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COMPARISON OF SUPERIOR STEEL BUILDING DESIGN SOLUTIONS FOR DIFFERENT LATERAL FRAME SYSTEMS AND COLUMN SHAPES

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Abstract

Superior design solutions of section sizes in seven-story steel buildings are obtained for three types of structural systems: (1) a space frame system with rectangular hollow structural section (HSS) columns (SFS), (2) perimeter frame systems (PFS) with I-shaped columns (PFSH), (3) PFS with rectangular HSS columns (PFSB). Moment connections are used in most beam-to-column connections in SFS, while they are limitedly used in the perimeter frames in PFS. SFS is a commonly used structural system in Japan, whereas PFSH is commonly used in other countries. In this research, structural characteristics of SFS, PFSH and additionally PFSB are evaluated for evenly rationally designed office buildings using an optimization algorithm. The superior solutions are derived by multiple start local search (MSLS), minimizing steel volumes. The solutions satisfy multiple requirements of the allowable stress design and ultimate lateral strength. The discrete design variables are the section sizes of grouped structural members. The problem has approximately 100 constraints and 40 variables. Dealing with these large numbers, the proposed MSLS algorithm works well and superior solutions are obtained for various types of building structures, such as moment frame, braced frame and mixed frame buildings, in the three types of structural systems, SFS, PFSH and PFSB. The main findings are as follows:

(1) Superior solutions for moment frame buildings are obtained for the base-shear coefficient of the ultimate lateral strength C_{QUN1} = 0.3 and 0.6. The value C_{QUN1} = 0.6 is given by referring to responses in the time-history analyses for very rare level 2 (L2) earthquake ground motions. PFSH can be advantageous for the moment frame building in terms of steel volume.

(2) Superior solutions of the braced frame building are obtained for C_{QUN1} = 0.35 and 1.0. The sections of the braces are steel pipes. The differences of steel volumes among PFSH, SFS and PFSB are relatively small. The steel volume is slightly smaller in SFS and PFSB, because axial forces are the primarily derived member forces under earthquake lateral load in these braced frame buildings and rectangular HSS columns are advantageous. The C_{QUN1} value needed for the L2 time-history analysis is nearly 1.0, which is very different from 0.35 required by the design standard.

(3) A comparison of the superior design solutions for mixed structures shows that the steel volume of SFS solution is larger than those of PFSs. Irregularity in the beam spans or different lateral systems in two horizontal directions causes an increase in the steel volume in SFS, depending on some critical constraints, such as uniform beam height in a single floor and column to beam strength ratio.

Superior design solutions are obtained by using the optimization algorithm but not based on engineers' personal experience. Therefore, although the number of cases studied in this research is limited, the discussion and findings comparing these different structural systems are of interest.

Keywords: steel structure, lateral frame, multiple start local search, ultimate lateral strength, time-history analysis

1. Introduction

Beam-to-column connections of steel buildings consist of two types: moment connections and pinned connections. The flanges of beams are rigidly connected to the columns in the moment connections, while they are not in the pinned connections. The lateral frames are composed of columns and beams with moment connections, and the gravity frames are composed of those with pinned connections. Steel buildings in Japan mostly have lateral frames, whereas in other countries including the US, lateral frames are limitedly placed typically in the perimeter frames separately from the gravity frames. The former system is called, in this paper, the space frame system (SFS) and the latter is the perimeter frame system (PFS). Rectangular hollow structural section (HSS) columns are normally used in the SFS, and I-shaped columns are used in the PFS. Past research [1,2] has focused on the differences of these systems; however, the buildings compared may not be evenly and rationally designed and discussion on the findings on their structural characteristics may not always be objective.

The authors proposed an algorithm to obtain superior design solutions for the SFS and PFS for seven-story office buildings and compared their structural characteristics [3,4]. The multiple-start-local-search (MSLS) approach was used to obtain the superior solutions, minimizing the steel volume with discrete variables of the section sizes. The solutions satisfy many structural design requirements in the building codes of allowable stress design and ultimate lateral strength. The superior solutions of SFS and PFS systems obtained through this algorithm are independent of designers' skills or experiences. Their structural characteristics were objectively discussed and dominant design requirements or constraints were identified. A comparison revealed that the steel volume of PFS is smaller than that of SFS. Also, the steel volumes of the superior solutions are smaller than the statistical average of the steel buildings in the same sizes. This fact confirms the effectiveness of MSLS.

