Investigation of the effects of different central steel brace type on nodal point connection detail and building behaviour

Farklı merkezi çelik çapraz tipinin düğüm noktası birleşim detayına ve bina davranışına etkilerinin incelenmesi

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Abstract

High strength and ductility properties of steel structures have been the reason for the preference against earthquake effects. However, it has observed that significant damages occurred in particularly the nodal point connection of the steel structures in the investigations made after the earthquakes. Therefore, it is very important to know the effect of different central steel brace and nodal point connection type on steel structure behavior. The main purpose of this study is to comparatively examine the effects on the behavior of steel structures of the different central braced types and nodal point connection detail. The findings obtained from structural analyses reveal that different central steel brace members contribute positively to the behavior of steel structures. This result shows that central steel braced that increase the performance of steel structures are very important. In Turkey, particularly in terms of the safety and performance of the steel structures to be constructed in earthquake zones is proposed to prefer steel structural systems with bidirectional central steel braces.

Keywords: Center steel braces, Design of steel structures, Sta-Steel, Steel structural systems

Öz

Çelik yapıların yüksek dayanım ve süneklik özelikleri deprem etkilerine karşı tercih edilme nedeni olmaktadır. Ancak depremler sonrasında yapılan incelemelerde çelik yapıların özellikle birleşim bölgelerinde önemli hasarların meydana geldiği gözlemlenmiştir. Bu nedenle farklı merkezi çelik çaprazların ve düğüm noktası birleşiminin türünün çelik yapı davranışına etkisini bilmek çok önemlidir. Bu çalışmanın temel amacı farklı merkezi çapraz tiplerinin ve düğüm noktası birleşim detayının çelik yapıların davranışlarına etkisini karşılaştırmalı olarak incelemektir. Yapısal çözümlemelerden elde edilen bulgular, farklı merkezi çelik çapraz elemanların çelik yapıların davranışlarına olumlu katkı sağladığını ortaya koymaktadır. Bu sonuç çelik yapıların performansını artıran merkezi çelik çaprazların oldukça önemli olduğunu göstermektedir. Türkiye'de özellikle deprem bölgelerinde inşa edilecek çelik yapıların emniyeti ve performansı bakımından iki yönlü merkezi çelik çaprazlara sahip çelik taşıyıcı sistemlerinin tercih edilmesi önerilmektedir.

Anahtar kelimeler*: Merkezi çelik çaprazlar, Çelik yapıların tasarımı, Sta-Steel, Çelik taşıyıcı sistemler*

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1. Introduction
1. Giriş

It is seen that the most common use of steel structures in Turkey is in industrial buildings. On the other hand, after the earthquakes that took place in recent years, the buildings have increased in the interest in the use of steel materials, too. This situation further increases the importance of constructing steel structures in accordance with the design and technique. Because the main feature expected from the structures built and/or to will be built in Turkey where almost all lands are at earthquakes hazard is that they are resistant to earthquakes. However, to minimize the structural damages resulting from earthquakes that will occur, it is necessary to correctly understand the earthquake behaviours of the structures in question and make appropriate designs. For this purpose, some researchers have investigated the behaviour of steel structural systems with different stiffness elements added to reduce the effects of especially earthquakes (Gönen, 1997; Tama, 2003; Ay et al., 2010; Kural & Zeybek, 2011; Özçelik, 2016; Yelgin & Bulut, 2016; Çavdar, 2017; Çavdar, 2019).

The selection and sizing of steel structural systems is related to the number of stories. Because as the number of stories increases, horizontal loads such as wind and earthquake acting on the structure to increase and these increasing loads are more effective than vertical loads. In cases where this horizontal load acting on the steel structure is more effective according to the vertical load, the importance of braced frame types more increases to meet the said horizontal loads. In the design of steel structures, frames with central steel braces can be used, as well as frames with external central steel braces, too. However, the easiest to design and most preferred especially in the construction of industrial structures are steel frames with central braces. But the ductility of central braced steel frames is lower than other types of frames (external central steel braced frames and moment-transferring steel frames). In addition, brittle fractures formed in the brace nodal point connections of the central braced frames are as the biggest problem. For this reason, the design of brace nodal point connections should be calculated according to the maximum force that can be transferred from the braces.

This article aims to determine the effects of the central steel brace types used in the design of steel structures in Turkey on the performance of the structure in question. For this purpose, by taking the moment-transmitting frame as the reference model (model 1), the X, V, Λ and / diagonal braced structural models were created according to the conditions of the Turkish Building Earthquake Code (TBEC, 2019) and the Regulation on the Design, Calculation and Construction Principles of Steel Structures (ÇYTHYDE, 2018). Structural analyses of the central braced steel structure models selected as an example was made with the Sta-Steel program (Sta-Steel, 2019). Thus, some conclusions were presented by comparing the obtained findings.

2. Frames with central steel brace by Turkey building earthquake regulations
2. Türkiye bina deprem yönetmeliğine göre merkezi çelik çaprazlı çerçeveler

Steel braced frames are systems that can form a central or external center and can carry horizontal loads. Central Steel Braced Frames (CSBF) systems are formed by connecting the axes of columns, beams and cross members forming the nodal point by overlapping. In addition, CSBF systems can be designed in two ways: ductility level limited (DLL) and ductility level high (DLH). External Center Braced Steel Frames (ECBSF) systems are formed by connecting the cross members to the nodal points as an external center. However, ECBSF systems can only be designed as DLH.

In CSBF systems with DLH, even if some of the brace members exposed to pressure effects are twisted, dimensioning in such a way that significant strength loss does not occur in the structural system should be done. On the other hand, CSBF systems are damping the earthquake energy to a significant extent when the braces subjected to pressure effects reach the yield strength by buckling and the braces subjected to tensile effects reach the yielding state due to tensile effects. Therefore, CSBF systems show very good plastic behaviour. In addition, since the brace members in these systems meet all the horizontal loads, all plastic deformations are expected to occur in these elements, while columns, beams, and nodal points are required to remain in the elastic region (Türk, 2016). However, since they are designed to withstand large dynamic effects, their ductility is lower than that of moment-transmitting frames. In the Turkish Building Earthquake Code (TBEC), which came into force in 2019, CSBF systems are given in Figure 1.
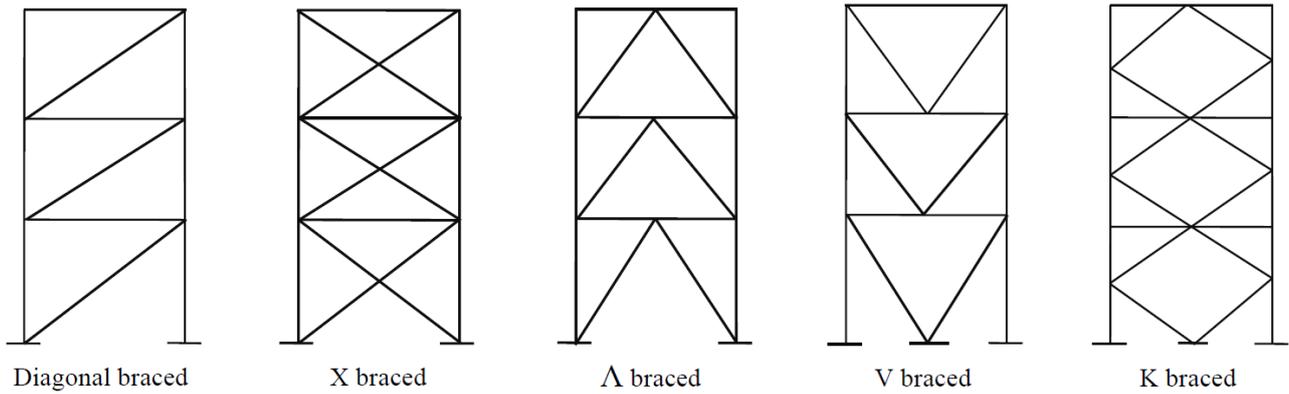
Figure 1. Types of CSBF according to Turkish Building Earthquake Code (TBEC, 2019)
Şekil 1. *Türkiye bina deprem yönetmeliğine göre merkezi çelik çaprazlı çerçeve (CSBF) tipleri (TBEC, 2019)*

