# Earthquake and Impulse Response Tests on a Steel Frame Strengthened by Fully Mechanical Braces

K. Ohi[I], K. Takanashi[II], S. J. Lee[I], Y. Shimawaki[I], H. Ohtsuka[I] and H. Imazu[I]

**ABSTRACT:** Effectiveness of steel bracing system installed with fully mechanical interfaces is experimentally demonstrated through a series of monotonic and cyclic loading tests. Also, sub-structuring pseudo-dynamic earthquake response tests are performed on a possible situation of a two-story braced frame upgraded by the proposed bracing system. An earthquake record and theoretical impulses are adopted as input excitations. The results show that an impulsive excitation acts more stringently on the occurrence of brace breaking.

## 1. INTRODUCTION

In recent years after the Kobe earthquake, seismic diagnosis, retrofitting, and upgrading projects for existing buildings in public use have been widely carried out in Japan to prepare a destructive earthquake in the future. A steel gymnasium in school is one of such buildings and particularly important, because they are expected to serve as a refuge after a destructive disaster (Ohi, 1998). A main earthquake resisting element in the longitudinal direction of a gymnasium is a vertical bracing system, and its brace joints usually have too small strength to enable yielding in body portion of the bracing member. Then, a possible upgrading technique is to replace an old bracing member by a new one or to install an additional bracing system. This paper proposes a installation technique of such a bracing system with fully mechanical interfaces without use of site welding, and examines the performance of steel buildings upgraded by such a system experimentally through quasi-static and dynamic loading tests and sub-structuring hybrid simulations under seismic loading.

## 2. LOADING TESTS ON AN ASSEMBLY OF COLUMN, INTERFACE AND BRACE

A test setup used in the monotonic loading tests here-in is shown in Figure 1. The loading tests are carried out on an assembly of a short column piece, a proposed interface, and a single angle brace, where they are placed with an angle of 45 degree to the laboratory floor, and loading is applied horizontally considering actuator capacity. The detail of used columns and proposed interface are shown in Figures 2 and 3. Three kinds of H-shaped columns, H-500×200x10x16 (depth, width, web thickness, and flange thickness), H-400×200×8×13 made of JIS SN400B grade steel, and H-250×250×9×14 made of JIS SS400 grade steel, are prepared as column specimen.
The interface between a bracing member and an existing steel frame consists of a tee stub and a vertical stiffening plate, which are both directly bolted to the H-shaped column web. The tee stub is cut from a hot-rolled H-shaped cross-section, H-500x200x10x16 made of JIS SN400B grade steel. A brace member is a single angle cross-section, L-65x65x6(side lengths, thickness) made of JIS SS400 grade steel, and it is connected to the tee web through five high-strength bolts (JIS S10T, 20mm in diameter). This joint design just meets the condition specified in the AIJ recommendation (Architectural Institute of Japan, 1998), that is, the nominal maximum strength at joints is 1.25 times the nominal yield resistance of brace body. The mechanical properties derived from tension tests on coupons are summarized in Table 1. The load-deformation curves transformed along brace member axis are shown in Figure 4. As for the two cases of H-400 and H-500 not stiffened, their column webs deformed and yielded. However, their resistances increased gradually and the breaking at bolt-hole deducted portion of brace member occurred

[I] Institute of Industrial Science, University of Tokyo

[II] Professor Emeritus, University of Tokyo

finally. In these two cases, the column webs are damaged considerably, but the total deformation capacity until breaking is greater than other three cases. On the other hand, the results of the stiffened cases are stiffer and stronger than the cases not stiffened. As for the H-250 case, even if it was not stiffened, the web deformation is small and similar to the stiffened cases. This is explained by short distances between column-web bolts and column flange. The dotted levels show the yield resistances predicted from limit analysis based on the yield line theory.

