Damage Control Design Based on Attenuation Mechanism by Unbonded Brace

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Summary

This paper shows a damage control design of 19-stories office building by ' Unbonded Brace '. Unbonded brace is a buckling-resistant structural member consisting of steel core members enclosed in a concrete-filled square steel tube. The unbonded braces consisting of steel core member using low yield strength steel become plasticized even at moderate seismic level. At large seismic level, the unbonded braces absorb a large quantity of seismic energy. Moreover, after a large earthquake, unbonded braces can be replaced as required.

Introduction

Since the Hanshin-Awaji Earthquake (1995.1.17) in Japan, studies on seismic energy isolation and damping structures abound and a number of attenuation mechanisms have been proposed. It is noteworthy that the realization of structures with high energy absorption is possible through early plasticization of seismic members (hysteretic damper).

In accordance with this concept, the authors proposed a damping structure. The damage control design for this building is based on attenuation mechanism produced hysteresis damping effect by unbonded brace using low yield strength steel (LYP100 : yield strength $\sigma y \fallingdotseq 100$ Mpa). This paper outlines the damage control design of a 19-stories office building, which was designed based on attenuation mechanism by unbonded brace and is currently under construction in Osaka.

1. Outline of the Building

Designed building is 19-stories steel structure as shown overall-view in Fig 1. As illustrated Fig 2 and 3, the building consists of main frames with three spans of 9.6 m, and hysteretic dampers (unbonded braces) are situated in common space located at north and south sides. The typical story heights are 3.85 m and 3.8 m, and the height of the building is 85.8 m.

This structure is furthermore characterized by:

(1) mega-structure frame taken advantage of rescue to enhance the hysteresis damping effect by unbonded braces,

(2) concrete filled tubes to enhance the capacity to absorb earthquake energy and the rigidity of 1 and 2 stories with taller height (4.8 m).