3. Study method
3. Çalışma metodu

The numerical applications of this study, it consists of 8 steel building models, symmetrical in plan and with three spans having each span 6 m in both directions, and 5 stories with each storey 3 m high. The structural systems of the formed building models consist of 8 different models as moment-transmitting frame and CSBF (/, X, V and Λ). In addition, it is accepted that the selected building models are symmetrical in both directions to avoid additional section effects due to structural torsion (Cengiz, 2020).

All columns HE 300B, main beams IPE 270, intermediate beams in the y-direction IPE 240 and x-direction IPE 220 of each steel structure model (moment transmitting frame, X braced frame, V braced frame, Λ braced frame and / diagonal braced frames) were created with profiles. In structural analyses, nodal point connections have modelled as rigid before forming bolted connections. In addition, the structural analyses of the CSBF building models were carried out according to the ZB local soil class (Cengiz, 2020). Other features of the steel buildings models selected as examples are given Table 1.

Table 1. Other structural features of the steel buildings models
Tablo 1. *Çelik bina modellerinin diğer yapısal özellikleri*

| Models	Frame type	All columns	Main beams	Intermediate beams x-direction	Intermediate beams y-direction	Brace profiles using in the outer axles	Explanation
model 1	Moment transmitting frame	HE 300B	IPE 270	IPE 220	IPE 240	-	see Figure 2a
model 2	Central X steel braced frame	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2b
model 3	Central V steel braced frame	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2c
model 4	Central Λ steel braced frame	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2d
model 5	Central / steel braced frame (Λ formed on the corner columns)	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2e
model 6	Central / steel braced frame (V formed on the corner columns)	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2f
model 7	Central / steel braced frame (// formed on the corner columns)	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2g
model 8	Central / steel braced frame (\\ formed on the corner columns)	HE 300B	IPE 270	IPE 220	IPE 240	R140x10 box	see Figure 2h
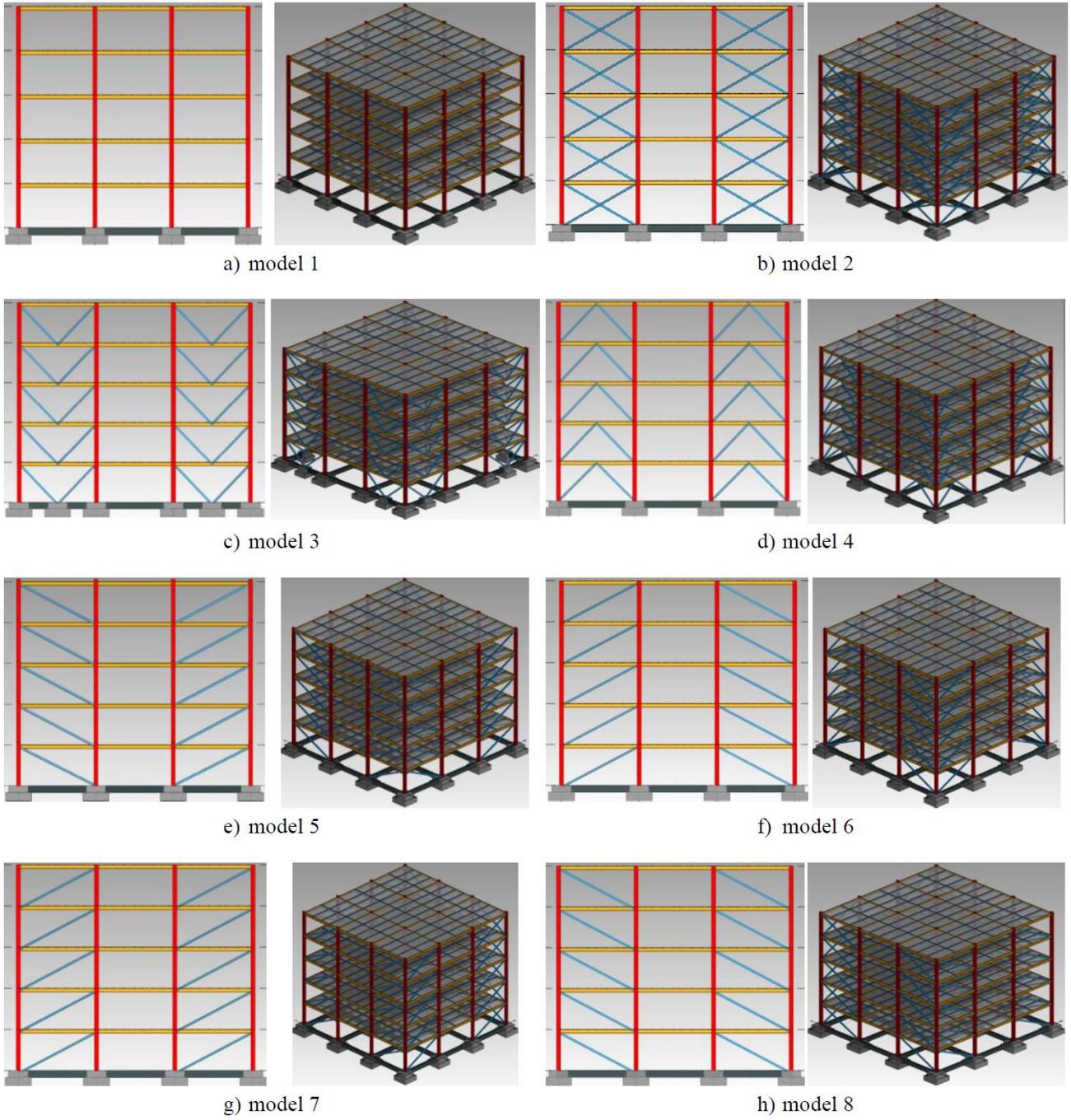
Figure 2. External axle section and 3D view of models
***Şekil 2.** Modellerin dış aks kesiti ve 3 boyutlu görünümü*