**Table 1 Mechanical properties of material used**

| Position | | Yield stress ($N/mm^2$) | Tensile strength ($N/mm^2$) | Elongation (%) |
|---|---|---|---|---|
| Angle | (L-65×65×6) | 323 | 464 | 24 |
| Tee-web | (H-500×200×10×16) | 322 | 460 | 27 |
| Tee-flange | (H-500×200×10×16) | 281 | 445 | 32 |
| Column-web | (H-400×200×8×13) | 349 | 460 | 24 |
| | (H-250×250×9×14) | 319 | 451 | 28 |
| Bolt | (M20-S10T) | 892 | 1060 | 14 |

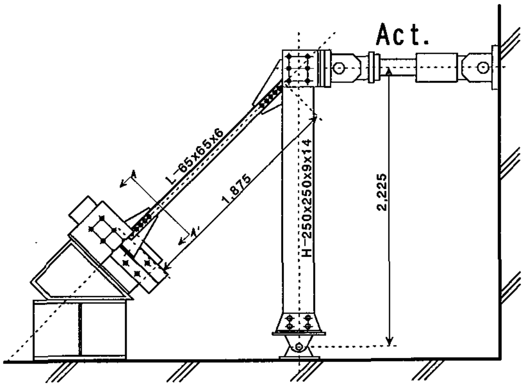


**Figure 1 Test setup of loading tests**

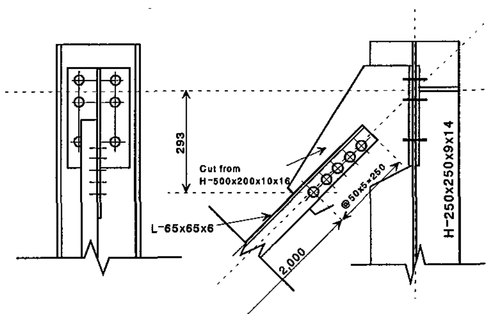


**Figure 2 Details of interfaces proposed**

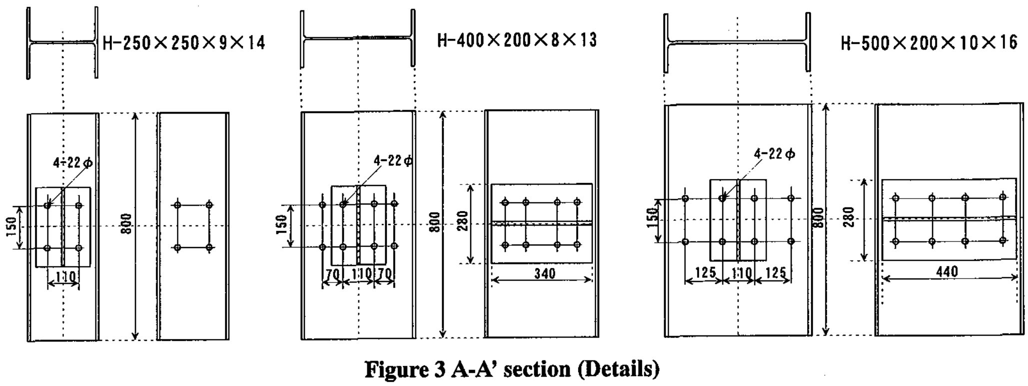
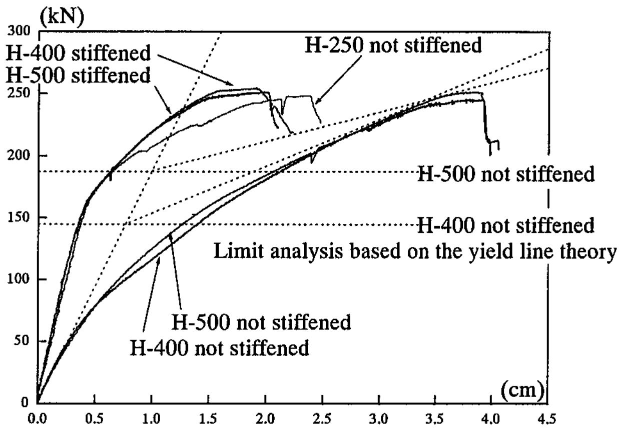
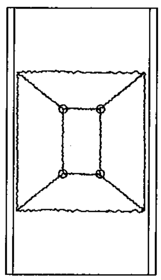


**Figure 3 A-A' section (Details)**



Yield line assumed to calculate dotted levels

**Figure 4 Test results**

## 3. QUASI-STATIC AND DYNAMIC LOADING TESTS ON BRACED STEEL FRAMES

A test setup commonly used in the loading tests on a braced frame as well as pseudo-dynamic tests is shown in Figure 5. A tested frame is simply supported on a rigid base block. H-shaped column is H-250 $\times 250 \times 9 \times 14$, identical to one of the columns tested in the previous section. In the previous test, small web deformation was observed, and then a stiffening plate is added here-in.

