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RESPONSE OF REINFORCED CONCRETE BUILDINGS DUE TO 1993 KUSHIRO-OKI EARTHQUAKE

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ABSTRACT

The 1993 Kushiro-oki Earthquake in Japan presented the same phenomenon as the 1962 Hiroo-oki earthquake, i.e., small damage of buildings was observed from a large-amplitude acceleration ground motion.Topographic and geological feature at Kushiro Meteorological Observatory (KMO) enlarged so much the short components of ground motion that the large records were obtained. Response of the nonlinear SDF system, based on the strength of existing buildings in Kushiro city indicated that the response lateral load of buildings must have not exceeded their lateral capacity in almost areas in Kushiro city. The response analysis of a building adjacent to KMO corresponded to small damage during the earthquake.

KEYWORDS

Kushiro-oki earthquake; Kushiro Meteorological Observatory; small damage; large acceleration; nonlinear response analysis; SDF system; strength of building; surplus capacity; frame model

INTRODUCTION

The 1993 Kushiro-oki earthquake, which took place at the east coast of Hokkaido on January 15, 1993, did not receive much public attention because another earthquake, the Hokkaido Nansei-oki earthquake occurred after six month bringing much greater damage and human losses. Furthermore, the tremendously disastrous earthquake, the 1995 Hyogo-ken-nanbu earthquake made former earthquakes escape public memory. From the earthquake engineering point of view, however, the Kushiro-oki earthquake presented an important problem; the relationship between recorded strong motion and earthquake damage, which was the same phenomenon as the 1962 Hiroo-oki earthquake. Ground acceleration exceeding 0.7 g recorded at the Kushiro Meteorological Observatory, while an investigation immediately after the earthquake indicated that building damage was small. Why was the damage of building so small for large acceleration record ?

A series of single-degree-of-freedom (SDF) nonlinear earthquake response analyses were carried out to correlate the observed damage and the response of structures. Yield resistance of the SDF system was determined according to investigated strength distribution of the existing buildings in Kushiro city. Estimated ground motions at several sites in Kushiro city, on the basis of the observed accelerograms, were used in the analyses as input ground motions. Furthermore, the earthquake response was studied for a five story building located adjacent to KMO at the site of which large ground accelerations were recorded.

OUTLINE OF DAMAGE

The 1993 Kushiro-oki earthquake of magnitude 7.8 occurred on January 15, 1993, at the epicentral distance 20 km to the south of Kushiro at the depth of 107 km. The focal distance was about 110 km. Damage by this earthquake was reported in Hokkaido and Aomori prefecture. Two persons were killed, 933 persons were injured, 3500 wooden dwellings were partially damaged or collapsed. In the Kushiro area, 12 wooden dwellings suffered total collapse, 50 partial collapse and 24900 partial damage. Many lifelines, railways and harbor facilities failed due to ground damage.

Damage investigation of reinforced concrete buildings was carried out by the first author and his group in and around Kushiro city immediately after the earthquake. The result is summarized in Table 1. Out of 37 reinforced concrete buildings, one building was partially collapsed, one was severely damaged and two were moderately damaged. The partially collapsed building was a fire station, whose observatory room at the top of the tower fell directly to the roof of the second story and crushed the part of the building. Major damage was a three-story high school building; columns along north corridor failed in shear. However, no damage or little damage was observed in the most reinforced concrete buildings. No damage was observed in reinforced concrete wall buildings. In steel buildings, partial fracture or buckling of braces in gymnasiums were observed.

Table 1 Damage Ranks of Investigated buildings.

Construction	No	Light	Minor	Medium	Major	Collapse	Total
Reinforced Concrete	4	21	8	2	1	1	37
Steel Encased Reinforced Concrete	1	1	1				3
Steel		2	2	1			5
Wooden				1			1
Total	5	24	11	4	1	1	46