Fig 1 Overall-View

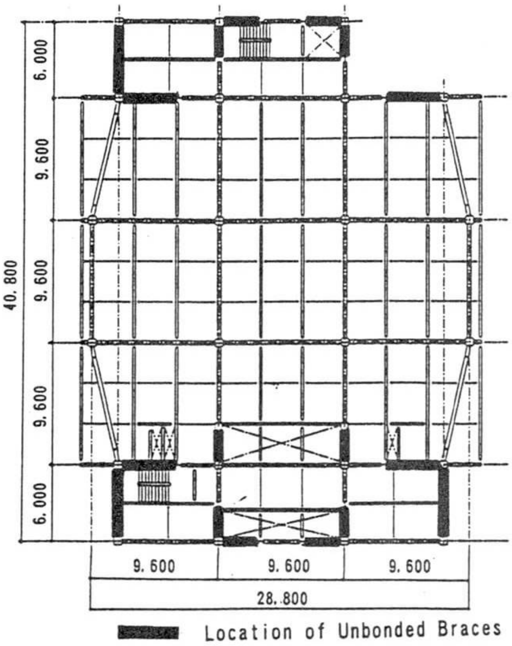

Fig 2 Typical Floor Plan

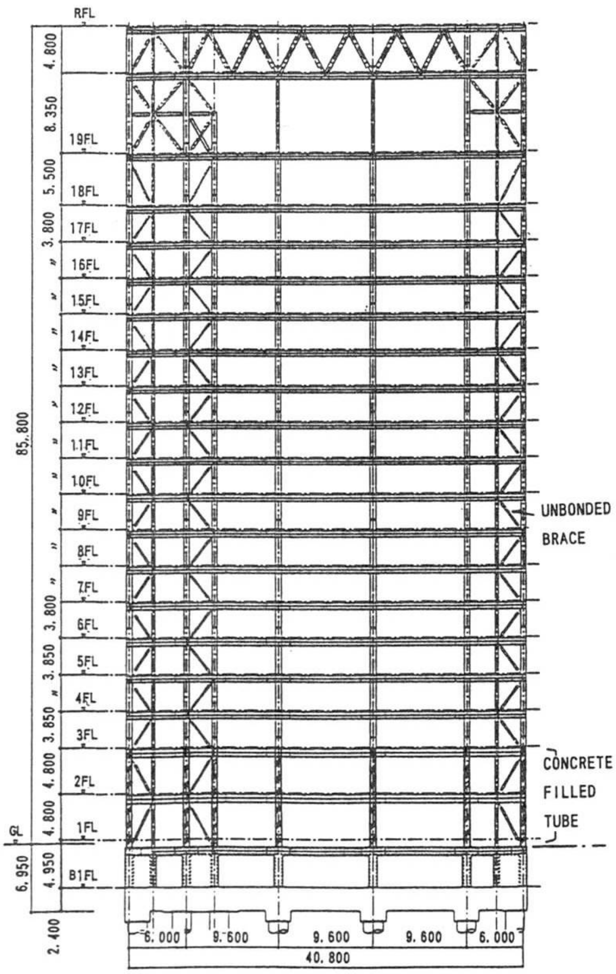

Fig 3 Framing Elevation (N-S dir.)

2. Seismic Design

2.1 Damping Mechanism

'Unbonded brace' is a buckling-resistant structural member consisting of steel core member enclosed in a concrete-filled square steel tube (Fig 4). The steel core member is coated with a non-bonding material, so that no axial force works on the concrete and the steel tube. Consequently, this brace shows stable hysteresises if the yielding load working on the core member is smaller than the buckling load of the steel tube. Besides the stable hysteresis, this brace shows excellent energy absorption. Because the steel core member is made of low yield strength steel (LYP100: $\sigma y \fallingdotseq 100$ Mpa) and the hysteresis energy of low yield strength steel is much greater than ordinary steel (Fig 5).

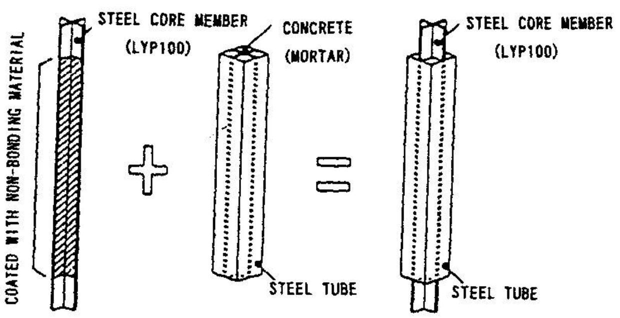

Fig 4 Unbonded Brace

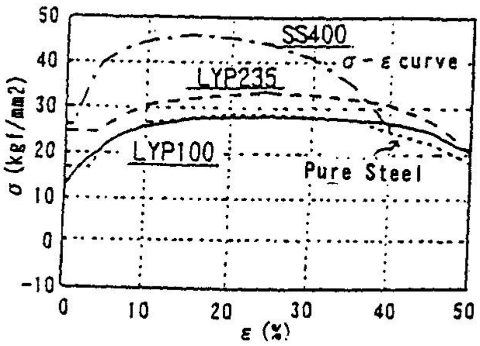

Fig 5 Stress-Strain Curve of Low Yield Strength Steel (LYP100)

2.2 Design Philosophy

In order to assure earthquake resistance of this structure, it was designed according to the following criteria. Table 1 gives the target values of structural performance obtained from the dynamic response analysis. The axial strength of unbonded braces was designed by aiming at that almost columns and beams are not yielding at seismic level 2 (the ground velocity of 50 kine). At that time, the end joints of unbonded braces are elastic.

