Seismic Performance of Concrete Filled Steel Tube Column Building Using Ultra High Strength Steel H-SA700

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***Abstract*—The ultra-high strength steel H-SA700 is a relatively new and environment-friendly structural steel with no requirement for intensive heat treatment during manufacturing. In this paper, the seismic performance of concrete filled steel tube (CFT) column building using ultra high strength steel H-SA700 was investigated analytically. For the time history response analysis, a three-dimensional seismic response analysis program STERA_3D was used. The target building is 50.4 by 46.0 meters, and 12 floors above the ground. The building model, using size reduced H-SA700 steel columns as opposed to the conventional steel ones used in the reference model, is able to secure seismic performance almost equal to that of the reference model. The model reduces the usage of steel material by about 20%, and the cross sectional dimension reduces by 10% to the reference model using conventional steel. Furthermore, the time history response analysis revealed that the columns on the upper floors of the building can be further reduced in size.**

***Index Terms*—ultra-high strength steel, concrete filled steel tube, H-SA700, time history response analysis**

I. INTRODUCTION

This research targets middle-high-rise buildings that are often planned for commercial purposes, and proposes an ultra high strength concrete filled steel tube (CFT) "H-SA 700" [1] of a high strength of 780MPa for building construction. Regarding the H-SA 700 steel, numerous studies [2]-[4] have been developed in the field of steel structure. Meanwhile, in the field of composite structures, it is limited to the basic research cases such as [5] application of CFT column and the construction examples [6] adapted to some columns of the lower floors of the high-rise building based on it. Again, it does not show the general design method of CFT column members of H-SA 700 steel and seismic performance verification example when the entire building is made of high elastic concrete filled steel tube (CFT). Therefore, the authors experimentally verified the seismic performance of CFT column members of H-SA 700 steel [7]. In addition, the design method of the column members was proposed [8].

In this paper, we will verify seismic performance by time history response analysis of buildings using H - SA 700 steel CFT column members to verify its implementation.

Manuscript received December 9, 2017; revised April 1, 2018.

II. OUTLINE OF EARTHQUAKE RESPONSE ANALYSIS OF BUILDINGS USING ULTRA HIGH STRENGTH STEEL CFT COLUMNS

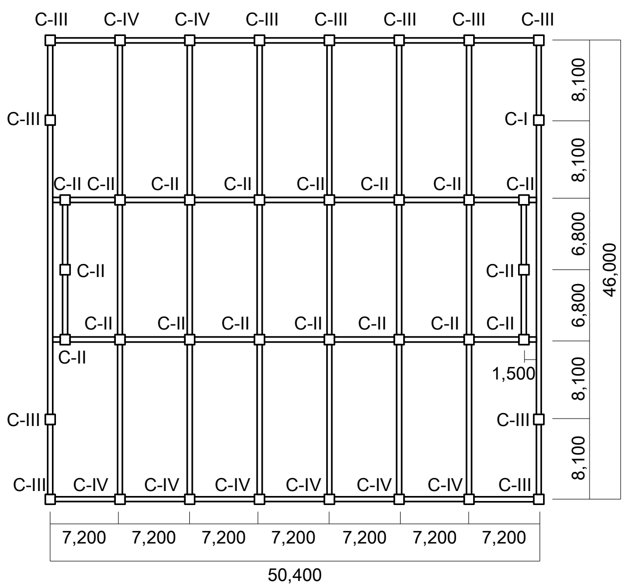

Figure 1. The ground floor plan of target building. (Unit: mm)

For the time history response analysis, a three dimensional seismic response analysis program STERA_3D [9] was used. The target building as shown in Fig. 1 is based on the "design example" described in "Recommendations for Design and Construction of Concrete Filled Steel Tubular Structures" in Japan [10] - appendix, and its outline is shown below. For further details, please refer to the CFT recommendations.

