SEISMIC DESIGN OF STEEL FRAMES CONSIDERING STRUCTURAL WEIGHT AND FABRICATION COST

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ABSTRACT :

The building structural engineers can't necessarily predict the fabrication cost from the structural weight appropriately. It is because the fabrication cost for steel members depends upon the complexity of the connections rather than the structural weight. In this study, simple fabrication time functions for steel building rigid frames are first shown. Next, three steel building frames (Nos.1, 2 and 3) satisfying Japanese seismic codes are designed. Members of a building frame are generally grouped together to have the same size. No.1 is designed so as to make the fabrication cost small by using small number of groups of member sizes; No.3 to make structural weight small by using large number of groups of member sizes; No.2 is the intermediate case. The fabrication time for 3 frames is estimated by the presented function and the total cost consisting of fabrication time and material cost is discussed.

KEYWORDS: Fabrication time functions, Structural design, Seismic design

1. INTRODUCTION

The building structural engineers can't necessarily predict the fabrication cost from the structural weight appropriately. It is because the fabrication cost of steel members depends on the complexity of the connections rather than the structural weight. Some fabrication time functions have previously been proposed and applied to a welded stiffened plates (Jarmai(2002)) and steel frames (Pavlovcic et al.(2004))

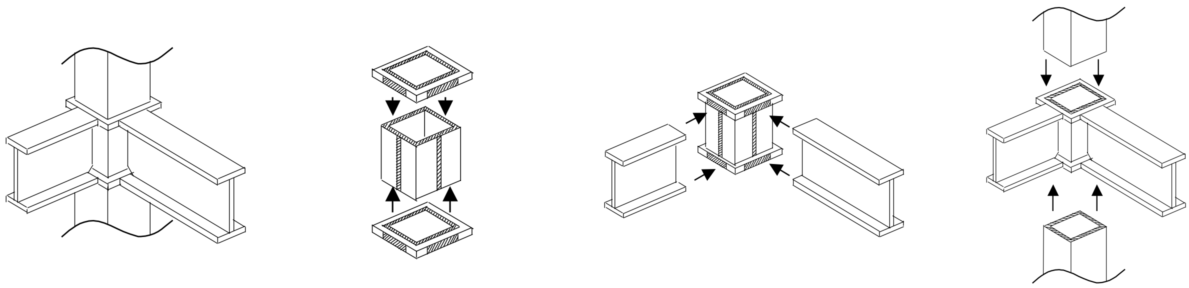
This paper presents a simpler fabrication time function for steel building rigid frames, which has been derived from questionnaires given to managers of three fabricating companies in Japan. The proposed function is based on the following assumptions.

(1) The total fabrication time includes the preparatory process time, such as cutting and drilling bolt holes, the assembly time of columns and beams, the welding time and the time of preparing shop drawings. (2) The preparatory process time is proportional to the number of diaphragms and beams. (3) The assembly time of columns and beams is proportional to the number of steel structural parts, such as columns, beams and diaphragms. (4) The welding time is proportional to the jointed sectional area of columns, beams and diaphragms. (5) The time of preparing shop drawings is proportional to the structural weight and the number of columns and beams of different sizes. (6) The painting, transportation and erection times are not considered.

Next, the coefficients of each function are presented, which are computed from the fabrication time data for four steel buildings in Japan, using the least squares method. Finally, three steel building frames (Nos.1, 2 and 3) satisfying Japanese seismic codes are designed. Members of a building frame are generally grouped together to have the same size. No.1 is designed so as to make the fabrication cost small by using small number of groups of member sizes; No.3 to make the structural weight small by using large number of groups of member sizes; No.2 is the intermediate case. The fabrication time for 3 frames is estimated by the presented function and the total cost consisting of fabrication time and material cost is discussed.

2. TYPICAL BEAM TO COLUMN CONNECTION IN JAPAN

Figure 1(A) shows typical H-beam-to-RHS-column connections in Japan. At the fabricating company, the box column connection is first welded to two through diaphragms by full-penetration welds, as shown in Fig. 1(B). The connection is welded to the flanges of the bracket by full-penetration welds and to the web of the bracket by fillet welds as shown in Fig. 1(C). Finally, the columns are welded to the connection by full-penetration welds as shown in Fig. 1(D). In the field, the bracket is connected to the beam by high strength bolts. This study deals with buildings having the beam-to-column connection shown in Fig. 1.



