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STRUCTURAL CHRACTERISTICS OF EXISTING SMALL SIZE MIDDLE RISE STEEL BUILDING STRUCTURES IN JAPAN

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SUMMARY

In Japan most popular structure for middle rise buildings is steel structure. In this paper structural investigation results about 92 existing small size middle rise steel building structures are described. Those are structural figures, main structural members, static and dynamic characteristics of the structures. Structural figures are size of each floor, framing arrangement, number of stories and so on. Size and thickness of steel member of main columns and beams are presented. As static characteristics of structures, shear coefficient of each story, natural period, ultimate lateral strength of each story, distribution of rigidity of each story, strength ratio of column to beam at each beam-to-column connection, are investigated. In order to get dynamic characteristics, elastio-plastic dynamic response analysis using 5 kinds of severe ground motions were executed about each structure. From those analysis response values about inter-story drift angle, factors of accumulated plastic deformation of each structural member are obtained. Through this investigation actual structural characteristics are clarified and it becomes clear that most of existing small size middle rise steel building structures in Japan which were built after the revision of building code in 1981 have sufficient earthquake resistant ability against sever earthquakes.

INTRODUCTION

In this paper, investigation results are summarized from following view points about the existing small size middle rise steel building structures.

·How are low or middle rise steel building structures designed in Japan?

·Do these structures have efficient earthquake resistant abilities?

Furthermore, the structural characteristics of those building structures in a severe earthquake ground motion are estimated by dynamic analysis.

The investigation was executed based on drawings and specifications about 92 existing structures of low or middle rise steel building structures which were built in an urban region in Japan (Utsunomiya and its vicinity) at 1993 and 1994 (MASUDA [1], MAEDA [2]). A follow-up same type investigation was carried out in 1998 (SUGAWARA [3]). The result of this follow-up investigation shows almost same tendency as these of 1993 and 1994 investigation. Therefore, the results of former investigation are reported in this paper, because number of investigated buildings are lager.

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Most of investigated items of structural characteristics shown in this paper are prescribed in design regulations of Japanese Building Standard Law (they are called building code). Basically those design regulations consist of elastic design (allowable stress design) and plastic design. In the elastic design each structural members are required to remain under elastic limit. In the plastic design ultimate lateral strength of the structure must be checked. But actually in usual middle rise building structures plastic design is exempted, in case three structural stipulations are checked. Those three stipulations prescribe prevention of local buckling of steel members, prevention of lateral buckling of beams and prevention of early stage fracture of main structural connections. Therefore the ultimate lateral strength and dynamic characteristics of investigated structures are checked in this paper.

2. RESULTS OF INVESTIGATION ABOUT EXISTING STEEL BUILDING STRUCTURES

2.1 Outline of buildings

Numbers of stories of investigated buildings are shown in Fig.1. About 87% of buildings, the numbers of stories are equal to or less than 6. Floor areas of investigated buildings are shown in Fig.2. The expression of horizontal axis means as follows. For example, 100-200 means a floor area is larger than $100m^2$ less than or equal to $200m^2$. About 88% of buildings, the floor areas of standard floor are equal to or less than $300m^2$. Fig.3 shows framing arrangement. In this figure, for example, 1×2 of horizontal axis means that short span direction of building arranges 1 span and longitudinal direction arranges 2 spans. 67% of buildings are arranged 1×2, 1×3 and 1×4. Average span length of short span direction is nearly 7.7m and that of longitudinal direction is nearly 5.6m. These results mean most steel buildings are small size, and this tendency almost accords with the tendency of steel building structures in Japan (Minister of Land, Infrastructure and Transport [4]).

Fig.4 shows usage of those buildings. 40% of investigated buildings are used as office or shop, or these combinations, and 42% are used as of dwelling or combination use of office or shop and dwelling.