Figure 6 shows the results of loading tests on the braced frame specimen. Quasi-static and dynamic loading tests are performed under a monotonic loading program as well as a cyclic loading program. All the braces were broken at the bolt-hole deducted portion of the angle member. Commonly in these loading tests, the column web plates are not damaged at all, and the proposed interface behave fairly well until the final failure.

In the dynamic tests, the velocity of loading apparatus is kept constant as fast as 50 cm/s (0.25 radian/s in the speed of story drift angle). The yield resistance of the bracing system under dynamic loading was found to be slightly greater than that observed under quasi-static loading, while the deformation capacity before breaking was considerably smaller. This is resulted from the increase of yield stress due to strain rate, which prevents yielding along the body-portion of angle member. Of course, the tensile strength at the bolt-hole deducted portion may increase due to strain rate, but it is not so great compared with the increase of the yield stress.

Under the cyclic reversals, the final deformation at breaking was slightly greater than the deformation under monotonic loading. Once a buckled angle member undergoes considerable flexural yielding in the middle portions, the plastic deformation does not recover to the straight line even if it is again pulled back to the former unloaded point. This phenomenon generates a small delay of recovering to the plastic tensile resistance after it reaches to the former unloaded point. Summation of these small delays slightly postpones the final breaking at the bolt-hole deducted portion in total.

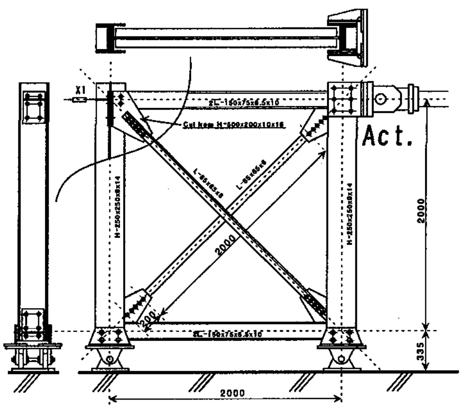



Figure 5 Test setup

**Table 2 Loading speed**

| | | |
|---|---|---|
| Monotonic loading | Quasi-static | 0.1 (cm/s) |
| | Dynamic | 50 (cm/s) |
| Cyclic loading | Quasi-static | 0.1 (cm/s) |
| | Dynamic | 50 (cm/s) |

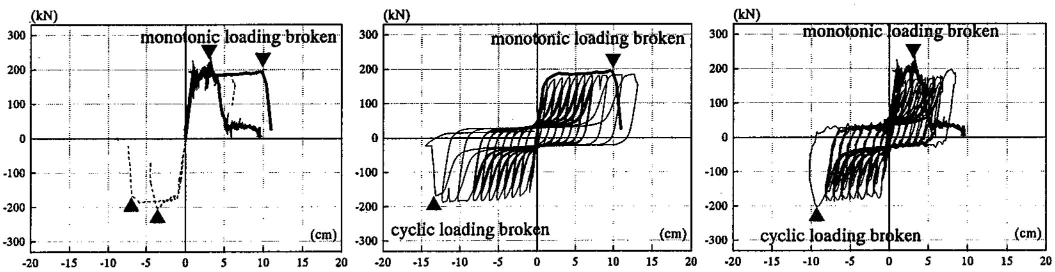



**Figure 6 Loading test results**

To remove such a delay, so-called 'skeleton curves' are identified from the cyclic test results in the following manner: only the portions of cyclic curves that exceed the previous resistance at last unloaded point are connected to make a skeleton curve. The skeleton curves identified are shown as dotted curves in Figure 6 and compared with the solid monotonic curves, quasi-static and dynamic, respectively. It is found that the deformation capacity in such a skeleton curve gives almost the same level with monotonic curve.

