

Article

Design and Seismic Performance of Tied Braced Frames

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Abstract: In this work, a tied braced frame (TBF) was developed to achieve uniform inelastic deformation in an eccentrically braced frame (EBF) by connecting links across the entire frame height with tie members. Herein, a TBF design method is proposed, considering a new lateral force distribution pattern. To better evaluate the seismic performance, and verify the design advantages of the TBF, nonlinear time-history analysis and fragility analysis were conducted using 6-, 10-, and 20-story TBF models designed using this method, as well as EBF models for comparison. It was found that the maximum inter-story displacement angles of the TBF model were reduced by 10%, 3.3% and 6.3% at the 84th percentile at 6, 10 and 20 stories, respectively, and the DCF values were also reduced by about 5.5%, indicating that the design of the TBF structure is more reasonable. The results revealed that the TBF models featured more uniform distributions of the normalized link shear forces and inter-story drift ratios, resulting in a better damage distribution and more ductile behavior. Furthermore, under earthquakes, the tie axial forces were similar to those calculated using the design equation, thereby indicating the reliability of the design method. Under the same seismic conditions, the PGA values of the TBF structure are about 10~15% lower at 50% exceedance probability compared to the EBF structure; the CMR values of the 6-story, 10-story, and 20-story models are 1.12, 1.09, and 1.06 times higher than those of the EBF structure, respectively. Notably, based on a comparison of the exceedance probability from the fragility analysis results for the TBF and EBF models, the TBF model exhibited better anti-collapse performance.

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1. Introduction

In high-rise buildings, due to the limited strength and stiffness of the light structure itself, it is more difficult to withstand lateral forces; therefore, high-rise buildings need to adopt lateral force resistance modes such as bracing to enhance lateral force resistance through structures such as steel bracing [1].

Among them, eccentrically braced frames (EBFs) have garnered extensive attention, owing to their excellent ductility and stiffness during earthquakes [2–6]. The structural form of EBFs is that the brace is eccentrically connected with the beam to form the link beam (link) [7]. Under frequent earthquakes, EBFs can provide large lateral stiffness and load carrying capacity. Under rare earthquakes, their capacity design principles ensure that inelastic deformations are concentrated in the links, whereas other members, such as the beams, columns, and braces, remain essentially elastic [8–10]. Therefore, EBFs not only have the advantages of concentrically braced frames (CBFs) to provide large lateral stiffness, but also provide sufficient ductility and energy dissipation capacity to ensure

that the structure will not collapse under earthquakes, which are suitable for multi-story buildings in high seismic intensity regions [11].

However, previous tests and numerical studies have indicated the possibility that the inelastic deformations in the links of the EBF are concentrated in several stories [12–14], resulting in excessive link rotations and a tendency for the formation of soft stories. Thus, the energy dissipation function of non-yielding links cannot be fully developed [15,16].

To overcome these drawbacks, Martini proposed a tied braced frame (TBF) in 1990 [17], as shown in Figure 1; in this structure, ties are added across the entire height of the structure, to achieve the design intention of activating the other links once yielding occurs in one link. Therefore, the shearing force can be transmitted by the links, and the links can function together, owing to their deformation compatibility, thus resulting in better seismic energy dissipation, damage distribution, and ductility of the braced frames.

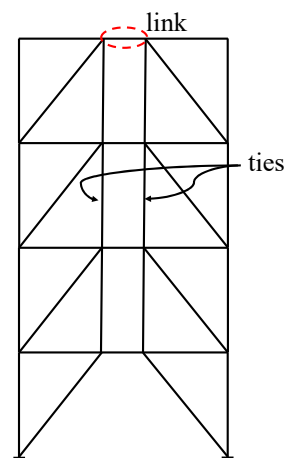


Figure 1. Tied braced frame.

Rossi et al. further investigated the seismic performance of the TBF, and proposed a design method in which seismic lateral forces were predicted based on the assumption of uniform plastic rotation in all the links; further, the members were designed according to the predicted forces. The method achieves an optimal collapse method characterized by uniform rotation of the links, maximizing the resistance capacity of each part of the structure, thus ensuring the overall stability and safety of the structure. [18]. Tremblay compared the seismic performances of TBF and EBF models, and found that Rossi's design method was conservative for structures with more than 20 stories [19]. According to Chen, the design of the TBF should fully consider the bending dynamic response of the elastic truss in higher order modes. The TBF consists of a pair of elastic vertical trusses connected with yielding links; thus, the internal forces in the elastic trusses should be divided into the force generated by the yielding of the links and the force generated by the bending of the truss under the higher modes [20,21]. However, this design procedure slightly underestimates the seismic forces on certain braces and ties, and its applicability to high-rise braced structures needs to be further evaluated [20]. On the whole, the existing design method of the TBFs is complicated, and the design is slightly conservative for high-rise structures. In addition, relatively little research has been conducted on the degree of damage and collapse performance of the TBFs.

Therefore, we need to study the properties of TBFs more comprehensively and deeply, to improve the design method of TBFs and promote their application. For this reason, this paper proposes a modified design method for TBFs based on Rossi's design method, by considering a new lateral force distribution pattern [18]. In order to prove the advantages of the tied braced steel frame the design method, TBF and EBF models with 6, 10, and 20 stories were designed and analyzed through a nonlinear time-history analysis and fragility analysis. Furthermore, the seismic performances of the TBF models were compared with those of the EBF models.

2. Design Method for TBF

2.1. Yield Mode

Under rare earthquakes, the ideal yield mode of an EBF occurs when the links yield and the other members are in an elastic state. For the TBF, owing to the deformation compatibility, the link shear forces are transmitted through the ties, such that the links undergo yield simultaneously. When the first mode of vibrations is dominant, the overall failure mode of the TBF model is as shown in Figure 2.

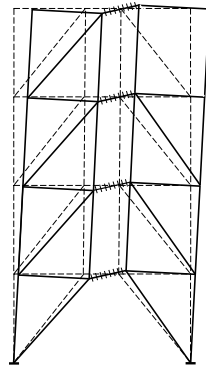


Figure 2. Overall failure mode of TBFs under the first mode of vibrations.

2.2. Seismic Lateral Force Distribution Pattern

Considering the most adverse force state, the assumption that all the links yield simultaneously is proposed [18], and a simplified braced sub-structure model is established, as shown in Figure 3a. Based on force equilibrium, when all the links yield, the seismic lateral forces on the brace span at each story can be calculated using Equation (1):

$$\sum_{i=1}^n F_i H_i = L \sum_{i=1}^n V_{li}, \quad i = 1, 2, \dots, n, \quad (1)$$

where n is the total number of stories of the structure, H_i is the height of the i -th floor above the base, F_i is the seismic lateral force on the i -th story, L is the length of the braced span, and V_{li} is the shear bearing capacity of the i -th story link.