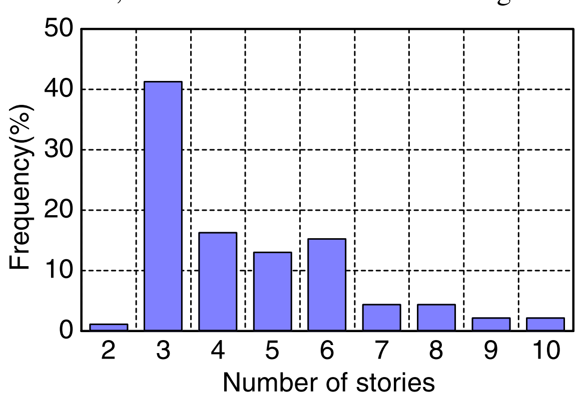

Fig.1 Frequency of Number of Stories

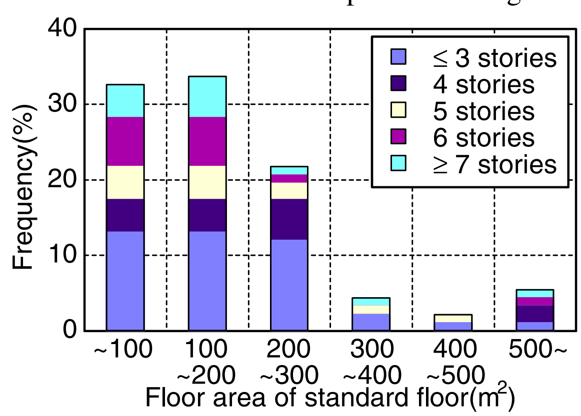

Fig.2 Frequency of Floor Area

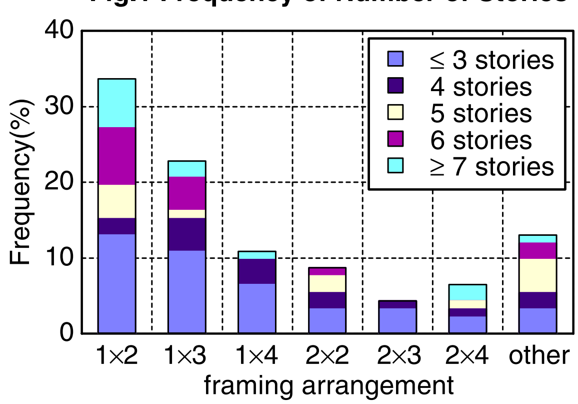

Fig.3 Frequency of Framing Arrangement

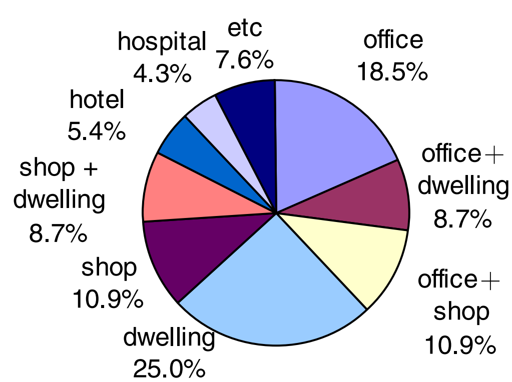

Fig. 4 Frequency of Usage of Building

2.2 Structural members of buildings

Fig.5 shows the situation of structural members used for structures of investigated buildings.
Columns (total numbers are 3488) are all cold-formed rectangular hollow section of STKR400 (mild steel level based on Japanese Industrial Standards) except that there are 16 circular steel tubes in one building, and 250mm ~ 450mm square members are mainly used. All of beams (4407 total numbers) are rolled wide flange steel of SS400. About width of flanges, 50% are 200mm, 26% are 300mm, and most of others are equal to or less than 175mm. The thickness of flange larger than 20mm is hardly used.