In this research, the superior design solutions with different structural systems and building structures are obtained for many structural design requirements including the ultimate lateral strength defined referring to the time-history analyses for level 2 (L2) earthquake records. Structural characteristics and responses for the L2 time-history analyses are examined from various aspects. The previous work [4] by the authors is extended and the properties are evaluated for a new structural system, PFSB, in which rectangular HSS columns are used in PFS. The traditional PFS using I-shaped columns is re-defined as PFSH for comparison with PFSB. SFS, PFSH and PFSB are examined for moment frame and braced frame buildings. Furthermore, mixed structures with the moment and braced frames are investigated. Influences of locations of the lateral frames and column shapes on the steel volume are evaluated under constraints on seismic performance, and the possibility of a new structural design approach is suggested.

2. Outline of Building Examined and Structural Design Approach

2.1 Outline of Building

A rectangular seven-story steel office building with the plan size 32.0 × 19.2 (m) is examined. The size of the building plan is the same as that examined in the previous work [4]; however, the column spacing is uniform at 6.4 m in this research. Moment frame and braced frame structures as shown in Fig. 1 are examined. The building is simplified in order to identify general structural characteristics. The solid triangles in Fig. 1 indicate moment connections and the others are pinned connections. All beam-to-column connections are moment connections and all frames are lateral frames in SFS, while four frames in the perimeters are the lateral frames and the others are the gravity frames in PFS.

Figure 2 shows the frame elevations for the braced frames. Concentrated braces are used. The geometry is the same for the moment frames except for the existence of the braces. Many braces are placed as seen in the elevation and they exceed the general design code. Thus, the flexural deformation of the multi-story braced frames is restrained and shear deformation is dominant. Therefore, additional axial forces in the columns in the braced frames under seismic lateral loads do not have much effect on the column design. The solid triangles in the columns in 2nd- and 5th-stories in Fig. 2 indicate the splices. The segments between the column splices are called "parts" and the members are grouped in each part. i.e. member sections are grouped in the parts. The names of columns and beams are shown in Figs. 1 and 2. GX2 and GY2 in the figures for PFS are pinned at the ends and designed only for the gravity load. However, these names are kept identical between the PFS and SFS for the sake of simplification.

The grouped member sections are shown in Table 1. The columns are rectangular HSS or I-shaped sections, and the beams and braces are I-shaped sections and pipes, respectively. The steel grade is assumed to be SN490 and the design standard strength (approximate nominal yield strength) is 325 N/mm^2.

