Paper No. 1164. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

DYNAMIC RESPONSE ANALYSES OF REINFORCED CONCRETE BUILDINGS DAMAGED IN THE 1993 KUSHIRO-OKI EARTHQUAKE OF M_{JMA} 7.8 IN JAPAN - DID THE BUILDINGS BEHAVE AS WE HAD EXPECTED? -

K. DAN *, M. KIKUCHI *, H. TANAKA **, and K. YASHIRO **

* Ohsaki Research Institute, Inc., 2-2-2, Uchisaiwai-Cho, Chiyoda-Ku, Tokyo, 100 Japan
** Tokyo Electric Power Company, 1-1-3, Uchisaiwai-Cho, Chiyoda-Ku, Tokyo, 100 Japan

ABSTRACT

The 1993 Kushiro-Oki, Japan, earthquake of M_{JMA} 7.8 stimulated much discussion among earthquake engineers in Japan as to whether the behavior of a structure calculated in the design process represents the actual behavior of that structure during an earthquake. Large peak horizontal accelerations of 711 cm/s^2 and 637 cm/s^2 were recorded on the ground at the Kushiro District Meteorological Observatory, but the damage to buildings in the city of Kushiro was rather slight, even though many buildings have been severely damaged by smaller or similar peak accelerations during other significant earthquakes. Dynamic response analyses of four reinforced concrete buildings, most severely damaged in this earthquake, were performed in this paper. The damage degree of the four buildings was different from one another. The analysis results verified that the behavior of structures calculated in the design process could indeed represent the actual behavior of those structures during an earthquake, when the input motions were evaluated appropriately and the structures were modeled accurately.

KEYWORDS

1993 Kushiro-Oki earthquake; strong ground motion; surface layer; soil-structure interaction; reinforced concrete building; dynamic response analysis; cracking; yielding; shear coefficient; ductility ratio.

INTRODUCTION

Large peak horizontal accelerations of 711 cm/s^2 and 637 cm/s^2 were recorded on the ground at the Kushiro District Meteorological Observatory during the 1993 Kushiro-Oki earthquake. The damage to buildings in the city of Kushiro was rather slight, even though many buildings have been severely damaged by smaller or similar peak accelerations during other significant earthquakes. This observation stimulated much discussion among earthquake engineers in Japan as to whether the behavior of a structure calculated in the design process represents the actual behavior of that structure during an earthquake. Dynamic response analyses of four reinforced concrete buildings at Kushiro Technical High School were performed in this paper to simulate the behavior of these structures during the Kushiro-Oki earthquake. These four buildings were most severely damaged in the earthquake, and the damage degree of the four buildings was different from one another. Before the analyses, the input motions to the buildings were evaluated on the

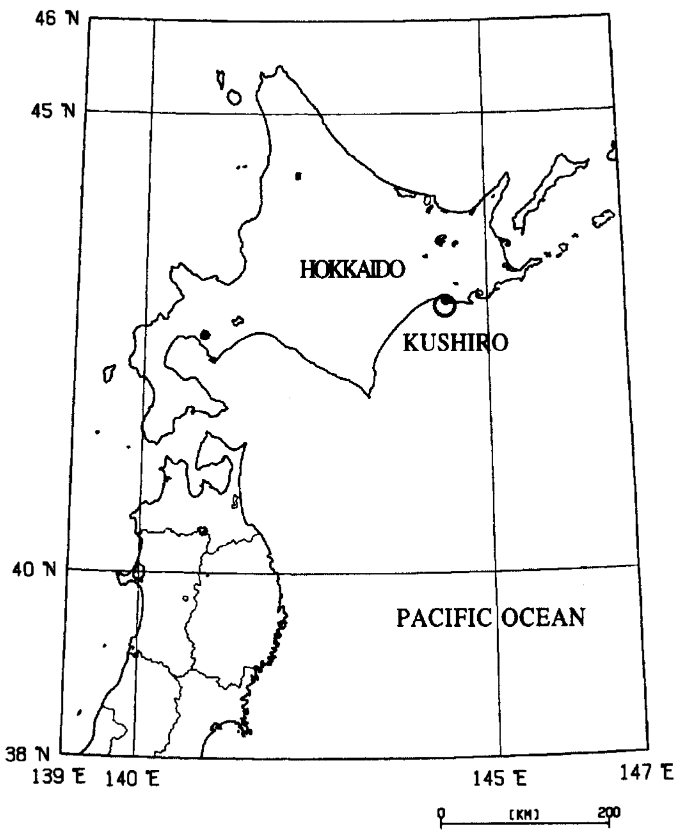


Fig. 1. The epicenter of the 1993 Kushiro-Oki, Japan, earthquake (M_{JMA} 7.8, focal depth 107 km) indicated by the open circle and the location of the city of Kushiro by the dot in the open circle. The seismic intensity was 6 in Kushiro on the JMA scale.