EARTHQUAKE GROUND MOTIONS

Acceleration Record

Kushiro city is located almost directly above the epicenter. Topographic feature of the city is divided roughly by Kushiro River, to a hilly region in the east and a plain in the west (Fig. 1). Ground acceleration exceeding 0.70 g (gravity acceleration) was recorded at the Kushiro Meteorological Observatory (KMO) which is located almost at the top of the hill east of the river (Fig. 1). Two strong motion accelerographs were activated at KMO; one was an 87-type electromagnetic system installed on the ground floor of the building, recording maximum acceleration of 815 cm/sec^2 in NS direction, and 919 m/sec^2 in EW direction. The other was a SMAC-MD type accelerograph installed on the ground of the station, recording maximum acceleration of 637 m/sec^2 in NS direction, and 711 m/sec^2 in EW direction. Although several other ground acceleration records were obtained near Kushiro city, the maximum accelerations 0.92 g or 0.72 g record at KMO were much larger than any other records. Acceleration response spectrum and velocity response spectrum of the record by SMAC-MD type accelerograph are compared with those of past large earthquakes in Fig. 2. The earthquake records of KMO was significantly large in short period range where the maximum acceleration is dominated and almost same or smaller level as the past large earthquake in the long period range where the maximum velocity is dominated.

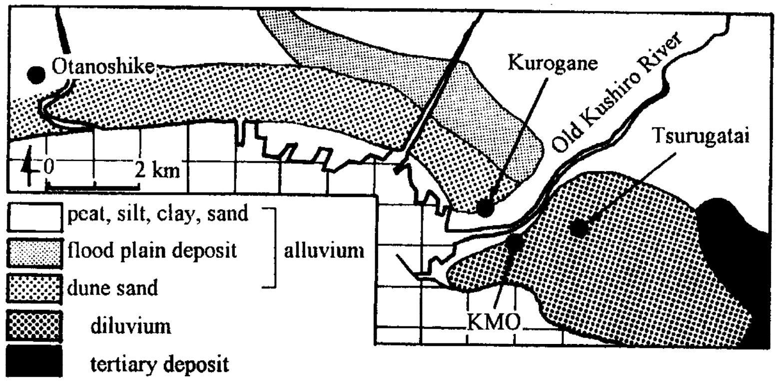

Fig.1 Outline of ground in Kushiro city.

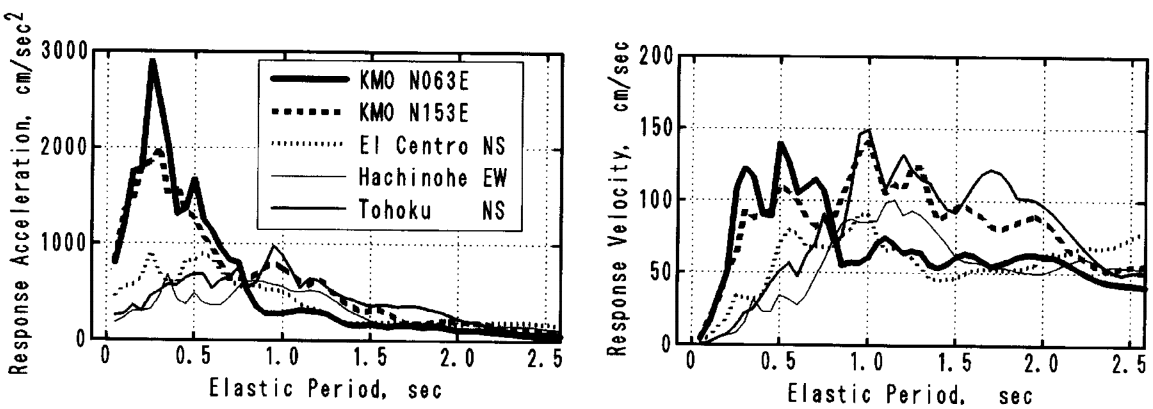

Fig.2 Response spectrum of the KMO record compared with past large earthquakes (0.05 Damping).

Estimation of Ground Motion

The bedrock input motion at KMO site was calculated by nonlinear ground response analysis based on the observed accelerograms and soil boring log data. Initially, a bed rock motion at KMO site was estimated from the observed SMAC-MD record by the analysis based on one-dimensional multiple reflection theory. A ground surface motion was simulated by nonlinear ground response analysis for the estimated bed rock motion. The simulated ground surface motion was compared with the observed one to modify the parameters in the analysis. The process was repeated to obtain the simulated ground surface motion well coinciding with the observed one. The site of KMO is located on a hill where stiff sandy layer is covered by relatively thin surface loam layer of several meters, thus it was clarified by the analysis that the relatively short period acceleration tends to be amplified.

