Design and Specification Compilation of a Modularized Prefabricated High-rise Steel Frame Structure with Inclined Braces Part II: Elastic-plastic analysis and Joint Design

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ABSTRACT

Modularized prefabricated steel structure has some obvious advantages, such as fast construction, industrial-scale production and environment-friendliness. Although it has been used for low-rise buildings, its applications in high-rise buildings are quite less. The elastic-plastic time-history analysis under rare earthquake conditions is performed on a 30-floor building. The changing law of the base shear force, the story drift angle, the stress, the damage characteristics, etc. are studied. According to the theoretical analysis, the finite element simulation and the model test, the design methods and the relevant formulas regarding the elastic and elastic-plastic properties of the beam-column connection joints, the column flange joints and the inclined brace joints are proposed in this paper. The control parameters for the structural design are also discussed. This paper provides an important reference for the research and design of the same type of modularized prefabricated high-rise steel structures, and the design method has been compiled into design specification.

KEYWORDS

Modular prefabrication; high-rise steel structure; structural design formula; elastic-plastic design for rare earthquakes; elastic-plastic design of joints

INTRODUCTION

The key difference between the prefabricated steel structure and the traditional steel structure is that the connection, which are connected on-site, are all connected by bolts. Therefore, the study of the performance of the joints under rare earthquake conditions is significantly important. Because the Pushover analysis may ignore the influence of the high-order vibration mode, the time-history analysis was performed to assure the safety of the bolted joint (Papanikolaou 2005). The time-history analysis could foresee the performance of the structure under certain earthquake conditions, but a potential earthquake is different from the input earthquake; therefore, multi-earthquake waves may be input for study (Thai 2011). The internal force of the bolted joint could be obtained by time-history analysis, which is the basis for the elastic-plastic design of the joint (Nguyen 2014). To rapidly connect a column to another column on site, the bolted flange joint is used for the column-column joint. The bolted flange joint has been used in tower buildings (Wang 2013; Willibald 2002); however, it is seldom used for high-rise and multi-rise steel structure buildings. To facilitate and speed up the connection between columns and beams are all bolted on site. The joint type is different than the present joint, which is a bolted, semi-rigid connection; the performance is quite different for this joint, and the influence of the joint on the structure is also different (Mohsen 2011; Kurt 2003; Shi 2006; Liu 2015). This paper mainly introduces the methods of structural elastic-plastic design and the design and experimental study of the connection joints, based on the T30 project. The design formula of the main joints was obtained and demonstrated by the finite element analysis and the experimental data.

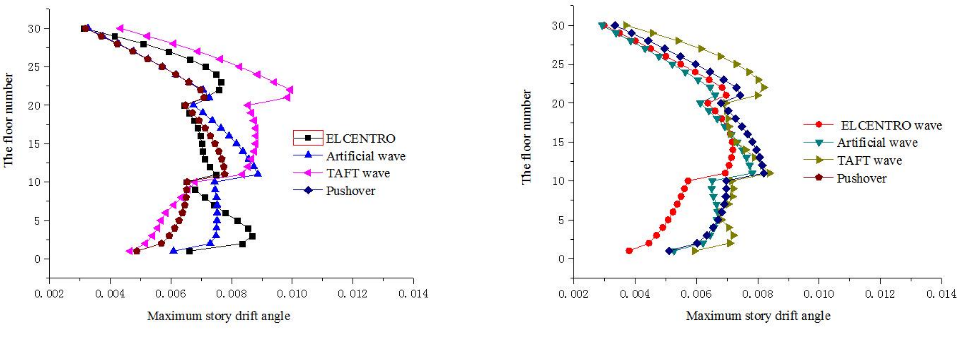
ELASTIC-PLASTIC TIME-HISTORY ANALYSIS UNDER RARE EARTHQUAKE CONDITIONS

Total base reaction

The curve of the total base reaction changing with the earthquake time step is obtained with the maximum acceleration peak value of 400 gal. By contrast, the total base reaction of the artificial wave and the ELCENTRO wave is greater than that obtained from the static pushover analysis. Whereas, the total base reaction of the TAFT wave is slightly greater than that obtained from the static pushover analysis.

Maximum story drift angle

As shown in Figure 1, the story drift angle mutates at the 11th and 21st layers, mainly due to the changing of the column and beam section at 11th and 21st story. The trend of the pushover analysis and the time-history analysis results is close, but the result is not identical. Because the elastic-plastic vibration of the pushover analysis mainly includes the action of the 1st and 2nd order vibration models; however, the seismic time-history analysis can arouse a much higher order vibration mode. The influence of the high-order vibration cannot be ignored for this type of structure.