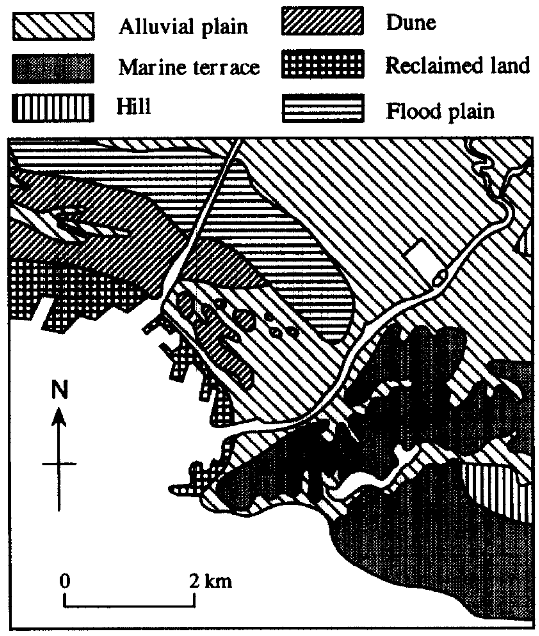


Fig. 2. The location of the Kushiro District Meteorological Observatory (KUS) and Kushiro Technical High School (KTH). Large peak horizontal accelerations of 711 cm/s^2 and 637 cm/s^2 were recorded on the ground at KUS during the 1993 Kushiro-Oki earthquake. Dynamic response analyses were performed in this paper to simulate the behavior of the four damaged reinforced concrete buildings at KTH.

ground at the school from the estimated base-rock motions at the observatory based on the strong-motion records.

THE 1993 KUSHIRO-OKI, JAPAN, EARTHQUAKE

Figure 1 shows the epicenter of the 1993 Kushiro-Oki earthquake. Its magnitude was reported by several different organizations: M_{JMA} 7.8 by Japan Meteorological Agency (JMA), *Ms* 6.9 by US Geological Survey, and *Mw* 7.8 by Harvard University. This event occurred 107 km deep in the Pacific plate. The aftershock distribution showed that the rupture zone was 30 to 40 km in EW direction and 15 to 20 km in NS direction. The fault rupture was estimated to have started at the lower plane of the Pacific plate, propagated horizontally to the east, and stopped near the upper plane. Dan *et al.* (1993) calculated the source spectrum and concluded that this earthquake had generated four times larger short-period (0 to 2 seconds) seismic waves, because of a very high stress drop, than the 1923 Kanto, Japan, earthquake of M_{JMA} 7.9.

The seismic intensity was 6 in Kushiro on the JMA scale. Large peak horizontal accelerations were recorded at the Kushiro District Meteorological Observatory (KUS in Figure 2): 711 cm/s^2 and 637 cm/s^2 on the ground observed by Building Research Institute, Ministry of Construction, Japan, and 919 cm/s^2 and 815 cm/s^2 on the first floor of the two-story building observed by JMA. Four reinforced concrete buildings at Kushiro Technical High School (KTH in Figure 2) were most severely damaged in this earthquake.