Ground motions at several sites in Kushiro city were estimated by nonlinear ground response analysis using the finally obtained bedrock motion at KMO site as the bedrock input in other sites. Three sites were selected to study the response of structures; i.e., Kurogane, Tsurugatai, and Otanoshike (Fig.1). Kurogane and Tsurugatai are built-up area in Kushiro city. The maximum acceleration and velocity of the estimated ground surface motions at these sites are listed in Table 2. Maximum accelerations, especially at Otanoshike which locates on thick soft deposits of western alluvium part of the city, were not so large as the observed one at KMO. Elastic response spectra of the estimated ground motions in case of 0.05 damping are compared with the spectrum of the observed KMO record as shown in Fig.3. Acceleration and velocity response of the estimated ground motions were significantly smaller than the response of the observed one at KMO.

Table 2 Observed and estimated ground motion

Location	Maximum acceleration, cm/sec^2		Maximum Velocity, cm/sec	
	N063E	N153E	N063E	N153E
KMO (observed)	711	637	33.5	42.3
Otanoshike (estimated)	147	164	37.6	45.5
Kurogane (estimated)	271	276	28.3	30.7
Tsurugatai (estimated)	367	389	21.2	16.7

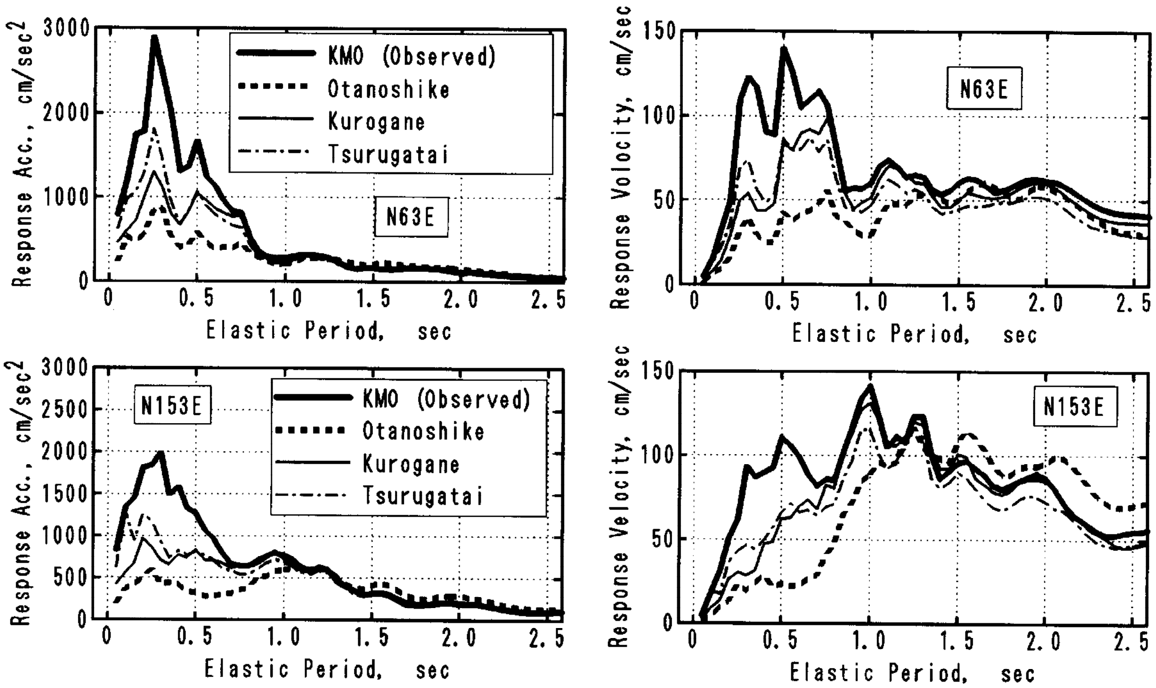

Fig.3 Response spectra of estimated ground motions (0.05 Damping)