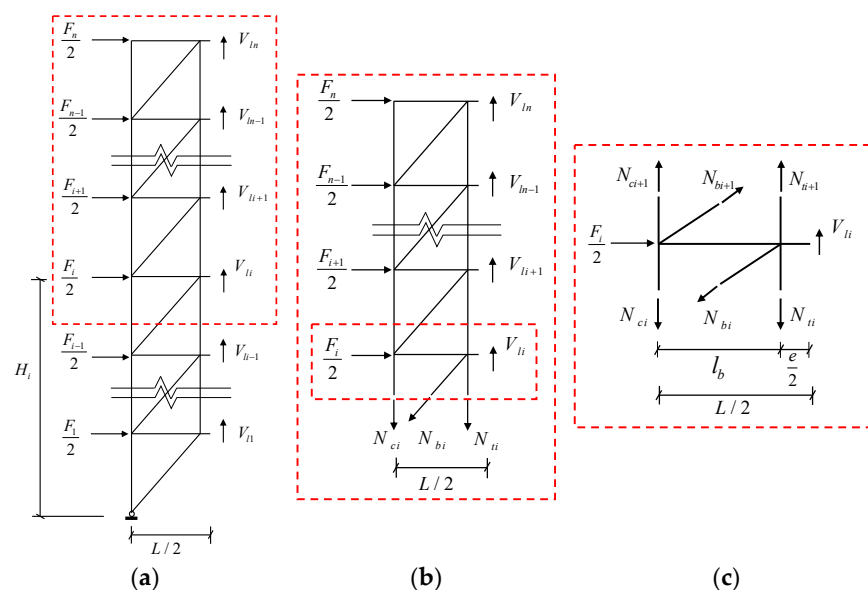


Figure 3. Simplified braced sub-structure model: (a) braced sub-structure; (b) force analysis of braces; and (c) force analysis of i -th story.

To obtain F_i , the distribution pattern of the seismic lateral forces needs to be determined first.

$$\beta_i = \frac{F_i}{F_1} \quad (2)$$

where β_i is the distribution pattern coefficient of the seismic lateral force.

Rossi's design method simplified calculation for the distribution pattern of the seismic lateral forces. The high-order mode action is simply calculated as multiplying the second mode by a coefficient, so that the equivalent seismic forces only need to be conceived as the combination of three sets of seismic lateral forces, which are a function of the first two modes of vibration of the structure [18].

The response spectrum method not only considers the relationship between the dynamic behavior of the structure with the ground motion characteristics, but also makes full use of the static theory [22]. Therefore, considering the effects of high-order vibration modes, in this study, it was assumed that the distribution pattern of the seismic lateral force is identical to the pattern calculated using the mode decomposition response spectrum method, which takes the number of modes required for the participating mass to reach 90% of the total mass.

2.3. Seismic Lateral Force Distribution Pattern

In order to simplify the calculation, the brace and ties are pinned, and the bending moment effect of the column is ignored. Therefore, the bending moment effect of the non-energy dissipation members is ignored during the design, and only the axial force is calculated.

As shown in Figure 3b,c, free bodies were selected from the top story to analyze the internal forces on the members; the axial forces on the braces, columns, and ties can be calculated using the following equations [18]:

$$N_{bi} = \frac{1}{2 \cos \alpha} \sum_{k=i}^n F_k, \quad (3)$$

$$N_{ci} = \sin \alpha \sum_{k=i+1}^n N_{bk} - \frac{e}{2l_b} \sum_{k=i}^n V_{lk}, \quad (4)$$

$$N_{ti} = -\sin \alpha \sum_{k=i+1}^n N_{bk} + \left(1 + \frac{e}{2l_b}\right) \sum_{k=i}^n V_{lk}, \quad (5)$$

where α is the slope between the brace and the beam, N_{bi} is the brace axial force on the i -th story, N_{ci} is the column axial force on the i -th story, N_{ti} is the tie axial force on the i -th story, e is the length of a link, and l_b is the horizontal length of the brace.

It was found that, during earthquakes, the deformation of high-rise structures is significantly affected by higher modes. The shear yield directions of the links on individual stories may be opposite to the design direction, as shown in Figure 4. Therefore, different to Rossi's design, in this work, when the structure exceeded 40 m, the height limit for which earthquake forces can be calculated using the base shear method provided in the Chinese seismic design code (GB50011-2016) [10], the tie axial force on the top story was corrected. The analysis results for the top story members are shown in Figure 4. The revised equation for calculating the tie axial forces on the top story can be obtained as follows:

$$N_{tn} = -\sin \alpha N_{bn} - \left(1 + \frac{e}{2l_b}\right) V_{ln} \quad (6)$$

Hence, the tie axial forces on the other stories can be obtained using Equation (7):

$$N_{ti} = -\sin \alpha \sum_{k=i+1}^{n-1} N_{bk} + \left(1 + \frac{e}{2l_b}\right) \sum_{k=i}^{n-1} V_{lk} + N_{tn} \quad (7)$$

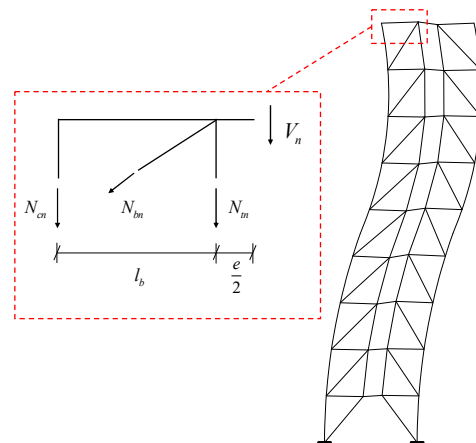


Figure 4. Influence of higher modes.

Referring to the requirements of the Chinese seismic design code (GB50011-2016) [10], the design value for the internal forces on non-energy dissipation members should be multiplied with amplification factors. The design axial force on the braces and ties can be calculated using Equations (8) and (9), as follows:

$$N_{bi,d} = \eta_b \frac{V_{li}}{V_i} N_{bi}, \quad (8)$$

$$N_{ti,d} = \eta_t \frac{V_{li}}{V_i} N_{ti}, \quad (9)$$

where $N_{bi,d}$ is the brace design axial force on the i -th story; $N_{ti,d}$ is the tie design axial force on the i -th story; V_i is the design shear force of the i -th story link; and η_b and η_t are the amplification coefficients of the brace axial force and tie axial force, respectively.