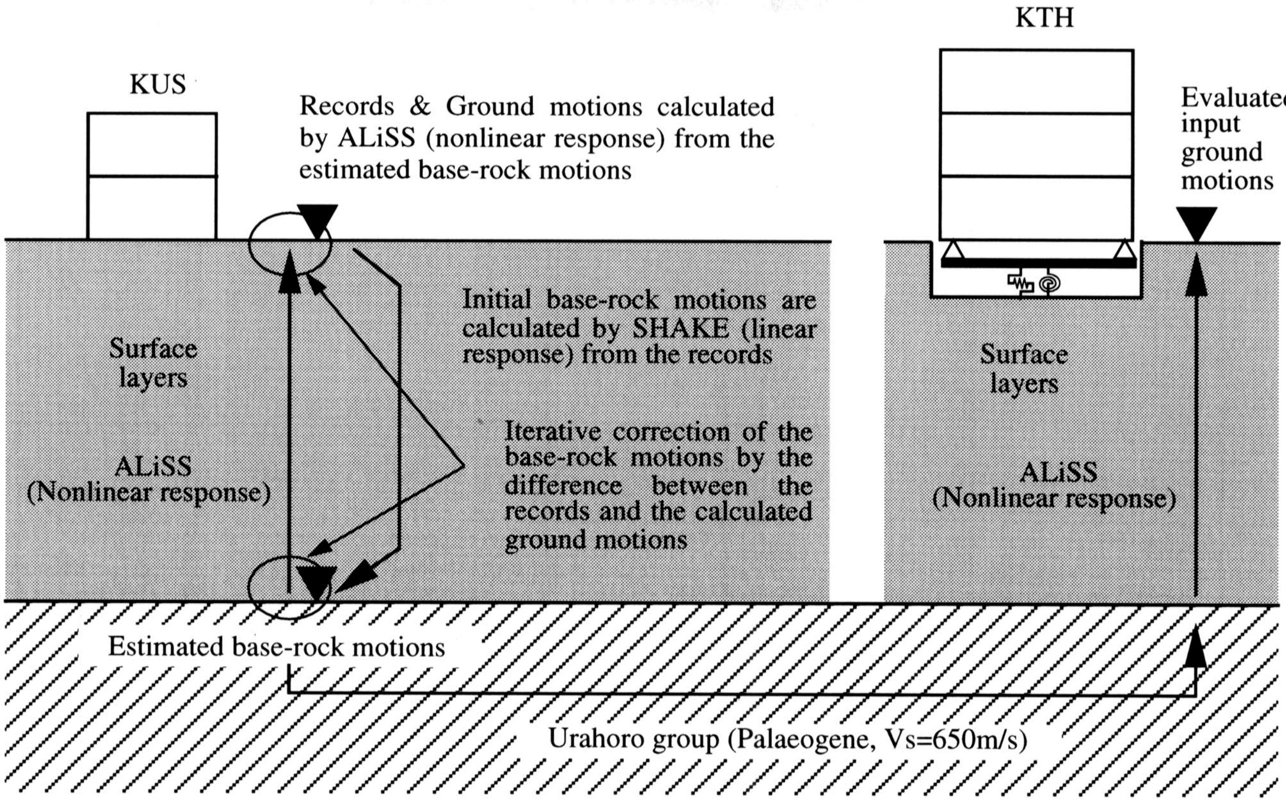


Fig. 3. Scheme for evaluating the input ground motions to the damaged buildings at Kushiro Technical High School (KTH) from the records on the ground of the Kushiro District Meteorological Observatory (KUS).

EVALUATION OF THE INPUT GROUND MOTIONS AT KTH

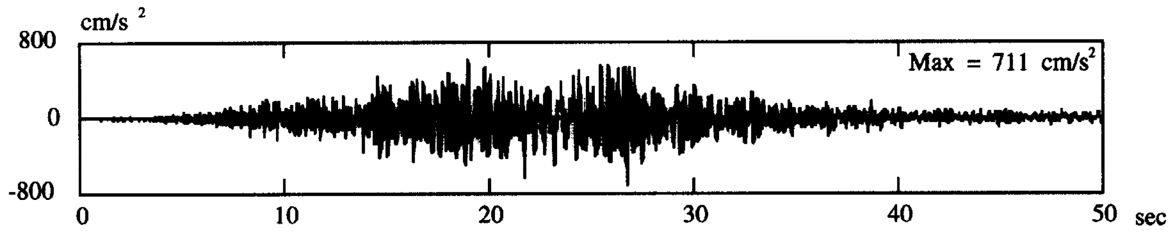
One of the most important subjects for the dynamic response analysis is the appropriate evaluation of input motions to the buildings. Because KUS and KTH are located on the same marine terrace as shown in Figure 2, and have a common base rock of the Urahoro group (Hokkaido Architect Association, 1982), two steps were taken for evaluating the input ground motions at KTH. In the first step, the base-rock motions at a depth of 20 m were estimated from the records on the ground at KUS by taking account of the nonlinear behavior of the soil of the surface layers. In the second step, the input motions to the buildings were evaluated on the ground at KTH by the nonlinear response analysis of the soil based on the estimated base-rock motions. The nonlinear response analysis of the surface layers was conducted by the ALiSS computer code (Fukutake *et al*, 1990). Figure 3 summarizes the scheme described above.

Figure 4 shows the ground motions recorded at KUS, the base-rock motions already estimated by Dan (1995), and the input ground motions evaluated at KTH. The order of shear strains of the soil was 0.1 % both at KUS and KTH. The predominant frequencies of the surface layers changed from about 4 Hz to about 3 Hz because of nonlinear effect of the soil.

