Pseudo-dynamic test on full-scale model of first-storey energy-concentrated steel structure with flexible-stiff mixed frames

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ABSTRACT: A new earthquake-resistant steel structure based on energy theory is presented. Before construction of an actual building to which this structural system was applied, a pseudo-dynamic test on an full-scale flexible-stiff mixed frame of the building was carried out. As a result, it was verified that the building satisfies an expected performance of this system in regards to energy absorption and affords sufficient seismic safety even during a severe earthquake.

1 INTRODUCTION

We have proposed a new earthquake resistant steel structure system, i.e.,the "first-story energy-concentrated structure with flexible-stiff mixed frames" based on the earthquake resistant limit design method derived from an energy theory. This structure aims to drastic reduction of earthquake input energy to the 2nd and upper stories by absorbing most of the total energy input exerted by a severe earthquake to a building at specific members of the first story. For this purpose, the yield-shear force coefficients of the 2nd and upper stories are set to be larger than that of the first story, and a flexible-stiff mixed structure is built in the first story. Stiff members absorb most of the earthquake input energy. Flexible members maintain elasticity and restoring force to restrict response deformation even under the severe earthquake.

Thus far, fundamental response characteristics of this structure had been studied from the energy theory viewpoint, by inelastic response analyses and inelastic shaking table tests on scaled models.

Recently, we had an opportunity to apply this system to an actual building. And to confirm behavior of an actual structure at the time of a severe earthquake, especially the energy absorption, we carried out a pseudo-dynamic tests on a full-scale flexible-stiff mixed frame of the first story which was a partial structure of the objective building.

2 OBJECTIVE BUILDING AND MODELLING

2.1 Objective building

The objective building is an experimental research facility applying the "first-story energy-concentrated structure with flexible-stiff mixed frames" (completed in September, 1988), which is a 5-story building with 2x2 span constructed on a type 3 ground(see Figure 1). The first story is designed as a flexible-stiff mixed structure composed of stiff members and flexible members. Stiff members are made of mild steel with large energy absorption capacity. The stiff members are selected so that they can absorb all the input energy of a severe earthquake. Tied beams and tied braces for the top of stiff members are designed using stiffer members than the stiff members so that interstory deformation of the first story is transmitted to the tops of the stiff members. Also, tied beams for the bottoms of the stiff members as well use stiffer members. The flexible members, which also function as main columns, are made of high-strength steel to afford large elastic deformation capacity and bearing capacity to vertical load.

The 2nd and upper stories are designed as an ordinary braced structure.

In order to concentrate most of input energy by a severe earthquake to the first story, distribution of yield-shear coefficients of these stories are designed to be 1.5 times or more than the optimum yield-shear coefficient distribution (see Figure 2).

2.2 Modelling

The dynamic model is made as a 5 mass system using equivalent shear springs. The dynamic properties are shown in Table 1.

In conducting the earthquake response test of the actual frames of the objective building, a partial framework with reduced degrees of freedom is adopted because of loading capacity of the testing apparatus and restriction of memory capacity of the control computer.
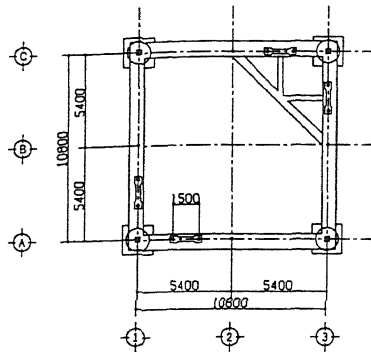
(a) Framing Plan of 1st Story (unit: mm)
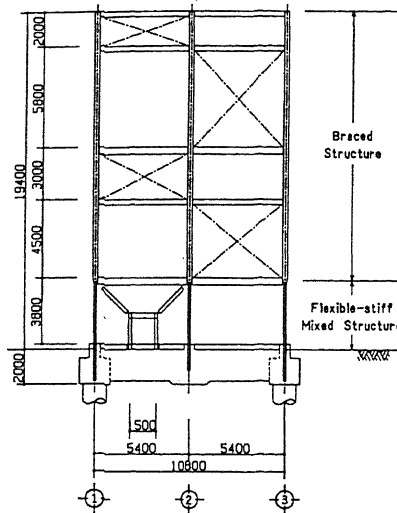
(b) Framing Elevation of Frame A (unit: mm)

Figure 1. Framing plan and elevation of objective building

As shown in Figure 1, the mass, stiffness and strength of in-plane frames A, C, 1 and 3 are nearly equal to each other, and in-plane frames B and 2 are not expected to resist earthquake load in design. The in-plane frame A was chosen as an objective of the test.