## 4. HYBRID RESPONSE SIMULATION COMBINED WITH LOADING TESTS

Hybrid responses of a 2-story fictitious frame were simulated by use of sub-structuring technique, where a computer-controlled loading tests is performed only on an additional bracing portion at the second story, where a bracing system is installed by use of the interface proposed. The profiles of the remaining portions of the frame other than the specimen are assumed to stay in possible ranges that is experienced in ordinary school gymnasiums. The overall hybrid model is illustrated in Figure 7. This 2-story frame represents a structural system in the longitudinal direction of an ordinary school gymnasium in Japan. The first story is a reinforced concrete wall structure, and the second story is steel bracing system. As for the first story, the amount of the reinforced concrete wall is enough and assumed to remain in elastic range. An existing bracing system at the second story has insufficient strength at its joint, and premature breaking will occur at the brace joint. To upgrade the seismic performance of the second story, an additional bracing system is installed with the interface proposed. Only a pair of additional new braces and its surrounding frame are loaded by computer-controlled actuator, and the restoring force measured from the test is reflected in the response analysis of the overall structural system after combined with the fictitious restoring forces of the remaining portions. The behavior of the fictitious old pair of braces is assumed to follow the hysteresis rule illustrated in Figure 8. At the moment that the resistance of the tension-side brace reaches to the yield strength, the brace is broken, and after breaking the compression-side brace carries only small post-buckling resistance. The yield resistance and the elastic stiffness are arranged as the same values with those of specimen bracing system. The situation is that the existing story resistance and stiffness are doubled by adding a new ductile bracing system. The dynamic properties of the overall system after upgrading is summarized in Table 4.

As shown in Table 3, two kinds of excitations are applied to the hybrid system: a theoretical impulse and an earthquake ground motion, which is the N-S component of El Centro 1940. The peak ground acceleration of El Centro 1940 is modified to 400 cm/s$^2$ and the record of 10 s in duration is used in the hybrid test. No scaling in time axis is considered. Figure 9 shows the response spectrum of energy input (in velocity expression) exerted by the modified earthquake record. Two theoretical acceleration impulses are also arranged to generate kinetic energy as much as the half of the average velocity level (50cm/s) and almost the same average level (100 cm/s) of the earthquake energy input. In the hybrid tests to theoretical impulses, free vibration is simulated from the condition that the initial velocity at each floor mass is set to the value, 50cm/s or 100cm/s.

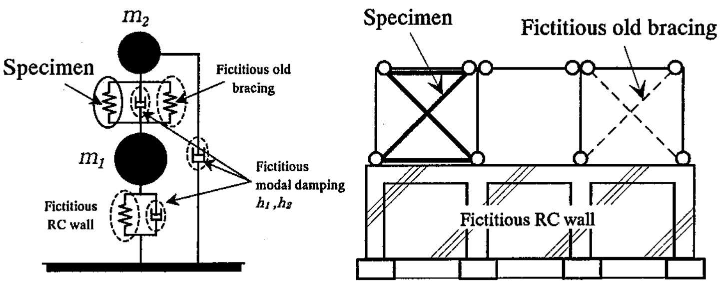



**Figure 7 Hybrid model analyzed**

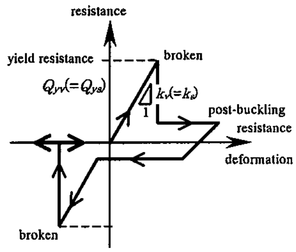



**Figure 8 Hysteresis model of fictitious old bracing**

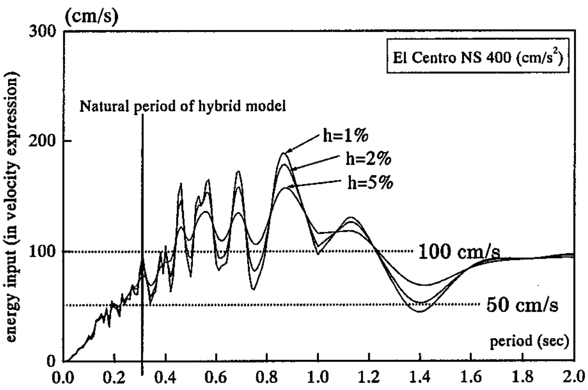



**Figure 9 Response spectrum of energy input**

(in velocity expression),El Centro NS 1940 400 cm/s$^2$

**Table 3 Input excitation used in the hybrid tests**

| Excitation | Magnitude arranged |
|---|---|
| El Centro NS 1940 | 400 cm/s$^2$ in PGA |
| Acceleration impulse I | 50 cm/s in velocity expression of energy input |
| Acceleration impulse II | 100 cm/s in velocity expression of energy input |

