARAMETRIC ANALYSIS OF THE PERFORMANCE OF STEEL-CONCRETE COMPOSITE STRUCTURES DESIGNED WITH TBDY 2018

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Abstract

Original scientific paper

In this study, the seismic behavior of steel-concrete composite buildings designed using ÇYTHYE 2016 and TBDY 2018 was investigated. For this purpose, composite moment resisting frame buildings with concrete filled steel tube columns and composite beams with 5, 10, 15 and 20 stories are modeled. Buildings are designed at high ductility (DCH) levels. During the design of the DCH class structures, the design was carried out for the ZA soil class for 0.79 g PGA in the region selected from the earthquake map given in the regulations. Within the scope of the study, SeismoStruct software was used during the design and performance evaluation of the structures. Incremental dynamic analyzes were used along with nonlinear static pushover analyses. In the static pushover analysis, uniform and triangular load distributions of the lateral load are adopted. In the dynamic analysis, 16 earthquake ground motions were obtained from AFAD earthquake acceleration databases according to the relevant design area and used. The variation of the seismic behavior of CMRFs depending on the variation of the floor number was investigated using nonlinear analysis results. Accordingly, the variation in lateral response, overstrength factors and ductility factors for CMRF structures are presented comparatively. In addition, the section deformation capacities were investigated during the IDR changes during dynamic and static nonlinear analyses. The behavior factor of all CMRFs, especially the CMRFs studied in the case study, demonstrated above-expected performance according to the design assumptions.

Keywords: *Composite moment resisting frame; concrete filled steel tube column; incremental dynamic analysis; nonlinear pushover analysis.*

TBDY 2018 İLE TASARLANAN ÇELİK-BETON KOMPOZİT YAPILARIN PERFORMANSININ PARAMETRİK ANALİZİ

Özet

Orijinal bilimsel makale

Bu çalışmada, ÇYTHYE 2016 ve TBDY 2018 kullanılarak tasarlanan çelik-beton kompozit binaların sismik davranışı incelenmiştir. Bu amaçla kat sayısı 5, 10, 15 ve 20 olan beton dolgulu çelik tüp kolonlu ve kompozit kirişlerinde oluşan moment aktaran çerçeve binalar modellenmiştir. Binalar yüksek süneklik (DCH) seviyelerinde tasarlanmıştır. DCH sınıfı yapıların tasarımı esansında yönetmeliklerde verilen deprem haritasından seçilen bölgede, 0.79 g PGA için ZA zemin sınıfı için tasarım gerçekleştirilmiştir. Çalışma kapsamında yapıların tasarımı ve performans değerlendirmesi sırasında SeismoStruct yazılımı kullanılmıştır. Doğrusal olmayan statik itme analizleri ile birlikte artımlı dinamik analizler kullanılmıştır. Statik itme analizinde yanal yükün düşeyde düzgün ve üçgen yük dağılımları uygulanmıştır. Dinamik analizde ise 16 deprem yer hareketi igili tasarım alanına göre AFAD veri tabanlarından elde edilerek kullanılmıştır. Kat sayısının değişimine bağlı olarak CMRF'lerin sismik davranışlarının değişimi doğrusal olmayan analiz sonuçları kullanılarak araştırılmıştır. Buna göre, CMRF yapıları için yanal tepki, aşırı güç faktörleri ve süneklik faktörlerindeki varyasyon karşılaştırmalı olarak sunulmuştur. Ayrıca dinamik ve statik doğrusal olmayan analizleri esnasında IDR değişimleri esnasında kesit deformasyon kapasiteleri araştırılmıştır. Tüm CMRF'lerin davranış faktörü, özellikle vaka çalışmasında incelenen CMRF'ler olmak üzere tasarım varsayımlarına göre performanslarının beklenenin üzerinde olduğu ortaya konmuştur.

Anahtar Kelimeler: *Kompozit moment aktaran çerçeve; beton dolgulu çelik tüp kolon; artımsal dinamik analiz; doğrusal olmayan itme analizi.*

1 Introduction

Strong ground motions caused by earthquakes can cause damage to buildings such as houses, workplaces, schools, hospitals where people spend part or all their daily lives, and these damages can cause serious material and moral problems on people. In the earthquakes that occurred in Turkey, for example, in Erzincan, Adana-Ceyhan,

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Afyon-Dinar and especially in the 1999 Marmara earthquake, a similar situation was encountered. In addition to the loss of life, heavy damage to our residential, commercial and industrial structures also brought a serious burden to Turkey's economy [1,2]. A large part of Turkey's surface area is located on active fault lines such as the regions where industry and population are concentrated, the North Anatolian Fault Line, the East Anatolian Fault Line, and the fact of earthquake is important for Turkey. Today, rapid, and important developments are being made about special building systems and elements produced or designed to dampen strong dynamic effects such as earthquake forces in structures and limit the damage that may occur. Intensive experimental and analytical studies are carried out for the production and design of such structural systems, especially in countries such as Japan, Italy, and America. Among the current studies conducted for this purpose, there are composite construction systems consisting of combining the superior properties of concrete and steel [3–10]. In this context, the production of load-bearing elements produced by working together of composite structures, namely concrete and steel, gains more importance in earthquake zones countries such as Turkey. In Turkey, where we live with the reality of earthquakes, it is important to design structures in accordance with the updated earthquake regulation.

Considering the characteristics of the building during the design, the coefficients at which the earthquake effects are reduced are recommended to the designer with national or international regulations. The suitability and validity of these factors depend on the suitability of the assumptions made about the designed system properties. On the other hand, it is known that the characteristics of earthquake loads acting on or likely to affect the structure are very variable and the behavior arising from the design features of the structure can be quite variable under the influence of these dynamic loads [11–13]. In addition, the definitions related to the design seismic force reduction factor are called the response factor (q factor) in the European code [14] and the response modification factor (R factor) in the American codes [15,16] and these are detailed according to the structure type. SEAOC guidelines interpret these factors [17] as a system quality factor, ie a system performance factor. Despite all these different definitions, although different design codes are used for the same building typologies, it is seen that there are small differences in these values used during the building design [8–10]. Minor differences in design R or q factors used in the calculations are due to changes in the building system and material-specific safety factors defined in design codes during the calculation of building materials and design loads. For example, when making earthquake calculations in national and international designs, the values of vertical load combinations and horizontal load factors can go up to 1.8 (ASCE 7-10 [18], ACI 318-14 [19] and AISC- 360-10 [20], Eurocode -2 [21]). In addition to the design features of the structures designed according to different standards from the previous studies on composite structures, the effects of the systems designed with different element section properties on the seismic behavior of the structure were investigated by the researchers. Various studies have been conducted to evaluate parameters related to these properties, connection points [22,23], shear interaction [24], number of floor effect [25] and seismic performance [26,27].