(a) X direction (b)Y direction

Figure 1. Maximum story drift angle.

Member stress

The maximum stress is less than the material ultimate strength during an earthquake. The maximum stress distributes on the web members and the chord members of the truss beam between the inclined braces. In addition, during an earthquake, the truss web member yields first, and the chord yields next; the stress of the column between the inclined braces is low, and it would not yield. Therefore, the entire structure meets the anti-seismic design principle of “strong column and weak beam" and "strong joint and weak component,” and overall collapse would not occur because of broken columns.

Member damage

During an entire earthquake, when the maximum stress of the component overpasses the yield strength, the component will be defined as damage. The relationship between the component damage and the seismic time step is shown in figure 2. It can be seen from the figure that the component damage rate is very low during an earthquake, with the maximum damage rate of the ELCENTRO wave, the artificial wave and the TAFT wave being less than 0.85%, 1.9% and 1.0%, respectively. Therefore, the entire structure is in a safe state.

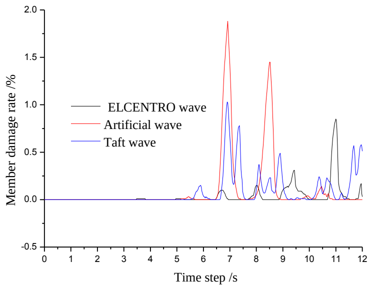


Figure 2. Curve of the member damage rate-earthquake time step.

DESIGN AND TEST OF JOINT

Design and test of column flange joint

As shown in Figure 3, flange joints are used to connect the column base of the main board with the columns. The 8M30 and 8M24 high-strength bolts are alternately arranged for the joints between the 1^{st} to 10^{th} stories. The 8M20 and 8M24 bolts are alternately arranged for the joints between the 11^{th} to 30^{th} stories. The high strength bolts are a S8.8 friction type connection. The bearing capacity design value of the joints should be greater than the maximum internal force design value of the column end under the dead load, the live load, the wind load and the frequent earthquakes to ensure that the high strength bolts would not slip. The calculation diagram of the column flange connections is shown in Figure 4. The elastic design should be calculated according to formulas (1) to (5), and meet the requirements of formula (1).

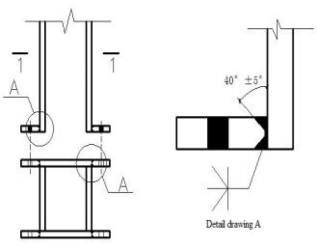


Figure 3. Column flange joint.

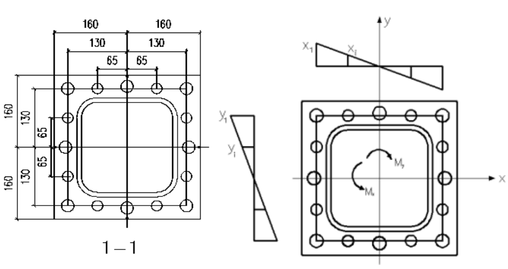


Figure 4. Calculation diagram.

$$(\frac{N}{\sum N_t^b}+\frac{\sqrt{V_x^2+V_y^2}}{\sum N_v^b}+\frac{M_x}{M_{tx}^b}+\frac{M_y}{M_{ty}^b})\gamma_{RE} \le 1 \quad (1)$$

$$N_{\mathrm{t}}^{\mathrm{b}} = 0.8P \quad (2)$$

$$N_{\mathrm{v}}^{\mathrm{b}} = 0.9\mu P \quad (3)$$

$$M_{tx}^b = \sum_{\mathrm{i=1}}^{\mathrm{n}} \mathrm{N_t^b y_{xi}^2 / y_{x1}} \quad (4)$$

$$M_{ty}^b = \sum_{\mathrm{i=1}}^{\mathrm{n}} \mathrm{N_t^b y_{yi}^2 / y_{y1}} \quad (5)$$

A calculation program was built to calculate all of the joints according to each load combination; the result was sorted from largest to smallest, according to the value of the left side of formula (1). The results show that the maximum value of the left side of formula (1) is 0.52, far less than 1.0, meets the design requirements with a high security margin. For some joints, the left side of formula (1) is negative, which indicates that the joints could also undertake the design load with the absence of a bolt on the flange joint because the axial pressure could enable the flange to remain tight enough to resist the design load.