The foundation view and dimensions of the steel building models considered in this study are shown in Figure 3. In addition, the semi-rigid nodal point details of the central steel braced frames are formed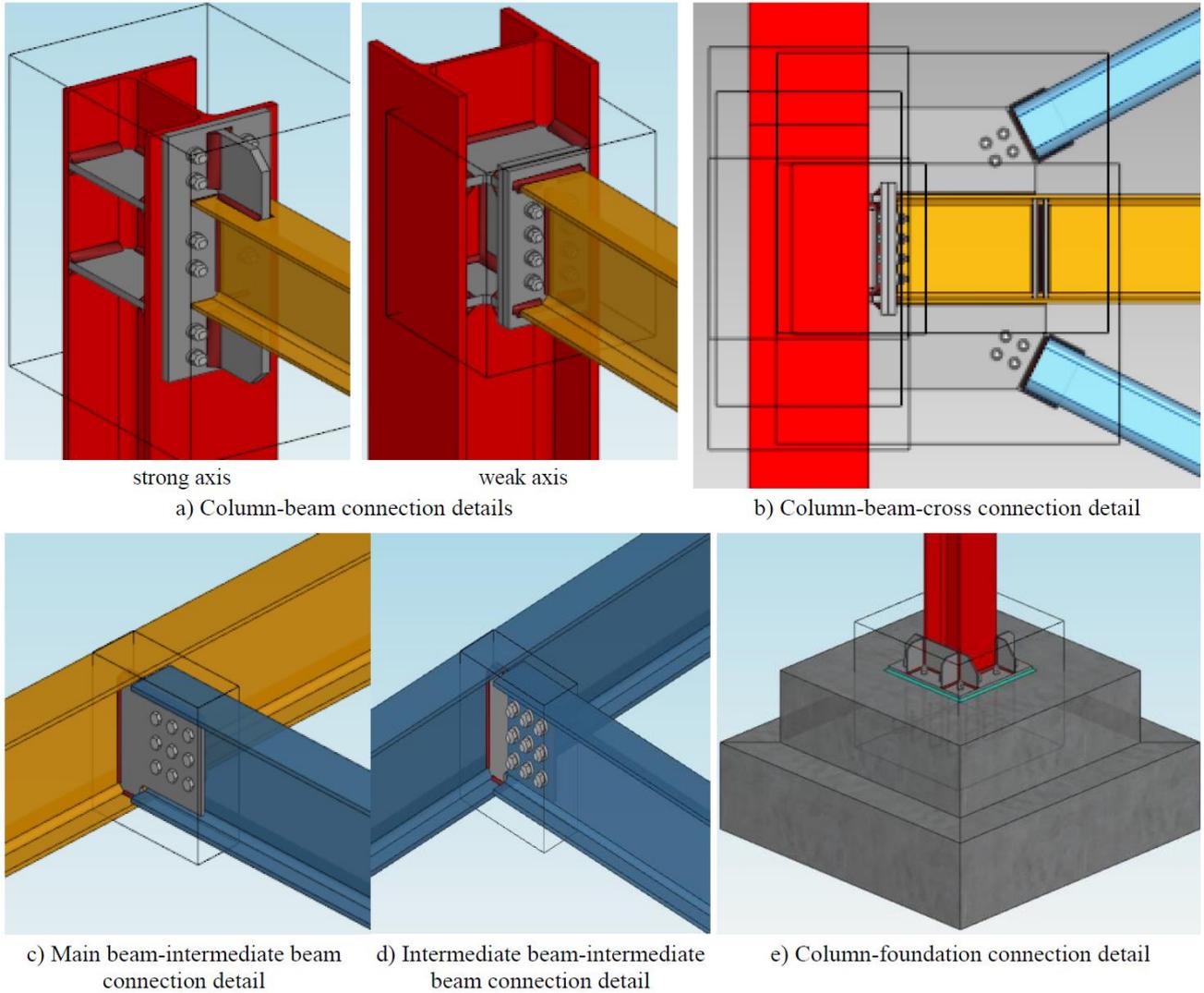 bolted and welded as shown in Figure 4. Plate thicknesses seen in this figure are 20 mm and weld thicknesses are considered as 14 mm.

Here, it would be useful to point out that the rigid column-beam nodal points are welded and the weld thickness in these nodal points is 14mm.
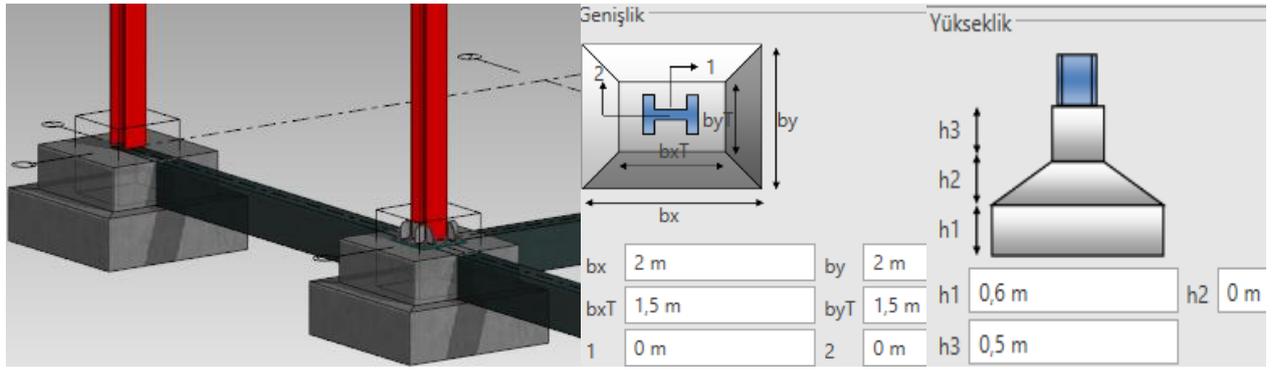
Figure 3. View and dimensions of the foundation system of steel building models
Şekil 3. Çelik bina modellerinin temel sisteminin görünümü ve boyutları

strong axis   weak axis
a) Column-beam connection details

b) Column-beam-cross connection detail

c) Main beam-intermediate beam connection detail

d) Intermediate beam-intermediate beam connection detail

e) Column-foundation connection detail

Figure 4. Details of nodal point created as bolted and welded of the considered frames
Şekil 4. Dikkate alınan çerçevelerin bulonlu ve kaynaklı olarak oluşturulan düğüm noktası birleşim detayları

3.1. Material properties of structural elements of steel building models

3.1. Çelik bina modellerinin yapısal elemanlarının malzeme özellikleri

European profiles were used for dimensioning the

structural elements. Accordingly, columns, beams and brace members have designed with S275 steel. Material properties related to S275 steel are given in Table 2. In addition, 8.8 bolt class has used in nodal point connections. The properties related to this bolt class are given in Table 3, too. Other

design parameters of the building models are also given in Table 4. The snow load given in Table 4 was selected according to the province of Kütahya (TS EN 1991-1-3, 2007). In addition, TS 498 regulation was used in the calculation of the wind load affecting the building models (TS-498, 1997).