- X direction: pure rigid frame structure, 7 spans, 50.4 m
- Y direction: pure rigid frame structure, 6 spans, 46.0 m
- Number of floors: 12 floors above the ground
- Maximum height: 57.9 m
- Building area: 1,954 m^2
- Area of description: 27,821 m^2
- Column members: CFT, H-SA 700 (yield stress σ_y: 700MPa) or BCP 325S (conventional steel, equivalent

doi: 10.18178/ijscer.7.2.92-98

to ASTM A992 in the U.S. and EN-10025 S355JR in Eurocode 3, σ_y: 325MPa), Square cross section

- Beam member: Steel structure, SN 490 B (conventional steel, σ_y: 325MPa), wide flange

Fig. 2 shows the target skeleton model of the building for analysis. In the time history response analysis, the tri-linear model of Fig.3 uses column member characteristics based on the reference [8]. Here we compared the seismic performance of a case, where conventional steel (BCP 325 S) is used for the column members as CFT, with the case, as proposed in this research, where H-SA 700 steel is used as CFT.

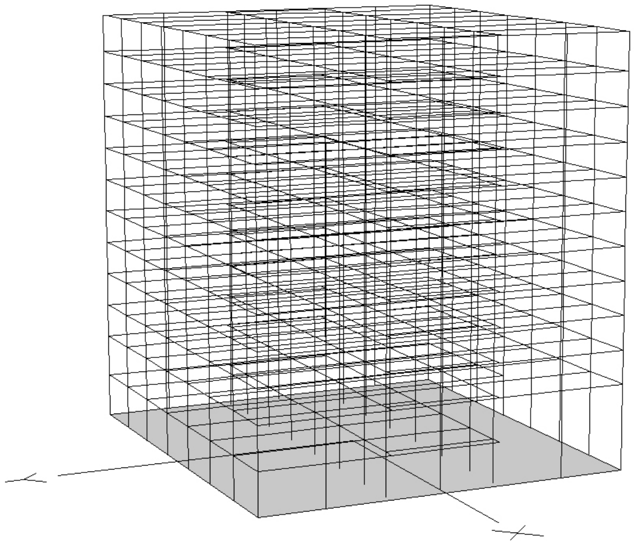

Figure 2. Target skeleton model of building for analysis.

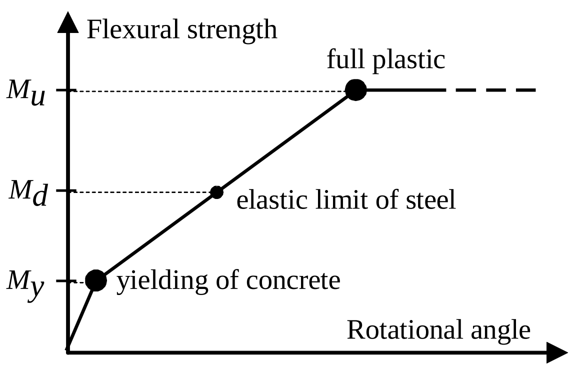

Figure 3. Flexural strength - rotational angle relationship of the CFT column member [8].

III. Column Cross Section Used for Analysis Model

In the design example of the CFT recommendations, four kinds of CFT column members were applied to the target building. The compressive strength, F_c of the filled concrete is 48MPa, and the measurements of column steel pipes are 650 x 25 (hereinafter, column I), 650 x 28 (column II), 650 x 32 (column III), and they are made of BCP 325 650 x 36 (column IV). The black solid line in Fig. 4 represents the flexural strength - rotational angle relationship of the above four types of columns (columns I to IV, hereinafter referred to as reference model).

On the other hand, we compared the seismic performance of the entire building with the CFT column members using the H - SA 700 steel proposed in this study (the compressive strength F_c of the filled concrete was set to 48MPa, which is the same as the reference model). Table I shows the cross-sectional dimensions of three models (A to C) of H-SA 700 steel CFT column in addition to the reference model. Moreover, the blue chain line in FIG. 4 shows the flexural strength versus the member angle of the model A, and similarly the red chain line represents model B, and the green chain line represents model C. The models A to C are based on the design method of H-SA 700 CFT column members proposed in reference [8].