Since it is confirmed by an earthquake response analysis that the 2nd and upper stories of this building reamin elastic even during a severe earthquake, the building is reduced to a 2 mass system by substituting an equivalent 1 mass for stories from the 2nd story up to the 5th story. Since the total energy input mainly depends on the total mass and fundamental natural period of the building, a 5 mass system is substituted to a 2 mass system so that the total mass and fundamental natural period of both systems may be equal. As a result, the mass and equivalent stiffness of the 2nd story of the 2 mass system are $M_2 = 0.2049$ tf.sec^2/cm and $_2k_{eq} = 25.59$ tf/cm, respectively. The dynamic properties of the first story of both systems
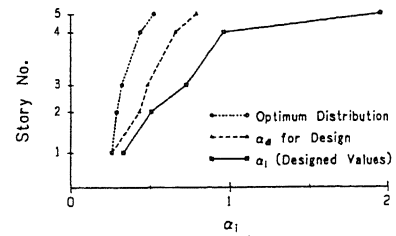

Figure 2. Yield-shear coefficient distribution of objective building

Table 1. Dynamic properties of objective building

Story No.		Story Weight cm	Stiff -ness tf/cm	Yield- Defor- mation cm	Yield- Shear Force tf
5		19.2	43.28	0.86	37.2
4		72.3	31.59	2.78	87.8
3		63.2	99.39	1.13	112.3
2		117.6	92.85	1.49	138.4
1	Flex.	129.3	5.00	14.95	74.8
	Stiff		63.86	0.91	58.1
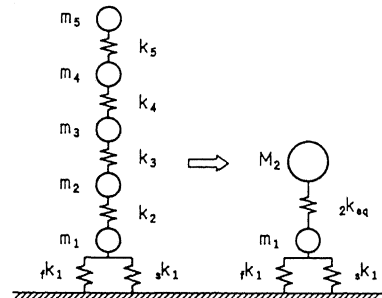
Figure 3. Modeling of objective building

$Qc = 2Qy/3$
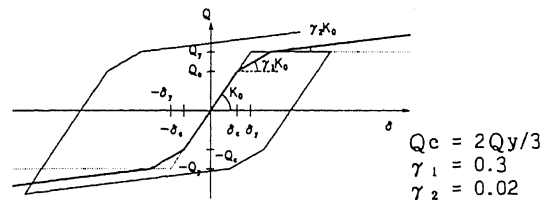$\gamma_1 = 0.3$
$\gamma_2 = 0.02$

Figure 4. Restoring-force characteristics of stiff members of first story

are the same (see Figure 3).

In order to check whether the 2 mass system model shown above can be appropriate, earthquake response analyses are conducted and the results are compared with the response results of the 5 mass system.