**Table 4 Dynamic properties of hybrid model**

| Mode | Natural Period (s) | Participation vector | | Model damping |
|---|---|---|---|---|
| 1 | 0.3 | (1.125 0.067) | | 0.02 |
| 2 | 0.1 | (-0.125 0.933) | | 0.02 |
| $k_1/k_2$ | $m_1/m_2$ | $k_{ES}$ (kN/m) | $Q_{YS}$ (kN) | $Q_{Y2}/m_2G$ |
| 15.65 | 2 | 19600 | 163 | 0.4 |

where $k_1/k_2$ :Story stiffness ratio, $m_1/m_2$ :Inertial mass ratio, $k_{ES}$ :Specimen elastic stiffness, $Q_{YS}$ :Specimen yield resistance, $Q_{Y2}/m_2G$ : Yield shear coefficient of the second story

Figures 10 and 11 show the time histories of the story drift at each story. Figure 12 shows the overall hysteresis loops at the second story. During these responses, the reinforced concrete wall at the first story remains elastic. Commonly in all the tests, fictitious old braces are first broken. In the response to El Centro 1940, both side of old braces are broken, while one of fictitious braces is broken to the impulse cases. Impulse responses are similar to monotonic loading process, but some higher (the second) mode vibration is mixed as observed in the impulse response to 50 cm/s. Additional bracing system works well in the response to El Centro 1940, and the upgraded system did not collapse, while the system collapsed due to the breaking of the additional brace against the impulse input of 100 cm/s. The reasons may be as follows: First, in the response to impulse, the energy is absorbed in a similar way to the monotonic loading, then the peak displacement becomes greater than the peak response to an earthquake when the amount of total energy to absorb is in the same level. Second, as experimentally shown in the previous section, apparent deformation capacity of bracing system under cyclic reversals is slightly greater than the deformation capacity under monotonic loading. Thus, impulsive excitation acts more stringently on the occurrence of brace breaking.

It is difficult, however, to discuss how much impulsiveness shall be considered in the earthquake resistant design of steel frames. It depends on the characteristics of earthquake motions considered in the design. For instance, Ohi (1991) observed two moderate events that cause two different inelastic responses in a braced frame model for seismic monitoring, as shown in Figure 10. The peak ground acceleration is almost the same in the two events, but one epicenter is located near to the observation site, while the other is located offshore in the ocean and relatively far from the observation site. In the former event, the hysteresis behavior seems almost a monotonic loading test result. With the high possibility of such an earthquake as the former event, we could not rely on the advantage of cyclic reversals.

**Table 5 Input energy converted into velocity**

| | El Centro NS 1940, 400 (cm/s$^2$) in PGA | Acceleration impulse I | Acceleration impulse II |
|---|---|---|---|
| Input energy converted into velocity | 188 (cm/s) | 50 (cm/s) | 100 (cm/s)<br>38 (cm/s)* |

*: Energy dissipation of the second story until collapse

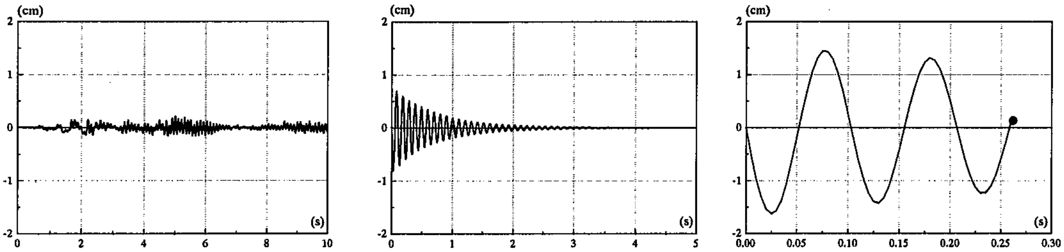


**Figure 10 Time histories of story drift at the first story**

(left :El Centro NS 1940 400cm/s$^2$, middle :impulse 50 cm/s, right :impulse 100 cm/s)