3. Structure Design and Modeling

3.1. Design of Buildings

Here, 6-, 10-, and 20-story TBF and EBF buildings with five spans were analyzed. The story height and span length were 3.9 m and 7.8 m, respectively. The link length (e) was 1 m, which remained constant across the height of the buildings, and the link was designated as a shear link with an e of less than $1.6 M_p / V_p$ (where M_p and V_p are the plastic moment strength and plastic shear strength, respectively). The braces, ties, and links were located in the middle span, as shown in Figure 5. Moreover, the buildings were assumed to be located in a region with a seismic intensity of VIII (design basic acceleration of ground motion: 0.2 g), seismic design group II, and site class III [10].

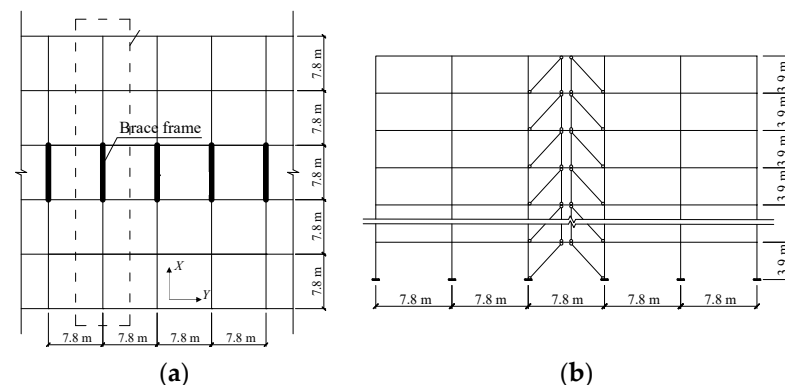


Figure 5. Plan and elevation of buildings: (a) plane view; and (b) elevation view.

For the design, the dead load and live load were considered as 8.25 kN/m^2 and 2.0 kN/m^2 , respectively. The nominal seismic loads were calculated using the mode-superposition response spectrum method, as proposed in the Chinese seismic design code (GB50011-2016) [10]. The load combinations for the strength design were considered according to the requirements of the Chinese seismic design code (GB50011-2016) [10]. The floors were assumed to behave as rigid diaphragms; thus, dynamic instability of the beams was prevented.

All members were H-shaped structural steel members, with a yield strength of 345 MPa , an elastic modulus of $E = 2.06 \times 10^5 \text{ MPa}$, and a Poisson's ratio $\nu = 0.3$. To simplify the design and analysis, only the plane model in the X-direction was selected as the analytical model, as shown in Figure 4. For TBF and EBF models with the same number of stories, the cross-sections of the links, beams, and columns are identical, and only the braces cross-sections of the TBF models are larger than the EBF models. The cross-sections of the mid-span for the 6-, 10-, and 20-story TBF and EBF models are shown in Appendix A: Tables A1–A3, and the steel consumption of the buildings are shown in Table 1. The steel consumption in the table refers to the amount of steel used for a single-braced span. For TBF and EBF models with the same number of stories, the steel consumption of a TBF structure is greater than that of an EBF structure.

Table 1. The steel consumption of structures (unit: t).

Example of Calculation	Structure	Steel Consumption
6-story	TBF	41.45
	EBF	37.54
10-story	TBF	100.65
	EBF	90.45
20-story	TBF	282.48
	EBF	240.05

3.2. Numerical Model

A three-dimensional finite element model was established using ABAQUS 6.14.4. The structural members were simulated using the “Timoshenko” beam element with three nodes. The link members are meshed according to a 0.1 m length of each cell, and the rest members are meshed according to a 0.5 m length of each cell. The vertical loads were simplified as mass points, which were then uniformly distributed on the beams, as shown in Figure 6. The material nonlinear kinematic hardening model was employed with an elastic modulus of $E = 2.06 \times 10^5 \text{ MPa}$, tangent modulus after yield of $E_1 = 0.01 E$, and Poisson's ratio of $\nu = 0.3$. The beam-to-column connections and the column bases were taken to be rigid connections considering the flexural restraint, and the brace-to-frame, tie-to-beam connections were taken to be ideally pinned. The out-of-plane imperfections of the braces and ties were considered as $1/500$ times their lengths, and the other members were restrained.

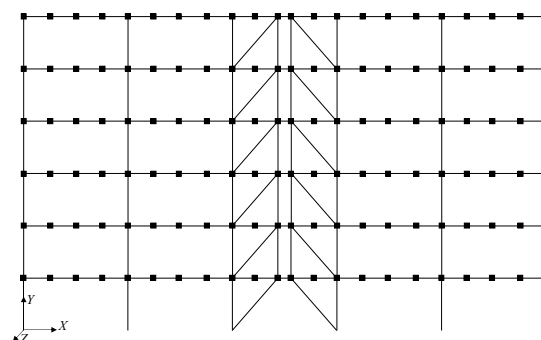


Figure 6. Numerical model for 6-story TBF model.

4. Seismic Records

To adequately evaluate the seismic performances of the TBF and EBF models, according to the building site class and seismic design group, nine far-field seismic records and seven near-fault seismic records were selected from the Pacific Earthquake Engineering Research Center, and one artificial seismic record were adopted as the input ground motions, which meet the requirements of time history analysis of the Chinese seismic design code (GB50011-2016) [10]. The details of these records are described in Table 2. The response spectra for the selected ground motion records were compared with the design response spectrum, as shown in Figure 7; the deviation between the mean value of seismic records and standard response spectra is within 20%, which can be used for a time-history analysis. To meet the requirements of the Chinese seismic design code (GB50011-2016) [10], the peak acceleration of each seismic record was adjusted to 70 gal and 400 gal for the time-history analysis, to simulate the conditions of frequent and rare earthquakes, respectively.

Table 2. Seismic records.