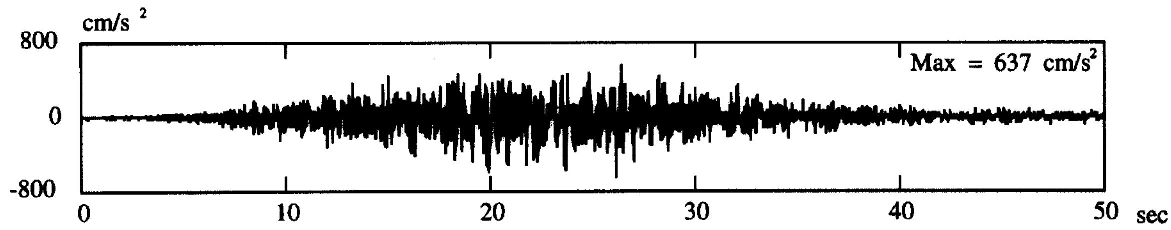
MODELING OF THE DAMAGED BUILDINGS AT KTH

Figure 5 shows the construction year of the four buildings at KTH and the damage degree in the 1993 Kushiro-Oki earthquake. None of the buildings may have enough column ties, because they were constructed before the Japanese seismic code was revised in 1971 to keep short columns away from collapse in shear. The minimum requirement of the base shear coefficient was 0.2 in elastic design at that time.

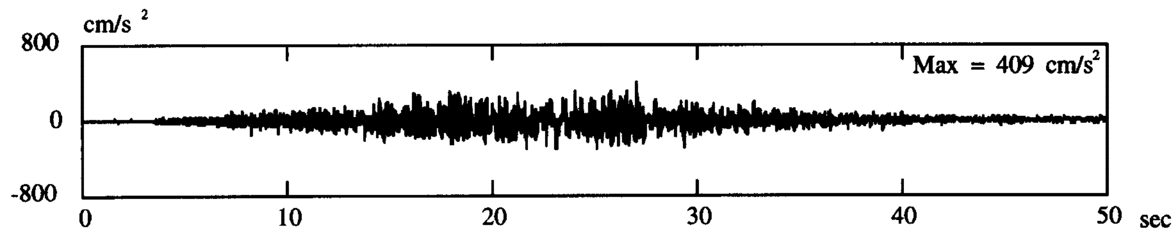
KDAN 4



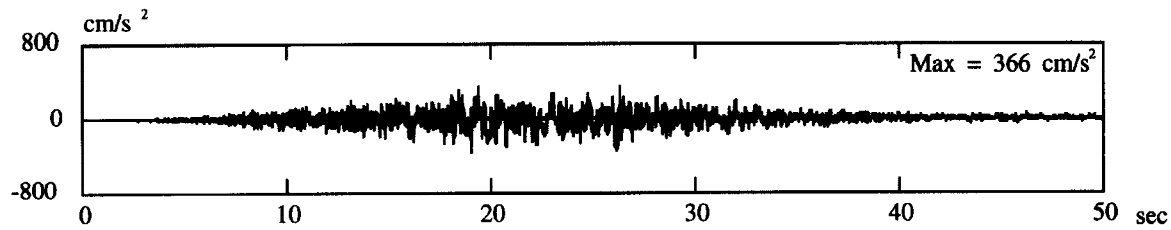
(a) Ground motion recorded at KUS (N063E)



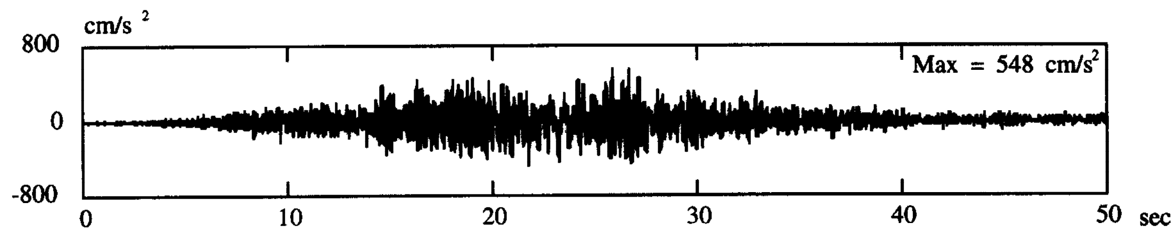
(b) Ground motion recorded at KUS (N153E)



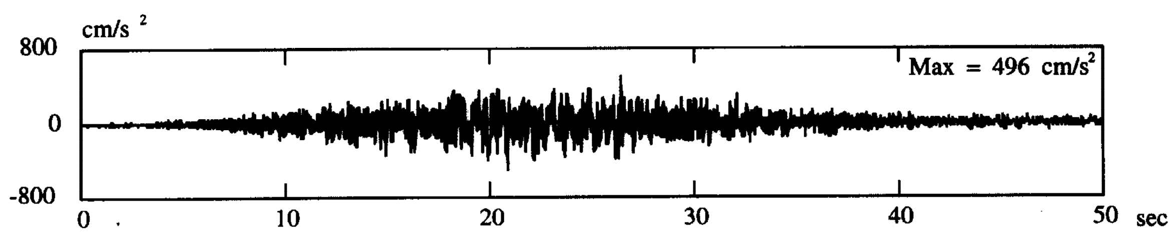
(c) Base-rock motion estimated at KUS (N063E)