Composite elements, where the superior properties of existing reinforced concrete or steel elements can be produced together thanks to appropriate engineering designs, and it is possible to combine the superior properties of both materials against earthquake effects, are frequently preferred in regions where intense earthquakes occur. It is of great importance that such structures can be applied and designed, especially in countries located between earthquake zones such as Turkey. When looking at the general design codes, national and international codes are constantly updated. When the updated national codes are evaluated, it is important to verify and re-evaluate the theoretical and practical information. Thanks to the design programs produced today, adaptive traditional material and element modeling techniques have become rapidly available. This is important in the transition of experimentally applicable models from element-based to whole-structure. For this reason, it has become easier to create models closer to reality thanks to developing modeling techniques and program capacities. Within the scope of this study, the behavior of the structures designed with TBDY 2018 and theoretically designed at the intersection of two different earthquake zones was analyzed analytically. After the design, it is aimed to help the designer in the evaluation of possible structural behaviors by examining the effects of design parameters on composite moment resisting frame (CMRF) structures in terms of dynamic effects by using nonlinear material and element methods. In this context, this case study was conducted on structures produced using the updated new national earthquake design code TBDY 2018 and national steel structure specification ÇYTHYE 2016. Evaluation of these behavior factors, which are used especially in earthquake regulations, by using different parameters within the scope of these two regulations, is important in terms of evaluating the response of the building to seismic effects. Within the scope of the study, the effects of these parameters were evaluated, especially in terms of elastic and inelastic behavior, with analytical studies used based on element and system.

Within the scope of the study, an analytical study of the behavior of composite structures under earthquakes has been examined. In the analyzed structural models, the analytical models of composite moment resisting frames (CMRF), the structural system consisting of IPE steel beams, also known as composite beams and square section (SHS) CFST columns, which are in full interaction with the slab, were examined. During the modeling of 5-, 10-, 15- and 20-story CMRF structures under earthquake loads, the designs were completed using the Regulation on the Design, Calculation and Construction Principles of Steel Structures 2016 (ÇYTHYE 2016) [28] and 2018 Turkey Building Earthquake Regulation (TBDY) [29].

Static pushover analysis (PO) and incremental dynamic analysis (IDA) were used to determine the earthquake performance of the buildings. The deformations of the elements in the CMRFs in the cross-sectional materials and the response of the system according to various parameters were evaluated. As a result, the performances of the buildings modeled within the scope of TBDY 2018 were examined comparatively with static and dynamic methods for a specific soil feature used in the design. A flowchart of the method followed is presented in Fig. 1.

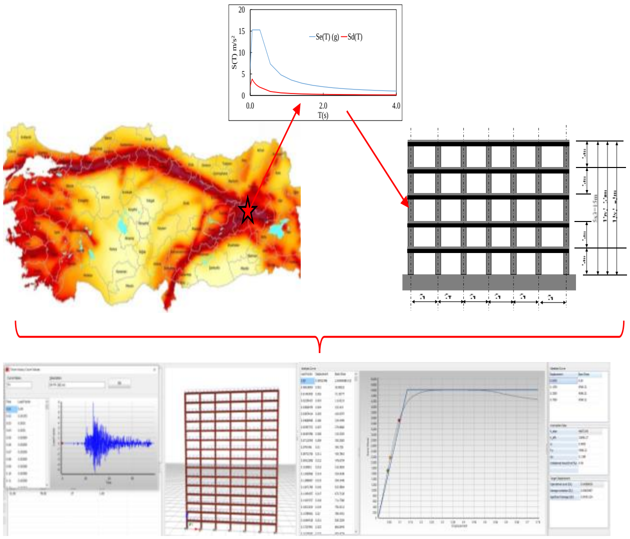


Figure 1. Flowchart of methodology.

2 Example Structures

The columns, which are the vertical carriers of the building, consist of elements obtained by filling the cores surrounded by SHS steel tube with concrete. These elements are defined as concrete filled steel tube, CFST, sectional composite columns in the literature. The beams were dimensioned by using elements with IPE type section in the analyzes made during the design under the influence of gravity loads and horizontally earthquake loads during the design. In all MRF systems, the designs are completed by assuming that the frame beams are fixed to the columns in a way that will transfer the moment fully.

The slabs consist of a solid section cast-in-place reinforced concrete slab system with full shear connection on the main beams. The columns are basically assumed to be rigidly fixed in both directions. The floor heights of the CMRF structures are used in the design, their total height from the ground is taken into consideration as 15, 30, 45 and 60m for the 5, 10, 15 and 20 story structures, respectively. Model buildings have a floor height of 3m on the first and subsequent floors. Modeled CMRF buildings have 6 spans in x and y directions, and each span is included in the calculations as 7m. In this case, the total width in both plan directions, that is, in the x and y directions, appears as 42m (Fig. 2).

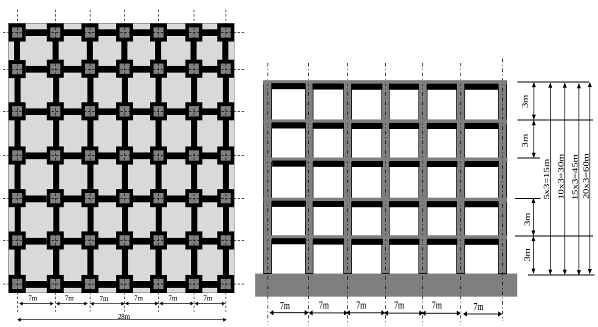


Figure 2. Schematic view of plane ad elevation view.