Table 2. Characteristic yield stress, F_y and tensile strength, F_u of S275 material
***Tablo 2.** S275 malzemesinin karakteristik akma gerilmesi, F_y ve çekme dayanımı, F_u*

Steel class	Characteristic thickness, t (mm)			
	t ≤ 40 mm		40 mm ≤ t ≤80 mm	
	F_y (N/mm²)	F_u (N/mm²)	F_y (N/mm²)	F_u (N/mm²)
EN 10025-2 S275	275	430	255	410

Table 3. Characteristic yield stress and tensile strength of bolt class
***Tablo 3.** Bulon sınıfının karakteristik akma gerilmesi ve çekme dayanımı*

Bolt class	Characteristic yield stress, F_{yb} (MPa)	Characteristic tensile strength, F_{ub} (MPa)
8.8	640	800

Table 4. Other design parameters of building models
***Tablo 4.** Bina modellerinin diğer tasarım parametreleri*

Building importance coefficient (residence and workplace), I		1
Structural system behaviour coefficients, (R_x ve R_y)		4
Live load participation coefficient (residences and workplace)		0.30
Longitude according to Turkey earthquake map		30.048
Latitude according to Turkey earthquake map		39.386
Short period map spectral acceleration coefficient (S_s)		0.8399
Map spectral acceleration coefficient for 1s period (S_1)		0.2122
Short period design spectral acceleration coefficient (S_{ds})		0.7559
Design spectral acceleration coefficient for 1s period (S_{d1})		0.1698
Earthquake ground motion level		DD2
Earthquake design class		DTS1
Strength excess coefficients (D_x ve D_y)	Moment transmitting frame	2.5
	Center braced frames	2
Foundation soil bearing capacity for ZB local soil class (kN/m²)		700
Foundation bedding coefficient for ZB local ground class (kN/m³)		100000
Equivalent earthquake load comparison coefficient (β)		0.80
Earthquake eccentricity (e)		0.05
Horizontal elastic design acceleration spectrum corner period, T_A (s)		0.04492
Horizontal elastic design acceleration spectrum corner period, T_B (s)		0.22457
Transition period to constant displacement in the spectrum of horizontal elastic design, T_L(s)		6
Snow load (kN/m²)		1.3
Live load (kN/m²)		2
Wind load (kN/m²)	up to 8 m high	0.5
	up to 20 m high	0.8

## 4. Discussion of findings obtained from structural analyses
4. Yapısal çözümlemelerden elde edilen bulguların tartışılması

In order to compare the behaviours of the central steel braced building models considered in this article, the critical nodal points and structural elements have been selected as in Figure 5.

The period and angular velocity values obtained from the structural analyses as rigid and semi-rigid of nodal point connections according to the mode superposition method of the considered DLL central steel braced building models are given in Table 5. From this table, it is seen that the period values obtained from model 1 for the first three modes are larger than the central steel braced building models, in spite of that the angular velocity (ω) values are smaller than the central steel braced building models. On the other hand, in all building models, the period values obtained from the structural analyses carried out with the rigid assumption of the nodal point connections are

larger than those obtained from the structural analyses carried out with semi-rigid assumption of the nodal point connections. This situation reveals that the period values are significantly reduced in all building models created by using central steel brace members. These findings obtained from structural analyses show that the period values of all building models with increased lateral stiffness by using central steel brace members decrease.
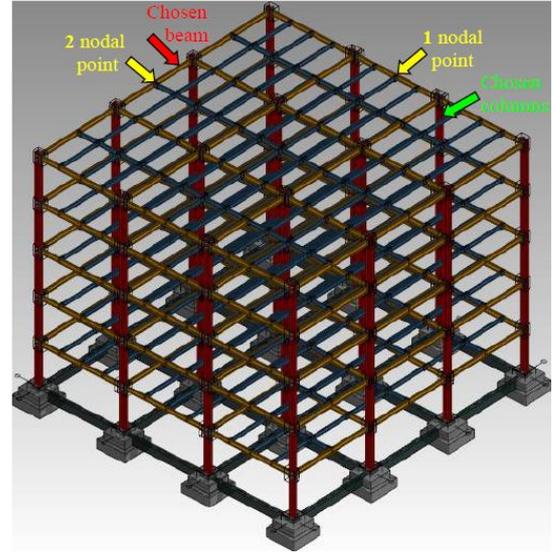
Figure 5. Selected nodal point and structural elements of steel building models
Şekil 5. *Çelik bina modellerinin seçilen düğüm noktası ve yapısal elemanları*

Table 5. Period and angular velocity values obtained from rigid and semi-rigid structural analyses of central steel braced building models
Tablo 5. *Merkezi çelik çaprazlı bina modellerinin rijit ve yarı-rijit yapısal çözümlemelerinden elde edilen periyot ve açısal hız değerleri*

Models		1. mod		2. mod		3. mod	
		T (s)	ω	T (s)	ω	T (s)	ω
Model 1	rigid connection	0.7889	7.964	0.6708	9.367	0.5876	10.693
	semi-rigid connection	0.4755	13.214	0.3929	15.992	0.3790	16.578
Model 2	rigid connection	0.5329	11.791	0.5016	12.526	0.3132	20.061
	semi-rigid connection	0.4551	13.806	0.3762	16.702	0.3207	19.592
Model 3	rigid connection	0.5440	11.550	0.5097	12.327	0.3159	19.890
	semi-rigid connection	0.4581	13.716	0.3789	16.583	0.3218	19.525
Model 4	rigid connection	0.5393	11.651	0.5035	12.479	0.3121	20.132
	semi-rigid connection	0.4588	13.695	0.3783	16.609	0.3212	19.562
Model 5	rigid connection	0.5540	11.341	0.5168	12.158	0.3228	19.465
	semi-rigid connection	0.4565	13.764	0.3752	16.746	0.3269	19.221
Model 6	rigid connection	0.5559	11.303	0.520	12.083	0.3251	19.327
	semi-rigid connection	0.4639	13.544	0.3812	16.483	0.3257	19.291
Model 7	rigid connection	0.5544	11.333	0.5197	12.090	0.3249	19.339
	semi-rigid connection	0.4599	13.662	0.378	16.622	0.3259	19.280
Model 8	rigid connection	0.5551	11.319	0.5185	12.118	0.3236	19.417
	semi-rigid connection	0.4599	13.662	0.3779	16.627	0.3258	19.285

The maximum base shear force values obtained from the structural analyses with the mode superposition and equivalent earthquake load methods for the earthquake effect in the x and y directions with the assumptions of rigid and semi-rigid of nodal point connections with the Sta-Steel program of the building models are given in Table 6. From this table, it is seen that the base shear force values obtained by assuming rigid the nodal point connections are greater than the base shear force values obtained by assuming the semi-rigid. On the other hand, the base shear force values obtained by

the equivalent earthquake load method of the considered building models are greater than those obtained by the mode superposition method in both the x and y directions. In addition, the highest base shear force values according to the mode superposition and equivalent earthquake load methods from the structural analyses carried out by assuming rigid the nodal point connections in both the x and y directions are obtained from model 2. But, from the structural analyses performed by assuming that the nodal point connections are semi-rigid, it is seen that the largest base shear

force values according to the mode superposition method are obtained from model 4 and according to the equivalent earthquake load method are obtained from model 2.