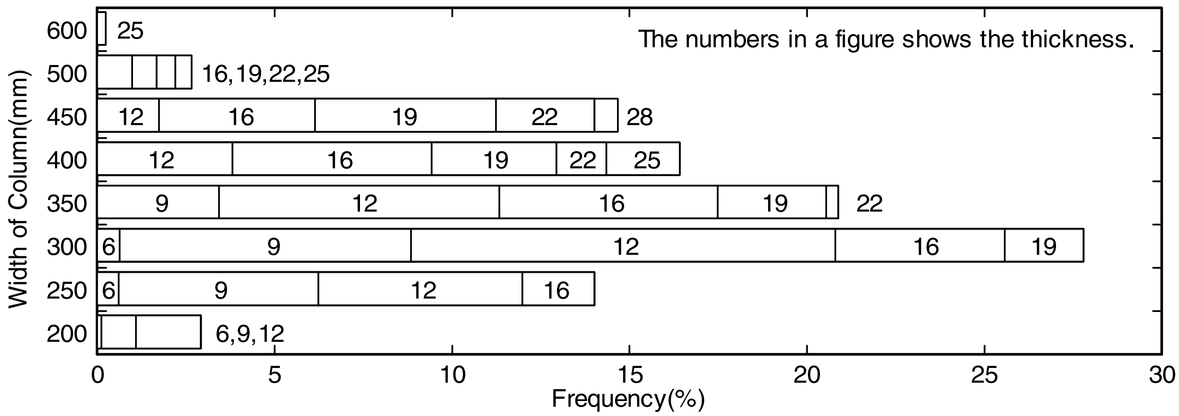

Fig.5-a Size of Columns

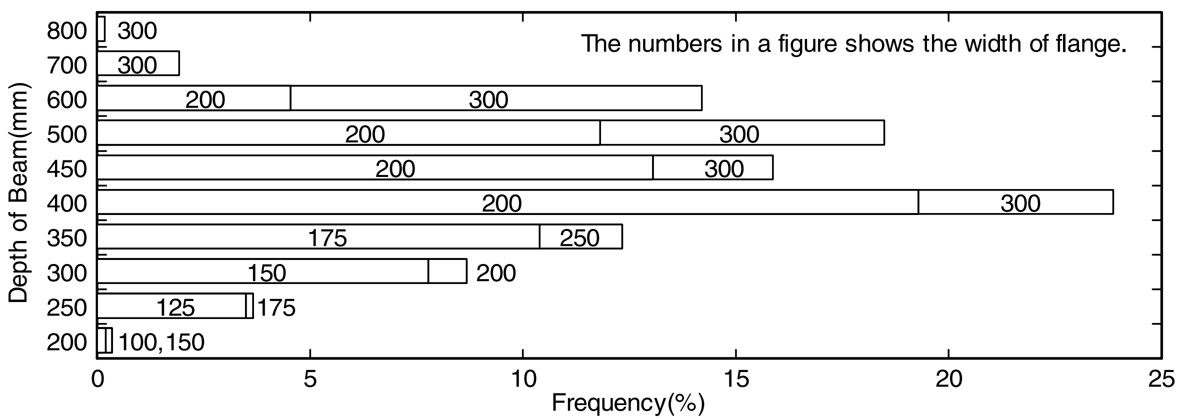

Fig.5-b Size of Beams

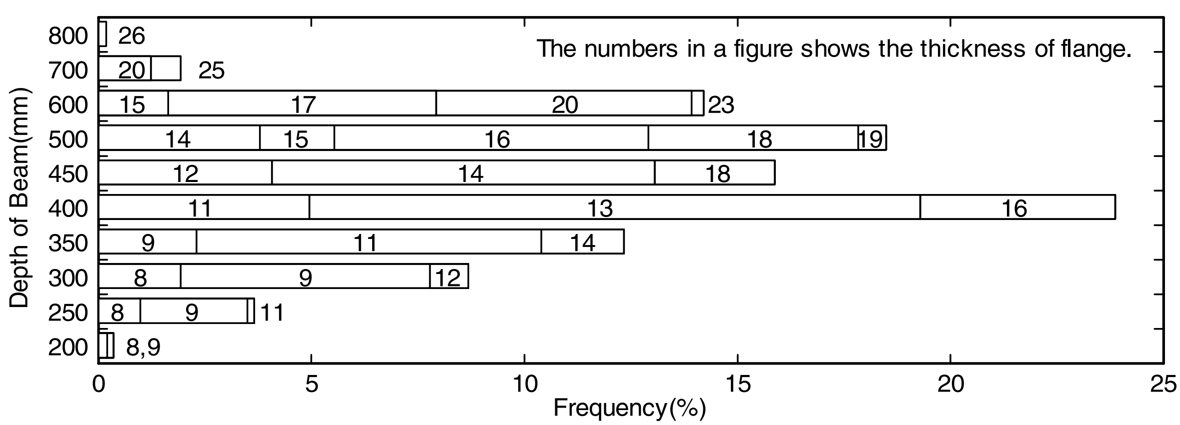

Fig.5-c Size of Beams

Fig.6 shows the using situation of width-to-thickness ratio of members. Expression in horizontal axis, for example, 21-23 means width-to-thickness ratio is larger than 21 less than or equal to 23. A limit of width-to-thickness ratio is stipulated to avoid local buckling before undergoing enough plastic behavior. About steel materials of 400N/mm^2 grade, a recommended value of width-to-thickness ratio is equal to or less than 33 in rectangular hollow section column and is equal to or less than 9 in flange of wide flange steel beam. Columns with width-to-thickness ratio lager than 33 are 19% of all. On the other hand, it is considered that a required value of width-to-thickness ratio from view point of desirous bending stress transfer at beam-to-column connection is equal to or less than 23 (AIJ [5]). Columns which do not satisfy this requirement are 39% of all. About 84% of beams, the width-to-thickness ratio of beam flanges are equal to or less than 9. Therefore, it is realized that most of members constituting structure of investigated buildings have enough plastic deformation ability.

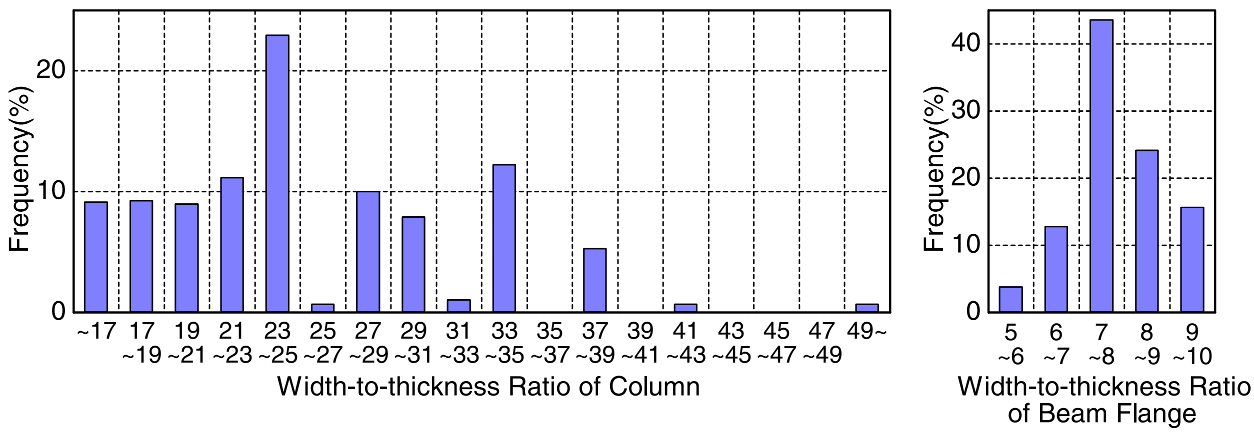

Fig.6 Frequency of Width-to-thickness Ratio of Members

2.3 Column-to-footing connections

The using situation of the type of column-to-footing connections is the only one part where a significant difference is shown between investigation of 1993 and 1994 and follow-up investigation of 1998.

The types of column-to-footing connections used in investigated buildings are shown in Fig.7 at each investigated year. In 1993's and 1994's investigation, embedded type column-to-footing connections were used in 56% buildings, and exposed type column-to-footing connections were used in 37%. In 1998's investigation, used frequencies of exposed type column-to-footing connections increase in comparison with 1993's and 1994's investigation. Most of these exposed type column-to-footing connections were constructed by special construction methods approved by Minister of Land, Infrastructure and Transport (MOLIT). According to this investigation result, column-to-footing connections of most structures are considered to be rigid connection.