Models of buildings consisting of CMRFs systems are positioned on the assumption that they can be constructed at a location with known ground values (Latitude: 39.298011° Longitude: 41.014378°) in Bingöl Province Karlıova District Yeşilyurt Mahallesi during the design phase. The assumed ground conditions for this geographic location are assumed to be the same as for soils of class ZA.

The steel class used during the design in the structural system elements was determined as S235 for structural steel class SHS section and IPE sections. Moreover, the concrete class used in the design in the concrete sections with composite beams and reinforced concrete sections was determined as C30 and reinforcing steel bars class were S420. Calculations for dimensioning the structural elements of buildings as CMRF and the analytical models used to evaluate their performance are given in Fig. 3. All of the analyzes made for the purpose of creating the analytical model and then making the designs and finally evaluating the performance were carried out with the SeismoStruct [30] computer software.

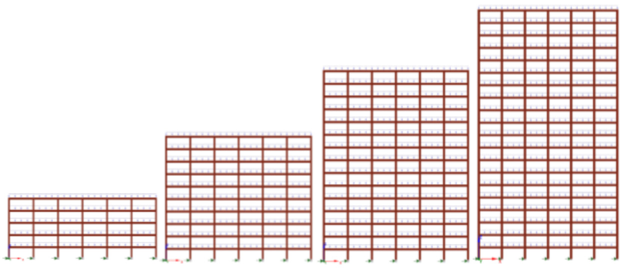


Figure 3. SeismoStruct program view of CMRFs.

2.1 SeismoStruct Program

SeismoStruct [30] computer program, which has the ability and computational capacity to use different nonlinear modeling techniques, was used to examine the earthquake effect of CMRFs designed within the scope of this study. If the SeismoStruct program is examined; While calculating the behavior of spatial frameworks during analysis with static and/or dynamic effects, they can analyze structural elements with the help of both material and geometrically non-linear models and calculate the response of the structure and its elements. In addition, seven different structural analyzes can be made with this software by using the technical features in the database of the software. To give a brief name to these analyzes; (i) dynamic under semi-permanent loading, (ii) static time-history analysis under semi-permanent loading, (iii) conventional pushover analysis, (iv) adaptive pushover analysis, (v) incremental dynamic analysis, (vi) modal analysis and (vii) static analysis (probably non-linear for the last two). Thanks to its large database, the software allows the use of elements with distributed elasticity and bulk plasticity evaluated on formulations based on force or displacement properties, and models are derived from theoretical and experimental data [31]. Although the defined numerical models work with different assumptions during the calculations, the basic input parameters used for these elements during the use of the models are the basic physical properties such as the cross-section geometry and uniaxial behavior of the materials used.

2.2 Structural Design

During the design of the CMRF system, for dimensioning the sections of beam and column elements, the section effects obtained from the SeismoStruct [30] computer software were used by using the loading conditions based on the ÇYTHYE 2016 and TBDY 2018 Regulations. For 5-, 10-, 15- and 20-story structures, CFST composite column sections SHS steel section members with dimensions of 450x12, 500x32, 600x40 and 750x50 (outside diameter x wall thickness) were used, respectively. For the beams, elements with IPE 350 section were selected. The dead loads of the materials in the cross-sections of the structural elements were calculated automatically by the computer software and were taken into account during the analyses. The total dead load remaining in the buildings is calculated as 3 kN/m^2 and the live load is 2.0 kN/m^2. Required elastic design spectral accelerations, which are used to determine horizontal earthquake loads in the field, are calculated automatically by the software according to the natural vibration period of the structure calculated during the analysis in the software. In the creation of the horizontal elastic design spectrum, DD-2 earthquake ground motion, which has a 10% probability of exceeding in 50 years, is taken into account. and also the local soil class was obtained depending on ZA. In order to create the horizontal elastic design spectrum, it is necessary to find the spectral acceleration coefficients and determine the ground effect coefficients. In accordance with the geographical location selected for the model building, spectral acceleration values were determined from the map by using Turkey Earthquake Hazard Maps [14]. In accordance with TBDY 2018 Section 2.3.3, local ground effect coefficients were obtained based on the local soil class and local ground effect coefficients for the short-term region and the local ground effect coefficients for the 1.0 second period. The dumping rate was taken as 5%. Using the Turkey Earthquake Hazard Maps [32], the short-period map spectral acceleration coefficient was read as SS= 1.947 and the 1.0 second map spectral acceleration coefficient as $S1$= 0.514. The highest ground acceleration was obtained as PGA=0.791g and the highest ground speed as PGV=60.469 cm/s.

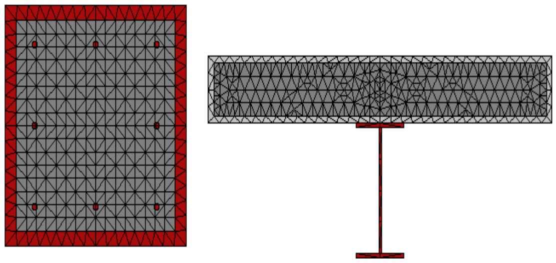
The features of the building structural system are MRF systems with CFST columns with high ductility, and the beams in these MRF systems are designed as composite elements with IPE section. In this case, in accordance with TBDY 2018 Section 4.3.2.2, structural system behavior coefficient R and extreme strength coefficient D can be used for steel structural systems in column systems with composite section. In this case, according to TBDY 2018 Table 4.1, for the buildings examined within the scope of the study, all earthquake effects are carrier systems designed as MRF systems and buildings that are carried by steel frames with high ductility level will be considered. In this case, the coefficients R=8 and D=3 will be taken as basis. The General Analysis Method was used to calculate the required strengths of the structural elements, and the Design with Load and Strength Coefficients Method was used for the dimensioning. As a requirement of this method, the axial and shear stiffnesses of all the members of the system (in this example, composite columns and frame beams) and the bending stiffnesses of the frame beams are reduced by multiplying by a factor. This coefficient was taken as 0.8 according to ÇYTHYE 6.2.3. The reduction coefficient applied to the bending stiffnesses of the composite columns was obtained as 0.8x0.8=0.64 according to ÇYTHYE 6.2.3(b) and 12.2.5(d).