Table 6. Maximum base shear force values obtained from rigid and semi-rigid structural analyses for earthquake effect in x and y directions of building models

Tablo 6. *Bina modellerinin x ve y doğrultularındaki deprem etkisi için rijit ve yarı-rijit yapısal çözümlemelerinden elde edilen maksimum taban kesme kuvveti değerleri*

Models	Connection type	Maximum base shear force values (kN)			
		Earthquake effect in x direction		Earthquake effect in y direction	
		mode superposition method	equivalent earthquake load method	mode superposition method	equivalent earthquake load method
model 1	rigid	84.871	101.19	96.17	113.67
	semi-rigid	40.272	48.934	48.321	61.398
model 2	rigid	138.17	176.74	159.92	187.75
	semi-rigid	62.459	79.935	72.606	96.699
model 3	rigid	131.46	167.54	148.31	178.83
	semi-rigid	62.489	74.885	75.681	90.544
model 4	rigid	130.49	169.39	151.69	181.44
	semi-rigid	64.362	74.341	78.162	90.161
model 5	rigid	127.73	162.86	138.62	174.57
	semi-rigid	59.059	73.508	67.727	89.448
model 6	rigid	129.88	162.28	140.04	173.50
	semi-rigid	57.044	72.304	65.181	87.987
model 7	rigid	129.76	162.38	139.09	173.23
	semi-rigid	58.579	72.939	67.261	88.736
model 8	rigid	129.97	162.54	140.07	174.01
	semi-rigid	58.277	72.898	67.004	88.714

The values of overturning force, overturning moment, moment against overturning and safety coefficient obtained from structural analyses for earthquake effects in the x and y directions with the assumptions of the rigid and semi-rigid of nodal point connections of the building models considered are given in Table 7. From this table, it is seen that the overturning force, overturning moment and anti-overturning moment values obtained by assuming rigid of nodal point connections in the x and y directions are larger than those obtained by assuming semi-rigid of nodal point connections. In addition, it is seen that the largest overturning force and overturning moment values by assuming both rigid and semi-rigid in the x and y directions are obtained from model 2, by contrast with the anti-overturning moment is obtained from model 3.

The maximum displacement and rotational values at 1 and 2 nodes from the structural analyses carried out with the assumptions of rigid and semi-rigid nodal point connections for the earthquake effect in the x and y directions of the DLL central steel braced building models are given in Table 8. From this table, the displacement values at 1 and 2 nodes obtained from model 1 from the structural analyses performed by assuming the nodal point connections as rigid are larger than the other building models considered. By contrast with, the displacement values at 1 and 2 nodes obtained from model 6 from the structural analyses performed by assuming semi-rigid of nodal point connections are larger than the other building models considered. In addition, the rotation values obtained from model 5 at 1 node for the earthquake effect in the x direction and from the model 6 at 2 nodes for the earthquake effect in the y direction are higher than the other building models. These findings reveal that the building models of the central steel braces significantly affect the displacement and rotational values.

Table 7. Overturning force, overturning moment, moment against overturning and safety coefficient values according to rigid and semi-rigid structural analyses for earthquake effect in x and y directions of building models
Table 7. Bina modellerinin x ve y doğrultularında deprem etkisi için rijit ve yarı-rijit yapısal çözümlemelere göre devrilme kuvveti, devrilme momenti, devrilmeye karşı moment ve güvenlik katsayısı değerleri

	Earthquake direction	Connection type	Model 1	Model 2	Model 3	Model 4	Model 5	Model 6	Model 7	Model 8
Overturning force (kN)	X	rigid	139.98	336.27	316.91	321.7	284.79	286.79	284.95	286.42
		semi-rigid	60.135	152.69	142.05	145.13	130.83	130.2	131.57	129.74
	Y	rigid	170.5	339.04	320.36	320.39	304.72	308.32	303.2	307.3
		semi-rigid	71.202	163.49	145.52	146.77	145.62	145.2	144.15	143.4
Overturning moment (kNm)	X	rigid	1336.6	3371.1	3181.5	3199.4	2788	2845.9	2824.5	2835.3
		semi-rigid	607.88	1531	1448.5	1449.8	1303.4	1299.7	1309.1	1294
	Y	rigid	1659.9	3362.6	3185.2	3155.8	3023.9	3026.5	2982.2	3023.2
		semi-rigid	731.66	1640.1	1475.7	1462.3	1451.6	1455	1442.9	1439.5
Moment against overturning (kNm)	X	rigid	60734	62604	66113	61890	61672	61676	61326	61667
		semi-rigid	24297	25730	29452	25228	25058	25058	25059	25058
	Y	rigid	60739	62607	66122	61972	61678	61678	61532	61672
		semi-rigid	24297	25730	29453	25228	25059	25058	25058	25058
Factor of safety	X	rigid	45.44	18.571	20.78	19.344	22.12	21.671	21.712	21.75
		semi-rigid	39.97	16.805	20.332	17.401	19.226	19.279	19.142	19.365
	Y	rigid	36.592	18.618	20.759	19.637	20.397	20.379	20.633	20.4
		semi-rigid	33.208	15.688	19.959	17.253	17.262	17.223	17.367	17.408

Table 8. Displacement and rotation values at 1 and 2 nodal points obtained from rigid and semi-rigid structural analyses for earthquake effect in x and y directions of models
Tablo 8. Modellerin x ve y doğrultularındaki deprem etkisi için rijit ve yarı-rijit yapısal çözümlemelerinden elde edilen 1 ve 2 düğüm noktalarındaki yerdeğiştirme ve dönme değerleri

Models	Connection type	Earthquake effect in the x direction 1 nodal point (18; 9; 15)		Earthquake effect in the y direction 2 nodal point (9; 18; 15)	
		Displacement δ_x (cm)	Rotation θ_y (rad)	Displacement δ_x (cm)	Rotation θ_y (rad)
Model 1	rigid	1.164	0	1.042	0
	semi-rigid	0.792	0.00128	0.678	0.00149
Model 2	rigid	0.993	0	0.950	0
	semi-rigid	0.853	0.00233	0.772	-0.00219
Model 3	rigid	1.004	0	0.962	0
	semi-rigid	0.831	0.00237	0.721	-0.00204
Model 4	rigid	1.004	0	0.945	0
	semi-rigid	0.830	0.00226	0.718	-0.00195
Model 5	rigid	1.015	0	0.984	0
	semi-rigid	0.844	0.00254	0.771	-0.00233
Model 6	rigid	1.01	0	0.987	0
	semi-rigid	0.857	0.00252	0.787	-0.00236
Model 7	rigid	1.003	0	0.983	0
	semi-rigid	0.844	0.0025	0.769	-0.00231
Model 8	rigid	1.004	0	0.981	0
	semi-rigid	0.845	0.00251	0.773	-0.00232

The maximum deflection and displacement values of the beam selected from the structural analyses performed with rigid and semi-rigid assumptions of the nodal point connections of the building models are given in Table 9.