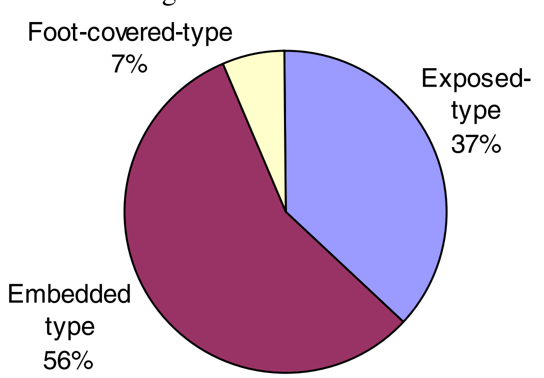

Fig.7-a Frequency of Usage about Types of Column-to-Footing Connections (1993,4)

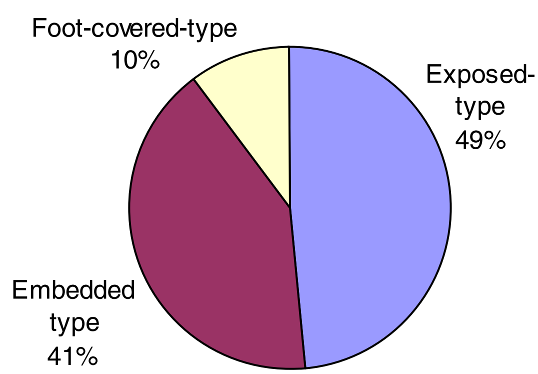

Fig.7-b Frequency of Usage about Types of Column-to-Footing Connections (1998)

3. STATIC CHARACTERISTICS OF SRTUCTURES

In the Japanese building code, seismic design load is decided using standard shear coefficient C_0. Standard shear coefficient C_0 is stipulated as $C_0 \geq 0.2$ (corresponding to nearly maximum speed 10kine equivalency of input earthquake ground motion) in the elastic design, and $C_0 \geq 1.0$ (maximum speed 50kine equivalency) in the plastic design. In this chapter, static structural characteristics are investigated about items demanded in the elastic design and the plastic design.

3.1 Natural period

The natural periods of investigated structures by eigenvalue analysis (exact period) are shown in Fig.8. Solid line in this figure is an approximate expression of natural period prescribed in the building code as shown Eq.(1).

$$T=0.03h \tag{1}$$

Here, T is approximate natural period(sec) and h is building height(m).

The exact natural periods of investigated buildings are distributed 0.5-1.5 second, and all of the exact natural periods exceed the approximate periods.

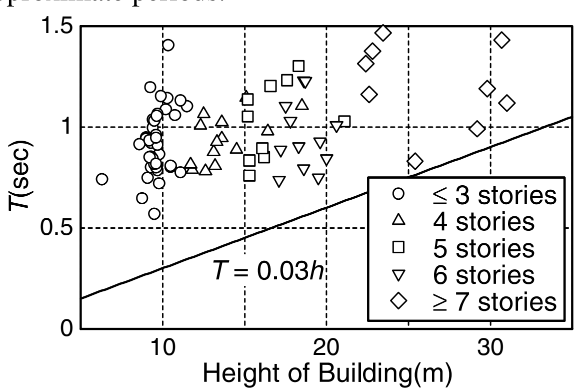

Fig.8 Natural Period

3.2 Story shear coefficient at elastic limit

It is required that story shear at elastic limit are smaller than seismic design load decided standard shear coefficient C_0 of elastic design. In this paper, story shear at elastic limit of each story of investigated structures are converted into a value having an explanation to be equal with standard shear coefficient (base shears coefficient conversion value ${}_e\alpha_i$), and earthquake resisting ability of building is examined with these values. Here, from the regulation of $C_0 \geq 0.2$ in the elastic design, ${}_e\alpha_i$ of investigated structures should be also equal to or larger than 0.2. Base shear coefficient conversion value ${}_e\alpha_i$ is defined in the following expressions.

$${}_e\alpha_i = \frac{Q_{yi}}{\sum W_i \cdot A_i} \tag{2}$$

$\sum W_i$: Total weight carried by i th story

Q_{yi} : Story shear at elastic limit of i th story

A_i : Distribution coefficient of story shear due to seismic load of i th story prescribed by the building code

Here, the story shear at elastic limit is calculated by node moment distribution method using nominal yield strength of materials. In addition, building weight is supposed to be 9.8kN/m^2 on standard floor and 10.4kN/m^2 on the top floor. These weights are thought to be about 10-20% over of the actual condition, but these values give safety side evaluation.

Distribution of the frequencies of ${}_e\alpha_i$ of each story of investigated structures is shown in Fig.9. The horizontal axis in this figure is divided by 0.1, and, for example, the expression between 0.6 and 0.7 means $0.6 \le {}_e\alpha_i < 0.7$. The structures with ${}_e\alpha_i$ smaller than 0.2 account for 16%. In addition, the structures whose ${}_e\alpha_i$ is smaller than 0.15 account for only 4%. Judging from about 20% overestimate of building weight, and actual yield strength of steel materials is about 1.1 times of the nominal value, it is supposed that ${}_e\alpha_i$ of most structure is more than 0.2 actually. In addition, ${}_e\alpha_i$ values of 41% of stories of investigated structures are between 0.2 and 0.3, and significant difference is not recognized by the number of stories. Judging from above facts, it is thought that story shear at elastic limit of most buildings satisfied the stipulation of the elastic design condition.