The natural vibration periods calculated at the end of the analyzes using SeismoStruct computer software were obtained as 0.816, 1.602, 2.354 and 3.098 s for 5-, 10-, 15- and 20-storey structures, respectively. Total CMRF weights were calculated from the software as 7582.7, 16127.3, 25490.5 and 36070.9 kN in the same order. In addition, with these data obtained, base shear forces were calculated as 420.1, 465, 487, and 512.5 kN from the software. According to the results of the analysis, it has been observed that the structural system of the building, which has a smooth geometry, does not contain any irregularities in the plan and vertically under the effects of earthquakes. It has been observed that the effective relative story drifts and second order effects do not exceed the limit values defined in article 4.9 of TBDY 2018. Columns are dimensioned in accordance with the Design, Calculation and Construction Principles of Steel Structures (ÇYTHYE) Regulation 2016 12.3.2. For the axial force-bending moment interaction diagram of the column cross-section, ÇYTHYE Table 12.5, in which the plastic stress distribution method is used, was used. In addition, in accordance with TBDY 2018 9.11.4.2, the levels of axial compressive forces of all composite columns meet the condition N_{dm}≤$0.40Pno$. N_{dm} is defined as, the largest of the axial compressive forces calculated under the joint effect of vertical loads and earthquake loads (by considering the live load reduction coefficients defined for live loads in TS 498 [33]. P_{no} is defined as, compressive force strength of composite element cross-section with bi-symmetry axes under axial load. The design of the moment-transferring frames was carried out in such a way that the columns were stronger than the beams at all beam-column joint points for each earthquake direction, in accordance with TBDY 2018 9.11.2.2.

For 5-, 10-, 15-, and 20-story structures, CFST composite columns cross-sections of 450x12, 500x32, 600x40 and 750x50 (outside diameter x wall thickness) SHS cross-section elements were used, respectively. The beams are specified as IPE 350. The reinforced-concrete slab thicknesses, on the other hand, are dimensioned as 150 mm.

2.3 Nonlinear Analytical Models of the CMRF System

The nonlinear behavior results of CMRF model structures designed using ÇYTHYE 2016 Regulation [28] and TBDY 2018 [29] were examined through analytical models developed using SeismoStruct [30] computer software. In the software, it is ensured that the analyzes are concluded by considering the structural and geometric secondary effects for all analyzes. Obtained results were evaluated comparatively. For the nonlinear behavior of columns and beams in CMRFs, element models based on the spread plastic behavior approach were used in the analysis. Accordingly, it is assumed that the fiber-shaped section model is used for the plastic behavior of beams and columns in the model and the plastic behavior spreads across the section and length section. Therefore, models consisting of a finite number of fiber elements in the cross-sections of beams and columns were used in these analyses. More importantly, it is assumed that full adherence is provided between the section elements in the concrete and steel section that make up the composite section. The properties of the plastic behavior in these regions are determined directly by the SeismoStruct software based on the material properties. If the analytical models defined during the analysis in the software packages are examined, it can be easily seen that it works with different assumptions depending on the principle used during the calculations. However, in the use of models for the realization of calculations, the basic calculation parameters used in the software input or the element to be calculated for the calculation input are the basic physical and mechanical properties such as the cross section and the geometry of this section, the uniaxial behavior of the material to be used. After defining the sections of the CFST column elements and composite beams used in the CMRF system within the scope of the study, the sections are included in the models as fiber elements as shown in Fig. 4.

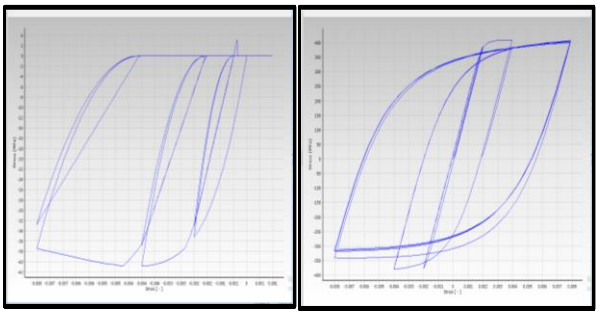


(a) (b)

Figure 4. Fiberized section views for (a) CFST columns and (b) composite beam.

Slabs are usually not directly included in the models during elastic analysis of the structural system produced during design but are assumed to form rigid diaphragms in each floor plane. The loads of the frame beams together with the vertical loads passing through the floors are defined as uniformly distributed loads. Characteristic material strengths were used in nonlinear behavior models of steel and concrete materials used in CMRF elements. The necessary coefficients to obtain these values, the data in TBDY 2018 Table 5.1 were used. Accordingly, the expected material strengths predicted for the characteristic compressive strength of concrete and the characteristic yield strength of the S235 steel class are accepted as $1.3fck$ and $1.5Fy$, respectively. In the model used for the nonlinear behavior of the steel material, a hardening coefficient of 0.005 was used.

Bilinear steel model is used for steel modelling in SeismoStruct [30] software and this is defined as "stl_bl" material model in the software. Tensile strength is neglected in the stress-strain curve of the concrete material. In addition, the "con_ma" material model in the software was used for the non-linear behaviour of the material while modelling the concrete in the SeismoStruct [30] software. Both models were developed for the cycling loading condition. The concrete material constitutive model image is shown in Fig. 5 (a) and steel material constitutive model image of the model is given in Fig. 5 (b).



(a) (b)

Figure 5. a) Concrete and b) steel models from SeismoStruct (2018).

During the non-linear analysis in the time-history (TH), each was performed using earthquake ground motions under a constant gravitational load. During the TH analysis, in which the earthquake effects are simulated, while calculating the gravity load values, the building floor weight, that is, the fixed loads, which are effective in the earthquake, plus 30% of the live load are included in the dead load, and the calculations are completed. The analyzes consist of two parts. In the first stage, PO analyzes consist of two parts: uniformly distributed horizontal loading (ULD-PO) and triangular horizontal loading (TLD-PO). The second phase is the application phase of incremental dynamic analysis (IDA) and TH earthquake ground motions. Earthquake ground motions consist of 8 ground motion pairs. When selecting the records of earthquakes, large-scale earthquakes between the North Anatolian fault line and the East Anatolian fault line surrounding the city of Karlıova, which is assumed to have been built, were used (Fig. 6).