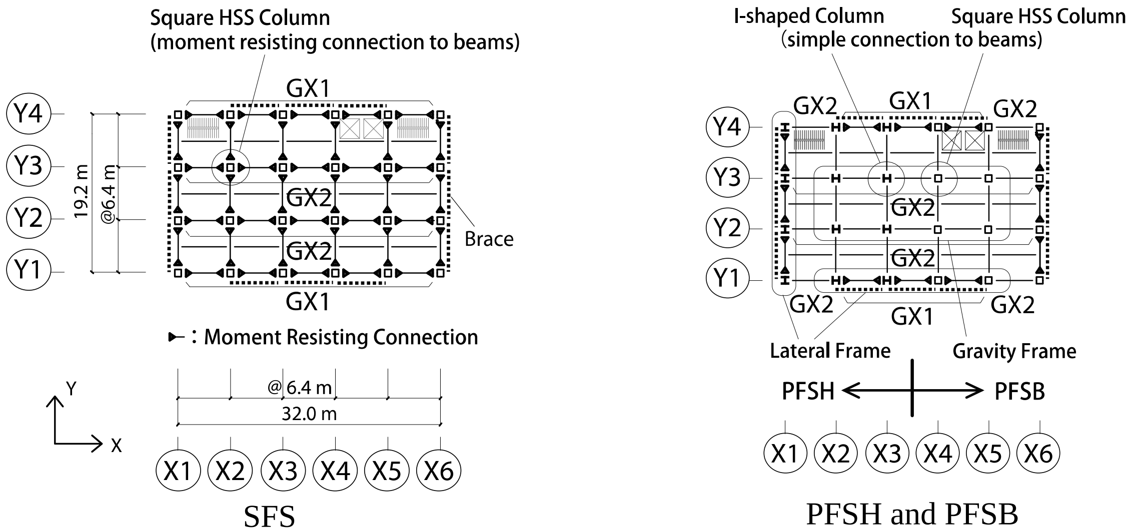


Fig. 1 – Floor framing plan

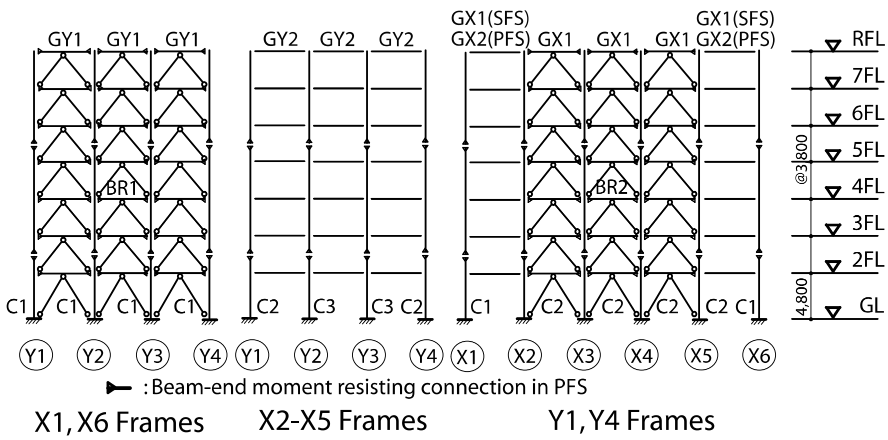


Fig. 2 – Framing elevations

Table 1 – Member grouping in stories

Part	Columns	Beams	Braces
3	Mid. 5th Flr. - 7th Flr.	6th Flr. - Roof	5th Flr. - 7th Flr.
2	Mid. 2nd Flr. - Mid. 5th Flr.	3rd Flr. - 5th Flr.	2nd Flr. - 4th Flr.
1	1st Flr. - Mid. 2nd Flr.	2nd Flr.	1st Flr.

2.2 Structural Design Approach

The two-step structural design approach is assumed for the structural design of the building, which is the allowable stress design for the gravity and seismic design loads (1st step design) and ultimate lateral strength design for earthquakes (2nd step design) [5]. Elastic analyses and inelastic pushover analyses are performed for the 1st and 2nd step designs, respectively. The vertical distribution factor A_i [6] is adopted. The vibration characteristic factor, R_t, and seismic zone factor Z [6] are both assumed to be 1.0. The seismic base-shear coefficient C_0 is 2.0 for the allowable earthquake design. Two types of the required ultimate lateral strength are defined for the 2nd step earthquake design, which are based on code-required strength taking into account the ductility of members (member ranks), and defined in reference to responses in time-history analyses for L2 earthquakes. For the code-required strength, shape factor F_{es} [6] is assumed to be 1.0, and the required base-shear coefficient C_{QUN1} is 0.3 and 0.35 for the moment frame and braced frame structures, respectively. These values are defined with the structural characteristic factor D_s with the ranks of the composing members A or B. On the other hand, C_{QUN1} is defined as 0.6 for moment frames and 1.0 for braced frames, referring to the responses in preliminary time-history analyses under the same conditions described below.

3. Superior Design Solutions by MSLS

3.1 Algorithm to Obtain the Superior Solutions

Superior design solutions of SFS, PFSB and PFSH are obtained by the multiple-start-local-search (MSLS) method [7]. The design variables are discrete section sizes under the conditions of the 1st and 2nd step design constraints. The objective function to be minimized is the steel volume. Feasible solutions, which satisfy all constraints, are first obtained from approximately 10^7 random combinations of design variables. In the next step, the ten best feasible solutions are assigned as initial solutions for MSLS. The superior solutions are defined as the best local optimal solutions obtained from the ten different initial solutions. The numbers of variables and constraints are approximately 40 and 100, respectively. The variables are the section sizes, and there are many complex constraints. Because the ratio of combinations satisfying all constraints with respect to random combinations is very low at 10^{-5} to 10^{-4}, probabilistic approaches such as Generic Algorithm are not effective to resolve this problem.

The step-by-step algorithm is assumed in MSLS. In the process starting from the initial solutions to the local optimal solutions, neighborhood solutions are examined around the tentative solutions at each step. The number of neighborhood solutions examined is set as the same as the number of variables. In the case that the objective function is improved and all constraints are satisfied in a neighborhood solution, then the tentative solution is replaced. When no better solution is found in the neighborhood, then the tentative solution is carried over to the next step. The discrete variables of section sizes are randomly increased or decreased by one or stay the same within the range of each variable. The number of steps is set as 3000. Therefore, the total number of neighborhood solutions is approximately $3000 \times 40 = 120000$. The constraints such as the width-to-thickness ratio are checked first without analyses and approximately 1/5 of the neighborhood solutions are analyzed.

