

Study on seismic capacity of super-high tensile steel braced frames

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ABSTRACT: Super-high tensile steel braced frame is a braced frame in which bracings consist of steel that has by far a greater strength than one normally used for a similar purpose. The basic concept of this type of frames is that even when the predictable maximum story drift due to seismic forces may cause other members (i.e., columns and beams) to enter the plastic range, bracing will follow such deformation remaining in the elastic range. Then, by the use of bracings of this type, a whole structure can be readily designed to fail in a mode of bending failure. Further, stable recovery characteristic and effective energy-absorbing capacity can be secured. In this paper, failure scattering effect on this frame is evaluated in terms of total energy absorbed, thus realizing its qualitative evaluation. As results of the incremental analysis, the bracings showed stabilized behaviors, remaining in the elastic range even at the ultimate stage.

1 FRAMES AND THEIR MECHANICAL CHARACTERISTICS

For the purpose of this present paper, the term "super-high tensile steel braced frames (Wakabayashi, Ukai 1969.)" is defined as braced frames in which bracings consist of steel (e.g., 100-kg/mm² tensile strength class steel) that has by far a greater strength than one normally used for a similar purpose.

The basic concept of this type of frames is that even when the predictable maximum story drift may cause other members (i.e., columns and beams) to enter the plastic range, bracings will follow such deflection remaining in the elastic range.

Validity of braced frames based on the concept mentioned above will be discussed in the following paragraphs.

① Stable recovery characteristic can be obtained.

Braced frames commonly in use pose a problem in that their rigidity and strength are lowered under repeated loadings, which is represented by "deterioration type envelope (loop) when graphically expressed. Unlike these, frames braced with super-high tensile steel ensure well stabilized envelopes as shown in Fig. 1.

② By the use of bracings of this type, a whole structure can be readily designed to fail in a mode of bending failure.

Where this type of frames is used for multi-story structures, it is believed to be preferable to

allow columns to have great rigidity and bearing strength compared with columns thus causing the frames to be of a beam-yield type.

The reasons why this is preferred are : it is comparatively easy to increase the plastic deformation ability of beams and thus to increase yield strain energies; since columns are generally subjected to high axial forces, their recovery characteristics tend to be affected by deterioration; failure of columns is conducive to failure of a whole building; and it is possible to ensure proper transfer of forces to upper and lower stories if columns are kept in a sound conditions and thus to alleviate concentration of structural damage.

For various reasons, however, it is not so easy to obtain positive beam-yield frames as it may seem. For instance, a balance of column strength and beam strength is lost if story heights are not uniform.

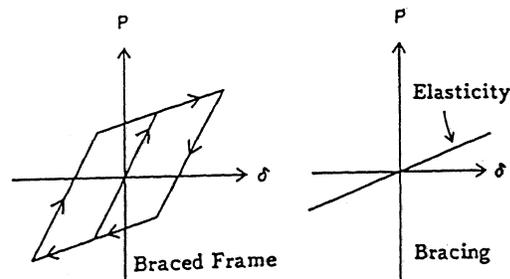


Fig. 1 Recovery characteristics

Such a balance of strengths is also lost by increase in strength and rigidity of beams due to beam-slab composite effects, by nonuniformity of the degree of precision of materials and workmanship or by nonuniform strength distribution. Further to this, high-rise buildings in these recent years are designed to have complicated plan and sectional configurations, and this makes it even more difficult to obtain a beam-yield type frames out of conventional rigid frames.

In the super-high tensile steel braced frames, bracings serve as a sort of mandrels which prevent structural damage from being concentrated on any one story and thus prevent that story from being deformed excessively.

As a result, the same effects as displayed by beam-yield type frames may be expected. (This makes it possible to prevent plasticization of any story to proceed beyond a certain point even if any column in the story may yield before beams and thus frames can be designed to have high structural safety. See Fig. 2.)

