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DYNAMIC RESPONSE OF THE NAMIOKA TOWN HOSPITAL BUILDING DAMAGED DURING THE 1983 NIHONKAI-CHUBU EARTHQUAKE

Masaomi TESHIGAWARA[1], Masaya HIROSAWA[2], Yoshikazu KITAGAWA[3], and Yutaka YAMAZAKI[4]

[1] Research Engineer, Production Dept. Building Research Institute, Ministry of Construction, Tsukuba, Japan.
[2] Director of IISEE, BRI, MOC, Tsukuba, Japan.
[3] Head of IISEE, BRI, MOC, Tsukuba, Japan.
[4] Head of Production Dept. BRI, MOC, Tsukuba, Japan.

SUMMARY

On May 26, 1983, an earthquake, named the Nihonkai-Chubu Earthquake, hit the Tohoku area of Japan, and the Namioka Town Hospital building, in Aomori-Prefecture, sustained a great deal of destruction by this earthquake. Seismic damage features of this hospital building with non-ductile members is explained by means of the dynamic response analysis in which the load-deformation characteristics of non-ductile members were appropriately represented by the strength-degrading model.

INTRODUCTION

On May 26, 1983, the Nihonkai-Chubu Earthquake, hit the Tohoku area of Japan. The magnitude of this earthquake was 7.7, and the epicenter of the earthquake was about 100km west of the sea coast. The Namioka Town Hospital, a five-story building, is one of a few reinforced concrete building which sustained a great deal of destruction by this earthquake. The third and fourth stories of this building were severely damaged to the extent that emergency repair and strengthening was required.

OUTLINE OF NAMIOKA TOWN HOSPITAL BUILDING

Outline of Structure The hospital building is located in Namioka-town, Aomori-prefecture, about 18km southwest of Aomori-city and about 150km east of the epicenter. The construction of this building started in 1968 and completed in 1970. Therefore, the building did not experience the 1968 Tokachi-Oki Earthquake. A perspective view and a key plan of the structure are shown in Figs. 1 and 2, respectively.

The Namioka Town Hospital building consists of two parts ; a one-two story low-rise part (A- to F-Frames) and a five story high-rise part (G- to J-Frames) with a one story basement. Since there is no expansion joint between the two parts, both parts are considered to be structurally connected with each other. Foundations of this building are supported by individual RC piles on a sandy layer. A list of cross sections of typical RC columns, walls and beams is shown in Fig. 3. Mechanical properties of materials used in this building is tabulated in Table 1. These values were obtained from testing of concrete cylinders and reinforcing bars taken out from the building.

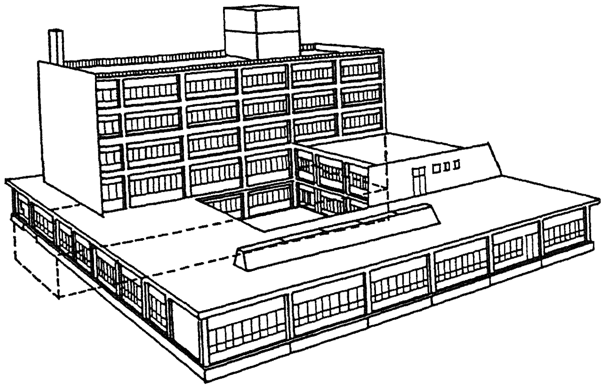


Fig. 1 Perspective View

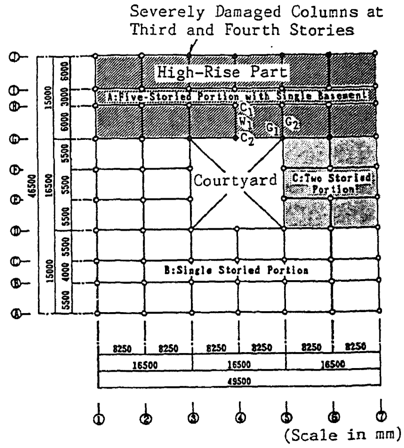


Fig. 2 Key Plan

Story		Columns C1	Columns C2
5	Section	500 × 500	500 × 500
	Main Bars	10−D22	8−D22
3	Section	600 × 600	600 × 600
	Main Bars	14−D22	12−D22
1	Section	700 × 700	700 × 700
	Main Bars	12−D22	12−D22

Story	Walls	W_1
5	Section	t=120
	Vertical	9φ @300
	Horizontal	9φ @300
	Corner	2−13φ
3	Section	t=120
	Vertical	@300
	Horizontal	9φ @300
	Corner	2−13φ
1	Section	t=200
	Vertical	9φ @250
	Horizontal	9φ @250
	Corner	2−D19