(d) Base-rock motion estimated at KUS (N153E)

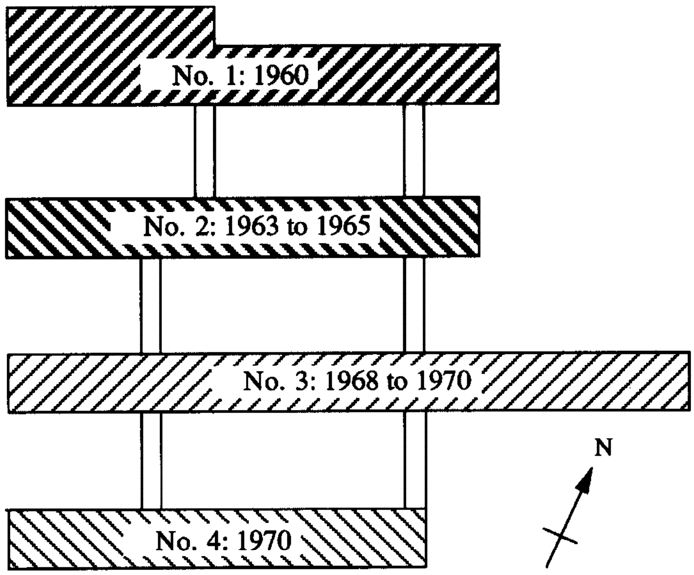


(e) Input ground motion evaluated at KTH (N063E) for the dynamic response analyses in EW direction



(f) Input ground motion evaluated at KTH (N153E) for the dynamic response analyses in NS direction

Fig. 4. The ground motions recorded at KUS, the estimated base-rock motions, and the input ground motions evaluated at KTH for the dynamic response analyses.



No. 1 building (three story): Moderate damage
Some very short columns on the 1F collapsed in shear, and some columns on the 2F cracked in flexure.

No. 2 building (three story): Severe damage
Many short columns on the 1F and 2F collapsed in shear, and some columns on the 3F cracked in flexure.

No. 3 building (three story): Slight damage
Some columns on the 1F to 3F cracked in flexure.

No. 4 building (two story): Little damage
Some columns on the 1F cracked in flexure.

Fig. 5. Construction year of the four buildings at KTH and the damage degree in the 1993 Kushiro-Oki earthquake.

Figure 6 shows representative damaged frames selected for the dynamic response analyses in this paper. They were modeled as an assembly of nonlinear elements, including sway and rocking springs to account for the effect of the soil-structure interaction. The sway and rocking springs, frequency dependent complex values, were calculated by the GRIMP2 computer code (Yoshida, 1994), and they were represented by the values at the first-mode frequency of the soil-structure interaction system. Nonlinear behavior of the soil was considered by equivalent-linear modeling; the shear-wave velocities of the surface layers were assumed to be 3/4 of those in the initial linear model, and the damping factors were assumed to be 7 % instead of 3 % in the initial linear model, because the order of the shear strains was 0.1 % and the predominant frequency of the surface layers changed from about 4 Hz to about 3 Hz in the evaluation of the input ground motions at KTH.

The columns and beams of the buildings were modeled by elements with rigid areas and rotational springs at both ends and axial and shear springs at the center (Giberson, 1969). The rigid area was taken within the surface defined by the other beams or columns connected at the joint, and it was extended when the columns were connected with spandrel walls over or under the openings. The earthquake-resistant walls were modeled as elements with two rigid beams at their upper and lower levels, two axial springs at both sides, and axial, shear, and rotational springs at the center (Kabeyasawa *et al*, 1983).

The hysteresis model proposed by Takeda *et al.* (1970) was applied to the rotational springs at the ends of the columns and the beams. The origin-oriented hysteresis model was applied to the shear spring at the center of the columns, the beams, and the walls. Both hysteresis models have a tri-linear skeleton curve. The first point of stiffness change indicates cracking of the concrete, and the second indicates yielding of the reinforcing steel. The stiffness after yielding was assumed to be 1/100 of the initial stiffness. The elastic model was applied to the rest of the axial, shear, or rotational springs.

The total weight of the building was distributed to the joints according to the area supported by the column at the respective joint. The damping factor was assumed to be proportional to the instantaneous stiffness with an initial value of 2 % at the first natural frequency of each frame model without the effect of soil-structure interaction.