Because the flange bolt connections have been used for the column, it is hard to realize the anti-bending equicohesive connections of the columns. To determine that the column flange connection joints would not fail under rare earthquake or extreme loading conditions, the elastic-plastic design of the column flange joints under these conditions based on seismic fortification intensity should be performed. The pushover or the elastic-plastic time-history analysis could be used for the elastic-plastic design. The calculation should be completed following formulas (6) through (11). When the two connected flanges do not slide relative to each other, formula (6) should be satisfied. When the two connected flanges slide relative to each other, the bolt connection changes into a bearing type connection, and formula (7) should be satisfied.

$$\eta_j(\frac{N_{pdz}}{\sum N_t^b}+\frac{\sqrt{V_{pdzx}^2+V_{pdzy}^2}}{\sum N_v^b}+\frac{M_{pdzx}}{M_{tx}^b}+\frac{M_{pdzy}}{M_{ty}^b}) \le 1 \quad (6)$$

$$\eta_j\sqrt{(\frac{\sqrt{V_{pdzx}^2+V_{pdzy}^2}}{\sum N_{vu}^b})^2+(\frac{N_{pdz}}{\sum N_{tu}^b}+\frac{M_{pdzx}}{M_{tux}^b}+\frac{M_{pdzy}}{M_{tuy}^b})^2} \le 1 \quad (7)$$

$$N_{vu}^b = 0.58A^b f_u^b \quad (8)$$

$$N_{tu}^b = A_e^b f_u^b \quad (9)$$

$$M_{tux}^b = \sum_{\mathrm{i=1}}^{\mathrm{n}} \mathrm{N_{tu}^b y_{xi}^2 / y_{x1}} \quad (10)$$

$$M_{tuy}^b = \sum_{\mathrm{i=1}}^{\mathrm{n}} \mathrm{N_{tu}^b y_{yi}^2 / y_{y1}} \quad (11)$$

According to the flange design formulas proposed in this paper, a program is built to design the column flange joint under the internal force of a rare earthquake achieved by a time-history analysis and a static pushover analysis. The calculation results of most joints could meet the requirements of the formula; the calculation results are sorted in descending order by the value of the left side of formula (6) or (7). All bolted flange joints meet the requirements of formula (6) and (7) by the pushover analysis. The pushover analysis shows the internal force of the largest joints is at the bottom of the story, which is different from that of the time-history analysis. The elastic-plastic time-history analysis under rare earthquake conditions indicates that flange joints meet the design requirements when the ELCENTRO wave is input. However, several joints do not meet the requirements when the artificial wave or TAFT wave is input. For such cases, the joint has to be strengthened by welding. The locations of the danger joints not only distribute at the bottom of the structure but also distribute at the middle and top of the structure, which indicates that the time-history analysis could include the influence of the high order vibration mode. Therefore, the rare earthquake time-history analysis is necessary for the flange joint, which could not be substituted by the static pushover analysis.

To study the bearing capacity and stiffness of the column flange joint in more detail and to verify the calculation formulas, a high-strength bolted flange connection model considering friction contact and the welded connections has been established. The load-displacement curve of the column top is shown in figure 5, which shows that the bolted connection stiffness of every segment is slightly smaller than that of the welded connections, but the difference is so small that it could be ignored. The stiffness difference between these two types of connections increases as the thickness of columns section increases. For segments 1 to 3, the difference is 0.3%, 1.1% and 1.6%, respectively, and the flange does not slip. The results also show that when the strength and stiffness are taken into account, the bolted flange connections between the columns could be assumed to be rigid connections.

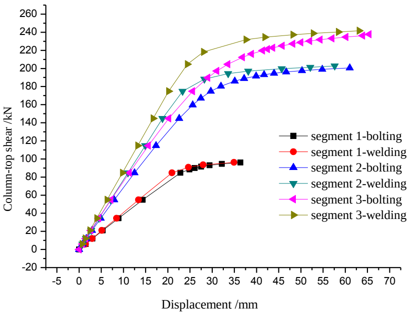


Figure 5. Load-displacement curves of the column top **Figure 6.** Model test of flange joint.