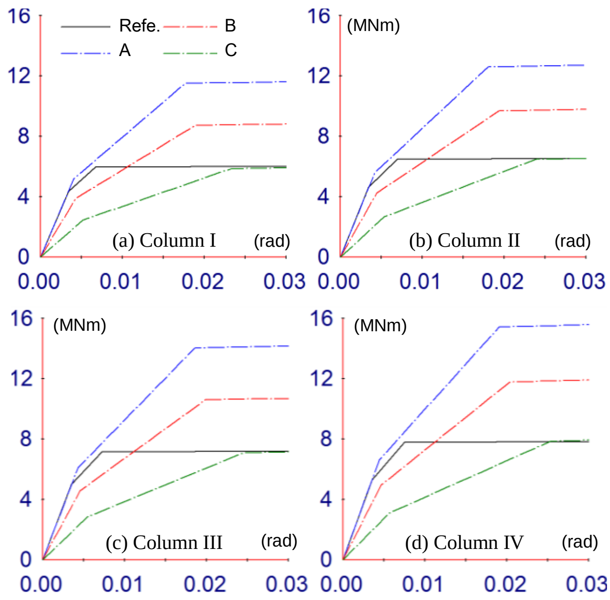

Figure 4. The flexural strength – rotational angle relationship of each CFT columns.

TABLE I. Area Section of CFT Column of Each Models

	Refe. Model	Model A	Model B	Model C
Steel	BCP325	H-SA700	H-SA700	H-SA700
Column I	650 x 25	650 x 25	600 x 22	500 x 22
Column II	650 x 28	650 x 28	600 x 25	500 x 25
Column III	650 x 32	650 x 32	600 x 28	500 x 28
Column IV	650 x 36	650 x 36	600 x 32	500 x 32

Here, the CFT column model A with H - SA 700 steel adopts the same cross section as the BCP 325 steel pipe of the reference model. Therefore, the initial rigidity of the member is equal to the reference model, and the ultimate flexural strength (full plastic moment M_u as shown in Fig. 3) is about twice the reference model using the BCP 325 steel. Model B is a case in which the cross section is adjusted so that the flexural strength M_d at elastic limit of H-SA700 steel is approximately equal to the ultimate flexural strength M_U of the reference model, and its full plastic moment reaches about 1.5 times the reference model. Model C is a case in which the cross section is adjusted such that the full plastic moment M_U of the H-SA700 member is equal to that of the reference model. Table II shows the comparison of the total weight of the steel materials of the column members for each of the four models.

IV. Static Incremental Analysis of Each Analysis Model

Fig. 5 shows the results of static incremental analysis of each model based on the AI distribution [11]. From the figure, in Models B and C, where the cross sections are relatively small with respect to the reference model, there is a tendency that the stiffness of the lower floor tends to be slightly lower. However, there is no extreme difference in the shear strength of each story, which is attributable to the fact that its story yield strength is mainly controlled by the beam yield, in the target building of the weak-beam type frames. In addition, in the vicinity of the intermediate story where deformation is dominant, there is no notable difference in rigidity as well as in the strength of each model.

TABLE II. Total Weight of Steel of Column Members

	Total weight	Comparison to reference model
Reference model	949.7 ton	1.000
Model A	949.7 ton	1.000
Model B	777.0 ton	0.818
Model C	635.3 ton	0.669