Story		G_1 End	G_1 Center	G_2 G J End	G_2 Center	G_2 H I End
R	Section	300 × 800		300 × 800		
	Top	4−D22	2−D22	3−D22	2−D22	4−D22
	Bottom	2−D22	3−D22	2−D22	6−D22	2−D22
4	Section	350 × 800		350 × 800		
	Top	6−D25	2−D25	6−D25	3−D25	5−D25
	Bottom	3−D25	2−D25	3−D25	4−D25	2−D25
2	Section	400 × 900		400 × 900		
	Top	7−D25	3−D25	6−D25	3−D25	6−D25
	Bottom	4−D25	3−D25	3−D25	3−D25	3−D25

Note : Hoop 9φ @200 Top & Base 9 φ @100
Diagonal Hoop 9φ @600

Note : Stirrup 9φ @300 Center
9φ @200 End

(Scale in mm)

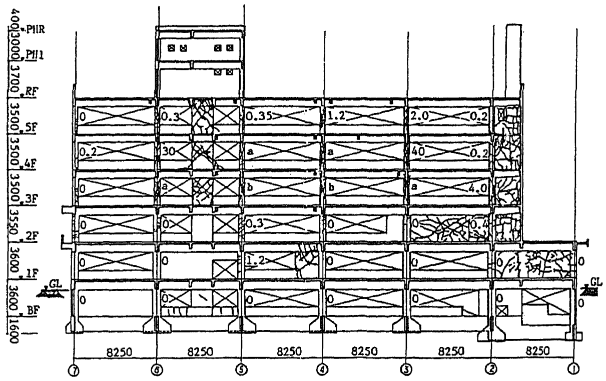
Fig. 3 List of Typical Members

Table 1 Mechanical Properties of Materials

Reinforcing Bars		Concrete	
Type	σ_y (kg/cm^2)	Fc(kg/cm^2)	Ec(kg/cm^2)
Main Re-Bar SD30	3370	180	1.3×10^5
9 φ SR24	2780		

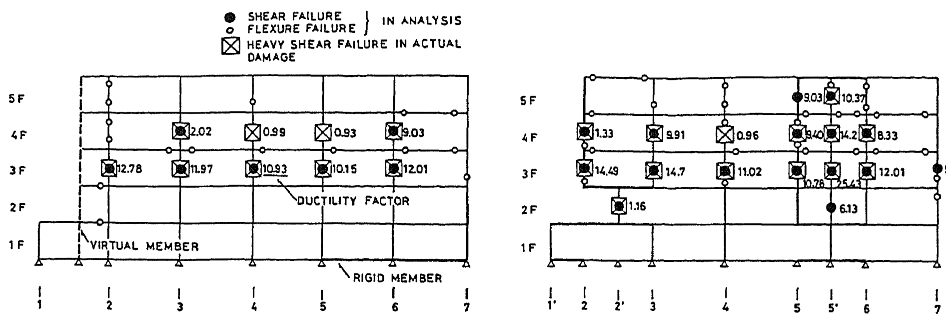
Outline of Structural Damage Observed cracking pattern of J-Frame as an example is illustrated in Fig. 4. Actual damage level of G- and J-Frames are denoted in Fig. 5. The outline of the structural damage of this building is as follows ;

a) Most of the third to fifth story interior columns in the exterior bay which have spandrel walls sustained severe shear failure. Among those columns, those having a large amount of longitudinal reinforcement also suffered from bond splitting failure.
b) Shear cracking occurred in the second to fifth story walls that were not counted in the design, and some of those walls exhibited shear failure.
c) A reinforced concrete chimney on the roof and parapets were inclined.
d) Local failure was observed in slabs of staircases.



note
numbers : max. residual crack width in mm
a : max. residual crack width is about 30mm
b : core concrete failure

Fig. 4 Cracking Pattern of J-Frame



a) G-FRAME

b) J-FRAME

Fig. 5 Failure Mode :
Strength-Degrading Model, NAMIOKA NS 225gal

DYNAMIC ANALYSIS

<u>Structure Model</u> Every column, beam and wall of A- to J-Frames are represented by beam models having rigid zones at both ends. Figures 5 a) and 5 b) show the frame models corresponding to G- and J-Frames. In these figure, actual damage level and results of analysis are also denoted. The following is taken into consideration for the idealization of the building.
a) Stiffness and strength of each column or beam member include those of wing walls and spandrel walls which are attached to the column or beam.
b) The inside of all joint parts surrounded by columns, beams and walls is considered as a rigid zone.
c) Since the basement is composed of relatively rigid members as compared with the upper structural members and, in addition, almost no damage was reported in the basement, a five story structure model without considering the basement is dealt with in the analysis. In this case, first floor beams below which shear walls exist in the basement are considered as rigid members.