Table. 1 Target Values of Structural Performance

Earthquake Ground Velocity	Target Values			
	Story Drift	Ductility Factor of Members		Strain of Members
		Beam	Column	Steel Core Member of Unbonded Brace
25 kine	$\leqq$ 1/200	$\leqq$ 1.0	$\leqq$ 1.0	$\leqq$ 0.005
50 kine	$\leqq$ 1/100	$\leqq$ 2.0	$\leqq$ 1.0	$\leqq$ 0.015

3. Seismic Response Analysis

3.1 Three-Dimensional Static Analysis

A three-dimensional nonlinear static analysis was carried out by the step-by-step method in order to identify the elasto-plastic characteristics of the designed frame, and confirm that the collapse mechanism is determined by flexural yield of beams. As shown in Fig 6 , unbonded braces were analyzed using three divided axial model consisting of ordinary steel (joint) and low yield strength steel (core member). The hysteresis curve of low yield strength steel to harden by repetition was approximated by the result of specimen test (Fig 7) . On the assumption that the floor is rigid, analysis was carried out by applying to the gravity center of each floor incremental load with the same distribution pattern as that of the design shear force.

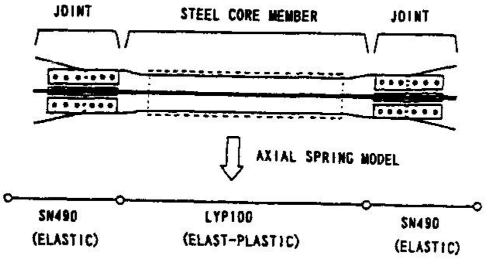

Fig 6 Modeling of Unbonded Brace

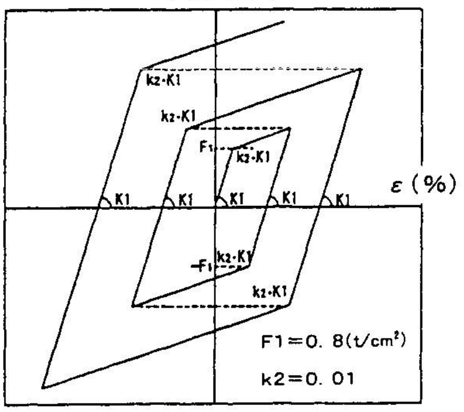

Fig 7 Hysteresis Curve of Low Yield Strength Steel (LYP100)

3.2 Time History Seismic Response Analysis

The time history seismic response analysis included: a shear-mode Lumped Mass System Response Analysis which was designed to grasp the response characteristics of the whole structure; a Plane Frame Response Analysis which was designed to grasp the elasto-plastic behavior of structural members (columns, beams, and braces).

In the Lumped Mass System Response Analysis, a model was defined that had 20 mass points (one for each story) on a rigid foundation. Internal viscous damping effect proportional to frequency was assumed, and the damping factor was assumed to be 2% for the first mode frequency. The maximum story drift angle for 25 and 50 kine responses were shown in Table 2 respectively, which are smaller than their respective target values, 1/200 and 1/100.

Table. 2 Maximum Story Drift angle by Lumped Mass System Response Analysis

Ground Velocity	X-direction	Y-direction
25 kine	1/245	1/208
50 kine	1/125	1/117

The Plane Frame Response Analysis was carried out using the wave recorded at the Hanshin-Awaji Earthquake (1995.1.17) with the velocity of 50 kine. Fig 8 shows the distribution of yield hinges produced in Y direction frame. The maximum ductility factor of beams was 1.16, and yield of columns was not observed. The maximum axial stress and strain of unbonded braces was 1.5 (t/cm^2), 0.005, respectively (Fig 9 and 10).

For investigating response at largest seismic level, the Plane Frame Response Analysis was carried out using the same wave with the velocity of 75 kine. The maximum ductility factor of beams was 2.6, the maximum strain of the steel core members in unbonded braces was 0.098, and yielding columns was not observed.

From the above results it was confirmed that the response values were within the target range of structural performance. The unbonded braces using low yield strength steel become plasticized even at moderate seismic level. At large seismic level, the unbonded braces absorb a large quantity of seismic energy.