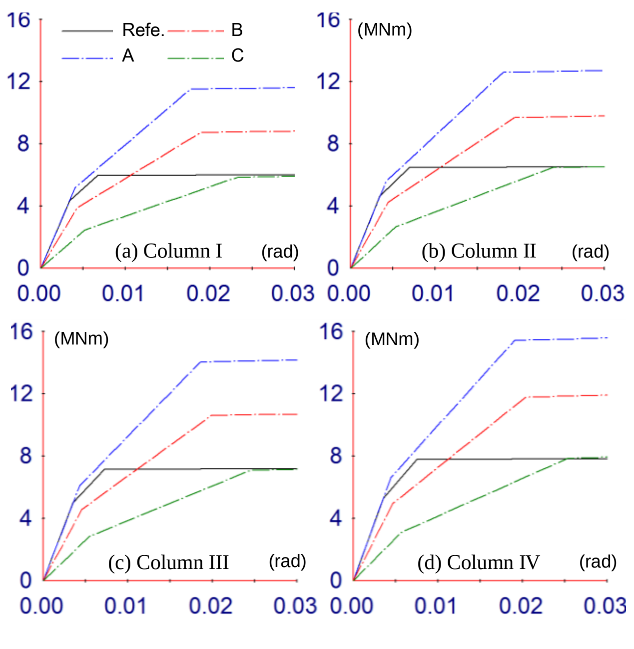

Figure 5. Result of static incremental analysis of each models.

V. Dynamic Analysis

Three types of The seismic waves (as shown in Fig. 6) were used: JMA-Kobe (NS component), ElCentro (NS component), Hachinohe (EW component), and the maximum ground motion speed was adjusted to 25 kine, 50 kine, and 75 kine respectively. Damping is assumed to be proportional to initial stiffness, and the damping constant is set to 2% for both primary and secondary. FIG. 7 shows the maximum deformation state of the reference building in time history response analysis with JMA-Kobe wave, 50 kine. The portion painted black in the edge of members of the skeleton model of the figure indicates that the plastic hinge is formed. From the figure, it can also be confirmed that the analytical model exhibits the fracture behavior of the weak-beam type frames as expected in the target building, and it is understood that no hinge is generated in the column members.

In Figs. 8 to 10 shows the distributions and comparisons of the maximum acceleration, the maximum story drift angle, and the residual displacement in the all-time history response analysis along the height direction for each model.

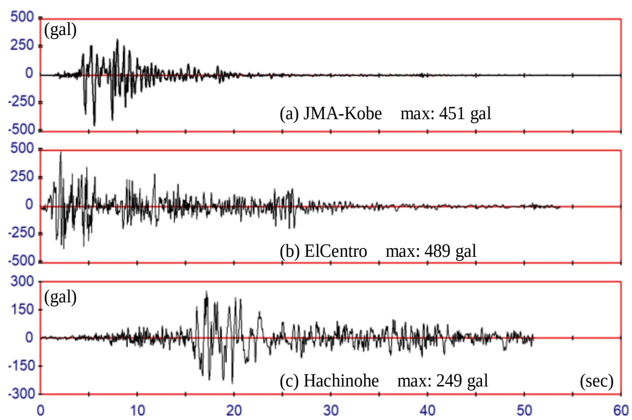

Figure 6. Input seismic waves. (either 50 kine)

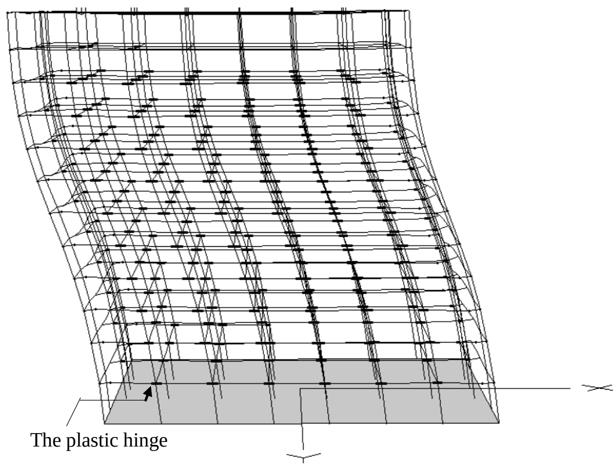

Figure 7. The maximum deformation state of the reference building. (JMA-Kobe, 50 kine)