(A) (B) (C) (D)

Figure 1 Typical beam to column connection

3. FABRICATION TIME FUNCTIONS (Sasaki et. al (2007))

In this study, the following function is used to predict the steel fabrication time:

$$TF=TP+TB+TW+TI \tag{3.1}$$

where TF represents the steel fabrication time, TP represents the preparatory process time, TB represents the assembly time, TW represents the welding time, and TI represents the time of preparing shop drawings.

The preparatory process consists of marking and drilling of diaphragms, marking, drilling and blasting of brackets and marking, drilling, blasting and flanging bevels of beams. The questionnaires indicated that the preparatory process time depends on the number of parts such as diaphragms, beams and brackets rather than structural weight. The following function is proposed to estimate the preparatory process time TP.

$$TP = KP \cdot (\sum_{i=1}^{nj} NP_{D_i} + \alpha_{PB} \cdot NP_B) \tag{3.2}$$

where N_{PDi} represents the number of diaphragms for the beam to column connection i, nj represents the number of beam to column connections, NP_B represents the number of beams and brackets and α_{PB} and KP represent the coefficients for evaluating the preparatory process time.

The questionnaires indicated that assembly time also depends on the number of parts rather than the structural weight. Therefore, function TB to estimate the assembly is expressed as follows:

$$TB = KB \cdot (\sum_{i=1}^{nj} NB_{0_i} + \alpha_{BC} \cdot NB_C) \tag{3.3}$$

where NB_{0i} represents the number of parts consisting of connection panels and brackets for the beam to column connection i, NB_C represents the number of columns and α_{BC} and KB represent the coefficients for evaluating the assembly time.

The questionnaires showed that the welding time depends on the sum of jointed sectional areas. The following function is proposed to estimate the welding time TW.

$$TW = KW \cdot \left(\sum_{i=1}^{nj} A_{Di} + \sum_{i=1}^{nbb} A_{BBi} \right) \tag{3.4}$$

where A_{Di} represents the jointed sectional area between the column and the diaphragm for the beam to column connection i, and A_{BBi} represents the jointed sectinal area between the column and the bracket, *nbb* represents the number of brackets and *KW* represents the coefficient for evaluating the welding time.

Since the time of preparing shop drawings depends on the number of sheets of shop drawings, the following function, based on the number of columns and beams, is proposed:

$$TI = KIc \cdot NIc + KIb \cdot NIb + KIg \cdot W \tag{3.5}$$

where *NIc* represents the number of shop fabricated column trees, *NIb* represents the number of beam groups having the same cross-sectional size, *W* represents the total structural weight of the frame and *KIc*, *KIb* and *KIg* represent the coefficients for evaluating the time of preparing shop drawings.

3. COEFFICIENTS TO EVALUATE THE FABRICATION TIME (Sasaki et. al (2007))

The values of α_{PB} and α_{BC} in Eqs.(3.2) and (3.3) were computed from questionnaires on fabrication time as follows.

$\alpha_{PB} = 2$, $\alpha_{BC} = 7$

The values of *KP*, *KB*, *KW*, *KIb*, *KIc* and *KIg* in Eqs. (3.2), (3.3), (3.4) and (3.5) were computed from the least squares approximation, based on recorded fabrication time data of just one fabricating company, as follows: *KP* = 0.85 (hours), *KB* = 1.07(hours), *KW* = 0.013 (hours/cm2), *KIb* = 2.87(hours), *KIc*= 4.12 (hours), *KIg* = 0.067 (hours/kN)

4. STRUCTURAL DESIGN AND COST EVALUATION

Three steel building frames (Nos.1, 2 and 3) satisfying Japanese seismic codes are designed. No.1 is designed so as to make the fabrication cost small; No.3 to make the structural weight small; No.2 is the intermediate case. No.1 uses small number of groups of member sizes to make the fabrication cost small. On the other hand, No.3 uses large number of groups of member sizes to make the structural weight small. The beam plan and framing elevations are shown in Fig. 2. Table 1 and 2 show the cross sectional size of columns and beams for 3 frames (Nos.1, 2 and 3). The ultimate story shear force under push-over analysis and the story drift angle under