Bending and shearing tests have been performed for the flange joints of columns. As shown in Figure 6, one end of the column is fixed, whereas the other end is applied with vertical shear to produce shear force and a bending moment at the flange joints. The test results show that for segments 1 to 3, the location of the damage is not on the flange; there is no relative sliding between the flange plates. It is demonstrated that the flange connections could be considered as rigid connections. With a combination of theoretical analysis, experimental results and the characteristics of the actual structure and considering that the flange joint is enclosed in the joint region by the inclined braces, it has been determined that the force of joint is smaller. Therefore, during the analysis process of the entire structural system, the bolted flange joints could be simplified as rigid connections to significantly reduce the computational time. The internal force of the joint would be obtained for the joint design as described above.

Design and test of beam-column joint

The truss beam-column joint of the structure has two types: bolted and welded. The welded connections could be calculated as rigid connections. The bolted connections between the end plate of the truss beams and the column belong to a non-typical, semi-rigid connection, which is first proposed in this paper. The thickness of the end plate, the type of plate, the arrangement and the number of bolts imposed large impact on the stiffness of the beam-column joints. Therefore, for the overall structural design, the bolted connections should not be assumed as rigid connections. Instead, such connections should be assumed as semi-rigid connections. Because of the complexity of semi-rigid connections, it is hard to obtain the accurate stiffness from a purely theoretical calculation or finite element analysis. Given this, full size tests on the bolted beam-column joint have been completed in the present paper, as shown in Figure 7. The bending moment-rotation curve of the joints is obtained through a series of tests. Under the design load, the rotational stiffness of the joint of segment 3 is 61525 kN ·m/rad, and the rotational stiffness of the joint of segment 2 is 43383 kN ·m/rad. These values are converted into spring stiffness and used in the theoretical analysis and the finite element calculation of the whole structure.

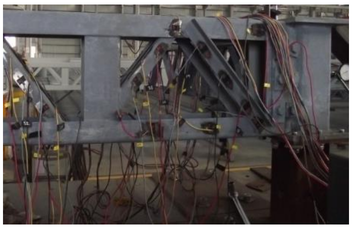


Figure 7. Model test setup

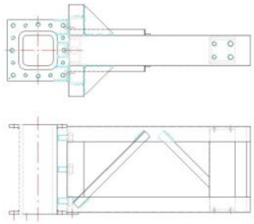


Figure 8. Structural diagrams

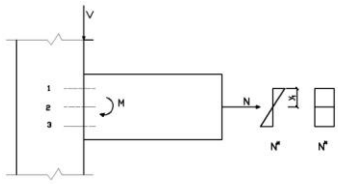


Figure 9. Calculation diagram

The bearing capacity design value of the beam-column joint should be greater than the combination internal force design value under the dead load, the live load, the wind load and the earthquake conditions. Moreover, the high-strength bolts should not slip. The structural diagram is shown in Figure 8, and the calculation sketch is shown in Figure 9. The joint design should be in accordance with formulas (12) to (14), respectively, and meet the requirements of formula (12).

$$\left(\frac{N_{\mathrm{v1}}}{N_{\mathrm{v}}^{\mathrm{b}}}+\frac{N_{\mathrm{t1}}}{N_{\mathrm{t}}^{\mathrm{b}}}\right)/\gamma_{\mathrm{RE}} \le 1 \quad (12)$$

$$\mathrm{N}_{\mathrm{v1}}=\frac{\mathrm{V}}{\mathrm{n}} \quad (13)$$

$$\mathrm{N}_{\mathrm{t1}}=N_{\max}=\frac{\mathrm{N}}{\mathrm{n}}+\frac{\mathrm{My}_1}{\sum_{\mathrm{i}=1}^{\mathrm{n}}\mathrm{y}_{\mathrm{i}}^2} \quad (14)$$

The mechanical behavior of all beam-column joints under various load combinations is calculated according to the formula. The maximum value of the left side of the formula is 0.86, which is far less than 1.0. Therefore, it meets the design requirements. The rare earthquake elastic-plastic design of the beam-column joint of the steel frame structure with inclined braces should be conducted according to the seismic fortification intensity. The internal force of the joint could be obtained by the pushover analysis and the elastic-plastic time-history analysis. The design should be conducted according the formulas (15) to (19). Formulas (15) and (19) should be met, or formulas (16) and (19) should be met. Formula (15) is used to verify whether the connecting surface will slip; if slip occurs at the surface of the bolt and hole connect, and the bolts are converted into bearing pressure type connections. Formula (16) is then used to verify whether the bolts are shear broken. Formula (19) is used to avoid the hole surface press damage.