No.	RSN	Earthquake Name	Year	Station Name	Magnitude	PGA (g)
1	06	Imperial Valley-02	1940	El Centro Array #9	6.95	0.281
2	36	Borrego Mtn	1968	LA-Hollywood Stor FF	6.63	0.133
3	68	San Fernando	1971	LA-Hollywood Stor FF	6.61	0.270
4	723	Superstition_Hills-02	1987	Parachute_Test_Site	6.5	0.455
5	752	Loma_Prieta	1989	Capitola	7.0	0.528
6	821	Erzican-Turkey	1992	Erzincan	6.7	0.515
7	828	Cape_Mendocino	1992	Petrolia	7.0	0.590
8	829	Cape_Mendocin	1992	Rio_Dell_Overpass-FF	7.0	0.385
9	900	Landers	1992	Yermo_Fire_Station	7.3	0.245
10	960	Northridge-01	1994	Canyon_CountryW_Lost_Cany	6.7	0.410
11	1165	Kocaeli-Turkey	1999	TCU065	7.5	0.220
12	1244	Chi-Chi-Taiwan	1999	CHY101	7.6	0.353
13	1503	Chi-Chi-Taiwan	1999	TCU065	7.6	0.603
14	1602	Duzce-Turkey	1999	Bolu	7.1	0.728
15	1605	Duzce-Turkey	1999	Duzce	7.1	0.348
16	1787	Hector_Mine	1999	Hector	7.1	0.337
17	—	Artificial Wave	—	—	—	—

5. Structural Response under Frequent Earthquakes

Under frequent earthquakes, the TBF models were in an elastic state, and out-of-plane buckling did not occur in the braces and ties. The normalized link shear forces in the TBF and EBF models were less than 1, where the normalized link shear force refers to the ratio of the maximum shear force during earthquakes to the shear yield force of the link (V/V_l), as shown in Figure 8, indicating the absence of a yielded link. It should be noted that the normalized link shear forces at the top and bottom stories in the TBF models were slightly larger than those in the EBF models, whereas those at the remaining stories were smaller.

Overall, the inter-story drift ratios were all lower than the limit value of 0.004 provided by the Chinese seismic design code (GB50011-2016) [10], as shown in Figure 9. Except for the larger inter-story drift ratios in the TBF models for the upper stories [23], the inter-story drift ratios for the other stories in the TBF models were essentially identical to those in the EBF models.

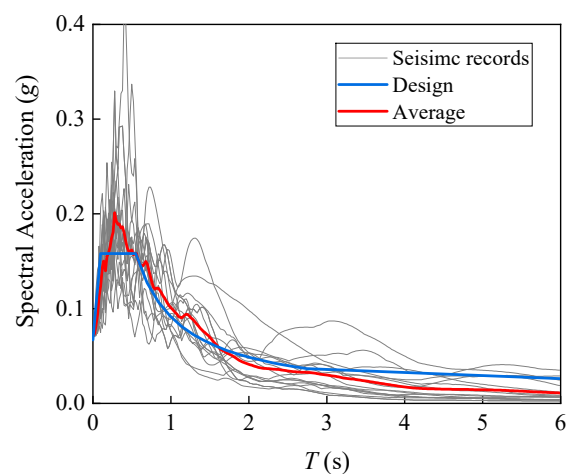


Figure 7. Response spectra of the seismic records.

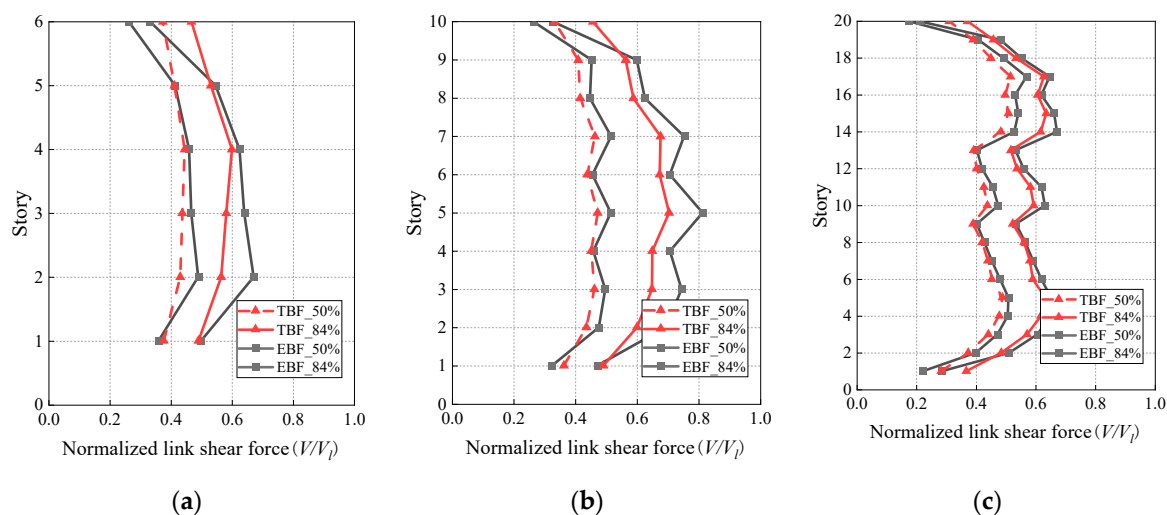


Figure 8. Normalized link shear forces under frequent earthquakes: (a) 6-story; (b) 10-story; and (c) 20-story.

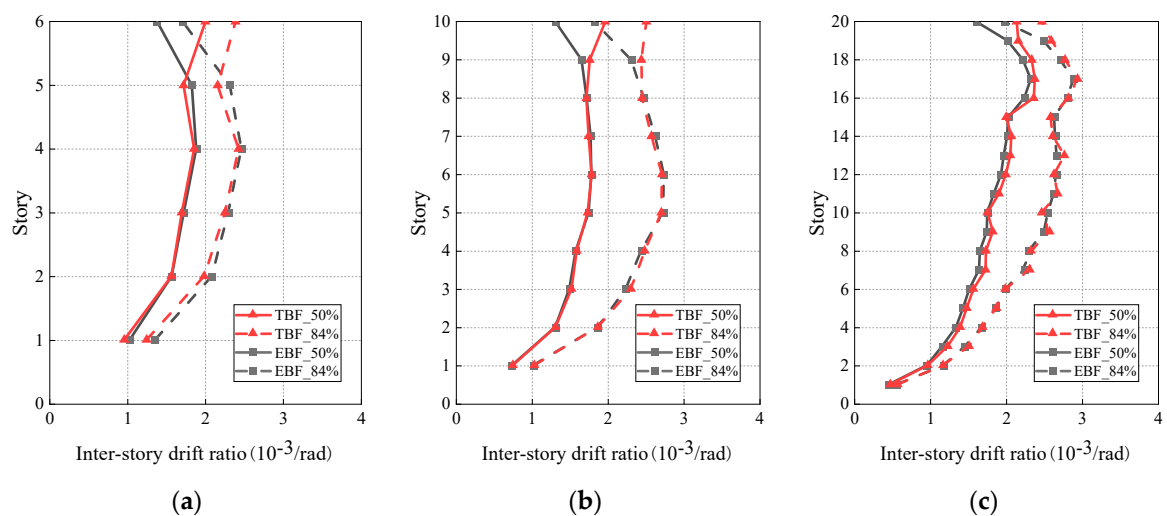


Figure 9. Inter-story drift ratios under frequent earthquakes: (a) 6-story; (b) 10-story; and (c) 20-story.