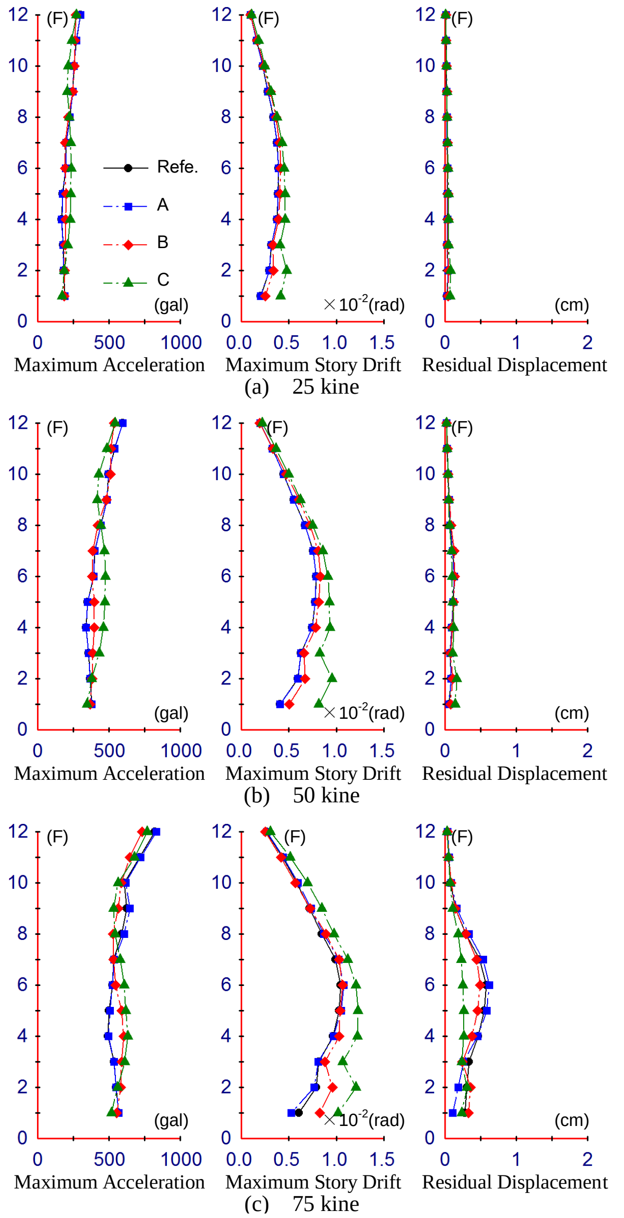

Figure 8. JMA-Kobe NS analysis result.