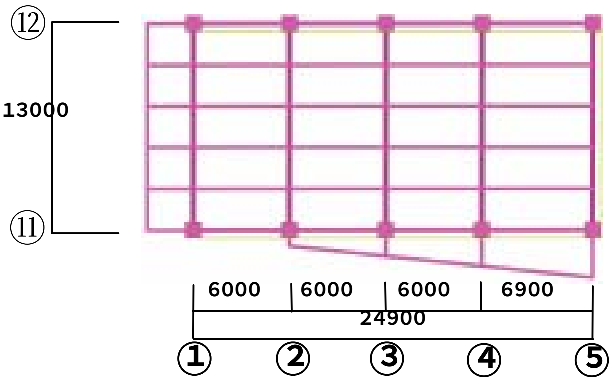


Figure 2(A) Beam plan

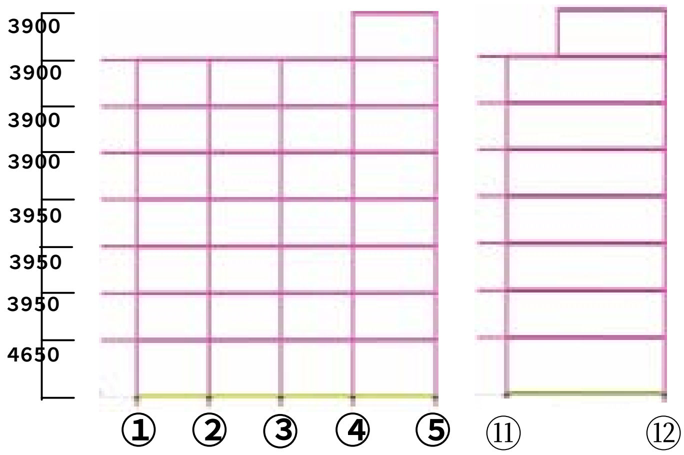


Figure 2(B) framing elevation

Table 1(A) Cross sectional size of columns for No.1 frame

	①	②	③	④	⑤
Roof				□500×19	□500×19
7th floor	□500×19	□500×19	□500×19	□500×19	□500×19
6th floor	□500×19	□500×19	□500×19	□500×19	□500×19
5th floor	□500×22	□500×22	□500×22	□500×22	□500×22
4th floor	□500×22	□500×22	□500×22	□500×22	□500×22
3rd floor	□500×25	□500×25	□500×25	□500×28	□500×28
2nd floor	□500×32	□500×32	□500×32	□500×32	□500×32
1st floor	□500×32	□500×32	□500×32	□500×32	□500×32

*□500×19 represents the box column whose depth is 500 (mm) and thickness is 19(mm).

Table 1(B) Cross sectional size of columns for No.2 frame

	①	②	③	④	⑤
Roof				□500×19	□500×19
7th floor	□500×19	□500×19	□500×19	□500×19	□500×19
6th floor	□500×19	□500×19	□500×19	□500×19	□500×19
5th floor	□500×22	□500×22	□500×22	□500×22	□500×22
4th floor	□500×22	□500×22	□500×22	□500×22	□500×22
3rd floor	□500×25	□500×25	□500×25	□500×28	□500×28
2nd floor	□500×32	□500×32	□500×32	□500×32	□500×32
1st floor	□500×32	□500×32	□500×32	□500×32	□500×32

Table 1(C) Cross sectional size of columns for No.3 frame

	①	②	③	④	⑤
Roof				□450×19	□450×19
7th floor	□450×19	□450×19	□450×19	□450×19	□450×19
6th floor	□450×19	□450×19	□450×19	□450×19	□450×19
5th floor	□500×22	□500×22	□500×22	□500×22	□500×22
4th floor	□500×22	□500×22	□500×22	□500×22	□500×22
3rd floor	□550×22	□550×22	□550×22	□550×22	□550×22
2nd floor	□550×22	□550×22	□550×22	□550×22	□550×22
1st floor	□550×22	□550×22	□550×22	□550×25	□550×25