To better compare the seismic performance, the coefficient of variation (*CV*) and drift concentration factor (*DCF*) were selected for the evaluation of the response discrepancies between the TBF and EBF models; *CV* was used to describe the discrete degree of the normalized story drift, and *DCF* was used to describe the uneven degree of deformation between stories, which can be calculated using Equations (10) and (11), as follows:

$$CV = \frac{\sigma}{\mu} \times 100\%, \quad (10)$$

$$DCF = \frac{\Delta_{\max}}{(\sum \Delta_i h_i) / H}, \quad (11)$$

where σ and μ are the standard deviation and average values of the dynamic response, respectively, $\Delta_{\max}$ is the maximum inter-story drift ratio of the structure, Δ_i is the maximum inter-story drift ratio of the structure for the i -th story, h_i is the i -th story height, and H is the total height of the structure. The smaller the values of *CV* and *DCF*, the smaller the discrete degree of the normalized shear force, and the more uniform the story drift.

The *CV* and *DCF* in the 50th and 84th percentiles of the normalized link shear force and inter-story drift ratios under frequent earthquakes are listed in Table 3. The discrete degrees of the normalized shear forces in the TBF models were smaller than those in the EBF models. Compared with the EBF models, the *CV* values for the 6- and 10-story TBF models were reduced by more than 50%, and by 30% for the 20-story TBF model. Although the *DCF* values in the 50th percentile for the 6- and 10-story TBF models were larger than those for the EBF models, they were smaller in the 84th percentile, indicating that the uneven degree of deformation in the TBF was less than that in the EBF under unfavorable conditions.

Table 3. *CV* and *DCF* values of the structures under frequent and rare earthquakes.

Earthquake			6-Story		10-Story		20-Story	
			50th	84th	50th	84th	50th	84th
<i>CV</i> (%)	Frequent	TBF	6.87	8.90	8.49	10.93	14.05	14.02
		EBF	19.05	20.71	17.15	20.39	21.64	20.90
		Deviation	(−63.93)	(−57.04)	(−50.52)	(−46.41)	(−35.07)	(−32.92)
	Rare	TBF	3.81	5.45	3.03	4.16	11.71	11.50
		EBF	16.25	14.57	16.73	16.97	21.32	20.63
		Deviation	(−76.55)	(−62.57)	(−81.89)	(−75.49)	(−45.08)	(−44.26)
<i>DCF</i>	Frequent	TBF	1.225	1.169	1.228	1.175	1.338	1.296
		EBF	1.201	1.209	1.181	1.231	1.377	1.302
		Deviation	(+2.00)	(−3.31)	(+3.98)	(−4.55)	(−2.83)	(−0.46)
	Rare	TBF	1.207	1.270	1.183	1.222	1.335	1.307
		EBF	1.278	1.347	1.238	1.293	1.357	1.380
		Deviation	(−5.56)	(−5.72)	(−4.44)	(−5.49)	(−1.62)	(−5.29)

Deviation = values of (TBF-EBF)/EBF.

In general, the distributions of the normalized link shear forces and inter-story drift ratios in the TBF across the entire height were more uniform than those in the EBF. This is because the ties can transfer link shear forces during earthquakes; owing to the compatibility of the ties, the internal force distribution was more reasonable in the TBF.

6. Structural Response under Rare Earthquakes

6.1. Shear Forces on Links

In the rare earthquake scenario, the links are designed as structural fuses for dissipating seismic energy through concentrated plastic deformations [24], and simultaneous yielding of the links across the height of the structure is expected for the TBF. Figure 10 shows the normalized link shear forces for the TBF and EBF models under rare earthquakes. Except for a few stories in the 20-story EBF model, the normalized link shear forces of the

other stories all exceeded 1.0, even reaching 2.0. The 50% quantile normalized link shear force distribution is in the range of 1.7 to 1.8, and the 84% quantile normalized energy-consuming beam shear force distribution is around 2.0. This indicated that plasticity was fully developed in the links, and that the design intention of energy dissipation via the links was realized.

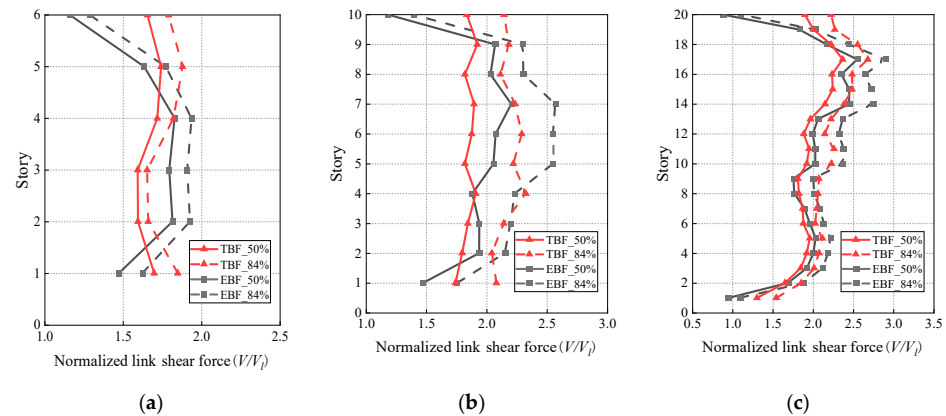


Figure 10. Normalized link shear forces under rare earthquakes: (a) 6-story; (b) 10-story; and (c) 20-story.

Similar to the distribution rule under frequent earthquakes, the distribution of the normalized link shear forces in the TBF across the height was more uniform than that in the EBF. As shown in Table 3, the CV values for the normalized link shear forces in the TBF models were smaller than those in the EBF models under rare earthquakes. In addition, the deviations in the CV values under rare earthquakes were larger than those under frequent earthquakes. This is because, with an increase in the earthquake intensity, the dynamic response of the structures increased, and the effect of the ties was more sensible.