In order to represent the non-linearity of members, one-component model consisting of a series combination of flexural springs at both ends and a shear spring at the center of the member is considered. Restoring force characteristics of both springs are, as shown in Figs. 6 and 7, given by a degrading tri-linear and an origin-oriented hysteresis rule, respectively. To represent brittle failure in shear deformation of an element due to external lateral force, the effect of strength degradation is considered after reaching its ultimate shear strength, and a structure model composed of such strength-degrading-type elements is referred hereinafter as the Strength-Degrading Model to distinguish it from an ordinary Origin-Oriented Model.

The stiffness matrix of the models is made assuming that moment distribution is inverted symmetry for columns and beams, and uniform for walls. All the values of the flexural yield moment, My, and the ultimate shear strength, Qu, of every member used here are those obtained through the aseismic diagnosis of the building (Ref.1).

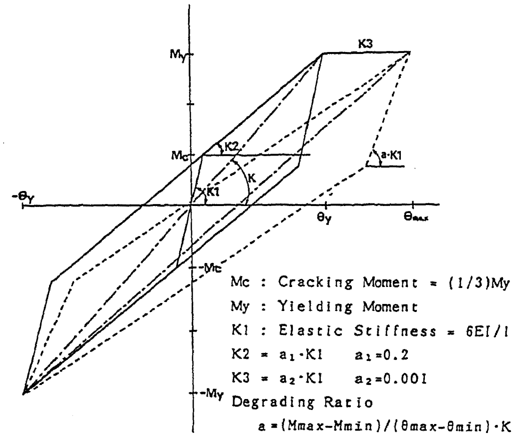


Fig. 6 Flexural Spring Model

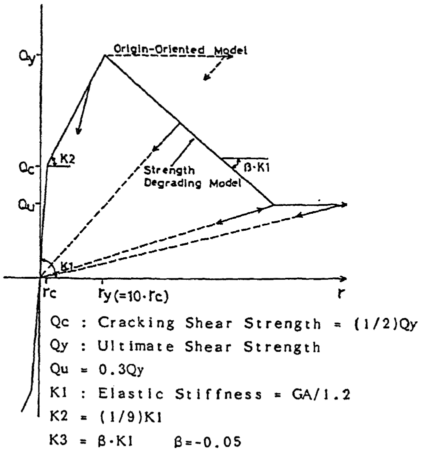


Fig. 7 Shear Spring Model

<u>Dynamic Characteristics and Input Ground Motion</u> The natural period of the model is T_1 = 0.352 sec., T_2 = 0.128 sec. and T_3 = 0.076 sec. for the first, second and third natural modes, respectively. The observed natural period for the first mode obtained from microtremor measurement of the damaged building in its longitudinal (X) direction is 0.72 seconds. The value of the viscous damping constant, h, taken through the analyses is 2 percent of the critical damping.

As input excitations to the building structure models, the NS-component of calculated earthquake motions at the ground surface of the building site concerned is used. This motion was estimated based on a dynamic response of soil layers, where bedrock ground motions observed during the earthquake at the Namioka Dam Site, which is distant about 7km east from the building site, was utilized as an input excitation. The maximum acceleration used in this dynamic analysis is 225 gals, which is approximately 15% larger in acceleration amplitude than the original one.

RESULTS

Figures 8 and 9 show story displacement response time histories of the Origin-Oriented and the Strength-Degrading Models, respectively. Comparing both time histories, it is recognized that the response of both models at all stories except the third one is quite similar to each other, while there exists a big difference at the third story. The maximum displacements of the former model are 0.92 and 1.40cm for the first and the latter halves respectively in the entire duration time, whereas those values of the latter model are 1.20 and 2.70cm for those two halves respectively. The Strength-Degrading Model gives 30% and 93% larger story displacements at the third story for the first and the latter half parts respectively in the entire duration time compared with the Origin-Orient Model.

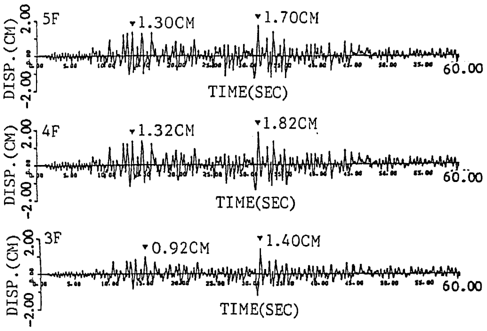


Fig. 8 Story Displacement Response of Origin-Oriented Model : NAMIOKA NS 225gal

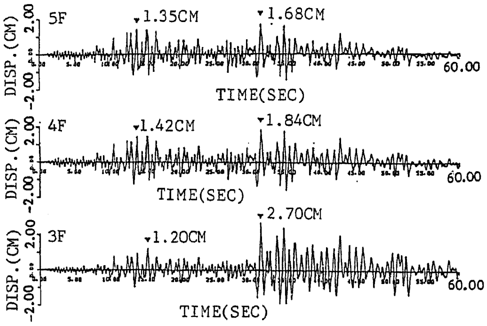


Fig. 9 Story Displacement Response of Strength-Degrading Model : NAMIOKA NS 225gal

As shown in Fig. 10, these maximum story displacement analytically obtained for the Strength-Degrading Model also gives quite good correspondence with the distribution of the maximum story displacements along the height of the building which were estimated from the observed maximum residual crack width on shear walls in the building.