STRENGTH OF R/C BUILDING IN KUSHIRO CITY

Ratios of shear wall area to the floor area of existing reinforced concrete buildings in Kushiro city were investigated by members of research committee established in Architectural Institute of Japan to evaluate the strength of buildings in the city. The number of building selected arbitrary for the investigation was 102. Base shear coefficient C_B of each building when shear stress of walls is postulated to reach the ultimate strength simultaneously was simply estimated by the following equation.

$$C_B = (25A_W + 7A_C) / W \quad (1)$$

where, A_W: Total area of shear walls at the first story (cm^2),
A_C: Total area of columns at first story (cm^2),
W: Weight of the building (kgf).

The average C_B value and the standard deviation σ for C_B distribution were very similar to the values derived from the following equations represented as a function of number of stories N, respectively.

$$C_B = 3.27\,N^{-0.91} \tag{2}$$

$$\sigma = 1.43\,N^{-1.02} \tag{3}$$

The average C_B values obtained from the investigation and the simulated C_B value by the equation (2) were plotted versus number of stories N in Fig.4. Damage due to earthquake must be larger for building with the lower capacity. The response in case of lower capacity was studied by using the base shear coefficient $C_B - \sigma$ and $C_B - 2\sigma$ which were also plotted in the same figure.

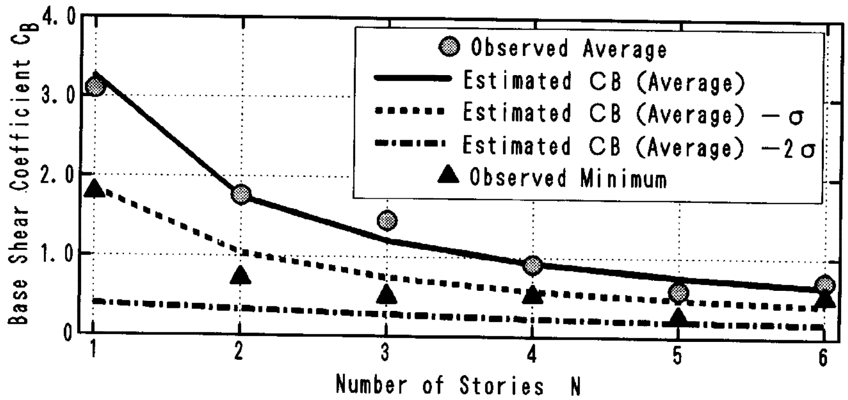

Fig.4 Observed and simulated strength of buildings in Kushiro city

STRENGTH OF BUILDING AND EARTHQUAKE RESPONSE

A series of single-degree-of-freedom (SDF) nonlinear earthquake response analyses were carried out to correlate the observed damage and the response of structures. Response ductility factor of SDF system was studied based on the strength of buildings in Kushiro city.

SDF System

Degrading-trilinear model was selected to simulate the restoring force characteristics of buildings idealized to SDF system. The skeleton curve under monotonically increasing lateral load was idealized by a trilinear relationship. In order to represent the characteristics of reinforced concrete building, the following constant parameters were assumed; i.e., story height h = 3.0 meter, the yielding deformation angle Ry = 1/150 rad at the roof level of the building, cracking resistance $Q_c = 0.33Q_y$ (Q_y: yielding resistance), the ratio of the yielding stiffness to the initial stiffness $\alpha_y = 0.33$, and the ratio of the post-yield stiffness to the initial stiffness $\beta = 0.01$. The yield resistance equivalent to the base shear coefficients C_B, $C_B - \sigma$, and $C_B - 2\sigma$ at every number of stories shown in Fig.4 were given to Q_y. The base of SDF systems was assumed to be fixed.