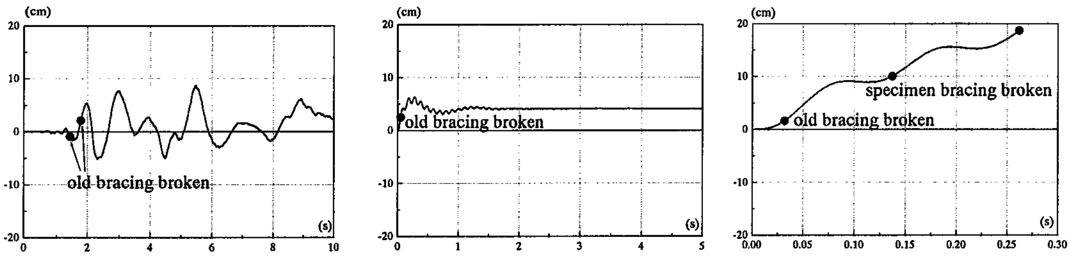



**Figure 11 Time histories of story drift at the second story**

(left :El Centro NS 1940 400cm/s$^2$, middle :impulse 50 cm/s, right :impulse 100 cm/s)

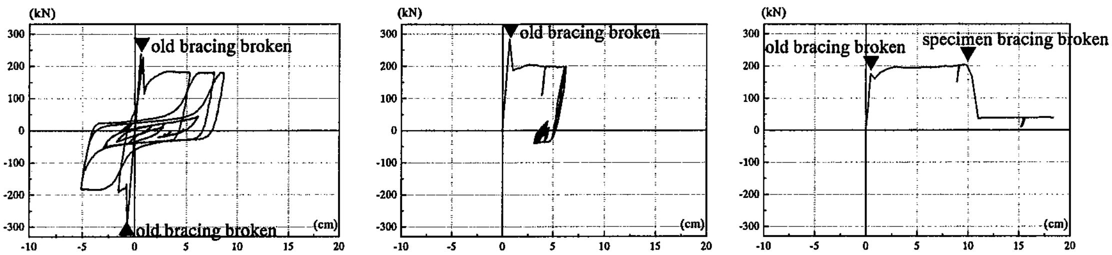



**Figure 12 Hysteresis loops at the second story**

(left :El Centro NS 1940 400cm/s$^2$, middle :impulse 50 cm/s, right :impulse 100 cm/s)

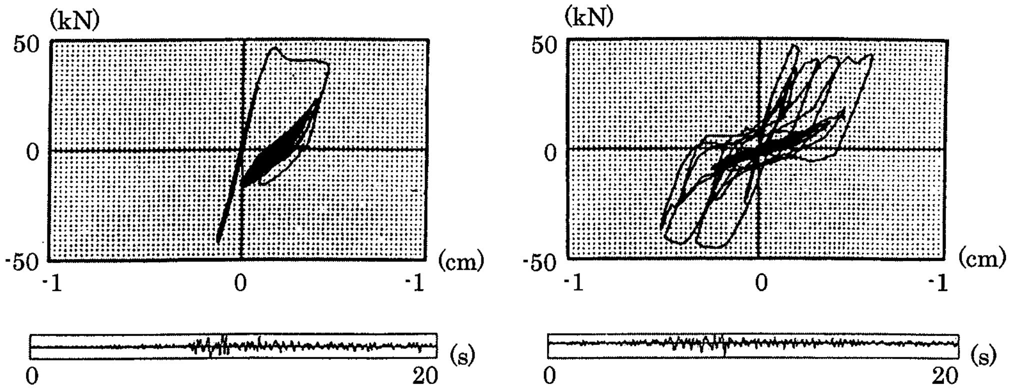



**Figure 13 Real inelastic responses of a braced frame to tow natural earthquakes**

(left: near-field event, October 1984 peak : $71cm/s^2$, right: far-field event, June 1986 peak : $74cm/s^2$)

## 5. CONCLUDING REMARKS

(1) Fully mechanical interface for additional bracing system is possible and behaves well to seismic loading. Such a choice of details will avoid a premature failure in the vicinity of site welded portions.

(2) The loading test results and the hybrid simulation results show that the occurrence of the breaking at the bolt-hole deducted portion is affected by loading speed and by the impulsiveness of the excitations as well. Further studies are needed to clarify these effects.


## ACKNOUWLEDGMENT

A part of this research is financially supported by R&D committee on Semi-rigidly Jointed Steel Frames, Kozai Club.


## REFERENCES


Architectural Institute of Japan: Recommendation of limit state design for steel structures, 1998

K.Ohi, K.Takanashi, Y.Homma: "Energy input rate spectra of earthquake motions," Journal of structural and constructional engineering, Transaction of AIJ, No.420, 1991.

K.Ohi and K.Takanashi: "Seismic diagnosis for rehabilitation and upgrading of steel gymnasiums," Engineering Structures, Vol.20, Nos.4-6, 1998.