As described above, the Strength-Degrading Model gives larger story displacements at the third story, particularly for the latter 35 seconds compared with the Origin-Oriented Model. This fact indicates that the duration time span greatly affects the response of a structure mainly composed of strength-degrading-type resisting elements. Based on the results discussed above, it is estimated that damage of the Namioka Town Hospital building would have remarkably decreased in comparison with the actual damage, if the 1983 Nihonkai-Chubu Earthquake were a single-shock-type earthquake.

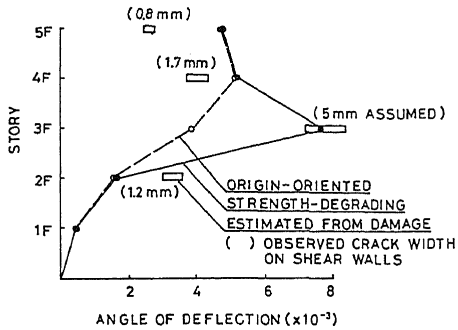


Fig. 10 Maximum Angle of Deflection NAMIOKA NS 225gal

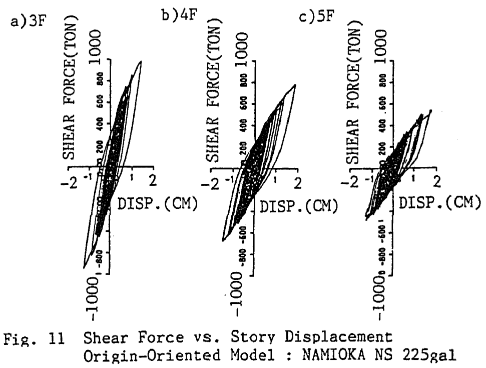


Fig. 11 Shear Force vs. Story Displacement Origin-Oriented Model : NAMIOKA NS 225gal

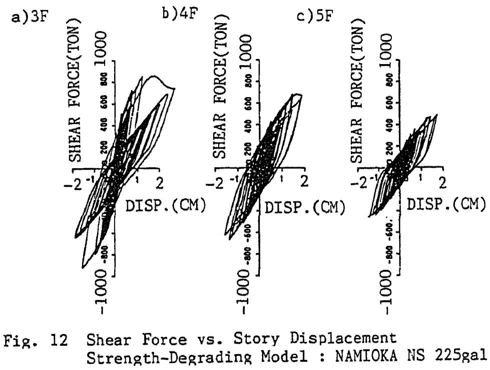


Fig. 12 Shear Force vs. Story Displacement Strength-Degrading Model : NAMIOKA NS 225gal

Figures 11 and 12 show story shear force vs. story displacement curves at the third to fifth stories obtained from the both models. In these figures, it is seen that stiffness at the third story of the Strength-Degrading Model compared with the one of the Origin-Oriented Model greatly decreases in the last part of the duration time due to the negative slope after reaching ultimate strength. This coincides with the fact that the vibration period of the third story response oscillation shown in Fig. 9 is initially 0.5 second approximately at small amplitude response in the first part of the whole duration time, and becomes about 0.9 second in the last part.

The final state on G- , and J-Frames of response of the Strength-Degrading Model is shown in Fig. 5. Almost all vertical elements at the third story of G- and J-Frames and about half of those at the fourth story sustain shear failure (denoted with the symbol●). Flexure failure (denoted with the symbol○) occurs at many fourth story horizontal elements of G- and J-Frames. In contrast with the analytical results, actual damage produced on vertical elements is also indicated in Fig. 5 with the symbol⊠. The figures in Fig. 5 denote analytically obtained ductility factors in shear deformation characteristics (standardized by displacement corresponding to its ultimate strength). Very good agreement between the analytical results and the actual damage is observed in these figures.

CONCLUSIONS

The Namioka Town Hospital building which is mainly composed of strength-degrading-type resisting elements shows very unstable behaviour in earthquake response. And dynamic response of the Strength-Degrading Model shows very good agreement with an actual damage features of this building. On the contrary, the Origin-Oriented Model in which no strength degrading effect is considered in shear-deformation characteristics of resisting elements gives insufficient results to explain actual damage reasonably.

It is estimated that damage of the Namioka Town Hospital building would have remarkably decreased in comparison with the actual damage, if the earthquake had been a single shock-type one.

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