The superior solutions are not globally optimal solutions; however, they are rationally obtained in the proposed design algorithm independent of engineers' experience or preference. This research aims to identify structural characteristics of steel frames with different building structures and structural systems by comparing the superior solutions. Therefore, obtaining strictly global optimal solutions is not the primary interest of this research. The superior solutions shown below can be improved, if the number of steps is increased; however, the decrease of steel volume in the superior solutions with 30000 steps in MSLS is less than 1%. Development of more effective algorithm may be a possible future direction of this research.

3.2 3D Frame Model

Three-dimensional (3D) frame models are created for the elastic analyses for the 1st step design and the inelastic pushover analyses for the 2nd step design. The modeling assumptions are essentially the same as the previous work [4]. The ultimate lateral strength calculated in inelastic pushover analyses is defined for the moment frames as the story shear force at which the maximum inter-story drift ratio first reaches 1.25%. Also, it is defined for the braced frames as the story shear force when the compressive axial forces first reach the buckling strength, which is assumed as 1.1 times the product of allowable temporary compressive stress and the cross-sectional area.

The buckling strength of beams is defined as the average of the tensile yield strength and buckling strength around the weak axis, by assuming that the upper flange of the beam is constrained by the slab and the lower flange is not. The axial forces of the beams are calculated as half of the sum of shear forces of K-shaped concentrated braces, which are supposed to be transferred by the axial force of the connecting beams. The buckling lengths are assumed as the member lengths between the connecting nodes.

3.3 Variables and Constraints

The discrete variables of section sizes are similarly defined as those in the authors' previous work [4]. Possible ranges of the variables are shown in Table 2. These variables of section sizes are defined from the list of standard rolled sections and built-up sections with steel plates with standard thickness. The discrete width and height of the sections in the columns and beams are defined every 50 mm. The thickness of standard pipe sections is varied with the diameters and may not be the same as the standard plate thickness; however, for the sake of simplicity, the set of thickness of the pipes is assumed to be the same as the set of standard plate thickness. The combinations of the flange width and thickness are defined as shown in Table 2, where the cross-sectional area of flange A_f is considered as an independent variable. The constraints of MSLS are essentially the same as those in the previous work [4]. GX2 and GY2 beams are supported with pins at their ends as shown in Figs. 1 and 2, and have the section H-400x200x8x13, which is the minimum I-shaped roll section carrying the gravity load and this section size is excluded from the variables.

Table 2 – Discrete MSLS variables

Symbols	Members	Parts	Discrete variable options
D_c	Rectangular HSS columns	Width	Every 50mm in 250-800mm
t_c		Thickness	*1 (excluding 9mm)
H_{wc}	I-shaped columns	Height	Every 50mm in 300-900mm
W_{fc}		Flange width	Every 50mm in 300-700mm
t_{wc}		Web thickness	*1
t_{fc}		Flange thickness	*1 (excluding 9mm and 12mm)
H_w	Beams	Height	Every 50mm in 300-1000mm
W_f		Flange width	Every 50mm in 200-400mm, *2
t_w		Web thickness	*1
t_f		Flange thickness	*1, *2
D_p	Braces	Diameter	318.5, 355.6, 406.4, 457.2mm
t_p		Thickness	*1 (including 6mm)

*1: Plate thickness options are 9, 12, 16, 19, 22, 25, 28, 32, 36 and 40mm.