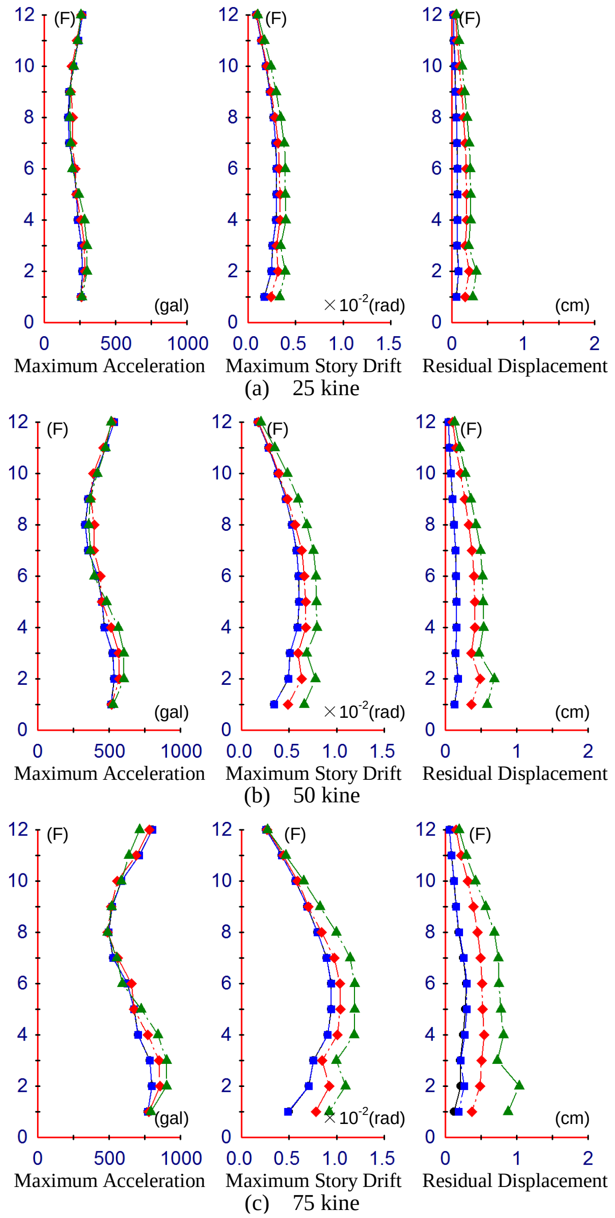

Figure 9. ElCentro NS analysis result.

In addition, Table III summarizes the degree of plasticity of the column base of the first floor inner columns in each analysis, "○" in the table indicates that the column base was in the elastic state through analysis, "△" indicates the elastic limit of the CFT columns is exceeded but does not reach to the full plastic moment M_u, and "×" indicates that the full plastic moment is reached.

In the analysis results of 25 kine velocity of each seismic wave (see Figs. 8 to 10 (a)), no significant difference is observed in any of the four models. This is because the columns are not plasticized in any of the models as shown in Table I. On the other hand, in the analysis result of 50 kine (see each figure of (b)), there is a difference depending on the model. Almost equal seismic performance is observed between the reference model in which the pillars are both elastic and 之 model A, while deformation is concentrated on the lower floor in model C where the column base has plasticized.

Since the column base of the target building is fixed, plasticization occurs only in the column base in the vertical direction of the column (in the other column end plasticizing does not proceed substantially because the connected beams precedes). This property can be confirmed more conspicuously in the analysis result of 75 kine (see each figure of (c)) i.e. in particular, the residual story drift angle on the first floor is about 1/500, and on the 5th floor it is about 1/370.

Each building model (A to C) using the H - SA 700 steel CFT columns targeted by this research, is compared with the reference model using conventional steel. First, in model A, in which the cross section is the same as the reference model, no significant difference is recognized between the input model of 75 kine (hereinafter referred to as level 3), which is unexpected in the current design, from the reference model.