K.DAN 6

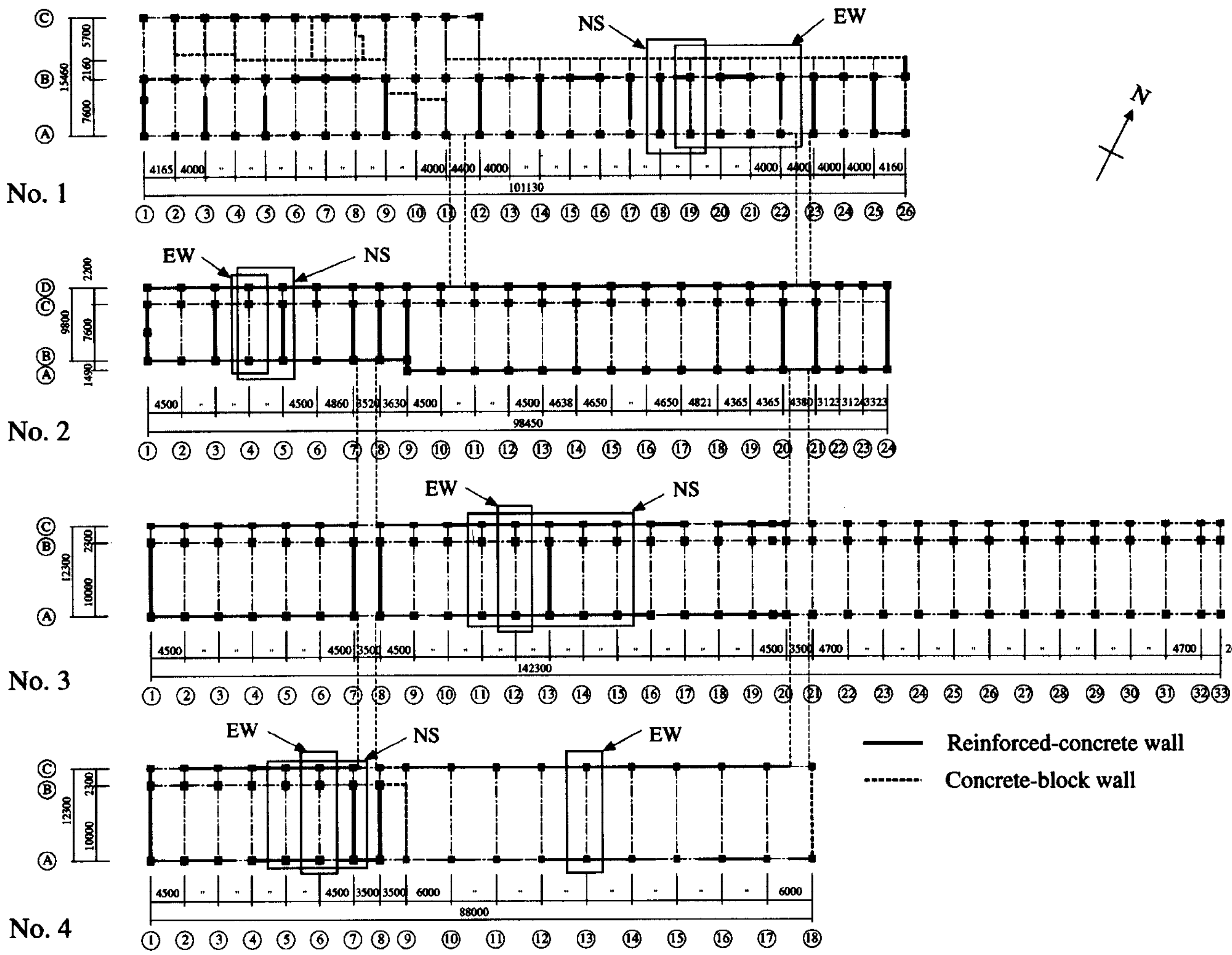


Fig. 6. Representative damaged frames for the dynamic response analyses, surrounded by the boxes, of the four buildings at KTH. The frames in the boxes indicated by EW and NS were selected for the input motions of Figure 4 (e) and (f), respectively. The sway and rocking springs connect the foundations to the soil to account for the effect of the soil-structure interaction.