Response of SDF system

Based on the strength distribution at every number of stories in Kushiro city, response of the SDF system was studied by using ground motions observed at the KMO site and estimated at other three sites as input motions. Response ductility factors, which were defined as a ratio of the maximum response deformation to yield deformation of the system, are plotted for each direction of the ground motions in Fig.5 and 6 . In case of the average strength (Fig.5.a, 6.a), almost response ductility factors did not exceed 1.0, except for the response by the ground motion at the KMO. Therefore, damage of the buildings with average strength in Kushiro city must be not severe. In case of the strength lower than the average by σ (Fig.5.b, 6.b), except for the ground motion at Otanoshike, the response ductility factors for the building higher than 3 stories high exceeded 1.0 and reached about 2.0 for 5 or 6 stories high. Thus, it was possible for the mid-rise buildings to suffer relatively severe damage. However, there are few mid-rise reinforced concrete buildings in Kushiro city. In case of the strength lower than the average by 2σ (Fig.5.c, 6.c), the maximum response was comparable for the three ground motions except that at Otanoshike. The response ductility factors significantly exceeded 1.0, the phenomenon which means that buildings with much lower resistance must suffer severe damage or have collapsed. Such was not observed in the damage investigation. Note that the strength distribution survey in Kushiro city indicated no building was lower than C_B(average)-2σ. Furthermore, the actual resistance of buildings in the city must be greater than the resistance estimated from equation (1), because of additional strength by non-structural elements which can be readily provided especially in low-rise buildings. Hence, in most areas of Kushiro city, response lateral resistance of buildings did not exceed their lateral capacity.

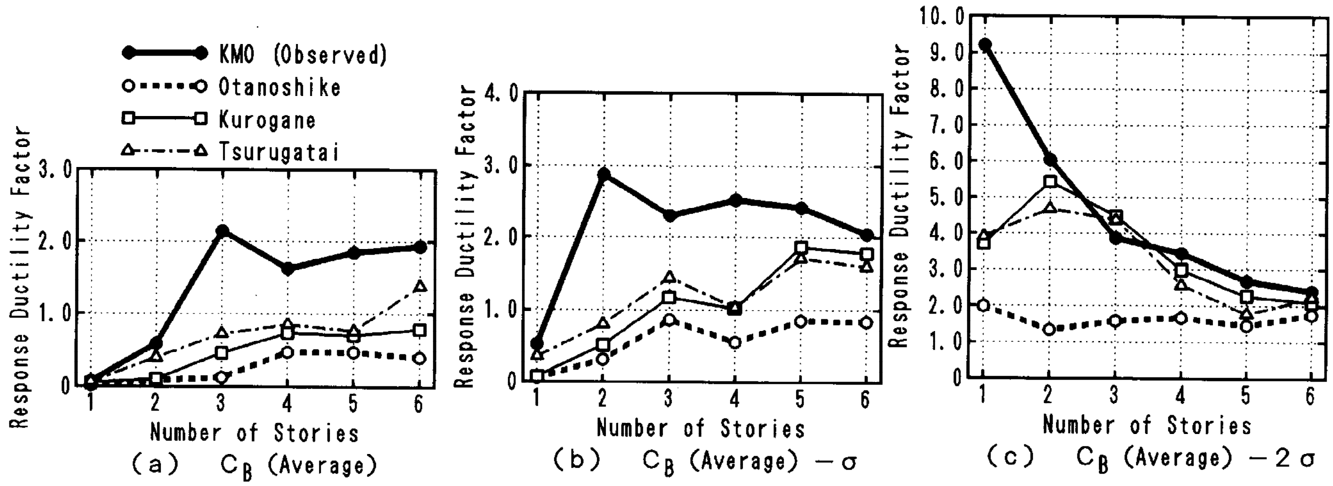

Fig.5 Maximum response of SDF system based on the strength of buildings in Kushiro city (N063E Direction, 0.05 Damping)