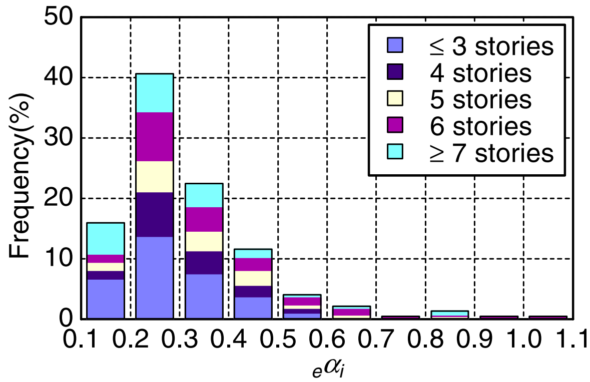

Fig.9 Frequency of ${}_e\alpha_i$

3.3 Inter-story drift angle

The stipulations of the elastic design include confirmation of inter-story drift angle for design seismic load being equal to or less than 1/200 (1/120 in some conditions of finishing materials). Distribution of the frequencies of ${}_{0.2}R_i$ is shown in Fig.10. Here, ${}_{0.2}R_i$ is the inter-story drift angle of i th story calculated using seismic load corresponding to standard shear coefficient C_0=0.2. The horizontal axis in this figure is a reciprocal number of inter-story drift angle ${}_{0.2}R_i$, and, for example, the expression between 200 and 280 means $1/280 < {}_{0.2}R_i \le 1/200$. About 49% of all stories ${}_{0.2}R_i$ are lager than 1/200, and about 10% are lager than 1/120. Judging from about 20% overestimate of building weight, actual ${}_{0.2}R_i$ are smaller than above mentioned results.

Distribution of the frequencies of weakest story in each building is shown in Fig.11. Here, the weakest story is defined as the story whose inter-story drift angle becomes greatest. It is clear that the weakest story of most buildings is appeared at middle story.

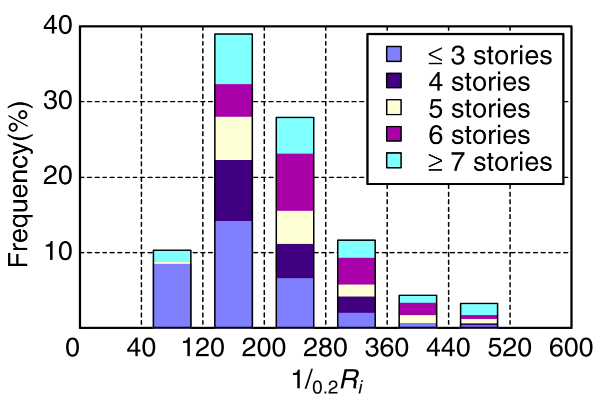

Fig.10 Frequency of Drift Angle of Each Story

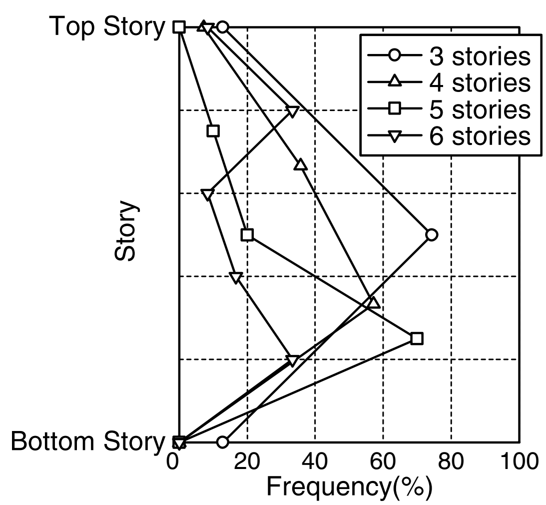

Fig.11 Frequency of Weakest Story

3.4 Story stiffness ratio

To prevent concentration of horizontal displacement and damage to some specified story in an earthquake, it is required to be designed that the distribution of stiffness of the building becomes as equal as possible along its height direction. In the building code, a story stiffness ratio R_s is defined by the follow expression with ${}_{0.2}R_i$, and it is required that each story stiffness ratio R_s is larger than 0.6.

$$R_s = \frac{r_s}{\overline{r_s}} \tag{3}$$

Here,

$$r_s = \frac{1}{{}_{0.2}R_i} \tag{4}$$

$$\overline{r_s} = \frac{\sum^{n} r_s}{n} \tag{5}$$

n : the number of building stories.

Fig.12 shows the distribution of the frequencies of the minimum R_s value of investigated structures. The horizontal axis in this figure divides by 0.1, and, for example, the expression between 0.6 and 0.7 means $0.6 \le R_s < 0.7$. It is clear that R_s of all structures are lager than 0.6. 70% of all structures are lager than 0.9 regardless of the number of stories. Judging from above results, investigated structures are designed to have almost equal story stiffness along its height.