Table 2(A) Cross sectional size of beams for No.1 frame

	⑪⑫ X direction	① Y direction	②③ Y direction	④ Y direction	⑤ Y direction
Penthouse	H600×200×12×19			H600×200×12×19	H600×200×12×19
Roof	H750×250×14×25	H750×300×14×22	H750×300×14×22	H750×300×14×22	H750×300×14×22
7th floor	H750×250×14×25	H750×300×14×25	H750×300×14×25	H750×350×14×28	H750×350×14×28
6th floor	H750×250×14×25	H750×300×14×25	H750×300×14×25	H750×350×14×28	H750×350×14×28
5th floor	H800×250×16×25	H800×350×16×25	H800×350×16×25	H800×400×16×28	H800×400×16×28
4th floor	H800×250×16×25	H800×350×16×25	H800×350×16×25	H800×400×16×28	H800×400×16×28
3rd floor	H800×250×16×25	H800×350×16×25	H800×350×16×28	H800×400×16×32	H800×400×16×28
2nd floor	H800×300×16×28	H800×350×16×25	H800×350×16×28	H800×400×16×32	H800×400×16×32

**H600×200×12×19 represents the H beam whose depth is 600 (mm), width is 200 (mm), thickness of web is 12 (mm) and thickness of flange is 19(mm).

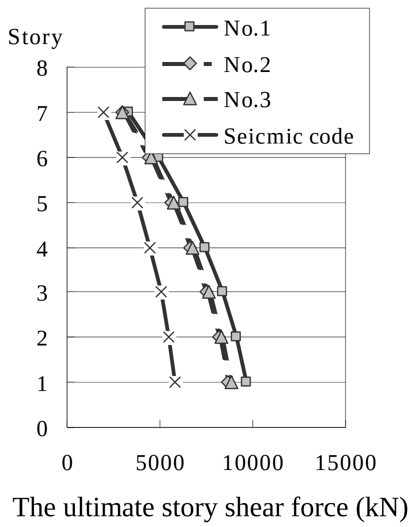
design seismic load of three frames are shown in Fig. 3. It is observed from this figure that No.1 frame has higher load capacity and story stiffness in X direction. It is because No.1 frame uses the same cross sectional depths in both X and Y direction. Figure 5 shows total structural weight for three frames. Figure 6 shows fabrication time for three frames. According to Fig.5, the structural weight for No.1 frame is the largest. On the other hand, the fabrication time for No.1 frame is the shortest, because the same cross sectional depths in both X and Y direction lead to simple connections. The fabrication time for frame No.3 is the longest because a large number of groups of member sizes lead to complex connections. The total cost function K can be expressed by the following equation (Jarmai & Farkas (1999)).

Table 2(B) Cross sectional size of beams for No.2 frame

	⑪⑫ X direction	① Y direction	②③ Y direction	④ Y direction	⑤ Y direction
Penthouse	H600×200×12×19			H600×200×12×19	H600×200×12×19
Roof	H600×200×12×19	H750×300×14×22	H750×300×14×22	H750×300×14×22	H750×300×14×22
7th floor	H600×200×12×22	H750×300×14×25	H750×300×14×25	H750×350×14×28	H750×350×14×28
6th floor	H600×250×12×25	H750×300×14×25	H750×300×14×25	H750×350×14×28	H750×350×14×28
5th floor	H650×250×12×22	H800×350×16×25	H800×350×16×25	H800×400×16×28	H800×400×16×28
4th floor	H650×250×12×28	H800×350×16×25	H800×350×16×25	H800×400×16×28	H800×400×16×28
3rd floor	H650×300×16×28	H800×350×16×25	H800×350×16×28	H800×400×16×32	H800×400×16×28
2nd floor	H650×300×16×28	H800×350×16×25	H800×350×16×28	H800×400×16×32	H800×400×16×32

Table 2(C) Cross sectional size of beams for No.3 frame

	⑪⑫ X direction	① Y direction	②③ Y direction	④ Y direction	⑤ Y direction
Penthouse	H600×200×12×19			H600×200×12×19	H600×200×12×19
Roof	H600×200×12×19	H750×300×14×22	H750×300×14×22	H750×300×14×22	H750×300×14×22
7th floor	H600×200×12×22	H750×300×14×25	H750×300×14×25	H750×350×14×28	H750×350×14×28
6th floor	H600×250×12×25	H750×300×14×25	H750×300×14×25	H750×350×14×28	H750×350×14×28
5th floor	H650×250×12×22	H800×350×16×25	H800×350×16×25	H800×400×16×28	H800×400×16×28
4th floor	H650×250×12×28	H800×350×16×25	H800×350×16×25	H800×400×16×28	H800×400×16×28
3rd floor	H650×300×16×28	H800×350×16×25	H800×350×16×28	H800×400×16×32	H800×400×16×28
2nd floor	H650×300×16×28	H800×350×16×25	H800×350×16×28	H800×400×16×32	H800×400×16×32