RESULTS OF THE DYNAMIC RESPONSE ANALYSES

The dynamic nonlinear response analyses, used in the structural design of tall or base-isolated buildings in Japan, were performed by the DAC3N computer code (Watanabe *et al.*, 1989). In Figure 7, the circles are the maximum shear coefficients in the EW direction during the Kushiro-Oki earthquake obtained by the dynamic analyses. The triangles show the ultimate earthquake-resistant capacity translated into the shear coefficients. The capacity was defined at the time when the building formed a collapse mechanism under horizontal force proportional to the seismic load in the present Japanese code by the pseudo dynamic analyses. The results show that the No. 1 and No. 2 buildings may have been damaged severely in the Kushiro-Oki earthquake.

Figure 8 shows some of the ductility ratios of the EW direction obtained here. The No. 1 building had suffered moderate damage in the Kushiro-Oki earthquake, and the analysis results showed that some very short columns on the first floor yielded in shear. The No. 2 building had suffered severe damage in the earthquake, and the analysis results showed that many short columns on the first and second floors yielded in shear. The No. 3 building had been damaged slightly, and the analysis results showed that some columns on the first and second floors yielded in flexure. The No. 4 building had been damaged little, and the analysis results showed that some columns on the first floor yielded a little in flexure.

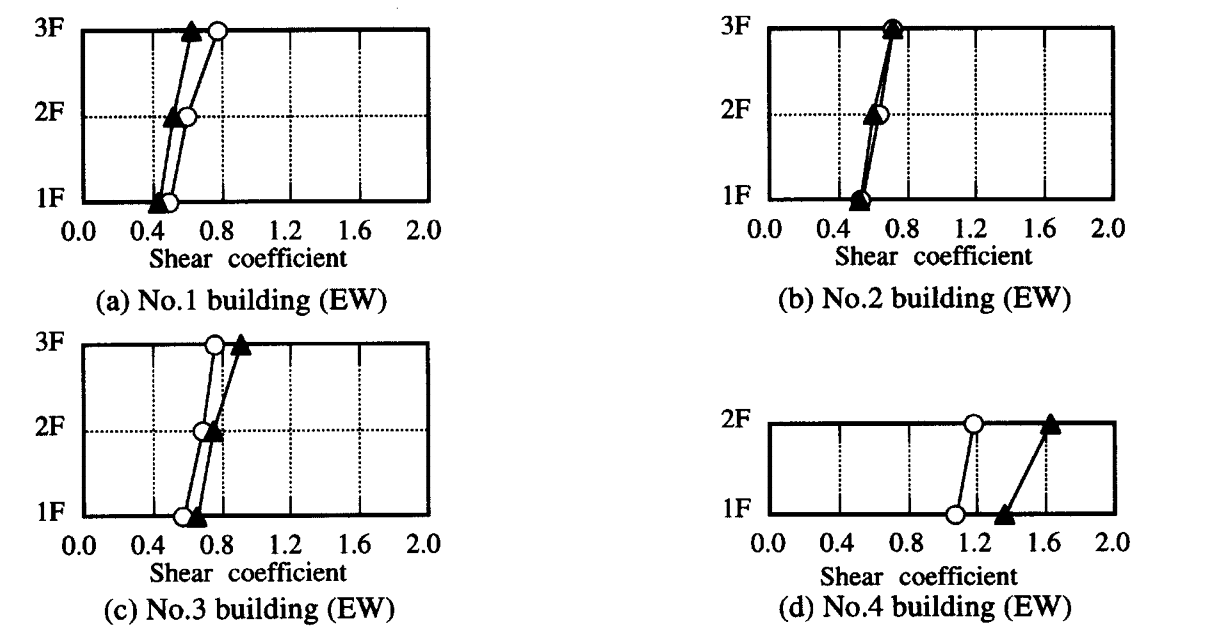


Fig. 7. Comparison of the maximum shear coefficients during the 1993 Kushiro-Oki earthquake obtained by the dynamic response analyses (circles) and the ultimate earthquake-resistant capacity calculated by the pseudo dynamic analyses (triangles).