If damage ratios of respective stories are kept about equal by utilizing the aforesaid mandrel effects, then highly effective frames may be expected in which strain energies are consumed efficiently throughout the entire stories.

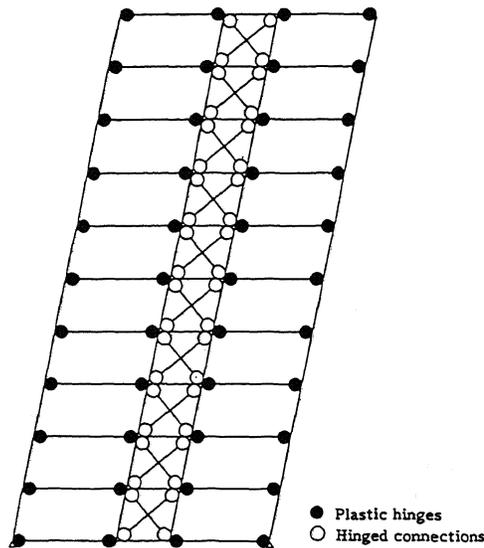


Fig. 2 Concept of state of yielding

2 EFFECTS OF SUPER-HIGH TENSILE STEEL BRACINGS

Static incremental analyses were conducted using the same frames as the author and his colleagues used for the actual building in order to ascertain quantitatively the injure deconcentration effects and the energy

absorption capacity of the frame braced with super high tensile steel.

As a means of evaluation, the area enclosed by the story shear force - relative story displacement curve and the story displacement axis (horizontal) and assumed to represent the energy absorbed by the respective stories. (See the hatched part in Fig. 3)

It should be noted that for the purpose of such incremental analysis, a certain external force distribution (i.e., a load distribution based on a seismic response analysis) was assumed and then the force was proportionally increased.

Comparison of the energy absorption by the two types of frames was made firstly by assuming that a relative story displacement angle of $1/75$ indicated the allowable limit of the frame at any story and by obtaining the load-deformation relationships for the respective frames at the time when the relative story displacement angle reached $1/75$. (Reference should be made to the area of the hatched part in Fig. 3 in which δ was taken as $1/75 \times h$ where h is a story height.)

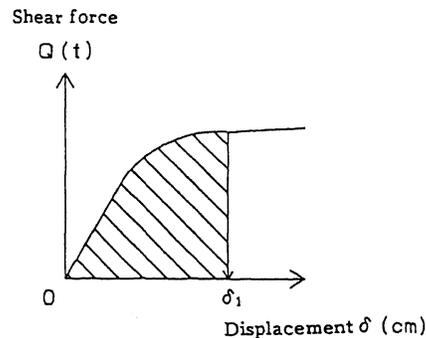


Fig. 3 Energy absorbed

Fig. 4 and Table 1 show the models used for the analysis and their members. As analyses parameters, three frame models, i.e., the frame without bracings (N-1), the braced frames as used for the actual building (B-1) and the frame in which the super high tensile steel bracings were made to have a cross sectional area twice as large as that of the actual bracings (B-2) were used.

Models:

- N-1 Frames without super high tensile steel bracings
- B-1 Frames provided with super high tensile steel bracings (as used for the actual building)
- B-2 Frames in which bracings on each story were made to have a cross sectional area twice as large as that of the actual bracings