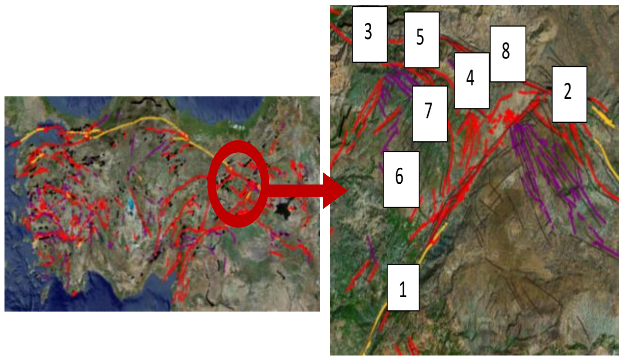


Figure 6. Selected ground motion schematic representation.

The application multipliers of the incremental effects were chosen as 0.05, 0.10, 0.20, 0.30, 0.40, 0.50, 0.60, 0.80, 1.00, 1.20, 1.40, 1.50, 1.75, 2.00, 2.30, 2.60, 2.90, 3.20, 3.50, 3.80, and 4.00. It is aimed that the total number of analyses will be 8x2=16. In this case, the seismic demand values and other calculated parameters were determined by taking the average of 16 IDA analysis results. The characteristics of the earthquake records used in the study are given in Table 1. The information about the earthquake movements in Table 1 was taken from the AFAD ground motion database [34].

Table 1. Properties of earthquake ground motions.

Record ID	Record Seq. #	Station ID	Event Date	Epicentral Distance (km)	Event Depth (km)	M_L/M_W	Component
TH-1 TH-2	2183	1133	1.05.2003	11.8	6	6.6/-	East West
TH-3 TH-4	2896	1206	25.08.2007	2.19	15.8	5.1/-	East West
TH-5 TH-6	23	2402	13.03.1992	12.82	23	6.1/-	East West
TH-7 TH-8	10099	1212	14.06.2020	16.2	8	-/5.7	East West
TH-9 TH-10	24	2402	15.03.1992	45.32	29	5.4/-	East West
TH-11 TH-12	1828	2306	25.06.2021	30.88	15.51	-/5.2	East West
TH-13 TH-14	6027	6202	2.12.2015	37	10.66	-/5.3	East West
TH-15 TH-16	2587	1208	14.03.2005	54.09	9.9	5.4/-	East West

In this study, the panel regions of the joints were modelled with the help of behavioural models originally developed by Della Corte et al. (2000) using the modified Richard-Abbott model. This model included in the SeismoStruct (2018) software includes this model, which can model all kinds of steel and composite connections (eg welded-flange bolted-web connection, extended end plate connection, recessed end plate connection, angled connection, etc.) thanks to its features. The calculation model basically has increasing and decreasing curve segments based on the moment-rotation relationship and calculating using this relationship. Contrary to the load case, the increasing and decreasing parts of the curve are created with different parameters that take the period into account in the model. The presence of both compressive and tensile reversible starting points with these parameters (i.e. initial stiffness, strength, post-limit stiffness, shape factor, compression-related calibration coefficients, damage rate, and isotropic hardening) and complex parameters can also be included in the analysis. The versatility of this type of modeling using SeimoStruct software has been validated using experimental data from previous studies, and researchers have proven that the element and material models used converge sufficiently to experimental studies when used in analytical models [8,35–39]. In addition, some parameters have been calibrated to achieve greater accuracy in modeling based on the application of the component method [40].

Using the existing earthquake records, each selected ground motion pair is scaled with the earthquake spectrum with a return period of 475 years, with a 10% probability of exceedance in 50 years, which is defined as a design earthquake. At the end of the analyses carried out under these ground motion records; the structures were mutually evaluated according to the design for the targeted performance parameters.

3 Results and Discussion

The response of CMRFs because of ULD-PO and TLD-PO analysis is shown in Fig. 7. In the graphs given, the horizontal axis is the ratio of the roof displacement to the building height, and the vertical axis is the ratio of the ground shear to the building weight. In 5-story CMRFs, IDA analysis presents a behaviour that lies between the first-mode dominant response and the higher-mode response. However, on the other hand, the IDA results obtained in 10-, 15- and 20-storey structures are parallel to the ULD-PO results, so it can be said that higher modes dominate in these structures [8,10,41]. The IDA was performed by using selected TH records to obtain the seismic response of the case study CMRFs. The dynamic behaviour of the structures is also plotted in Fig. 7.

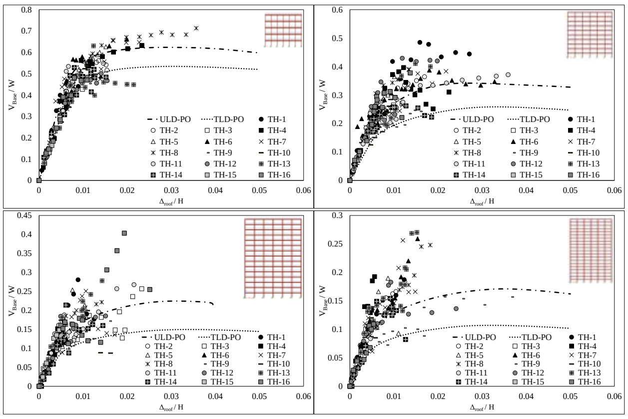


Figure 7. PO and IDA graphs for CMRF structures a) 5, b) 10, c) 15, and e) 20 stories.