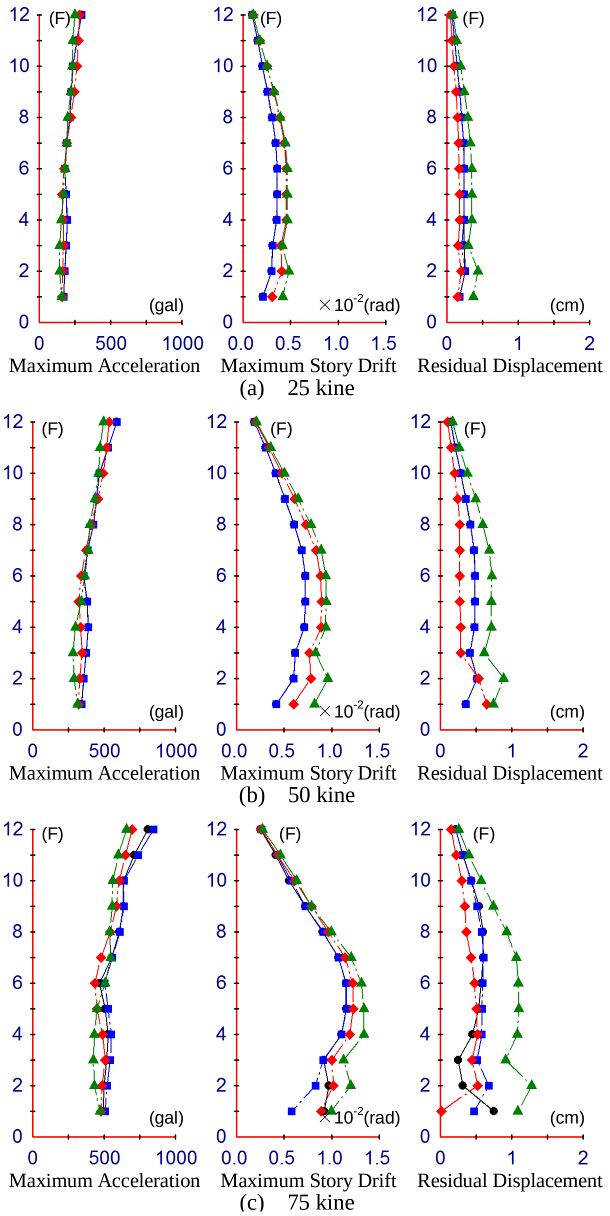

Figure 10. Hachinohe EW analysis result.

This is because the column cross-sectional ultimate flexural strength of this model reaches about twice the reference model, while the influence on the total strength (base shear coefficient) of the entire building is predominant due to the yielding of the beam end. Accordingly, in conventional normal design, even if the type of steel used for the column member is changed from ordinary conventional steel to ultra high strength steel to increase the ultimate flexural strength and the elastic deformation ability of the member, there is no substantial improvement in the seismic performance of the building.

In model B, in which the cross-section is adjusted so that the flexural strength M_d at elastic limit of H-SA700 steel is approximately equal to the full plastic moment M_u of the reference model, — there is a tendency that the response becomes somewhat excessive compared with the reference model as the input level increases.

TABLE III. DEGREE OF PLASTICITY OF FIRST FLOOR INNER COLUMN BASE IN EACH ANALYSIS MODEL

		Refe. Model	Model A	Model B	Model C
25 kine	JMA-Kobe	○	○	○	○
	ElCentro	○	○	○	○
	Hachinohe	○	○	○	○
50 kine	JMA-Kobe	○	○	○	△
	ElCentro	○	○	○	△
	Hachinohe	○	○	△	△
75 kine	JMA-Kobe	×	○	○	△
	ElCentro	×	○	△	△
	Hachinohe	×	△	×	△

○: Elasticity, △: Elastic limit to Full plastic, ×: Full plastic

However, its degree is small and the degree of plasticity of the column members is not so different from the reference model (see Table III). Therefore, in the secondary design (ex. Horizontal load carrying capacity design, hereinafter referred to as level 2), we recommend design which takes the flexural strength M_d at the time of plasticization of H-SA700 steel into consideration.

Conversely, in model C, in which the cross-section is adjusted so that the full plastic moment M_u of the material is equal to the reference model, since deformation concentration and prominence of the lower story portion are particularly recognized at the input of 50 kine or more, it is difficult to apply as it is to the design of a real building. However, responses other than the lower story do not differ much from model B noted previously. This seems to be due to the fact that the portion where plasticization occurs in the pillar member of the target building is limited to the column base.

As shown in Table III, in models A to C, in which the H-SA 700 steel was applied to the CFT column members, the column members did not reach the full plastic state except for some analysis results. This is because, in addition to applying ultra high strength steel having high elastic deformation capability, particularly in Model C, the apparent rigidity is reduced due to the reduction of the cross sectional dimensions, and as a result, the elastic deformability of the member is further improved.