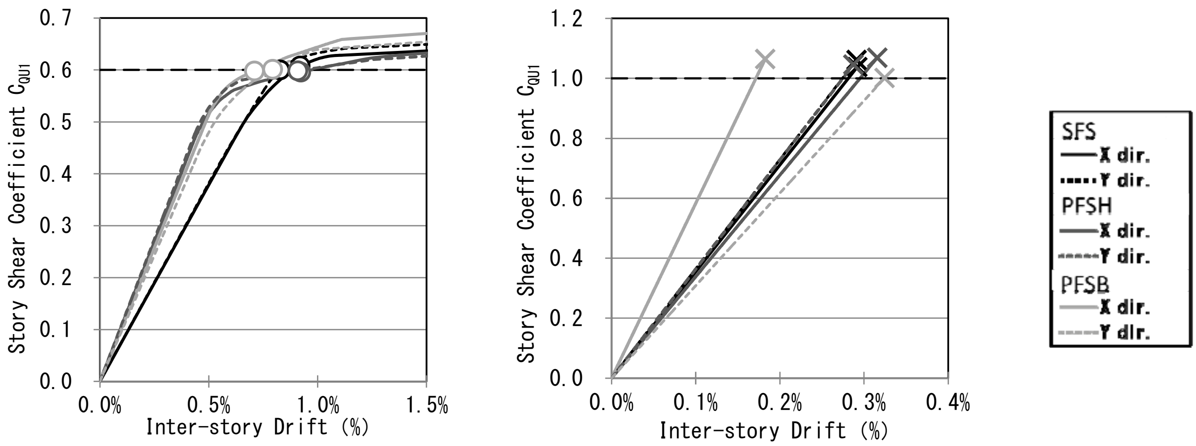
*2: Combinational options of the flange width and thickness in beams are shown below.

	1	2	3	4	5	6	7	8	9	10	11	12
W_f (mm^2)	150	150	200	200	250	250	250	300	300	300	350	350
t_f (mm^2)	12	16	16	19	19	22	25	25	28	32	32	36
A_f (10^3 mm^2)	1.8	2.4	3.2	3.8	4.8	5.5	6.3	7.5	8.4	9.6	11.1	12.6

4. Evaluation of Superior Solutions

4.1 MSLS Analysis Results

Figure 3 shows the relationships between the base-shear coefficient and inter-story drift ratio in the 1st story. These relationships are obtained by the pushover analyses using Midas [8]. The blank circle and cross marks in the plot indicate the ultimate lateral strength. Most of the base-shear coefficient of the ultimate lateral strength, C_{QU1}, is greater than C_{QUN1} = 0.6 for the moment frames and C_{QUN1} = 1.0 for the braced frames. In order to save the computational cost, no iterative calculation is performed at each incremental step in the pushover algorithm in MSLS, which is different from the algorithm in the commercial software Midas. Consequently, C_{QU1} is slightly smaller than C_{QUN1} = 0.6 for the moment frames; however, the shortage is less than 2%. As Fig. 3 shows, the values of C_{QU1} and C_{QUN1} are close. Therefore, the constraints for the ultimate lateral strength have much effect on the superior solutions. C_{QUN1} = 0.6 for the moment frame and C_{QUN1} = 1.0 for the braced frame are 2.0 and 2.9 times as much as C_{QUN1} = 0.3 for the moment frame and C_{QUN1} = 0.35 for the braced frame of the code-required values. Therefore, the temporary X- and Y-directional loads are not dominant for the seismic design. The maximum inter-story drift ratios are 0.29-0.36% (PFSB-SFS) for the moment frame structures and 0.08-0.09% (PFSH-SFS) for the braced frame structures. These values are smaller than the 1st step design requirement of 0.5%. Also, the maximum stress ratios are relatively small, and are 0.60-0.65 (PFSB-SFS) for the moment frame structures and 0.32-0.34 (PFSB- PFSH) for the braced frame structures.



Moment frame structures

Braced frame structures

Fig. 3 – Relationships between base shear coefficient and inter-story drift ratio of moment and braced frame structures

4.1 Steel Volume

Moment Frame Structures

The superior solutions of moment frame structures with C_{QUN1} = 0.3 are primarily dependent on the constraints of the allowable stress design including limit of inter-story drift ratio (0.5%), while those with C_{QUN1} = 0.6 are primarily dependent on the constraints of the ultimate lateral strength. The steel volumes are smaller in the order of PFSH, SFS and PFSB, both with C_{QUN1} = 0.3 and 0.6. The steel volumes with C_{QUN1} = 0.6 are 22-29% greater than that with C_{QUN1} = 0.3. The section of secondary beams is assigned as H-400x200x8x13 and additional 30% steel is included for miscellaneous parts such as stiffeners and brackets for exteriors. The total steel weight per unit area (kg/m^2) is 113.0 (SFS), 106.2 (PFSH) and 122.6 (PFSB) with C_{QUN1} = 0.3, and 135.4 (SFS), 133.8 (PFSH) and 153.2 (PFSB) with C_{QUN1} = 0.6. The steel volumes of PFSH are smaller, because the column strength and stiffness can be adjusted with fewer restrictions by changing the I-shaped column section sizes. Also, it is effective for the structure with limited lateral frames to carry the seismic loads. In SFS, the height of beams in each floor is assumed to be uniform; however, this constraint does not have much effect on increase of the steel volume, mainly because column spacing is uniform in both the X- and Y-directions. In this research, yielding of panels in I-shaped columns is not incorporated and additional steel plates reinforcing the panels may be required; however, their steel volume would be small compared with the total volume. Considering that the steel volume of the horizontal stiffeners beam-to-column connections in the rectangular HSS column is not taken into account, detailed evaluation for the volume of reinforcing plates for the panels is not necessary. Since the ultimate lateral strengths of the superior solutions with C_{QUN1} = 0.6 are almost the same among SFS, PFSH and PFSB, and the maximum inter-story drift ratios and ductility factors are not significantly different. Hence PFSH can be regarded to be advantageous with less steel volume.