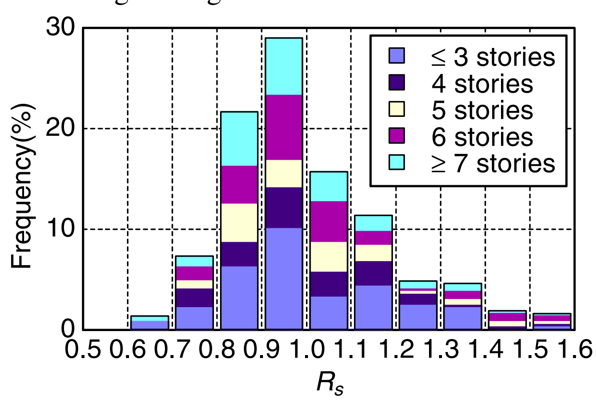

Fig.12 Frequency of R_s

3.5 Horizontal load-carrying capacity

In order to prevent collapse of structures in a severe earthquake, it is stipulated in principle that horizontal load-carrying capacity Q_u of structures (ultimate lateral strength) should be larger than necessary horizontal load-carrying capacity Q_{un}. Here, Q_{un} is the required lateral strength specified by the building code. In other words, if Q_u/Q_{un} is lager than 1.0, the ultimate strength of structure is considered to be enough for a severe earthquake. In this paper, Q_u is calculated by load incremental method and is lateral strength of each story when maximum drift angle reached to 1/50rad of any story. In addition, about calculation of Q_{un}, ductility factor of each story is assumed to be around 4 (D_s=2.5).

Relationship of Q_u/Q_{un} and drift angle ${}_{0.2}R_i$ of each story is shown in Fig.13. A tendency of decrease of Q_u/Q_{un} with increase of ${}_{0.2}R_i$ is recognized generally. As for the investigated structures, most structure with ${}_{0.2}R_i$ <1/200 correspond with Q_u/Q_{un} ≥1.0. Therefore, it is considered that a limit of drift angle is effective to preserve of horizontal load-carrying capacity.

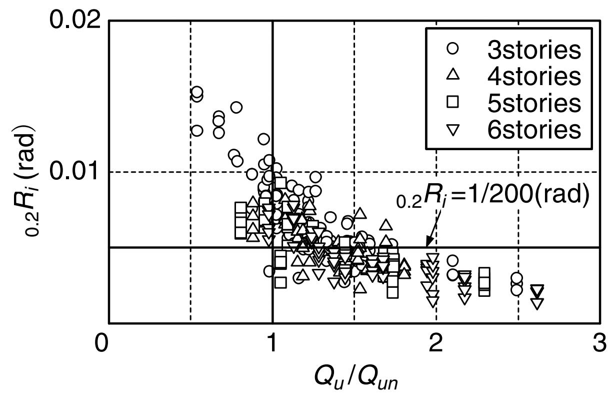

Fig.13 Q_u/Q_{un} - $_{0.2}R_i$ Relationship

3.6 Strength ratio of column to beam

Strength ratio of column to beam is not prescribed in the building code. However, to preserve in earthquake resistant ability of building, it is desirable that collapse mechanism of building becomes total collapse mechanism. To form total collapse mechanism, it is necessary to design structures that column members should be considerably stronger than beam members at each beam-to-column connection. As a guideline to prevent collapse of column, the strength ratio of column to beam R_c at each beam-to-column connection, defined Eq.(6), is desirable lager than 1.5 except top floor (BCJ [6]).

$$R_c = \frac{\sum {}_cM_p}{\sum {}_bM_p} \tag{6}$$

Here, ${}_cM_p$ and ${}_bM_p$ is the full plastic moment of column and beam.

The strength ratio of column to beam are calculated at every beam-to-column connection. Distribution of the frequencies of R_c is shown in Fig. 14, according to the every position of beam-to-column connections. Horizontal axis in this figure divides by 0.2, and, for example, line in between 1.2 and 1.4 means $1.2 \le R_c < 1.4$. From this figure it is clear that R_c of most connections are lager than 1.5 except top floor. Judging from above mentioned conditions, it is estimated that collapse type of most structure are total collapse mechanism.