6.2. Inter-Story Drift Ratio

The maximum inter-story drift ratios for the TBF and EBF models under rare earthquakes were lower than the limit value of 0.02 [10], as shown in Figure 11, indicating that the collapsing phenomenon did not occur. Compared with the EBF models, the maximum inter-story drift ratios across the height in the 84th percentile for the 6-, 10-, and 20-story TBF models were reduced by 10%, 3.3%, and 6.3%, respectively. Moreover, as shown in Table 2, the DCF values for the TBF models were smaller than those for the EBF models under rare earthquakes. In the 84th percentile for the TBF models, the DCF values decreased by approximately 5.5%, indicating that the distribution of the inter-story drift ratios across the height for the TBF was more uniform.

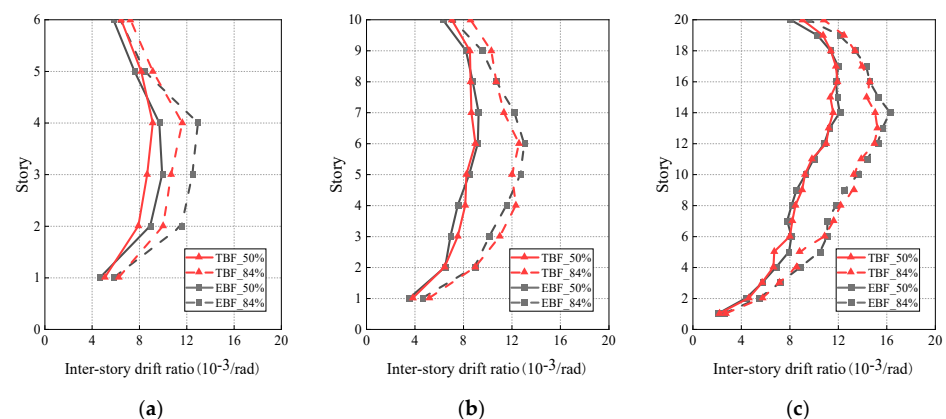


Figure 11. Inter-story drift ratios under rare earthquakes: (a) 6-story; (b) 10-story; and (c) 20-story.

6.3. Axial Forces on Ties

The tie axial forces for the TBF models under rare earthquakes are shown in Figure 12. Regarding the effects of the second mode, the force demand is the highest near the mid-height of the building. Higher values were also observed for the top and bottom heights of the buildings, owing to the effects of the third and higher modes. The actual tie axial force distribution trends are nearly identical to the intended distribution.

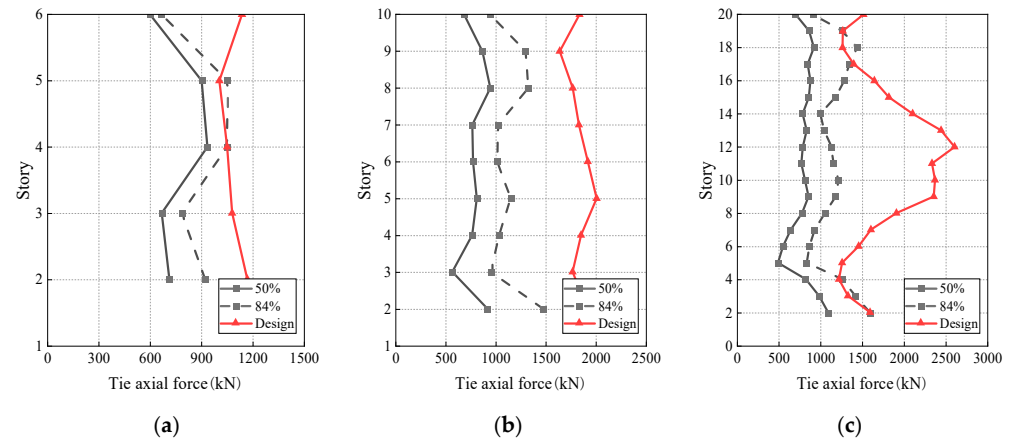


Figure 12. Tie axial forces under rare earthquakes: (a) 6-story; (b) 10-story; and (c) 20-story.

To better evaluate the design of the tie axial forces, the normalized tie axial forces in the TBF models under rare earthquakes were calculated, as shown in Figure 13. The normalized tie axial force refers to the ratio of the maximum axial forces during earthquakes to the design tie axial force ($N/N_{t,d}$), which is calculated using Equation (9). For the 6-story TBF model, the maximum normalized values across the height in the 50th and 84th percentile were 0.90 and 1.05, respectively. For the 10-story TBF model, the maximum normalized values across the height in the 50th and 84th percentile were 0.53 and 0.79, respectively. Furthermore, for the 20-story TBF model, the maximum normalized values across the height in the 50th and 84th percentile were 0.74 and 1.14, respectively. The normalized tie axial force values were less than 1.0 under the ground motions, and the normalized tie axial force was between 0.6 and 1.0 for 84% of the quantile. In general, the 50th and 84th percentile normalized values were below 1.0, indicating that the design forces for the ties in this study were reasonable.

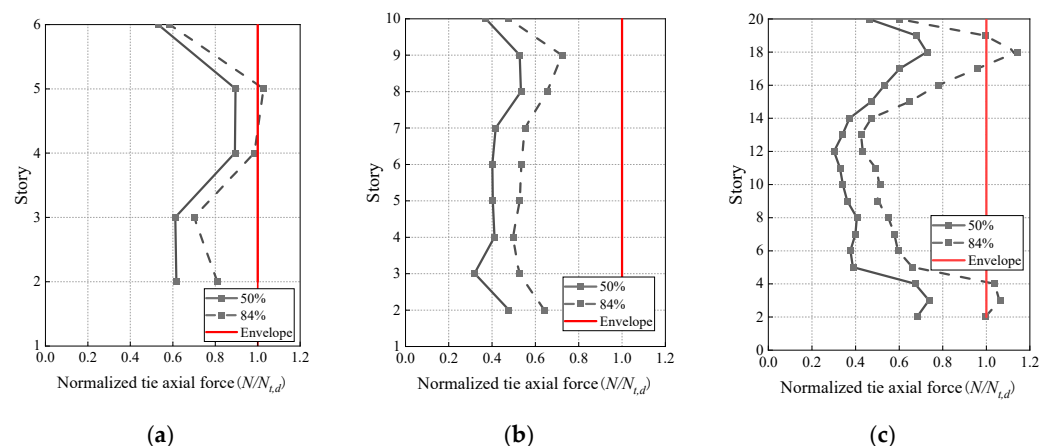


Figure 13. Normalized tie axial forces under rare earthquakes: (a) 6-story; (b) 10-story; and (c) 20-story.