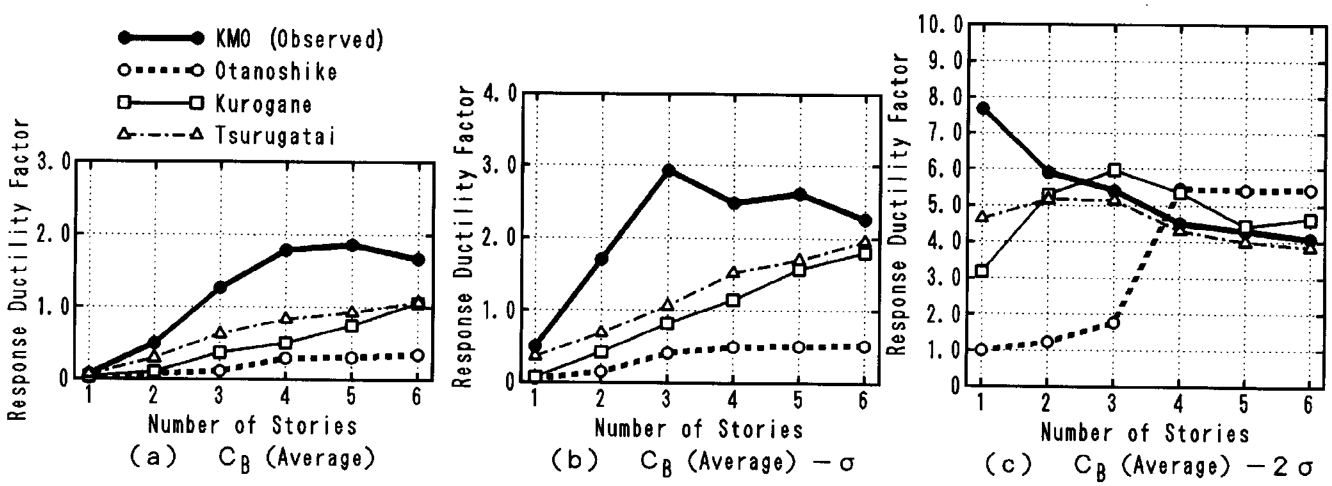

Fig.6 Maximum response of SDF system based on the strength of buildings in Kushiro city (N153E Direction, 0.05 Damping)

DYNAMIC RESPONSE ANALYSIS OF A DAMAGED BUILDING

Damage investigation revealed no buildings with severe damage or collapse even in the vicinity of KMO. A damaged reinforced concrete building located adjacent to KMO building was studied by dynamic earthquake response analysis.

Outline of the building and its damage

The building is a five story reinforced concrete office building constructed in 1970. Typical structural damage was shear cracks in exterior walls and columns with open windows in the longitudinal direction as shown in Fig.7. This type of damage was observed remarkably at the lower three stories. In addition, many fixed window glasses were broken also at the lower three stories. the damage represents that response deformation was relatively large in lower stories in the building.

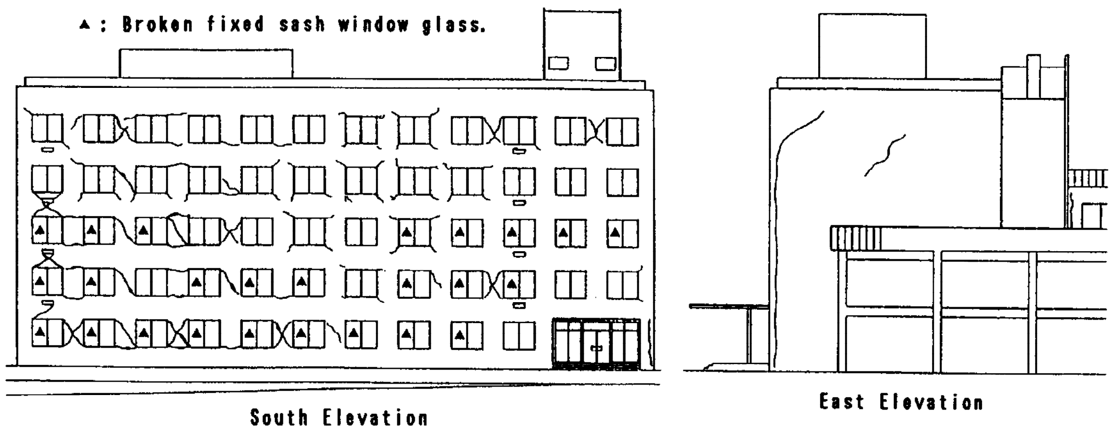

Fig.7 Damage of the five story building adjacent to KMO

Dynamic Response Analysis

The building was analyzed for nonlinear static and dynamic earthquake analysis in its longitudinal direction using supposed ground motion in the site to investigate maximum drift in each story. The longitudinal direction coincides with N063E direction. Based on data of soil boring and PS logging, the KMO record was converted to ground surface acceleration in the site.