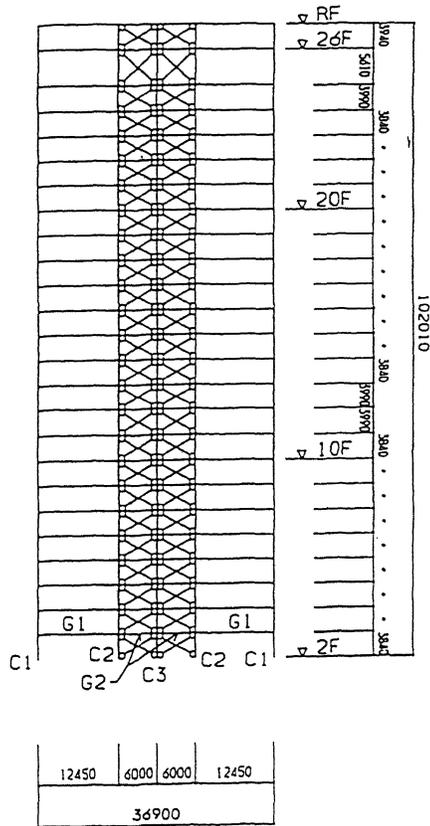


Fig. 4 Model used in analysis

2.1 Static elastoplastic analysis

(a) Analysis method

For the purpose of the static elastoplastic analysis, the entire space frame was dissolved into a number of planar frames and analysis was made for each such planer frame.

As for the load, the shear force to be carried by respective frames under the design seismic load was increased in increments in the X-direction as shown in the figure below.

(b) Elastoplastic characteristics of members

The elastoplastic characteristics of the members were established for the analyses as follows:

(Columns and Beams)

The conditions for the yielding of columns and beams are given by the following formula and their post-yielding behaviors are assumed to follow the plastic flow theory.
H-shapes, Box-shapes

$$\text{If } N/N_r \leq 0.125$$

$$M = M_p$$

$$\text{If } N/N_r > 0.125$$

$$M = 1.14 (1 - N/N_r) M_p$$

Table 1 Member list

Columns section			
Floor	C 1	C 2	C 3
26-22	H-400×400×13×21	H-400×400×13×21	H-400×400×13×21
21-19	H-406×403×16×24	H-406×403×16×24	H-406×403×16×24
18-16	H-406×403×16×24	H-418×417×30×30	H-406×403×16×24
15-13	H-406×403×16×24	H-428×422×35×35	H-410×407×20×26
12-10	H-410×407×22×26	H-438×427×40×40	H-414×409×22×28
9-7	H-410×407×22×26	H-458×427×40×50	H-418×417×30×30
6-2	H-414×409×22×28	H-478×427×40×60	H-428×422×35×35

Beam section			
Floor	G 1	G 2	
	(With inner column)	(Inner)	
R	FPL-22×300 WPL-16×706	FPL-22×300 WPL-16×706	FPL-19×300 WPL-16×712
26	FPL-22×300 WPL-16×706	FPL-22×300 WPL-12×556	FPL-19×300 WPL-16×712
25		FPL-25×350 WPL-16×706	FPL-19×300 WPL-16×706
24-22	FPL-22×300 WPL-16×706	FPL-22×300 WPL-12×556	FPL-19×300 WPL-16×712
21-13	FPL-22×300-350 WPL-16×706	FPL-22×300 WPL-16×706	FPL-19×300 WPL-16×712
12		FPL-25×350 WPL-16×706	FPL-22×300 WPL-16×706
11-3	FPL-22×300-350 WPL-16×706	FPL-22×300 WPL-12×556	FPL-19×300 WPL-22×712

Brace section			
Floor		B-1	B-2
26	33Ø P C bar	× 2	× 4
25	27Ø P C bar	× 4	× 8
19-24	33Ø P C bar	× 2	× 4
11-18	27Ø P C bar	× 4	× 8
2-10	30Ø P C bar	× 4	× 8

where, N_y : Axial force at yielding ($A \times \sigma_y$)
 M_p : Full plastic moment ($Z_p \times \sigma_y$)

The yielding of the columns and beams due to shears are governed by the $Q-\gamma$ relationship in the figure to the left, in which:

G: Shear elastic modulus (810 t/cm²)

Q_y : Yield shear strength ($= A_s + \tau_y$)

A_s : Cross section area subject to shear

τ_y : Yield shear stress ($\sigma_y/\sqrt{3}$)

Recovery of bracings is governed by the $N-\mu$ relationship shown in the figure to the left, in which:

N_t : Tensile strength of the bracing

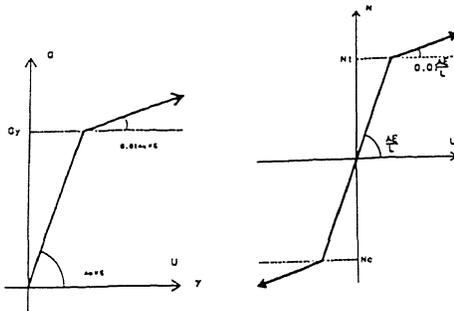
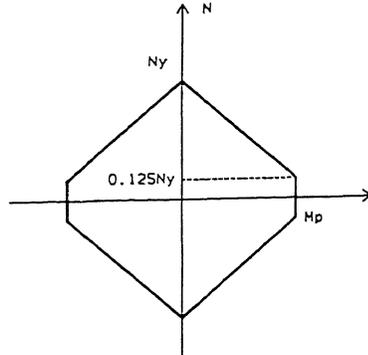
N_c : Compression strength of the bracing

(c) The full plastic moment (M_p) on the member was increased in consideration of the force moment.

(d) The yielding conditions used for the analysis include the consideration of the permanent bending moment on the beams and the permanent axial force on the columns.

(e) The material strength was taken as 1.1 times the F -values.

(f) The slabs were taken into account in computing the rigidity of the beams but they were disregarded in computing the strength of the beams.



2.2 Elastic limit strength $QY1$

For steel frames, the story shear force which caused any member belonging to a certain story to attain full plastic moment was regarded as the elastic limit strength of that story.

2.3 The first flexion point

The first flexion point is taken as the point at which the first branch of the hysteresis characteristic reaches the elastic limit strength.

2.4 The second branch rigidity $K2$

For determining the captioned rigidity for the steel frames, the response results as obtained by the preliminary response analysis at the time of a Level 2 earthquake is taken into account and the rigidity is so established as to have characteristics very similar to those seen in the neighborhood of the maximum response.

2.5 Ultimate strength $QY2$

Shearing forces as confirmed for respective frames by the static load incremental analysis were taken as the ultimate strengths.

2.6 The second flexion point

The second flexion point is taken as the point at which the second branch of the recovery characteristic reaches the ultimate strength.

2.7 The third branch of rigidity ($K3$)

The third branch of rigidity ($K3$) is assumed to be zero (i.e., $K3 = 0$).

2.8 Plastic coefficient μ

The datum point is taken as the first flexion point of the skeleton curve.

2.9 Hysteresis rule

Hysteresis rule of tri-linear type (normal) was used.

For external force distribution, the design seismic load distribution was used. This distribution was obtained from the response computation which was based on the optimum yield shear force distribution. Cumulative plastic energies for respective stories were computed and such energies for the super-high tensile steel braced frames were compared with those for the pure rigid frames on a story by story basis.

3. CONCLUSION

The states of deformation and plastic hinge developments as obtained by the static analysis are as shown in Figs. 5 and 6. It can be known from these figures that at a load factor of 1.325, B-1 Model had no plastic hinges whereas in Model N-1 plastic hinges were developed in all the stories from the lowermost through the 17th story with the deformation also at a fairly advanced stage.

Some observations on energy absorption will be made below.

Incidentally, in all of the three models, the relative story displacement angle of 1/75 times that story height was exceeded first at the 11th story; therefore, the energy absorption by the respective frame models will be considered on the basis of the load - deformation curves up to the point the said displacement angle was exceeded.

Fig. 7 through -9 show the load-deformation relationships for the principal stories.

Next, Table 2 indicates the energy absorbed by the respective frames.

From these, the bracings may reasonably be expected to display a satisfactory interstory force transfer function.