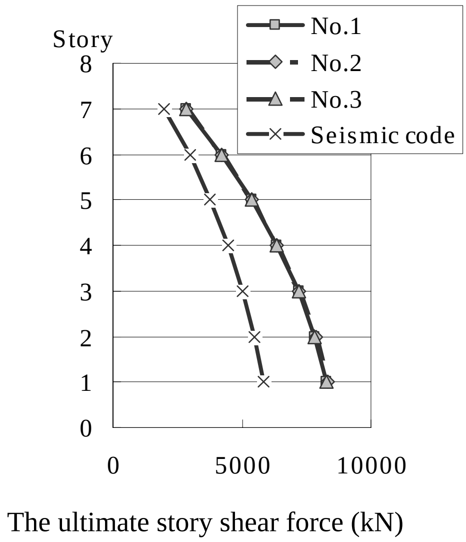


Figure 3(A) The ultimate story shear force (X direct.) Figure 3(B) The ultimate story shear force (Y direct.)

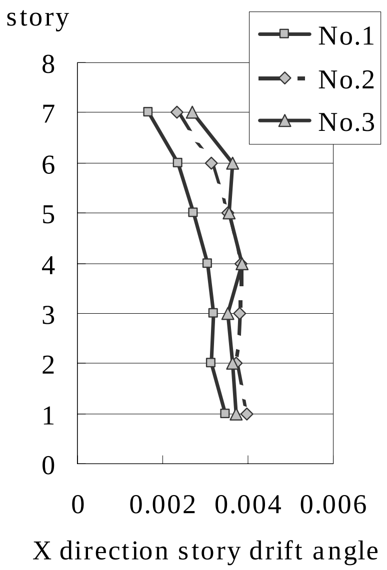


Figure 4(A) X direction story drift angle

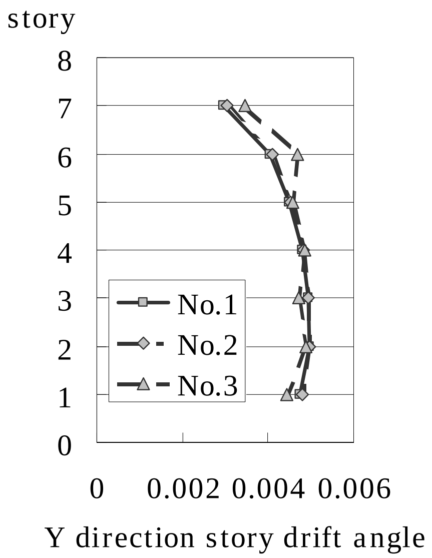


Figure 4(B) Y direction story drift angle

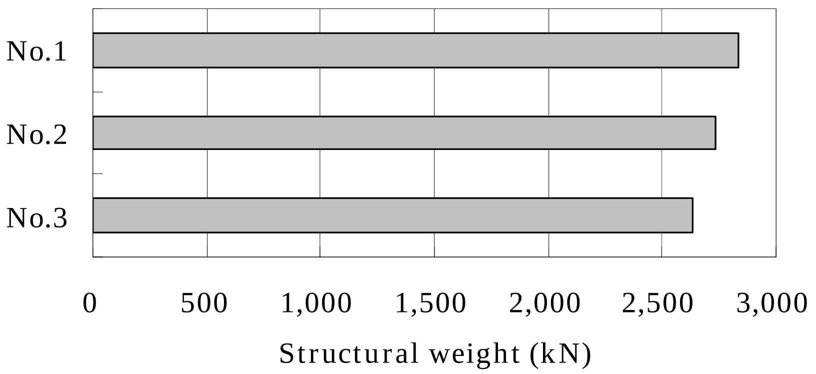


Figure 5 Structural weight

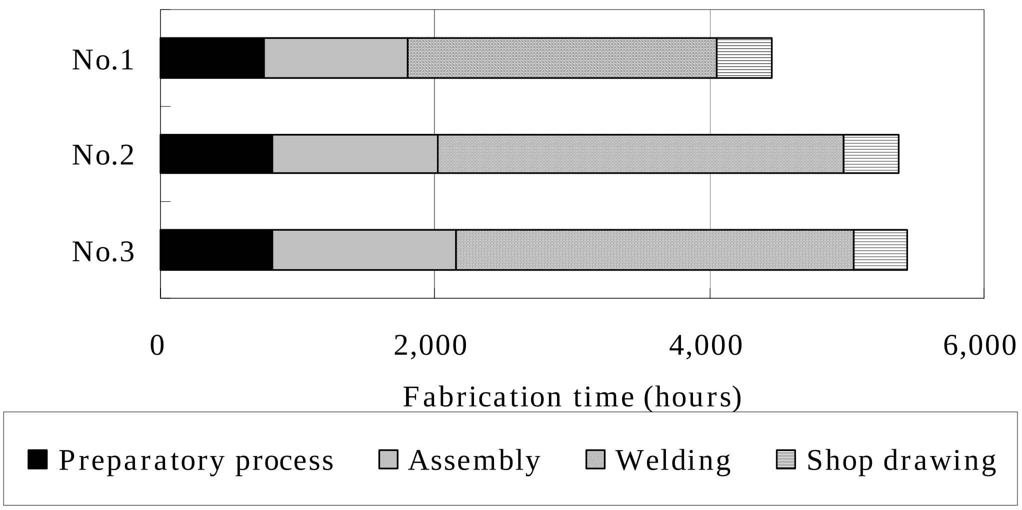


Figure 6 Fabrication time

$$K = k_m \cdot W + k_f \cdot TF \quad (4.1)$$

where W is the structural weight, and km and kf are the material cost factor and labor cost factor, respectively. Eq. (4.1) can be written in the following form (Jarmai & Farkas (1999)).

$$\frac{K}{k_m} = W + \frac{k_f}{k_m} \cdot TF \quad (4.2)$$

Figure 7 shows the relationship between K/km and kf/km for Nos.1, 2 and 3. Frame No.3 gives the smallest total cost when kf/km is less than around 0.2. Frame No.1 gives the smallest total cost when kf/km is greater than around 0.2. This figure indicates that the less weight frame (No.3) does not necessarily have the minimum cost, and that appropriate grouping of member sectional size (No.1) leads to a lower total cost in many cases.