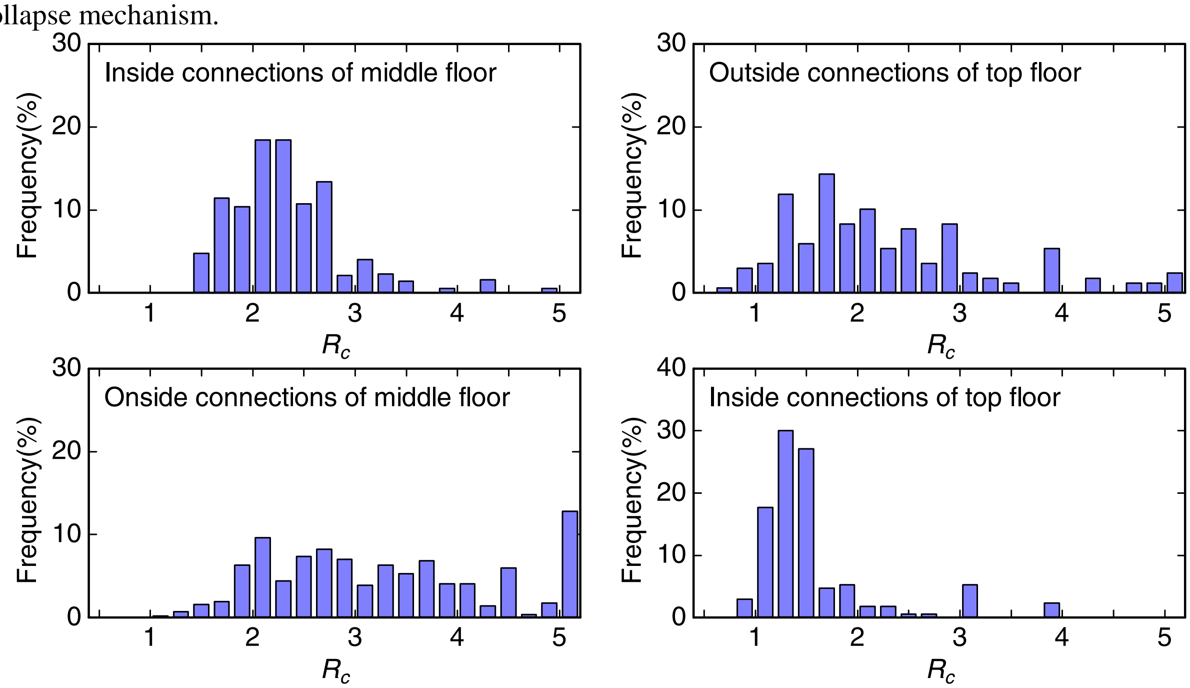

Fig.14 Frequency of R_c

4. DYNAMIC CHARACTERISTICS OF STRUCTURES

Dynamic response analysis was performed to estimate the damage in a severe earthquake (MAEDA [7]). The computer program club.f (OGAWA [8]) is used in this analysis, which deal with the combined non-linear analysis of plane steel frames and which can take account of the elasto-plastic deformations of panel zone of beam-to-column connections.

The analysis conditions are as follows.

· Yield strength of steel materials is nominal value of 235N/mm^2.

· A strain hardening coefficient of column and beam member is 2%, and a strain hardening coefficient of panel zone is 1%.

· Column-to-footing connections are rigid.

· 2% viscous damping factor is set as the stiffness proportional damping.

· Five earthquake ground motion shown in table 1 were selected for this analysis. The maximum accelerations of these earthquake ground motions were set in order that the equivalent velocities of damage energy V_{dm} (AKIYAMA [10]) of each structure are 150kine (corresponding to nearly maximum speed 50kine equivalency of input earthquake ground motion).

Table 1 Input earthquake ground motions

Input Earthquake Ground Motion	Duration
EL CENTRO NS	20sec
HACHINOHE EW	20sec
KOBE NS	20sec
TAFT EW	20sec
wv_elns *	30sec

* Artificial ground motion used for seismic resistant design (Yi Hua Huang, [9]).

4.1 Inter-story drift angle

Relationship of ${}_e\alpha_i$ and ${}_{dy}R_i$ (maximum response of inter-story drift angle of each story) is shown in Fig.15. The relationship of ${}_e\alpha_i$ and the maximum value of ${}_{dy}R_i$ is almost inversely proportional relationship, and this relationship is expressed by Eq.(7), and shown by solid line in Fig.15.

$$ {}_{dy}R_i = \frac{1}{150}\cdot\frac{1}{{}_e\alpha_i} \tag{7}$$

In addition, the ${}_{dy}R_i$ of the story whose ${}_e\alpha_i$ is larger than 0.3 is smaller than 1/50(0.02rad), in this analysis.

Relationship of ductility factor μ_i and ${}_e\alpha_i$ of each story is shown in Fig.16. Ductility factor μ_i is defined by the following expressions.

$$ \mu_i = \frac{{}_{dy}R_i}{R_{yi}} \tag{8}$$

The μ_i of the almost all story whose ${}_e\alpha_i$ is larger than 0.2 is smaller than 4 in this analysis. Because the width-to-thickness ratio of almost all members used for the investigated structures satisfy the stipulated value, μ_i of each story of the structures are expected to be 4-5.

Therefore, it is thought that most of the story whose ${}_e\alpha_i$ is larger than 0.2 have enough seismic resistant ability.

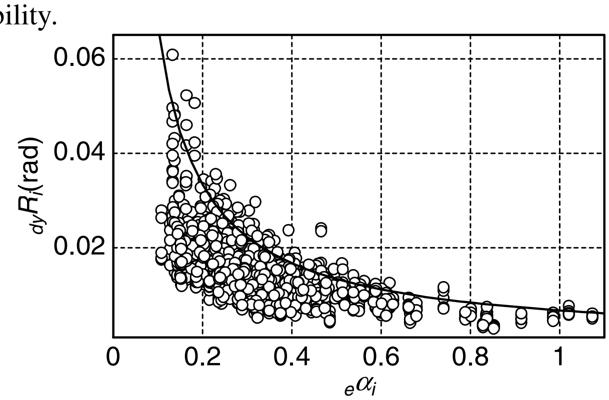

Fig.15 ${}_e\alpha_i$ - ${}_{dy}R_i$ Relationship

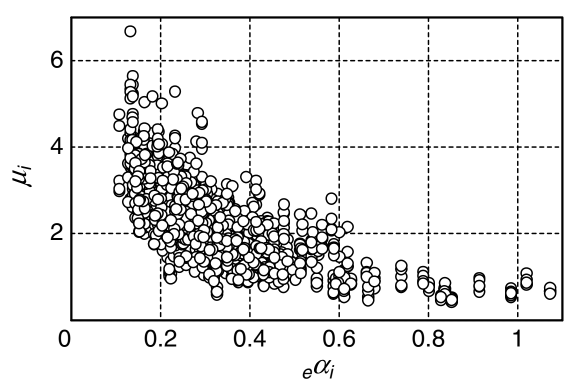

Fig.16 ${}_e\alpha_i$ - μ_i Relationship

4.2 Estimated damage of each member

The situation of damages of panel zone, column and beam of middle story is estimated using accumulated plastic deformation factor η of each member.

The accumulated plastic deformation factor η is defined by the following expression.

$$\eta = \frac{W}{M_p \cdot \theta_p} \tag{9}$$

Here, W is the total cumulative absorption energy of each member, M_p is the full plastic moment of each member (panel) and θ_p is the elastic limit rotation angle.