3.1. Ductility Factor

This factor identifies the behavior that the behavior is making without significant reduction in maximum strength under seismic horizontal forces. The ratio of ductility in the literature is widely used in use the elastic deformation rating of a structural system under the influence of a particular earthquake or horizontal load. The mathematical definition of this ratio is expressed as follows as ductility ratio of displacement function [8–10,42]. That is, the displacement ductility ratio μ (ductility demand) can be expressed as:

$$\mu = \frac{\Delta_u}{\Delta_y} \quad (1)$$

Yield and ultimate displacement values are Δ_y and Δ_u, respectively, in Eq. (1). As a result of IDAs, it was calculated from Fig. 7 as 1.81, 1.86, 1.42 and 1.74 for 5-, 10-, 15- and 20-story CMRF structures, respectively. The μ values obtained as a result of the analyses with TLD-PO were approximately 17% larger for the 5-, and 15-storey structures compared to those obtained with IDA. On the other hand, the μ value for the 10-story structure is 10% lower for TLD-PO than for μ calculated from the IDA results, while it is almost the same for the 20-story structure (0.8% difference). Considering the analyses made with ULD-PO, the μ value in 5-, 10-, and 15-storey buildings is 12%, 27% and 6% higher, respectively, than the values obtained from IDA. However, the μ value obtained from ULD-PO of the 10-story structure is 8% smaller than that obtained from IDA. While the μ value obtained from the dynamic method (IDA) was 1.71 on average, this value was calculated as 1.85 and 1.80 for the static methods, ULD-PO and TLD-PO (Fig. 8).

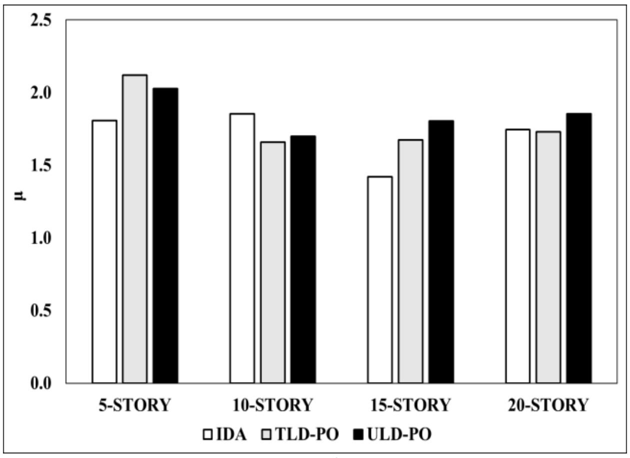


Figure 8. Ductility factor variation.

3.2. Overstrength Factor

When describing the nonlinear response of structures, the load-displacement relationship is often assumed to be elasto-plastic. Within the scope of this study, the structural extreme strength factors expressed by the following equation were calculated from the Figure obtained with IDAs for each structure:

$$\Omega_{\mathrm{d}} = \frac{V_{\mathrm{y}}}{V_{\mathrm{d}}} \qquad (2)$$

Yield and design base shear values are displayed as V_y and V_d, respectively, in Eq. (2). As a result of experimental and theoretical studies conducted by researchers for Ω_d, an important performance parameter of the building, it has been shown that this factor plays an important role in protecting buildings from collapse in the face of severe earthquakes. It has been reported in the literature that this factor for steel and reinforced concrete structures varies between 1.8 and 6.5 for long-term and short-term structures [13]. In this study, IDA results showed that the Ωd factors of CMRFs reached 10.97, 12.53, 13.23 and 12.23 values in 5-, 10-, 15- and 20-storey structures, respectively. As a result of the analysis, the Ω_d values obtained with TLD-PO are approximately 10%, 21% and 14% smaller for 10-, 15- and 20-storey structures, respectively, than those obtained with IDA. On the other hand, when the Ω_d value is calculated for the 5-storey building, it is slightly (in the order of 1%) calculated from the IDA results for TLD-PO. Considering the analyses made with ULD-PO, the Ωd value for 5-, 10-, 15- and 20-storey buildings is 13%, 35%, 45% and 43% smaller, respectively, than the values obtained from IDA. While the Ω_d value obtained from the dynamic method (IDA) was calculated as 12.2 on average, this value was calculated as approximately 8 and 11 for the results of the static methods, ULD-PO and TLD-PO (Fig. 9).

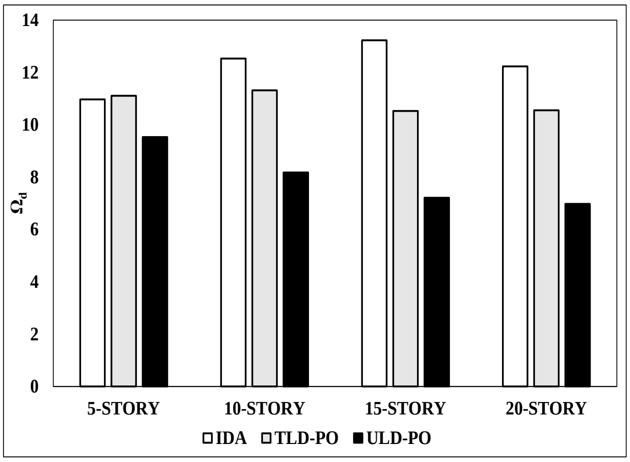


Figure 9. Overstrength factor variation.

3.3. Inherent Overstrength Factor

Elnashai and Mwafy [43,44] recently suggested a measure of response termed 'inherent overstrength factor. Inherent overstrength factor (Ω_i) is formulated as below;

$$\Omega_{\mathrm{i}} = \frac{V_{\mathrm{y}}}{V_{\mathrm{e}}} \qquad (3)$$

Yield and elastic base shear values are given as V_y and V_e, respectively, in Eq. (3). The suggested measure of response Ω_i reflects the reserve strength and the anticipated behaviour of the structure under the design earthquake. Clearly, in the case of $\Omega_i \geq 1.0$, the global response will be almost elastic under the design earthquake, reflecting the high overstrength of the structure. If $\Omega_i < 1.0$, the difference between the value of Ω_i and unity is an indication of the ratio of the forces that are imposed on the structure in the post-elastic range [13]. When the values obtained within the scope of the study were examined, the values of the Ω_i parameter were obtained as 1.39, 1.57, 1.65 and 1.53 for 5-, 10-, 15- and 20-story CMRF, respectively.