$$\eta_j\left(\frac{N_{v1}^{dz}}{N_v^b}+\frac{N_{t1}^{dz}}{N_t^b}\right)\le 1 \qquad (15)$$

$$\eta_j\sqrt{\left(\frac{N_{v1}^{dz}}{N_{vu}^{b}}\right)^2+\left(\frac{N_{t1}^{dz}}{N_{tu}^{b}}\right)^2}\le 1 \quad (16)$$

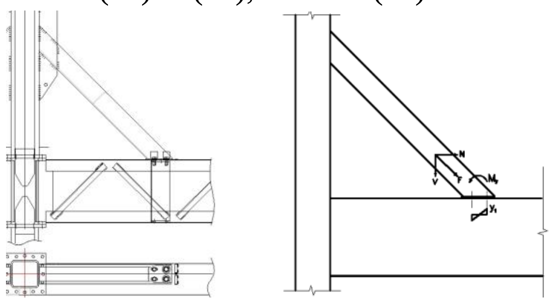
$$N_{v1}^{dz}=\frac{V_{pdz}}{n} \qquad (17)$$

$$N_{t1}^{dz}=N_{max}=\frac{N_{pdz}}{n}+\frac{M_{pdz}y_1}{\sum_{i=1}^{n}y_i^2} \quad (18)$$

$$N_{v1}^{dz}<N_{cu}^{b}=d\sum t(1.5f_u)/1.2 \qquad (19)$$

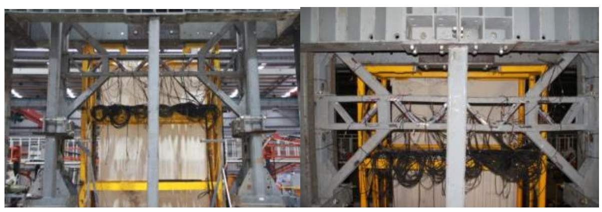
Design and test of inclined brace joints

As shown in figure 10 (a), the inclined braces are welded connected to the column at one end and are bolted connected to the truss at the other end; both of the joints are considered rigid connections. The calculation sketch is shown in Figure 10 (b). The welded connection is designed according to the *Code for seismic design of buildings* (GB50011-2010) and the *Code for design of steel structures* (GB50017-2003). Only elastic design is needed, based on the maximum internal force combination. To simplify the calculation, the weld is designed based on the tension equicohesive of the inclined braces section. By contrast, the high strength bolt friction type connection is used for the bolted joint of the inclined braces according to formulas (12) to (14); formula (12) should be met.



(a) Structural sketch (b) Calculation sketch

Figure 10. Inclined brace joint



(a) Before the load (b) Failure mode

Figure 11. Frame test under vertical and horizontal loads.

The rare earthquake conditions of the elastic-plastic design are conducted on the bolted connections of the prefabricated steel frame structure with inclined braces. The internal force of the joint is obtained by the pushover analysis and the elastic-plastic time-history analysis. The design is conducted according to the above formulas (15) to (19); formulas (15) and (19) should be met, or formulas (16) and (19) should be met. The joint design program under rare earthquake conditions was built according to the formula above. The calculation results meet the formula, which shows that the inclined brace joints are reliable and would not break under rare earthquake conditions. The seismic design principle “strong joint and weak component” is guaranteed. This conclusion is verified by the frame test, as shown in Figure 11. The bolted joints of the inclined braces do not damage or slip, and no visible deformation under ultimate horizontal load is indicated. In summary, the bolted joint is not need to realize an equicohesive connection, but it is needed only to assure that the joints do not damage under rare earthquake conditions.

CONCLUSIONS

(1) The rare earthquake elastic-plastic time-history analysis considers that the influence of the high order vibration mode could not be substituted by the pushover analysis. The elastic-plastic time-history analysis is necessary for the modularized prefabricated steel structure to avoid progressive collapse of the whole structure caused by damage of the joints.

(2) The columns and column bases are connected by flanges, which not only could meet the strength requirements but also could be assumed to be rigid connections if they are designed according the method proposed in this paper.

(3) The beam-column joints are semi-rigid connections; the connecting stiffness of the joint that is not enclosed by an inclined brace has significant influence on the design of the entire structure. The accurate stiffness should be obtained by a test before the structural integral analysis.

(4) The column-column joint, the beam-column joint and the inclined brace joint are difficult to realize an equicohesive connection by a high-strength bolt, which is required by Chinese code. However, to assure structural safety, checking the ultimate strength of the joint under rare earthquake conditions is enough, according to the formulas suggested in this paper.

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