Moreover, numerical analyses on unbonded braces fixed in a frame were made using the non-linear finite element method. The maximum strain of the end in steel core at maximum axial deformation was 0.016. It was confirmed that unbonded braces fixed in a frame can absorb a large quantity of seismic energy.

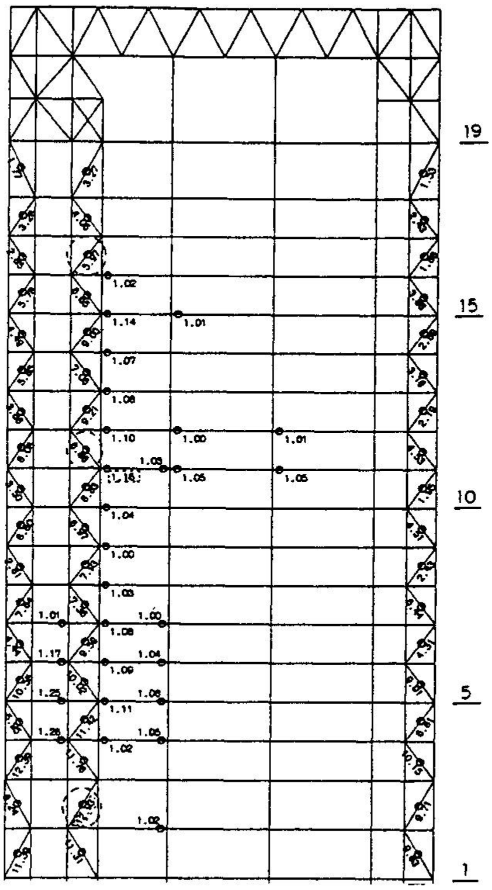

Fig 8 Distribution of Plastic Hinges (Y-dir. 50 kine)

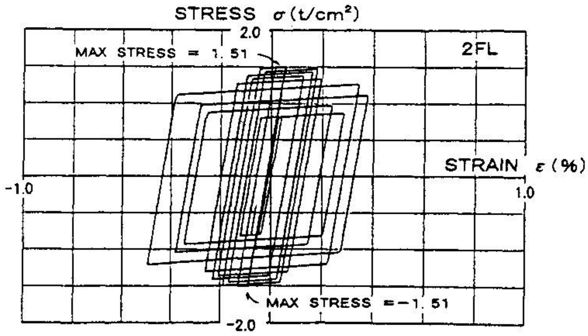

Fig 9 Stress-Strain Curve of Steel Core Member (50 kine)

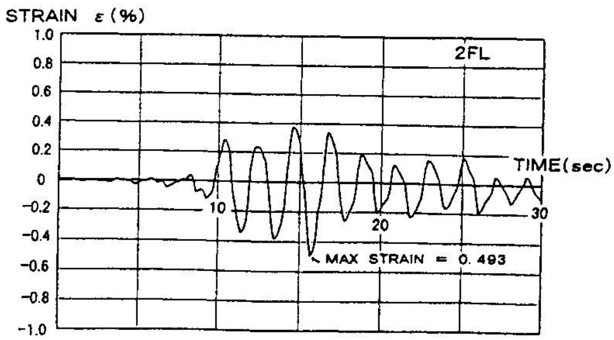

Fig 10 Strain Time Response of Steel Core Member (50 kine)

4. Conclusion

The authors designed 19-stories office building which unbonded braces using low yield strength steel was first extensively adopted. According to this attenuation mechanism by unbonded braces, columns and beams was little damaged at large earthquake. Moreover, after a large earthquake, unbonded braces can be replaced as required.

In view of the results of the seismic response analysis, the authors believe that the building designed according to the hysterisis damping effect by unbonded braces using low yield strength steel will prove to be excellent in seismic resistance.

In closing, the authors believe that this damage control design will help to develop 'Structural response control'.

References

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