Each accumulated plastic deformation factor of panel zone, column and beam is shown in Fig.17 about middle story. Here, η_p means η of panel zone, η_c means η of column and η_b means η of beam.

The reason why the results are shown only about middle story is that it was thought that most of weak story of building appear in middle story, judging from Fig.9.

At the inside connections, the damage of panel zone are larger than other members, and the damages of most columns and beams are very small because η is smaller than 1. At the outside connections, the damages of beams are larger than other members, and the damages of most columns and panel zones are very small.

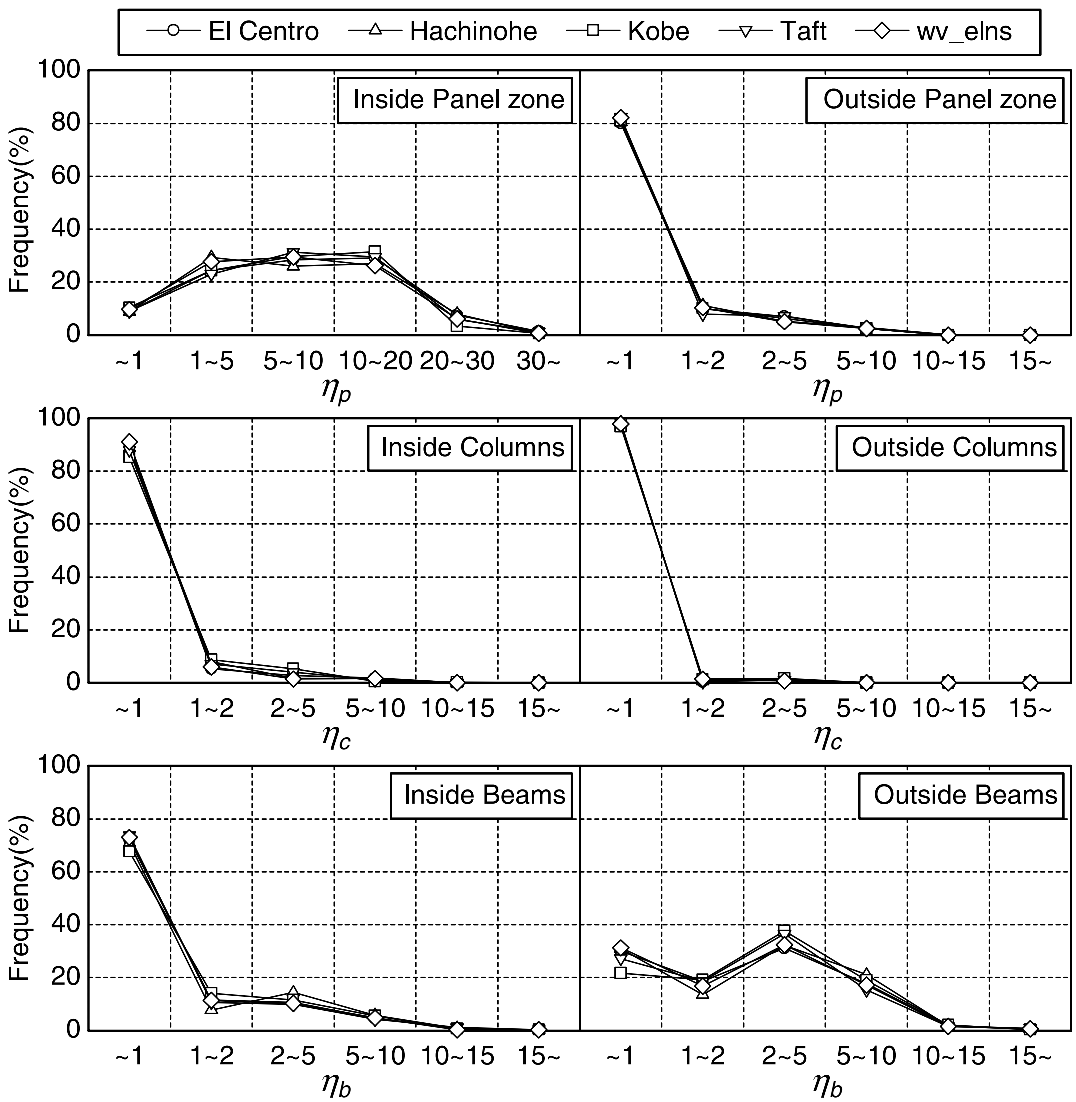

Fig.17 Damage of Structural Members

5. CONCLUSIONS

The results of the investigation about low or middle rise steel building structure built in a local city in Japan and the consideration of seismic resistance ability of those structures are summarized as follows.

1) In general low or middle rise steel building structure, cold-formed rectangular hollow sections are used for columns, and rolled wide flange steels are used for beam, and steel material of structural members are 400N/mm^2 grades.
2) Considering about the distribution situation of strength ratio of column to beam and considering about the results of dynamic analysis, it is clear that most of columns do not reach plastic range under severe earthquake. Therefore, it is supposed that the collapse mechanism of structures is total collapse mechanism.
3) Considering about the width-to-thickness ratio of structural members, ductility factor of most stories of structures preserves 4-5. On the other hand, results of dynamic analysis show that the ductility factor of the almost all structures whose ${}_e\alpha_i$ are lager than 0.2 is smaller than 4. Judging from above mentioned facts, it is clear that the structures whose ${}_e\alpha_i$ is lager than 0.2 preserve enough plastic deformation ability.

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