Table 9. The maximum deflection and displacement values of the selected beam from the structural analyses with the rigid and semi-rigid acceptance of the nodal point combination of the building models
Tablo 9. Bina modellerinin düğüm noktası birleşimlerinin rijit ve yarı-rijit kabulüyle yapısal çözümlemelerinden seçilen kirişinin maksimum sehim ve yerdeğiştirme değerleri

Models	Connection type	Beam length L (mm)	Maximum deflection (mm)	Maximum displacement (mm)	Limit deflection ($L/300$)
Model 1	rigid	6000	1.1577	1.8958	20
	semi-rigid	5562	0.34035	0.47405	18.54
Model 2	rigid	6000	1.1495	1.8466	20
	semi-rigid	5562	0.31634	0.45269	18.54
Model 3	rigid	6000	1.0738	1.7802	20
	semi-rigid	5562	0.3076	0.4499	18.54
Model 4	rigid	6000	1.2324	1.8736	20
	semi-rigid	5562	0.41366	0.56269	18.54
Model 5	rigid	6000	1.1574	1.8873	20
	semi-rigid	5562	0.34557	0.48007	18.54
Model 6	rigid	6000	1.1035	1.7994	20
	semi-rigid	5562	0.31537	0.45034	18.54
Model 7	rigid	6000	1.1022	1.8216	20
	semi-rigid	5562	0.33417	0.47037	18.54
Model 8	rigid	6000	1.1365	1.8535	20
	semi-rigid	5562	0.33141	0.46712	18.54

From this table, it is seen that the maximum deflection and displacement values obtained by assuming the nodal point connections as rigid are greater than those obtained by assuming the semi-rigid nodal point connection. In addition, the largest deflection values with both rigid and semi-rigid assumptions are obtained from model 4, by contrast with the displacement values are obtained from model 1 with the assumption of rigid and model 4 with the assumption of semi-rigid.

The displacement distributions at the storey levels along the building height from the structural analyses with assumptions of the rigid and semi-rigid of the nodal point connections for earthquake effect in the x and y directions of the building models are given in Figure 6-9, respectively. From these figures, it is seen that the displacement distributions at the storey levels of the building models having central steel brace members are smaller than the displacement distributions obtained from model 1. In addition, the displacement distributions obtained by assuming that the nodal point connections are rigid are larger than the displacement distributions obtained b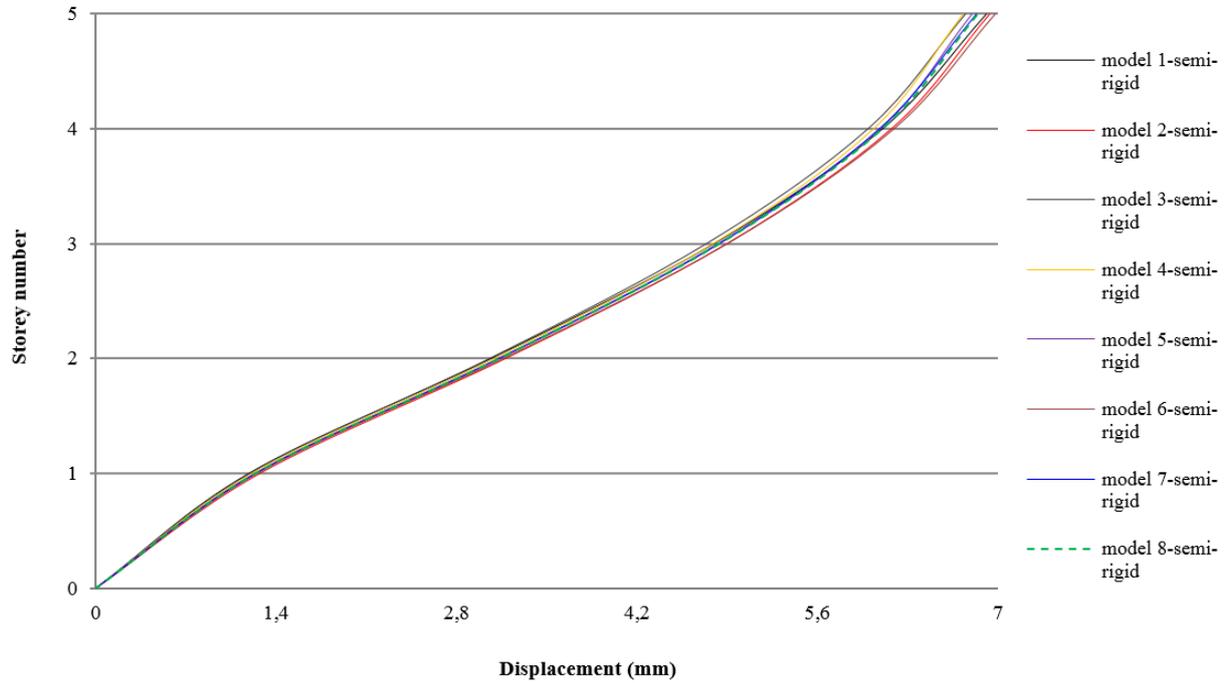y assuming that the nodal point connections are semi-rigid. These findings prove that the displacement distribution values obtained by assuming that the nodal point connections of the building models with different central steel braces are both rigid and semi-rigid have decreased significantly compared to model 1 that the reference model. In other words, this situation reveals that these building models perform better than the reference model.