Based on structural design document, the entire building was idealized to a frame model simulating its actual stiffness and strength in the longitudinal direction. Columns, beams and nonstructural walls between windows were replaced by line elements with rigid zones, nonlinear flexural springs in both ends and nonlinear shear springs. In the exterior frames, the influence of spandrel walls, hanging walls and column side walls on stiffness and strength of members was considered. Participation width of slab to beam stiffness and strength was supposed to be one tenth of clear span length. Reinforced concrete walls of staircases or elevator shaft were also idealized to wall model constituted by line elements with nonlinear shear, axial and bending springs. Takeda model was chosen as hysteresis model for restoring force characteristics of every nonlinear spring. Horizontal linear constrain springs were added to base node in frame model to consider interaction between ground and structure.

Roof horizontal displacement and base shear coefficient by the static analysis is shown in Fig.8. Dashed line in the figure represents base shear coefficient adopted in the seismic design of the building. Horizontal capacity of the building was about 0.8 in base shear coefficient and was about four times the design base shear. Various surplus strengths contributed to the increase of horizontal base carrying capacity of the building. Distribution of maximum response story drift by the dynamic analysis is shown in Fig.9. Dashed line in the figure represents the response of frame model constituted only by columns and beams whose capacity was supposed to be the original design capacity not considering surplus strength. The response drift in each story of the frame model with existing stiffness and strength represented by a solid line, was significantly smaller than that of the frame with the original design capacity. It is considered that the response drift less than 0.5 per cent represents minor damage as observed. The distribution of response drifts which were larger in lower stories also corresponds to the behavior during the earthquake.

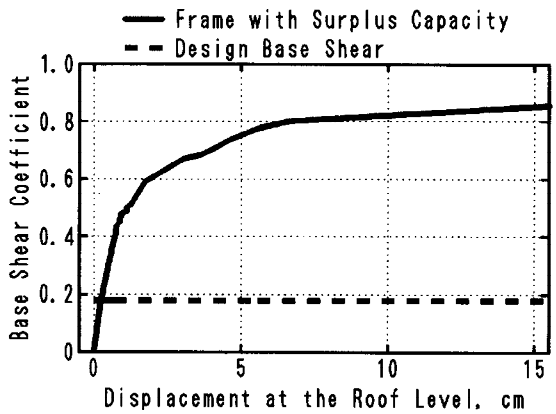

Fig.8 Static response of the frame model

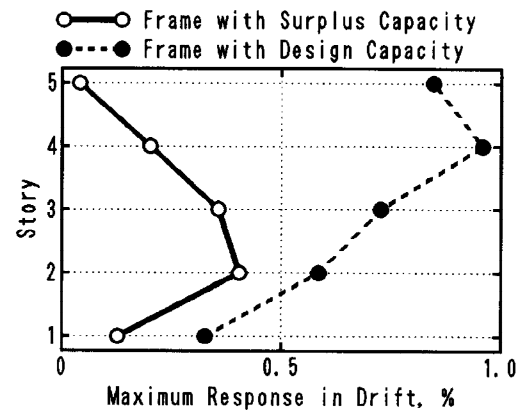

Fig.9 Dynamic response of the frame model

CONCLUSIONS

Very large accelerations exceeding 700 cm/sec^2 were recorded by the 1993 Kushiro-oki earthquake. Observed damage, especially in reinforced concrete buildings, was not so much as to be inferred by these records. The problem of large acceleration and small damage was studied in this paper.

Topographic and geological feature at the Kushiro Meteorological Observatory enlarged so much the short components of ground motion that the large records were obtained. Nonlinear ground response analysis represented that the earthquake power on other sites in Kushiro city must be not so large. Response of the nonlinear SDF system, based on the strength of existing buildings in Kushiro city and using the estimated ground motions at several sites, indicated that the response lateral load of buildings must have not exceeded their lateral capacity in most areas of Kushiro city. Furthermore, the response drift of a building adjacent to KMO corresponded to small damage during the earthquake, in case of the analysis in which the surplus capacity was idealized. Thus, one of the reason for so small damage was the lateral resistant capacity of buildings in Kushiro city.

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