Comparison of the total energy absorbed by the respective frames indicates that 8.9 % and 16.9 % more energies were absorbed by the braced frames B-1 and B-2 respectively than the unbraced rigid frame N-1.

The cross sectional areas of the bracings used for the frames B-1 and B-2 were very small being only 28.3 sq.cm and 56.6 sq.cm respectively, but the effects of these bracings on energy absorption were very great. Thus, the data obtained indicates that these bracings are highly effective.

Table 2 Energy absorbed

Floor	Unit: T-cm				
	N-1	B-1	B-2	(B-1)/(N-1)	(B-2)/(N-1)
26	83.4	100.2	125.4	1.202	1.503
25	372.5	349.0	395.9	0.937	1.063
24	429.1	409.7	467.6	0.955	1.090
23	538.7	515.8	565.9	0.958	1.050
22	641.8	650.0	650.2	1.013	1.013
21	602.6	621.4	680.3	1.031	1.129
20	674.6	716.4	743.1	1.062	1.102
19	734.0	803.2	795.8	1.094	1.084
18	804.1	974.8	1014.3	1.212	1.261
17	894.5	1111.4	1211.9	1.242	1.355
16	879.6	1097.1	1204.6	1.247	1.370
15	877.8	1100.5	1216.6	1.254	1.386
14	936.8	1176.7	1308.4	1.256	1.397
13	963.3	1211.4	1347.4	1.258	1.399
12	823.4	1039.5	1151.6	1.263	1.399
11	1093.7	1342.1	1506.1	1.227	1.277
10	1102.1	1264.1	1363.9	1.147	1.238
9	1040.6	1168.0	1253.5	1.122	1.205
8	1075.5	1177.6	1261.1	1.095	1.173
7	1073.5	1157.3	1223.9	1.078	1.140
6	964.8	1022.9	1036.8	1.060	1.075
5	972.6	1015.4	1068.2	1.044	1.098
4	938.0	1009.0	1070.8	1.026	1.089
3	984.9	942.5	995.8	0.957	1.011
2	1423.4	862.5	857.3	0.606	0.606
Total	20970.0	22838.6	24516.2	1.089	1.169

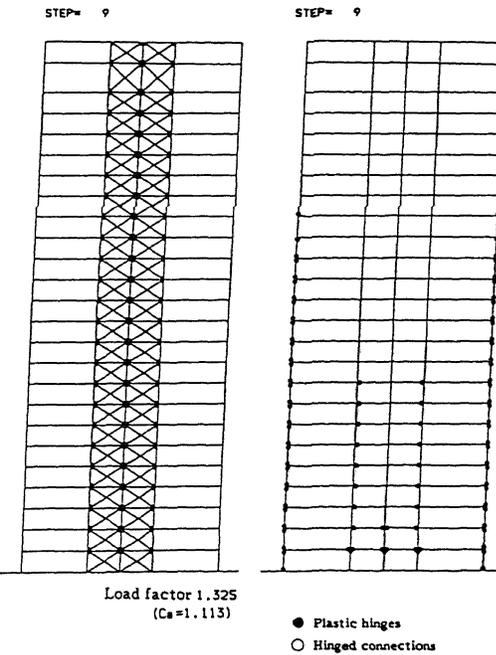
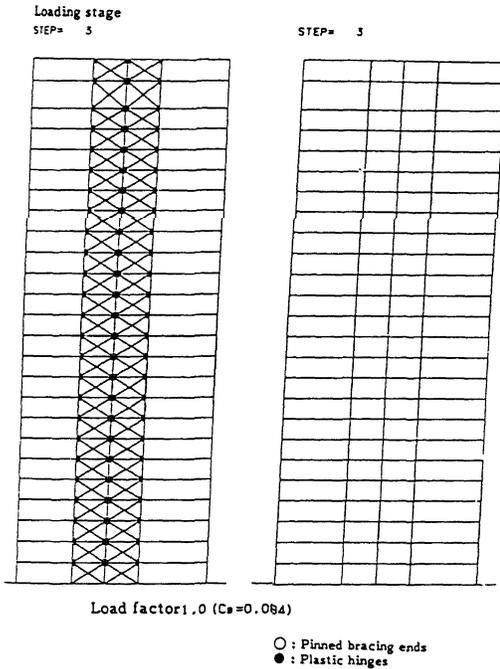