Braced Frame Structures

Contrary to the moment frame structures, the primary forces induced by the seismic load in braced frame structures are axial forces. The difference in the steel volume among the structural systems, SFS, PFSH and PFSB, is relatively small. I-shaped columns are disadvantageous against buckling, and the steel volume in PFSH is slightly larger as shown in Fig. 4.

Braces carry most of the seismic lateral load in the braced frame structures; therefore, the sizes of members in the gravity frames (C3, GX2 and GY2 in Figs. 1 and 2) are not dependent on the seismic loads. When the beams are designed for the gravity load, the mid-span flexural moments and deflections are smaller due to moment connections to the columns; however, there is less composite effect with the floor slabs. Beam cambers are commonly used in the steel buildings in the US and other countries but not in Japan. The composite effect and cambers are not considered in this research. The superior design algorithm against the gravity load can be improved in future research.

The column spacing 6.4 m is shorter than that in standard office buildings, and the beam sections are small. Consequently, the strong-column-weak-beam constraint does not have much effect on increase of the column sections. This may partly account for less significant difference between SFS and PFS. Increasing the column spacing from 6.4 m to 9.6 m, the steel volume in SFS becomes slightly less than the others. Compared to PFSB, the advantage of reducing steel material in beams by moment connections to columns outweighs the disadvantage of increase of column sections under the strong-column-weak beam constraint condition.

The steel volumes of the superior solutions with C_{QUN1} = 1.0 are 70-90% larger than those with C_{QUN1} = 0.35. The steel weight per unit area (kg/m^2) calculated in the similar manner as the moment frame structures is 68.3 (SFS), 76.2 (PFSH) and 73.3 (PFSB) with C_{QUN1} = 0.35, and is 116.8 (SFS), 120.2 (PFSH) and 116.8 (PFSB) with C_{QUN1} = 1.0.

Composite Effect

Composite effect of steel beams is not taken into accout in the superior solutions. The effect is more beneficial for PFS than SFS, because there are simply supported beams in the gravity frames in PFS, where slabs are under compression in the whole length of beams. Taking into account the composite effect, the steel volume in PFSH and PFSB shown in Fig. 4 is reduced by 2.4 m^3 in both moment and braced frame structures. The ratios of this reduction are 8-11% and 14-19% of steel volume of beams in moment and braced frame strucutres, respecitively. Also, the ratios are 4-6% and 5-8% of total steel volume in moment and braced frame strucutres, respecitively. The steel volumes of the superior solutions for PFSB of braced frame structures are reduced to 26.3 m^3 and 48.0 m^3 from 28.7 m^3 and 50.4 m^3 with C_{QUN1} = 0.35 and 1.0, respectively, and are the smallest among SFS, PFSH and PFSB. This study implies potential advantage of use of HSS columns for PFSB of braced frame structures.