As a result of the analysis, the Ωd values obtained with TLD-PO are approximately 10%, 21% and 14% smaller for 10-, 15- and 20-storey structures, respectively, than those obtained with IDA. On the other hand, when the Ω_i value is calculated for the 5-storey building, it is slightly (in the order of 1%) calculated from the IDA results for TLD-PO. Considering the analyses made with ULD-PO, the Ω_i value for 5-, 10-, 15- and 20-storey buildings is 13%, 35%, 45% and 43% smaller, respectively, than the values obtained from IDA. While the Ω_i value obtained from the dynamic method (IDA) was calculated as 1.54 on average, this value was calculated as approximately 1 and 1.36 for the results of the static methods, ULD-PO and TLD-PO (Fig. 10). These values show that structures can easily withstand elastic deformation design earthquakes.

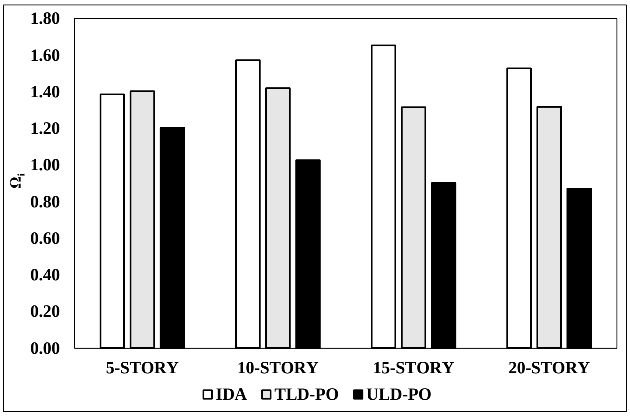


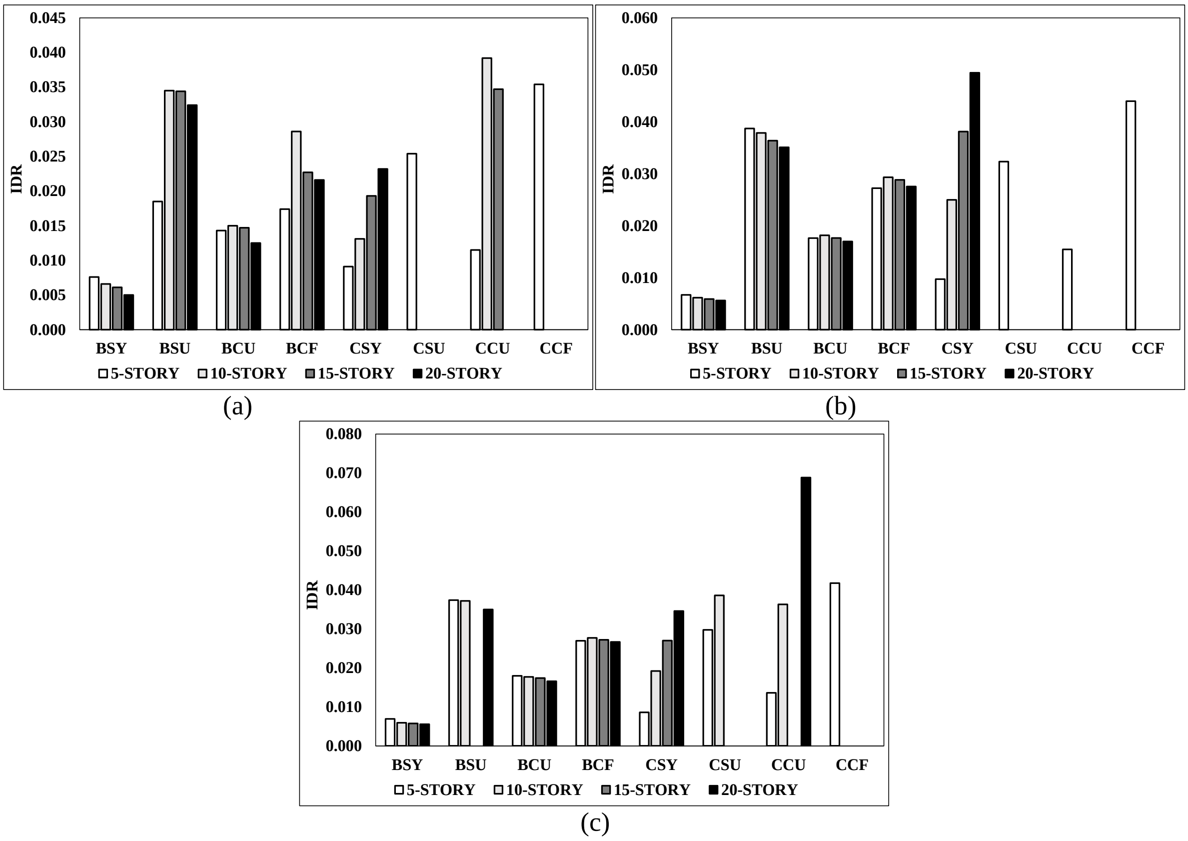
Figure 10. Inherent strength factor variation.

3.4. Composite Section Capacities

In this part of the paper, the examination of the deformations in the structural elements because of the non-linear dynamic analysis carried out in the systems formed by the CMRF structures is presented. The deformation states and definitions that occur in the mentioned structural elements are summarized in Table 1. The deformations in the CFST column cross sections in the CMRF system and in the sections of the composite beams, which are formed by the combination of IPE section and solid slab, are examined within the scope of this section. During the examination, IDR (interstory drift ratio) values were taken into consideration. The cross-sectional deformations obtained during IDA and given in the Table 2 were evaluated within the 3 parameters mentioned above. Obtained results are presented in the Fig. 11.

Table 2. Deformation states and definitions

Deformation	Definition
BSY	In the composite beam, the steel has reached yield elongation at the outermost fiber.
CSY	In the composite column, the steel has reached yield elongation at the outermost fiber.
BSU	Steel reached its ultimate capacity in the composite beam.
CSU	Steel reached its ultimate capacity in the composite column.
BCU	Concrete reached its ultimate capacity in the composite beam.
CCU	Concrete reached its ultimate capacity in the composite column.
BCF	In the composite beam, the concrete converged to the elongation at crushing limit.
CCF	In the composite column, the concrete converged to the elongation at crushing limit.