Fig. 5 Plastic hinges caused

Fig. 6 Plastic hinges caused

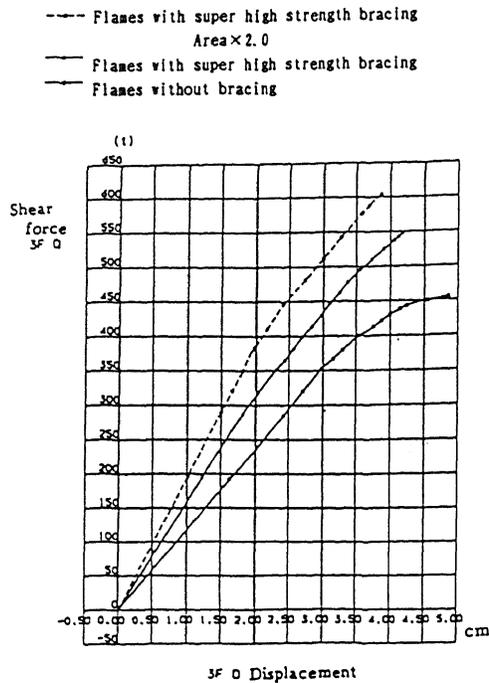


Fig. 7 Load-displacement relation

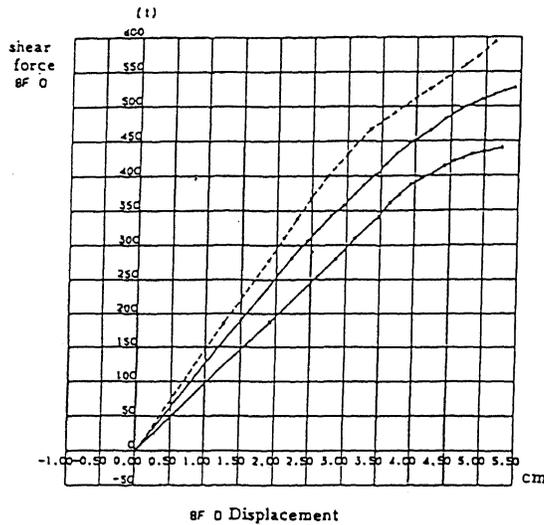


Fig. 8 Load-displacement relation

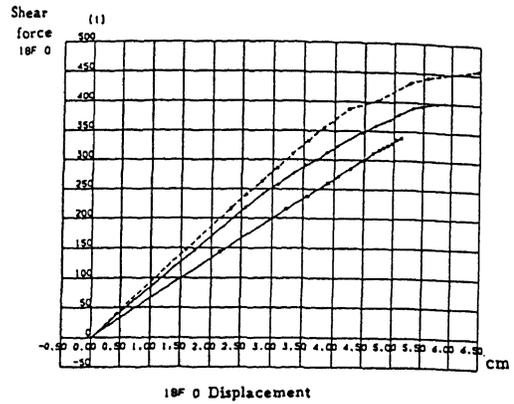


Fig. 9 Load-displacement relation

In the design of actual buildings, it is difficult to cause beams to yield before columns if conventional rigid frames are used as has been mentioned previously. When a thought is given to this point, the application of the frame braced with super high tension steel to high-rise buildings is believed to make effective contribution to the aseismic structural design of such building.

REFERENCES

- Wakabayashi, K., Ukai, K. et al. 1969. Study on Structural System of KTC Building, Vol: 1 Structural Design. *Procedure of AIJ Convention, August 1969*: pp. 1105-1117