VI. RECOMMENDED DESIGN AND PERFORMANCE OF CFT COLUMN USING ULTRA HIGH STRENGTH STEEL

As mentioned in the previous chapter, in this research it is recommended to use flexural strength, M_d at the elastic limit of H-SA700 steel in level 2 design. However, as shown in Table II, in model B based on the design, the amount of steel used for the column members has decreased by only less than 20% with respect to the reference model, and the sectional dimensions are only reduced by about 10%. On the other hand, in the middle and high floors where the column members are not plasticized, even when the maximum yield strength, M_u of the members is used in the design, the response difference from the reference model is small. Therefore, in this chapter, the columns of the lower floors (here, the first and second floors), which may be plasticized under large amplitude at the time of plasticization of the H-

SA700 steel tube— their flexural strength M_d, and full plastic moment M_u of the upper floors need to be verified for seismic performance.

For the column cross section of the verification model (hereinafter referred to as Model D), Model B in Table II is applied to the first floor and the second floor, and Model C is applied to the third to twelfth floors. Fig. 11 shows a comparison between the reference model and the above verification model D for the maximum acceleration, the maximum story drift angle, and the distribution of the residual drift in the height direction for respective buildings of 75 kine of the JMA-Kobe wave. Although the maximum relative story drift angle is somewhat larger close to the intermediate stories where the drift is maximum, the responses of both are roughly familiar in general, and the concentration and prominence of the lower story drift, which was remarkable in Model C, are sufficiently reduced. The total weight of steel in Model D is 668.2 tons, which is 70% of the reference model. Model D, design method of flexural strength of columns is different between lower floors and other floors, can reduce the total weight of the steel material of the pillar members over 10% compared to Model B, which is directly linked to the reduction of the construction cost.

VII. Conclusions

Earthquake response analysis of a highly elastic concrete-filled steel tube construction building, using ultra high strength H-SA 700 targeted by this research, was conducted using a three-dimensional earthquake response analysis program. In the model in which the cross sectional dimensions are the same and the ultimate flexural strength of member is about twice as compared with the conventional reference model using ordinary conventional steel for the column members, no remarkable improvement was observed in earthquake resistance. This is attributable to the fact that the influence on the total strength (base shear coefficient) of the entire building is predominant due to the yielding of the beam end, and the influence of the column member on the tensile stress of the entire building is limited.

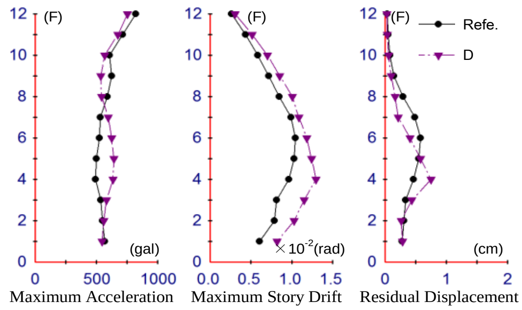

Figure 11. Dynamic analysis result of Model D. (JMA-Kobe NS 75kine)

The model with the cross section adjusted so that the flexural strength M_d at elastic limit of H-SA700 steel is equal to the full plastic moment M_u of the reference model, and is able to secure seismic performance almost equal to that of the reference model. However, the model reduces the usage of steel material by about 20%, and the cross sectional dimension reduces by 10% to the reference model using conventional steel.

In the model in which the cross-section is adjusted so that the full plastic moment M_u of the H-SA 700 steel CFT column is adjusted to be equal to the reference model, the response increases gradually with the increase of input wave, and the deformation concentrates on the lower floors. It also provides prominence due to the fact that plasticization progresses only at only one column base part of all the column members of first floor, and plastic hinges are formed at the time of a large earthquake. In this research, we propose a method to design the pillars of the lower floor of the building by using the value of the flexural strength M_d at elastic limit of H-SA700 steel, and the columns of the upper floor by using the full plastic moment M_u. This method is preferable as it is possible to reduce the amount of steel while suppressing building response during earthquake.

Acknowledgment

This paper reports a continuing research supported in part by the Japan Iron and Steel Federation and JSPS Grant Number 15K18152 (Principal Investigator : Kazuhiro Hayashi).

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