7. Fragility Analysis

7.1. Development of Analytical Fragility Curves

Fragility curves are representations of conditional probability [25,26], which indicate the probability of meeting or exceeding the limit state level under a given input ground motion intensity measure (IM). This conditional probability can be expressed as follows [27]:

$$P_f = \Phi\left(\frac{\ln[\bar{D}/\bar{C}]}{\sqrt{\beta_C^2 + \beta_D^2}}\right) = \Phi\left(\frac{\ln[\alpha(IM)^\beta/\bar{C}]}{\sqrt{\beta_C^2 + \beta_D^2}}\right), \quad (12)$$

where $\Phi(\cdot)$ is the standard normal distribution function; $\bar{C}$ is the corresponding engineering demand parameter (EDP) of a specific limit state; $\bar{D}$ is the median value of the structural response; β_C and β_D are the standard deviations of $\bar{C}$ and $\bar{D}$, respectively; and α and β are regression parameters. The most commonly used indicators of earthquake intensity are peak ground acceleration (PGA) and spectral acceleration $S_a(T_1, 5\%)$. Both metrics can be used to describe the intensity of ground shaking. It is shown that the correlation between PGA and structural seismic demand parameters is significantly better than that of $S_a(T_1, 5\%)$ in the long-period segment [28]. The excellent period of the average acceleration response spectrum of the ground shaking records selected in this paper is around 0.5 s, and the basic period of the arithmetic cases is greater than 1.0 s. Therefore, the index of IM was defined as the peak ground acceleration (PGA). The amplitude of the PGA increased by 200 gal for each seismic record, and this continued until it reached 1600 gal. The maximum inter-story drift ratio ($\Delta_{\max}$) of the structure was chosen as the index for EDP [29].

According to the Chinese seismic design code (GB50011-2016) [10], four limit states—the slight, moderate, extensive, and collapse limit states of a steel structure—are defined according to the different levels of the $\Delta_{\max}$ (i.e., 0.4%, 1%, 2%, and 4%, respectively) under seismic excitations.

7.2. Fragility Curves

The fragility curves are shown in Figure 14. The initial slope of the slight limit curve was large, indicating that the TBF and EBF were prone to slight failures during earthquakes. When the PGA was less than 400 gal, the structures mainly suffered slight or moderate damage; in this case, the probability of extensive damage was significantly small (less than 20%), and the probability of collapse was minuscule. It should be noted that, when the PGA was greater than 400 gal for the extensive curves and 600 gal for the collapse curves, the slope of the curves increased significantly, indicating that the probability of extensive damage and collapse for the TBF and EBF increased rapidly.

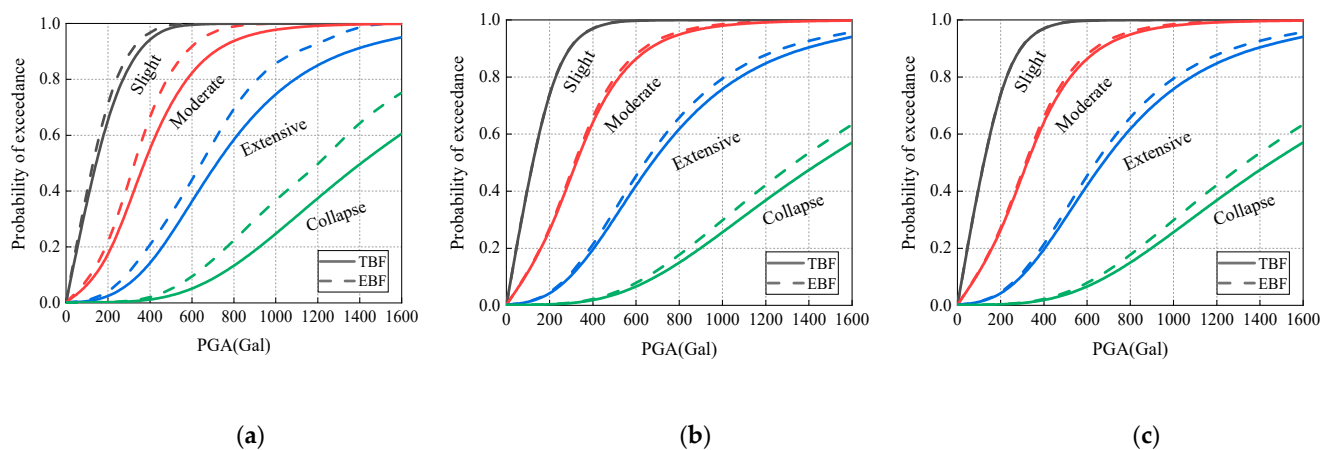


Figure 14. Fragility curves: (a) 6-story; (b) 10-story; and (c) 20-story.

For the 6-, 10-, and 20-story models in each limit state under the same PGA, the exceedance probability for the TBF models was lower than that for the EBF models. The PGA values at 50%, which exceeded the probability for the four limit states and reflected the average anti-collapse ability of the structure. The higher the PGA value, the stronger the anti-collapse performance of the structure. For the slight and moderate limit states, there was little difference between the TBF and EBF models. For the extensive damage and collapse states, the PGA values at 50% probability of exceedance for the TBF models were increased by 10% and 15%, respectively, indicating that the TBF has better seismic performance and better resistance performance than the EBF.

In addition, Table 4 shows the collapse margin ratio (CMR) for the TBF and EBF models, which indicates the ratio of the PGA values at 50% probability of exceedance for the extensive limit state to the PGA value under rare earthquakes [28]. The CMR values for the TBF models were higher than those for the EBF models; they were improved by 1.12, 1.09, and 1.06 times for the 6-, 10-, and 20-story models, respectively, indicating that the TBF has better collapse-resistant performance.

Table 4. Collapse margin ratio of structures.