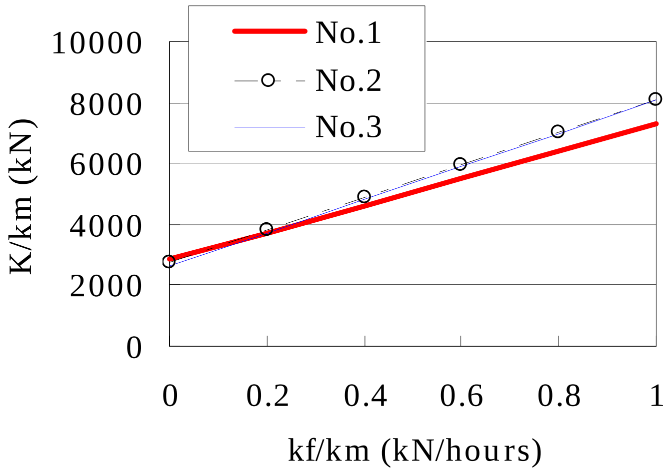


Figure 7 The relationship between K/km and kf/km

5. CONCLUSIONS

In this study, simple fabrication time functions for steel building rigid frames have been first shown. Next, three steel building frames (Nos.1, 2 and 3) satisfying Japanese seismic codes have been designed. No.1 have been designed so as to make the fabrication cost small by using small number of groups of member sizes; No.3 to make structural weight small by using large number of groups of member sizes; No.2 is the intermediate case. The fabrication time for 3 frames has been estimated by the presented function and the total cost consisting of fabrication time and material cost has been discussed. These examples indicated that the minimum weight frame does not necessarily produce the minimum cost, and that appropriate grouping of member sizes leads to a lower total cost in many cases.

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