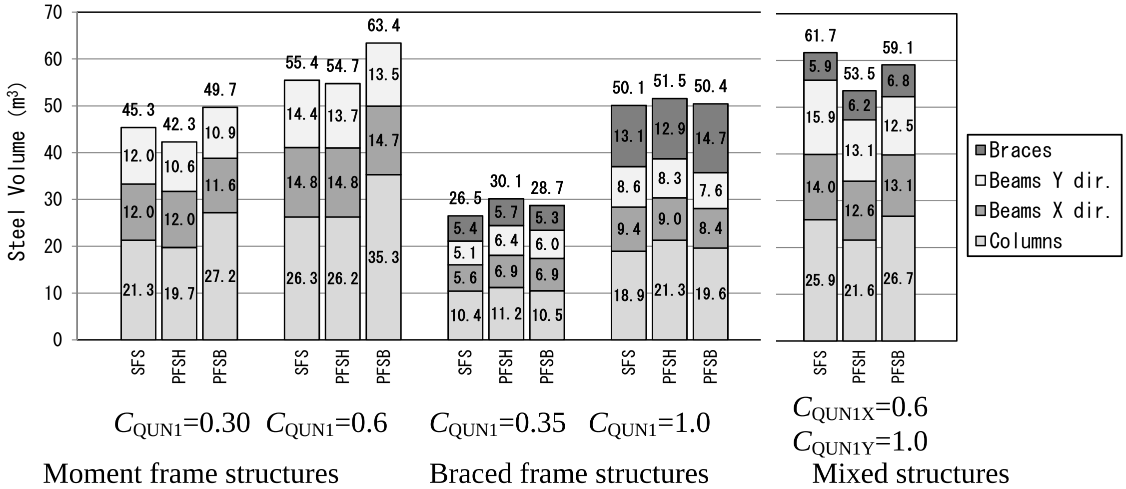


Fig. 4 – Comparison of steel volume

5. Mixed Structures

The structures evaluated in the previous sections have the same building structures (i.e., moment frame or braced frame) in the X- and Y-direction in the diagram in Fig. 1. In this section, buildings with different structures in the X- and Y-directions with non-uniform column spacing are examined.

5.1 Compositions and Grouping

As shown in Fig. 5, there is no column on the X2-X5 axes at the Y2 axis, and the column spacing is 12.8 m between the Y1 and Y3 axes. These long-span beams are called GY2, and beams between Y3 and Y4 are GY3. These beams are pin supported at the ends; therefore, these are secondary beams in PFSH and PFSB, and have the minimum rolled sections H-750x250x12x25 and H-400x200x8x13, respectively, for carrying the gravity load. In SFS, GX1s in the Y4 axis are renamed as GX2, and C2 at the X2-X5 axes are renamed as C3. The torsional deformation can be controlled by making the lateral frame in the Y1 axis stiffer than the frames in the Y3 and Y4 axes.

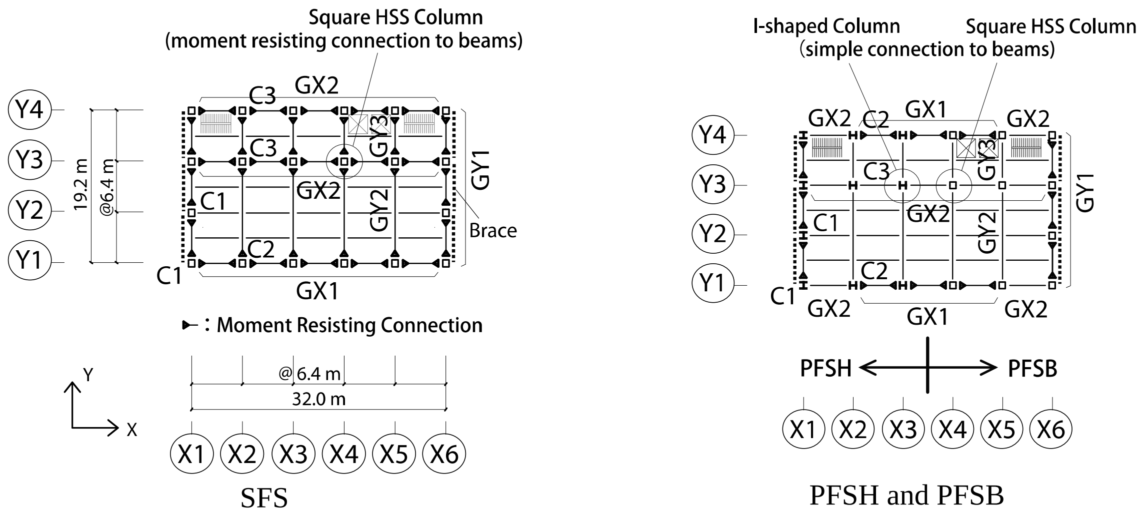


Fig. 5 – Floor framing plan of mixed structure

5.2 Superior Solutions for Mixed Structures

C_{QUN1} for the X-direction, which is the moment frame structure, is renamed and assigned as C_{QUN1X} = 0.6. Also, C_{QUN1} for the Y-direction, which is the braced frame structure, is renamed and assigned as C_{QUN1Y} = 1.0. The superior solutions of the mixed structure in SFS, PFSH and PFSB are obtained. The ultimate lateral strengths mostly satisfy the required strengths of C_{QUN1X} and C_{QUN1Y}. In the time-history analyses, the maximum inter-story drift ratio R_{MAX} is about 1.5 % in the X-direction.