The variation of the earthquake load with the central steel brace ratio, from the structural analyses performed with rigid and semi-rigid assumptions of the nodal point connections of the building models is given in Figure 10. From this figure, it is seen that the earthquake load increases with the increase of the central steel brace ratio. In addition, the earthquake load values obtained by assuming that the nodal point connections are rigid are greater than the earthquake load values obtained by assuming that the nodal point connections are semi-rigid. This finding reveals how important the nodal point connections are in steel structures and that they significantly affect the earthquake load values.
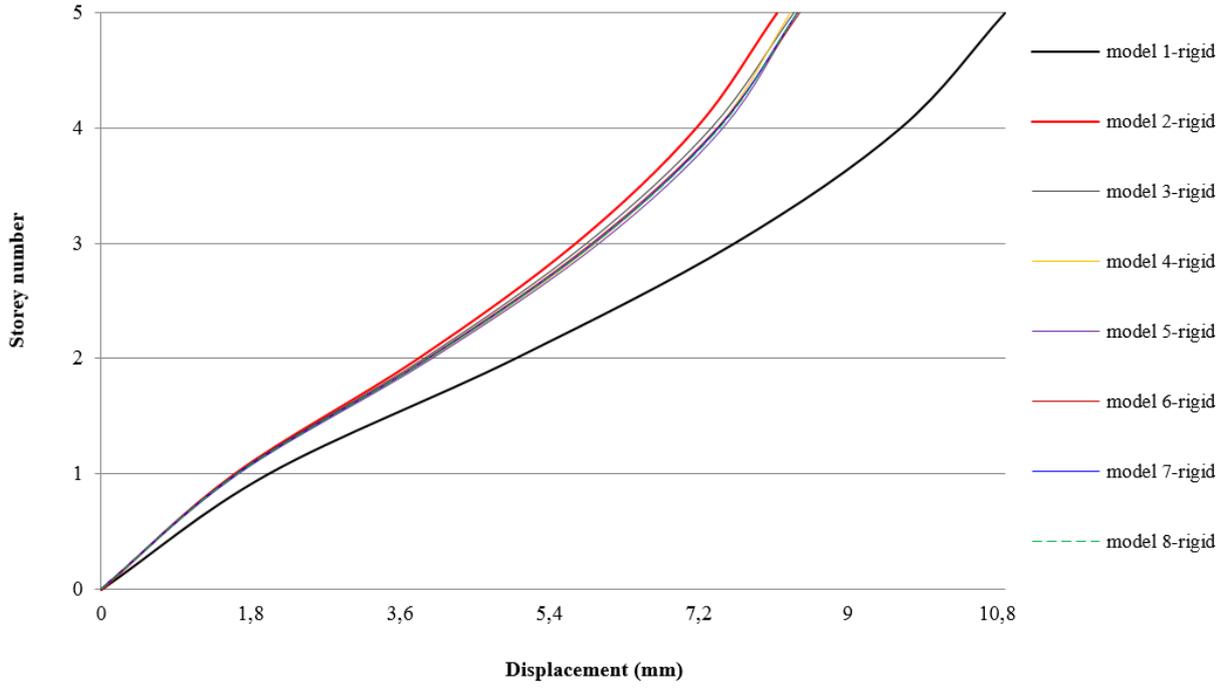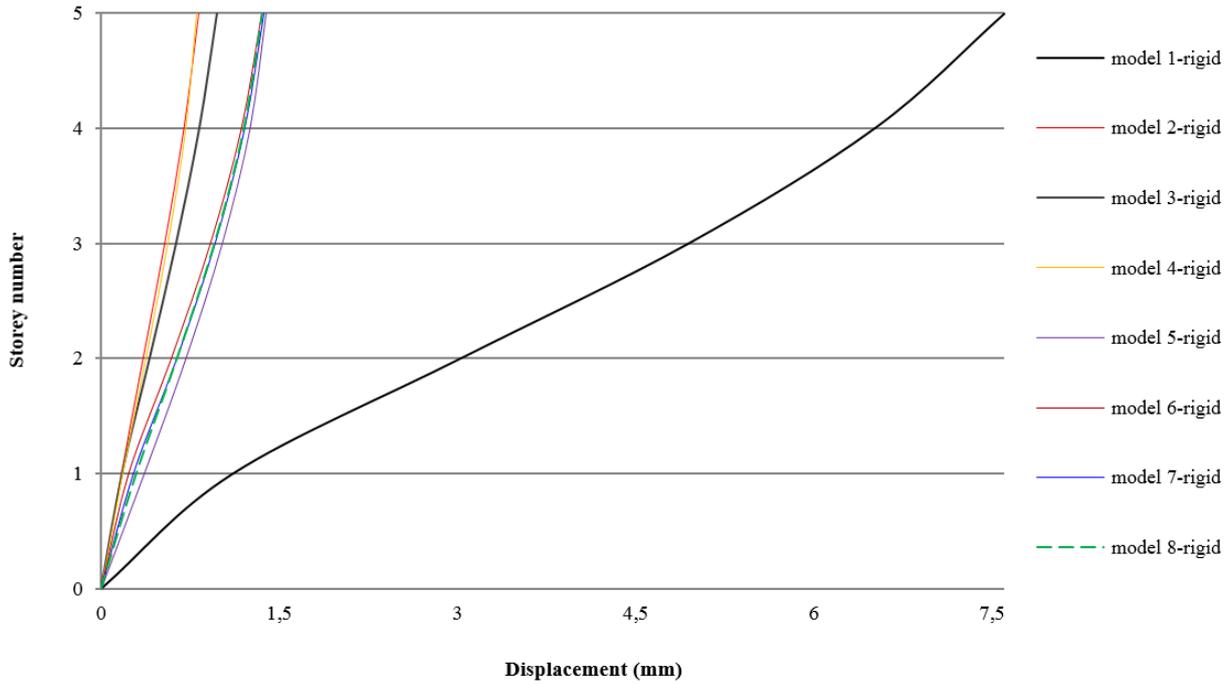
Figure 6. The displacement values at the storey levels of the column selected for the x-direction earthquake effect, assuming that the nodal point combination of the building models is rigid
Şekil 6. Bina modellerinin düğüm noktası birleşimlerinin rijit olduğu kabulüyle x doğrultusundaki deprem etkisi için seçilen kolonunun kat seviyelerindeki yerdeğiştirme değerleri
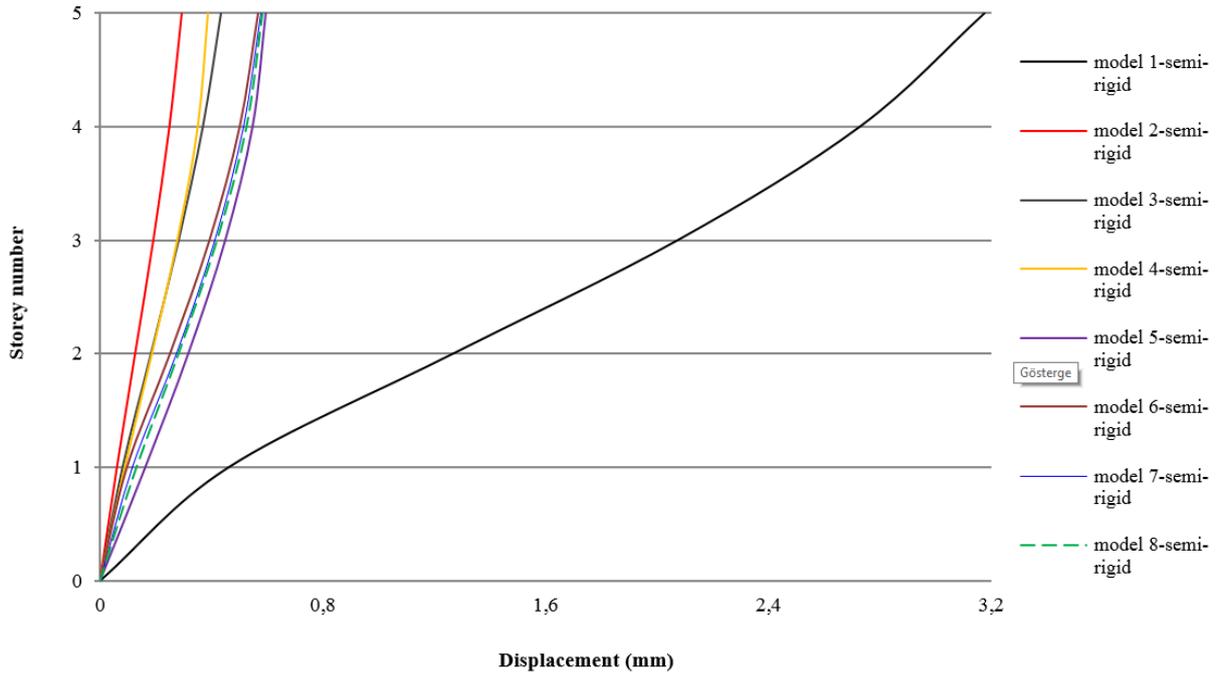

Figure 7. The displacement values at the storey levels of the column selected for the x-direction earthquake effect, assuming that the nodal point combination of the building models is semi-rigid
Şekil 7. Bina modellerinin düğüm noktası birleşimlerinin yarı-rijit olduğu kabulüyle x doğrultusundaki deprem etkisi için seçilen kolonunun kat seviyelerindeki yerdeğiştirme değerleri