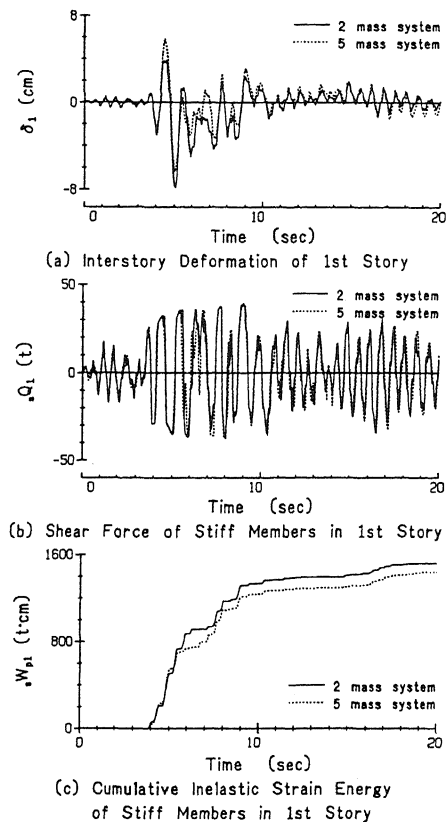A tri-linear type, shown in Figure 4, is

(a) Interstory Deformation of 1st Story

(b) Shear Force of Stiff Members in 1st Story

(c) Cumulative Inelastic Strain Energy
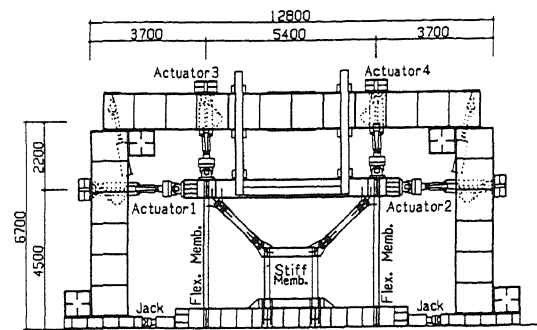of Stiff Members in 1st Story

Figure 5. Time histories of responses

Figure 6. Test specimen and loading system

strain energy of stiff members, slight difference can be seen between both in the range 6.0 - 20.0 sec. However, the difference is 5.7% at most at the end of an earthquake, and it is mainly due to difference of the total energy input.

As described above, response results of both systems coincide favorably with each other in spite of difference of the number of mass points.

3 TEST METHOD

3.1 Specimen

The 2 mass system model as described in Section 2 is chosen as a specimen. The behavior of the 2nd story is numerically simulated with the use of an analysis program. The first story of the designed building is composed of 2 stiff members and 3 flexible members. However, the flexible members are confirmed to remain elastic even during a severe earthquake as the result of the response analysis. Beacuse of that, one flexible member is omitted, to which no stiff members are attached, and evaluation is performed only by the numerical analysis program. After all, as shown in Figure 6, the specimen of this test is designed as a partial structure of the objective building, which is composed of 2 flexible members, 2 stiff members, 2 tied beams and 2 tied braces which connect stiff members, and the 2nd story girder and a footing beam. All members of the specimen are the same members as those used for the objective building, except for the 2nd story girder and a footing beam. Flexible members are made of high-strength steel HT80(Yield stress σ_y = 8.81tf/cm^2, Ultimate stress σ_u = 9.15tf/cm^2, Ultimate strain ε_u = 24%) and seamless steel pipes (ϕ 153.5 x 25.1 mm), and stiff members are seamless pipes (ϕ 165.0 x 18 mm) made of mild steel STK400(σ_y = 2.78tf/cm^2, σ_u = 4.72tf/cm^2, ε_u = 38%), and other members are made of SS400(σ_y = 2.70tf/cm^2, σ_u = 4.31tf/cm^2, ε_u = 32%).
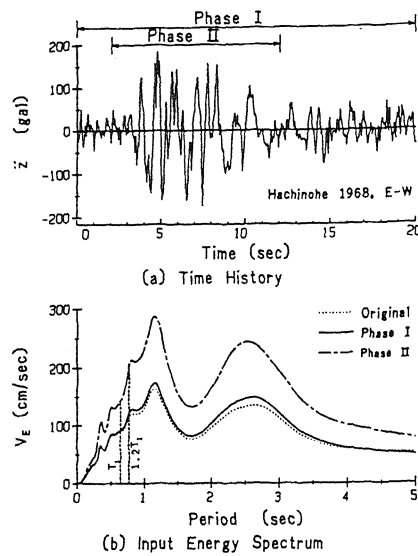
The 2nd story girder is made of H-shaped

used for the restoring force characteristic of stiff members, which takes into account strain hardening and Bauschinger effect. A elastic perfectly plastic type is used for flexible members. The numerical analysis is carried out using the Newmark-β method (β = 1/4), and damping is disregarded.