(a) (b) (c)

Figure 11. Section deformation with IDR a) IDA, b) TLD-PO and c) ULD-PO.

Deformations in composite beams were investigated as BSY, BSU, BCU and BCF. The IDR values observed in the limit values are calculated and presented in the Fig. 11. Deformations to BSY limit values were observed at IDR values of 0.0076, 0.0066, 0.0061 and 0.005 calculated for 5-, 10-, 15- and 20-storey buildings, respectively, from the IDA analysis. ULD-PO results, on the other hand, showed that these deformation values were 9%, 10% and 6% lower for 5-, 10- and 15-storey structures, respectively, compared to the IDR values obtained from IDA. Also, in the 20-story structure, the ULD-PO results are 11% greater than the IDR values obtained from IDA. The BSU deformation limit was found to be 0.0185, 0.0345, 0.0344 and 0.0324 IDR values in IDA analysis for 5-, 10-, 15- and 20-storey buildings, respectively. On the other hand, looking at the results obtained with TLD-PO, it is 109%, 10%, 6% and 8% greater than the IDR values obtained from the IDA results for BSU. For ULD-PO, no BSU deformation was observed in the 15-story structure. In 5-, 10- and 20-storey buildings, it is 102%, 8% and 8% higher, respectively, according to the IDR values calculated from IDA. The BCU deformation occurring in the concrete section of the composite beams reaches the limit state at the end of the IDA when the IDR values are 0.0143, 0.015, 0.0147 and 0.0125 in 5-, 10-, 15- and 20-storey structures, respectively. These IDR values calculated from ULD-PO analysis are 26%, 18%, 18% and 33% greater than those calculated from IDA for 5-, 10-, 15- and 20-storey structures, respectively. In addition, TLD-PO analysis results show that these deformations occur at IDR values larger than the IDA result, and this difference is calculated to be 23%, 21%, 20% and 36%, respectively. The other deformation, BCF deformation, which occurs in the concrete section of composite beams, reaches the limit state because of IDA when IDR values are 0.0174, 0.0286, 0.0227, 0.0216 in 5-,10-, 15- and 20-storey buildings, respectively. These IDR values calculated from ULD-PO analyses are 55%, 20% and 23% greater than those calculated from IDA for 5-, 15- and 20-storey structures, respectively. In the 10-storey structure, it is 3% smaller. In addition, TLD-PO analysis results show that these deformations occur at IDR values larger than the IDA result, and this difference is calculated to be 56%, 3%, 27% and 28%, respectively (Fig. 11).

The deformation states occurring in CFST elements are presented as CSY, CSU, CCU and CCF. For CSY deformations limit state IDA analysis, the IDR value was calculated as 0.0091, 0.0131, 0.0193 and 0.0232 for a 5-, 10-, 15- and 20-storey building, respectively. In ULD-PO analyses, on the other hand, the IDR values at which this deformation occurs are 47%, 40% and 49% larger in 10-, 15- and 20-storey buildings, respectively, according to IDA analyses. However, the IDR value at which this deformation occurs is 6% smaller in a 5-storey building. Considering the CSU deformation, it was observed only in 5-story structure in IDA analysis, 5- and 10-story structure in ULD-PO analysis, and only 5-layer structure in TLD-PO. The IDR value at which this deformation occurs in ULD-PO and TLD-PO in the 5-story structure is 17% and 27% higher, respectively, compared to the IDA analysis. In ULD-PO, on the other hand, when the IDR value is as high as 0.04, it is in the limit state. As a result of IDA, the CCU deformation limit state occurring in the concrete cores of CFST elements was found to be 0.0115, 0.0392 and 0.0347 in 5-, 10- and 15-storey structures, respectively. While it is not observed for IDA in the 20-story structure, the IDR value was calculated as 0.07 in the 20-storey structure in the analysis made with ULD-PO. In TLD-PO analysis, while the IDR value was 0.0154 in the 5-story structure, the CCU deformation limit was formed. CCF, on the other hand, is seen in 5-story structures in IDA, ULD-PO and TLD-PO analysis and the IDR values seen were calculated as 0.035, 0.042 and 0.044, respectively (Fig. 11).

4 Conclusion

In this study, the performance evaluation of the structures designed with the ÇYTHYE 2016 and TBDY 2018 design codes when they are built on solid and hard rocky soils (ZA group soil) was examined by evaluating the deformations occurring in the sections of the elements as well as various performance parameters. For this, static pushover analysis and incremental dynamic analysis were performed. PO analysis made with two different lateral load patterns. It has been observed that the structures have reached the global yield limit with the effect of design earthquakes, and therefore, a behavior dominated by intense plastic behavior on element basis has begun to occur. When the obtained data are examined, the following conclusions can be reached:

- When the μ values are examined, the obtained values are greater than 1.4. This shows that CMRF structures can adequately absorb earthquake effects by ductile horizontal displacements.
- For buildings designed as CMRF, the Ωd parameter calculated using IDA analysis using selected regional earthquakes was obtained as at least 10.97. In addition, the lowest score in PO analysis is 6.98. As a result, they have higher performance factors than calculations with R=8 design factor.
- When the Ωi values are examined, it is seen that almost all of the obtained values are greater than 1 for the IDA results. In this case, the structures show that they will absorb earthquake energy thanks to their flexible behaviour.
- When the composite beams are examined according to the IDR change, it is seen that the behaviour is within the elastic limits when the IDR value is 0.005. When the IDR value is above 0.02, the plastic behaviour is dominant in composite beams. However, this situation emerges as a decreasing IDR value as the number of floors increases. In addition, when the IDR value in beam sections approaches 0.015, the plastic behaviour observed in concrete emerges.
- In addition, when CFST columns are examined according to IDR change, it can be said that when the IDR value is 0.01, the behaviour is within elastic limits and then plastic deformations occur. Plastic behaviour is common in CFST columns when the IDR value is above 0.03. However, this situation emerges as an increasing IDR value as the number of stories increases.

Declaration

Ethics committee approval is not required.

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