Figure 8. The displacement values at the storey levels of the column selected for the y-direction earthquake effect, assuming that the nodal point combination of the building models is rigid
Şekil 8. *Bina modellerinin düğüm noktası birleşimlerinin rijit olduğu kabulüyle y doğrultusundaki deprem etkisi için seçilen kolonunun kat seviyelerindeki yerdeğiştirme değerleri*

Figure 9. The displacement values at the storey levels of the column selected for the y-direction earthquake effect, assuming that the nodal point combination of the steel building is semi-rigid
Şekil 9. *Bina modellerinin düğüm noktası birleşimlerinin yarı-rijit olduğu kabulüyle y doğrultusundaki deprem etkisi için seçilen kolonunun kat seviyelerindeki yerdeğiştirme değerleri*
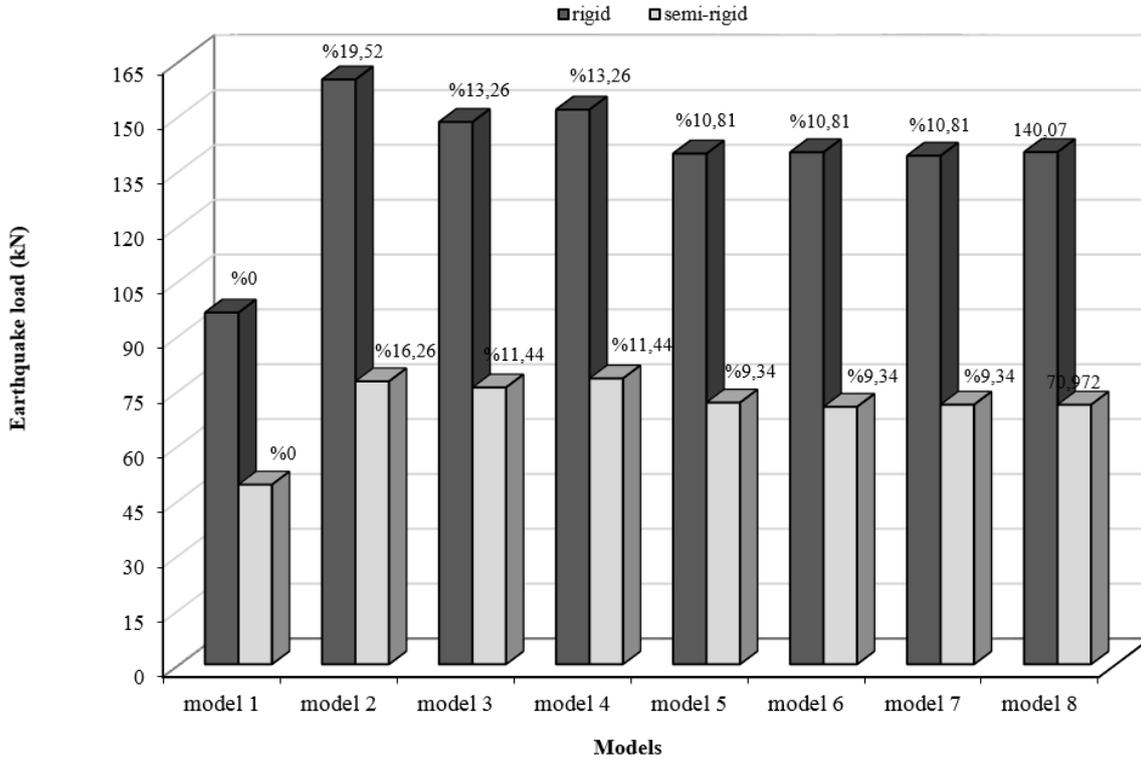
Figure 10. Earthquake load values of building models according to different central steel cross ratios, assuming that the nodal point combination is rigid and semi-rigid

Şekil 10. *Düğüm noktası birleşimlerinin rijit ve yarı-rijit olduğu kabulleriyle farklı merkezi çelik çapraz oranlarına göre bina modellerinin deprem yükü değerleri*

5. Conclusions and recommendations
5. Sonuçlar ve Öneriler

The main conclusions and recommendations from this study are summarized below.

- From the structural analyses, the period values obtained from model 1 for the first three modes are greater than the models with central steel braces. By contrast with, angular velocity values are obtained smaller than the central steel braced models. This result reveals that the central brace members, which increase the lateral stiffness of the models, decrease the period of the structure in question, thus making a positive contribution to the behaviour of the structure.

- In all building models, the period values obtained by assuming that rigid of nodal points are larger than those obtained by assuming that the nodal points are semi-rigid. This result reveals that nodal point details are very important in the design of steel structures.

- The base shear force values obtained by the equivalent earthquake load method in both the x and y directions of the building models are

greater than those obtained by the mode superposition method. Assuming that the nodal point connections of the building models are rigid, the maximum base shear force value obtained from model 2 by mode superposition and the equivalent earthquake load methods in the x-direction is 62.8% and 74.66% greater than model 1, respectively. In addition, the maximum base shear force value obtained from model 2 by mode superposition and the equivalent earthquake load methods in the y-direction is calculated larger 66.29% and 65.17% according to model 1, respectively. Assuming that the nodal points are semi-rigid, the maximum base shear force value obtained from model 4 by mode superposition method in x and y directions is 59.82% and 61.76% compared to model 1, respectively, and the maximum base shear force value obtained from model 2 by equivalent earthquake load method is 63.35% and 57.49% greater compared to model 1, respectively.

- As a result of the structural analyses, the base shear force values obtained by assuming that rigid of nodal points are greater than the base shear force values obtained by assuming that the nodal points are semi-rigid. This result reveals

that rigid nodal points will meet greater base shear force and transferring the moment.

- Findings obtained from the structural analyses; it is reveals that the building model with two-way central steel brace members behaves better than the taken into account other building models. This result shows that two-way central steel brace members significantly increase the performance of the structure.

- The overturning force, overturning moment and anti-overturning moment values obtained by assuming the nodal points as rigid are larger than those obtained by assuming the semi-rigid nodal points.

- Displacement distributions at storey levels in both x and y directions of different central steel braced building models are smaller than those of the moment-transmitting building model. This result reveals that different central steel brace members that increase the lateral stiffness of the structure increase the performance of the structure in question.

- The displacement distributions obtained at the story levels from the structural analyses performed by assuming that the nodal point connections are rigid in both the x and y directions are greater than the displacement distributions obtained by assuming that the nodal point connections are semi-rigid.

- As a result of the structural analyses carried out with the assumption that the nodal points are both rigid and semi-rigid, the earthquake load values increase with the increase in the central steel brace ratio. In addition, the earthquake load values obtained by assuming that the nodal points are rigid are greater than the earthquake load values obtained by assuming that the nodal points are semi-rigid. This result shows that nodal point connections affect earthquake load values, too.

- The largest displacement value of the selected nodal points (for 1 and 2 nodal points) from the structural analyses which is performed by assuming rigid of the nodal point connections of the building models is obtained from model 1. By contrast with, the largest displacement value from the structural analyses performed with the assumption of semi-rigid is obtained from model 6. This result shows that the central steel brace types significantly reduce the displacement values of the considered models

compared to the moment-transferring frame, except for model 6 in the semi-rigid joint.

- It is recommended to use two-way (X) central steel brace members in the design and construction of steel structures that will be built in Turkey when the findings of this study are examined.

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