Taking into account the ground condition of the construction site, the major portion(0.0 - 20.0 sec) of Hachinohe 1968 EW wave is employed as an input earthquake wave, and 120 cm/sec is set as the pseudo-velocity of the total energy input of the severe earthquake level. The maximum input acceleration is 205 cm/sec^2. The time increment Δt is set to be 1/200 sec.

As the result, the total energy input (pseudo-velocity V_E) is 119.6 cm/sec for the 5 mass system and 122.6 cm/sec for the 2 mass system, showing favorable coincidence with difference of 2.4% only.

Figure 5 shows the time histories of the responses of the first story of both systems.

For the response interstory deformation of both, waveform of the 2 mass system shifts to the negative side in the range 4.5 - 8.0 sec in the 5 mass system. However, they almost coincide. Response shear force of stiff members coincides very much for both and with respect to time. As for cumulative inelastic

(a) Time History

(b) Input Energy Spectrum

Figure 7. Input earthquake wave

(a) Interstory Deformation of 1st Story

(b) Shear Force of Stiff Members in 1st Story

Figure 8. Time histories of responses

section. Its flexural stiffness is equal to a composite girder of the objective building, which is composed of a H-shaped steel girder and RC slab.

3.2 Loading method

The motion equation of the test model is given in a time increment form as follows.

$$[M]\{\Delta \ddot{y}_n\}+\{\Delta F_n\}=-[M]\{1\} \; \Delta \ddot{z}_n \qquad (1)$$

In these equations, left subscripts indicate the story (mass) number. $_1F_n$ is horizontal force obtained by 4 actuators, and $_1k_f$ is stiffness of the virtual flexible member of the first story and it is equal to a value of one flexible member determined by pre-loading of the specimen. $_1k_{p-\Delta}$ is a negative stiffness due to P-Δ effect effected by building weight shared by the virtual flexible

$$_2F_n= \; _2k_{eq} \; (_2y_n-_1y_n) \qquad (2)$$

$$_1F_n= \; _1F_b+(_1k_f+_1k_{p-\Delta})_1y_n-_2F_n \qquad (3)$$

where, $[M]$, $\{y\}$ and $\ddot{z}$ are the mass matrix, relative displacement vector and input earthquake acceleration, respectively. $\{F\}$ is the restoring force vector and given by a test result. In this test, the damping term is disregarded. A right subscript n represents a time step. The numerical analysis uses the central difference calculas.

In this test, one flexible member of the first story and each of the 2nd story are taken as virtual specimens. In order to evaluate them by the numerical analysis program only, restoring force of the equation (1) is set as follows.
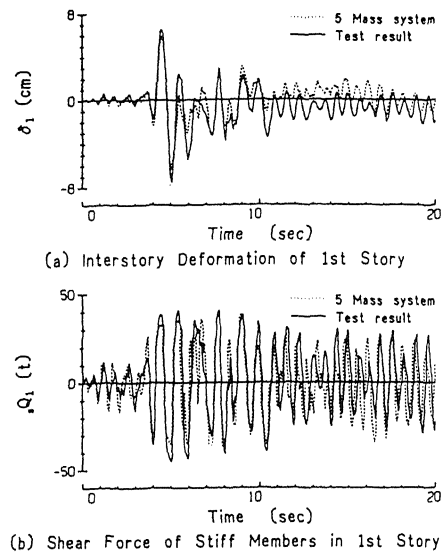
member. Thus, $_1k_f$ =0.938 tf/cm and $_1k_{p-\Delta}$ =-0.189 tf/cm.

The loading system is also shown in Figure 6. For vertical design load shared by each of flexible members subjected to long-term load(right column 89.9t, left column 53.4t), the load is applied by actuators No. 3 and No. 4.

The application of the load to the specimen is controlled in the following way.

The information about the value of the displacement, which is obtained by a numerical analysis, is given to the actuator No. 1. At the same time the information about the force which is necessary to cause the displacement by actuator No. 1 is sent to actuator No. 2. Also, horizontal force component of the actuators No. 3 and No. 4 is input to $_1F_b$ during the numerical analysis for the pseudo-dynamic test for control.