	TBF	EBF	CMR_{TBF}/CMR_{EBF}
6-story	1.78	1.58	1.12
10-story	2.08	1.90	1.09
20-story	1.67	1.58	1.06

8. Conclusions

To improve the design method of the TBFs and promote its application, a novel design method was proposed based on the ideal failure mode of the TBFs. Nonlinear time-history and fragility analyses were performed on 6-, 10-, and 20-story TBF and EBF models. The following conclusions were drawn:

- (1) Considering a new lateral force distribution pattern, the seismic lateral force on the bracing span is calculated by establishing the equilibrium relationship between bending moment and the shear bearing capacity of link. Hence, the suggestion design equations of non-energy dissipation members, such as the braces, columns, and ties, were given.
- (2) Under the rare earthquake, compared with the EBF, the TBF exhibits better seismic performance, with a 10%, 3.3% and 6.3% reduction in the maximum inter-story drift ratio, and an approximately 5.5% reduction in the DCF value for the 84th percentile of 6, 10 and 20 stories for the TBF model, respectively. These results showed that, due to the deformation compatibility of ties, the distributions of the normalized link shear forces and the inter-story drift ratios were more uniform for the TBF. Further, plasticity was fully developed in the links, and the design intentions of simultaneous yielding and energy dissipation via the links, were realized.
- (3) The modified design equations of the tie are reliable. The axial forces of the ties exhibited the same distribution trends as the actual axial forces of ties under rare earthquakes; moreover, the maximum axial forces of the ties were reliably evaluated using the revised equations.
- (4) The fragility analysis results showed that TBFs feature better seismic performance and better collapse-resistant performance than EBFs. Under the same earthquake conditions, the PGA value of TBF models with 50% exceedance probability is about 10% to 15% lower than that of EBF models. In particular, under the extensive and collapse damage states, the TBF performed considerably better than the EBF. The collapse margin ratios for the TBF were also higher than those for the EBF. The CMR values of the TBF models are also higher than those of the EBF model by a factor of 1.12, 1.09 and 1.06 for the 6-story, 10-story and 20-story models, respectively.

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Appendix A

Table A1. Cross-sections of mid-span for 6-story TBF and EBF models (unit: mm).

Story	Link	Beam	Column	Brace		Tie
				TBF	EBF	
6	H300 × 280 × 10 × 18	H450 × 300 × 14 × 18	H400 × 300 × 12 × 16	H220 × 220 × 12 × 16	H200 × 220 × 12 × 14	H220 × 220 × 12 × 16
5	H350 × 280 × 10 × 18	H450 × 300 × 14 × 18	H400 × 300 × 12 × 16	H220 × 220 × 12 × 16	H200 × 220 × 12 × 14	H220 × 220 × 12 × 16
4–3	H380 × 300 × 12 × 18	H450 × 300 × 14 × 18	H500 × 400 × 16 × 22	H300 × 250 × 14 × 18	H250 × 250 × 12 × 18	H220 × 220 × 12 × 16
2	H400 × 300 × 12 × 18	H480 × 300 × 14 × 18	H600 × 450 × 20 × 24	H300 × 250 × 14 × 18	H250 × 250 × 12 × 18	H220 × 220 × 12 × 16
1	H400 × 300 × 12 × 18	H480 × 300 × 16 × 20	H800 × 550 × 24 × 28	H300 × 250 × 14 × 18	H250 × 250 × 12 × 18	—

Table A2. Cross-sections of mid-span for 10-story TBF and EBF models (unit: mm).

Story	Link	Beam	Column	Brace		Tie
				TBF	EBF	
10–9	H280 × 220 × 8 × 16	H450 × 350 × 18 × 22	H400 × 300 × 12 × 16	H220 × 220 × 12 × 16	H220 × 220 × 12 × 16	H250 × 250 × 12 × 16
8–7	H320 × 220 × 10 × 18	H450 × 300 × 16 × 20	H600 × 450 × 16 × 22	H300 × 300 × 14 × 18	H300 × 300 × 14 × 18	H250 × 250 × 12 × 16
6–5	H350 × 250 × 12 × 18	H450 × 300 × 16 × 20	H700 × 500 × 22 × 26	H350 × 300 × 14 × 20	H300 × 300 × 14 × 18	H250 × 250 × 12 × 16
4–3	H450 × 250 × 12 × 18	H450 × 300 × 16 × 20	H750 × 600 × 26 × 30	H350 × 350 × 14 × 22	H350 × 300 × 14 × 20	H250 × 250 × 12 × 16
2	H450 × 250 × 12 × 18	H450 × 300 × 16 × 22	H850 × 700 × 28 × 36	H400 × 350 × 14 × 22	H350 × 350 × 14 × 20	H250 × 250 × 12 × 16
1	H450 × 250 × 12 × 18	H450 × 300 × 16 × 22	H1000 × 750 × 28 × 36	H400 × 350 × 14 × 22	H350 × 350 × 14 × 20	—

Table A3. Cross-sections of mid-span for 20-story TBF and EBF models (unit: mm).

Story	Link	Beam	Column	Brace		Tie
				TBF	EBF	
20–19	H220 × 220 × 8 × 16	H400 × 300 × 16 × 22	H400 × 350 × 16 × 20	H250 × 230 × 14 × 16	H250 × 230 × 14 × 16	H220 × 220 × 12 × 16
18–17	H280 × 220 × 8 × 16	H400 × 300 × 16 × 22	H450 × 400 × 20 × 24	H250 × 230 × 14 × 16	H250 × 230 × 14 × 16	H220 × 220 × 12 × 16
16–14	H280 × 220 × 10 × 18	H400 × 300 × 16 × 22	H600 × 480 × 24 × 28	H300 × 250 × 14 × 18	H300 × 250 × 14 × 18	H220 × 220 × 12 × 16
13–12	H300 × 250 × 12 × 18	H400 × 300 × 16 × 22	H700 × 550 × 26 × 30	H300 × 250 × 14 × 18	H300 × 250 × 14 × 18	H300 × 250 × 14 × 18
11–10	H350 × 250 × 12 × 18	H400 × 300 × 16 × 22	H900 × 780 × 34 × 42	H350 × 320 × 14 × 22	H350 × 320 × 14 × 22	H300 × 250 × 14 × 18
9–8	H400 × 250 × 14 × 20	H400 × 300 × 16 × 22	H900 × 880 × 34 × 42	H350 × 350 × 14 × 22	H350 × 320 × 14 × 22	H300 × 250 × 14 × 18
7–6	H400 × 250 × 14 × 20	H400 × 300 × 16 × 22	H1200 × 900 × 36 × 48	H350 × 350 × 14 × 22	H350 × 320 × 14 × 22	H300 × 250 × 14 × 18
5–4	H420 × 250 × 14 × 20	H450 × 300 × 16 × 22	H1200 × 1000 × 46 × 56	H400 × 350 × 14 × 22	H350 × 350 × 14 × 22	H220 × 220 × 12 × 16
3–2	H420 × 250 × 14 × 20	H450 × 300 × 16 × 22	H1500 × 1200 × 50 × 62	H420 × 350 × 14 × 22	H350 × 350 × 14 × 22	H220 × 220 × 12 × 16
1	H350 × 250 × 12 × 18	H450 × 300 × 16 × 22	H1500 × 1200 × 50 × 66	H420 × 350 × 14 × 22	H350 × 350 × 14 × 22	—

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