The steel volume of the mixed structure is also shown in Fig. 4. The volumn is smaller in the order of PFSH (87%), PFSB (96%) and SFS (100%), where the ratios with respect to the volume of SFS are shown in parentheses. Apart from the moment frame structure, the steel volume of SFS is slightly larger than that of PFSB, possibly because there are some constraints that increase the steel volume for irregular configuration. In particular, the uniform beam height constraint increases beam sections including the 12.8 m long-span Y2 beam, and the strong-column-weak-beam constraint increases column sections.

6. Conclusions

In this research, superior solutions for a seven-story office building with different lateral frame locations and column shapes were obtained using the multiple-start-local-search (MSLS) method, and their structural characteristics were investigated. The objective function of the method is to minimize the total steel volume considering the grouped section sizes as discrete variables. The superior solutions satisfy the requirements in the 1st step allowable stress design and 2nd step ultimate lateral strength design. An algorithm to obtain the solutions with many constraints and discrete variables was demonstrated.

Superior solutions were obtained for three types of structural systems with different lateral frame locations and column shapes, which are (1) the spaced frame system (SFS) with rectangular HSS columns and with lateral frames in all frames, (2) the perimeter frame system (PFS) with I-shaped columns and with lateral frames in perimeter frames (PFSH), and (3) PFS with rectangular HSS columns (PFSB). Superior solutions were obtained for three types of building structures: moment frame, braced frame and mixed. The following findings were obtained:

(1) The superior solutions for the moment frame structures were obtained for the required base-shear coefficient of ultimate lateral strength, C_{QUN1} = 0.3 and 0.6. C_{QUN1} = 0.6 was defined referring to the responses of time-history analyses against L2 earthquake ground motions; however, all superior solutions of the three structural systems do not satisfy practically common structural design criteria of 1.0% inter-story drift with 1.4-1.5% of the maximum values. The ductility factors are lower than 3.0. The steel volume of the solutions with C_{QUN1} = 0.6 is approximately 1.3 times greater than that with C_{QUN1} = 0.3. The steel volume is smaller in the order of PFSH, SFS and PFSB. Since the seismic performance in these solutions is nearly equivalent in the ultimate lateral strength and maximum ductility factors against L2 earthquakes, PFSH would be more rational in moment frame structures than SFS, which is a popular system in Japan.

(2) The superior solutions for the braced frame structures were obtained for C_{QUN1} = 0.35 and 1.0. The differences in the steel volume among the structural systems, SFS, PFSH and PFSB, are relatively smaller than those with moment frame structures. The steel volume of superior solutions with C_{QUN1} = 1.0 is approximately 1.8 times larger than that with C_{QUN1} = 0.35. The steel volume in PFSH is slightly greater than that of the others. Axial forces are the primary additional structural member forces under the seismic loads, and rectangular HSS columns are advantageous.

(3) The first natural periods of the superior solutions are 0.92-1.05 sec. for the moment frame structures and 0.46-0.51 sec. for the braced frame structures. C_{QUN1} = 0.6 for the moment frame structures and C_{QUN1} = 1.0 for the braced frame structures are defined referring to the response in time-history analyses against L2 earthquakes, and these values are significantly higher than the code required values. Although the seven-story buildings examined can be designed by the ultimate lateral strength calculations or time-history analyses, the lateral strength required in these two design approaches is

significantly different.

(4) The steel volume in SFS is larger and the volume in PFSH is smaller in the mixed structures. Apart from the moment frame structures, the steel volume of SFS is larger, because contraints on the uniform beam height and strong-column-weak-beam could increase it under the irregular configuration.

Ths research examined a seven-story office building with moment frame, braced frame and mixed structures. The findings from this limited number of case studies are not sufficient to understand the general structural characteristics of these structures. However, superior solutions were obtained by using the design algorithm independent of engineers' personal experience and skills. Therefore, the discussion and findings comparing these different structural systems do have merit.

7. Acknowledgements

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