The input earthquake wave is Hachinohe 1968 EW wave the same as in Section 2.2 (see Figure 7). The time incrementΔt is 1/200 sec considering the stability of the solution.

Loading of the pseudo-dynamic test is performed for the following 2 phases.

1) Phase I: A severe earthquake level in the design; pseudo-velocity of total energy input is V_E = 120cm/sec, and the duration time is t_0 = 20 sec. The maximum input acceleration is $\ddot{z}_{max}$ = 205 cm/sec^2.

2) Phase II: An input level where flexible members reach their elastic limit; V_E = 200 cm/sec, t_0 = 10 sec and $\ddot{z}_{max}$ = 350 cm/sec^2.

4 TEST RESULTS AND DISCUSSION

In this paper, only the results of Phase I are discussed.
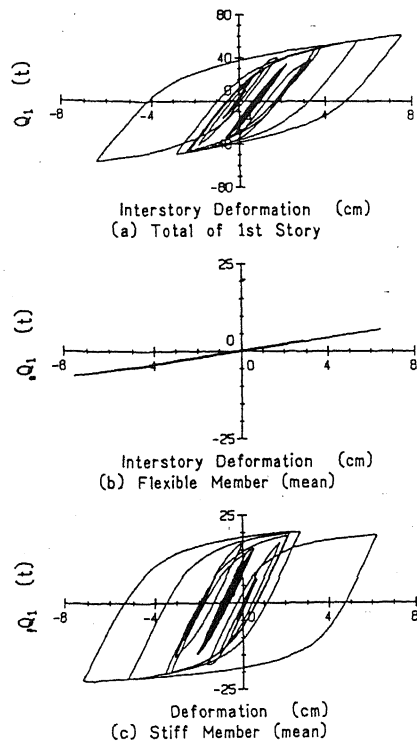
Figure 8 shows the time history of response

Interstory Deformation (cm)
(a) Total of 1st Story

Interstory Deformation (cm)
(b) Flexible Member (mean)

Deformation (cm)
(c) Stiff Member (mean)

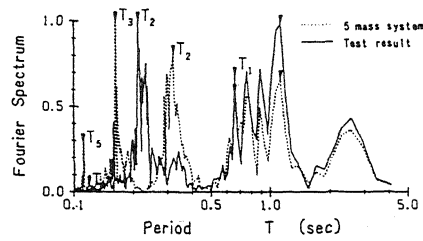Figure 9. Shear-force vs interstory deformation relationship

Figure 10. Fourier spectrum of response displacement of first story
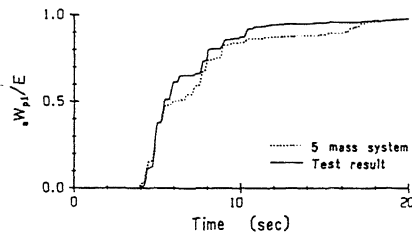
Figure 11. Time history of cumulative inelastic strain energy of stiff members of first story

interstory deformation and response shear force of stiff members. For the response interstory deformation, the maximum deformations of the positive and negative sides are 6.47cm and -7.54cm, respectively. These values are a little larger compared with 5.86cm and -6.40cm as values obtained by the analysis of the 5 mass system, and drift to the negative side from 10 sec and afterwards, but the test result and analytical result show favorable correspondence. For the response shear force of stiff members, test result is slightly larger than the analytical result of the 5 mass system because the yield point of stiff members is larger than the design value and increase of strength due to strain hardening is large, but the response waveforms coincide very well.

Figure 9 shows the shear force-deformation relationship of the entire of the first story and of stiff members and flexible members. A stable loop with large energy absorption is plotted for stiff members. The plastic zone develops to the 25cm range from their ends before the end of testing and the maximum strain value at points 5cm inside from their ends reach $20802\,\mu$, but no deterioration phenomena like cracking or local buckling are observed. The shear force-deformation relationship of flexible members shows an elastic behavior, the maximum strain value is

$2391\,\mu$ at points 5cm inside from their ends and there is sufficient surplus of 57% to the elastic limit value, $4180\,\mu$, of the material.

Figure 10 shows a Fourier spectrum of response displacement of the first story. A peak appears in the secondary natural period of the specimen in the short-period region, unlike the property of the analytical result of the 5 mass system, but the value is very small. On the other hand, in the fundamental natural period (0.660 sec) and longer period region, both the test result and the analytical result present peaks which indicate expansion of the period accompanying the plasticization of stiff members, and both correspond very well with each other. Influence of a substantial natural period, which is a little longer than the fundamental natural period, is predominant, and the test result coincides very well with the analytical result of the 5 mass system.

Hence, test results are evaluated from the energy viewpoint.

The pseudo-velocity V_E of the total energy input is 123 cm/sec and coincides very well with a forecast analytyical value.

The total sum of absorbed energy is 1490 t.cm and it well agrees with the total energy input, 1554 t.cm, with an error of -4.2%.

The ratio of energy absorption of stiff members to the total absorbed energy is 98.6% and indicates that most of energy is absorbed.

Thus, most of the total energy input during a severe earthquake is absorbed as cumulative inelastic strain energy of stiff members, and elastic vibrational energy can be disregarded.

Figure 11 shows the time history of the cumulative inelastic strain energy of stiff
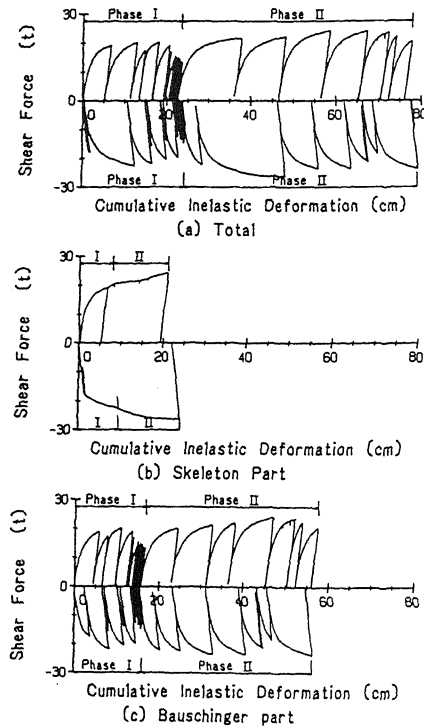
Figure 12. Cumulative inelastic deformation of stiff members of first story

members. The test result coincides well with the analytical result of the 5 mass system.

Figure 12 (a) shows the shear force-cumulative inelastic deformation(CID) relationship of stiff members separately on the positive and negative sides. The values of CID of the positive and negative sides are 22.4cm and 23.8cm, respectively and are almost equal. The residual deformation, 1.4cm, i.e., difference of both values, is small.

Figure 12 (b) and (c) show a skeleton part and a Bauschinger part of Figure 12 (a). For the skeleton part, the values of CID on the positive and negative sides are 6.4cm and 7.9cm, respectively. For the Bauschinger part, the values of CID on the positive and negative sides are 34.2cm and 36.0cm, respectively. Scale of the CID quantity of the Bauschinger part is 2.16 times that of the skeleton part.

For the cumulative inelastic strain energy of stiff members, the quantity is almost equal both on the positive and negative sides, presenting very efficient energy absorption. For each member as well, the absorbed energy of the Bauschinger part is 1.5 - 2.0 times as large as that of the skeleton part.

5 CONCLUSIONS

To confirm the actual behavior of the

structural system, we carried out a pseudo-dynamic test on an actual framework of a building to which this system was applied. As a result, the following are verified.

1) The behavior of an entire building during an earthquake can thoroughly be simulated by a pseudo-dynamic test using a full-scale substructure of the 5-story designed building.

2) The stiff members of the first story can absorb most of earthquake input energy to a building as a cumulative inelastic strain energy. The flexible members remain elastic.

3) The quantity of CID and the cumulative inelastic strain energy are almost equal on both the positive and negative sides.

4) For stiff members, energy absorption of the Bauschinger part is approximately twice as large as that of the skeleton part, and it can improve the energy absorption capacity of members.

5) The objective building shows stable behavior with much ductility even against an input energy of a severe earthquake. This structural system satisfies sufficient seismic safety.

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