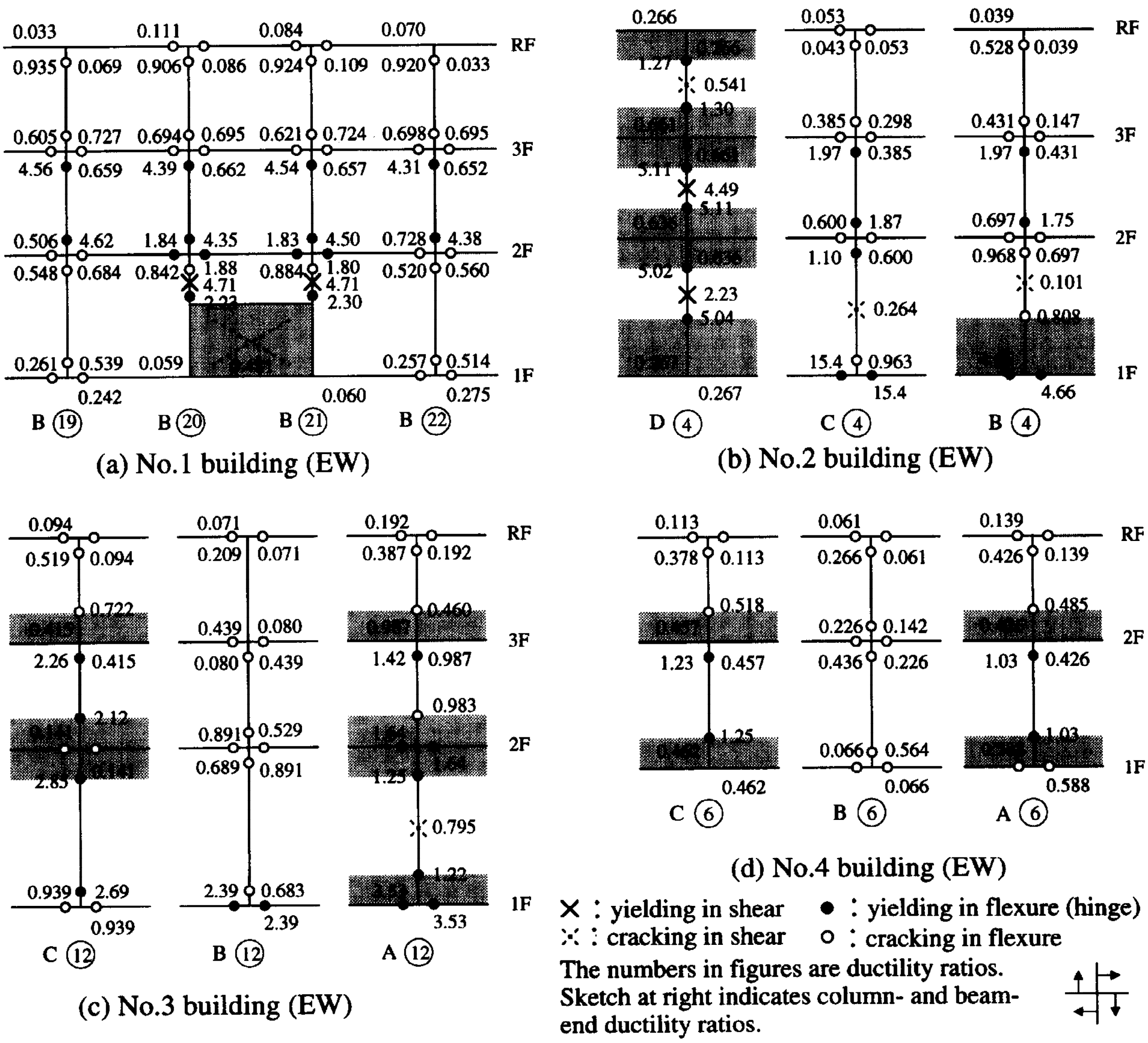


Fig. 8. Ductility ratios obtained by the dynamic response analyses.

K. DAN 8

CONCLUSIONS

The dynamic response analyses of the four damaged buildings in this paper verified that the behavior of structures calculated in the design process could indeed represent the actual behavior of those structures during an earthquake, when the input motions were evaluated appropriately and the structures were modeled accurately. However, larger peak accelerations and velocities were recorded during recent destructive earthquakes, such as the 1994 Northridge, USA, earthquake and the 1995 Hyogo-Ken Nambu, Japan, earthquake. This observation showed that even larger seismic force might have acted on buildings, resulting in huge life and property losses. For a further study, it is necessary to evaluate the input motions during these earthquakes and to model the buildings independently, as we did here, and the point is to simulate the behavior of little damaged buildings as well as that of severely damaged ones.

ACKNOWLEDGMENTS

The authors wish to express their appreciation to Mr. T. Kashima of Building Research Institute, Ministry of Construction, Japan, for the digital data of the strong-motion records on the ground at the Kushiro District Meteorological Observatory and to Mr. M. Hirose of Kushiro Technical High School for the data of soil investigation at the school. The acknowledgment is extended to Prof. O. Joh and Prof. Y. Goto of Engineering Faculty, Hokkaido University, for the detailed information of the damaged buildings at Kushiro Technical High School.

REFERENCES

Dan, K., K. Watanabe, M. Kikuchi, and M. Ebihara (1993). Structural damage potential of strong-motion records at the Kushiro District Meteorological Observatory from the 1993 Kushiro-Oki earthquake (*M* 7.8). *Journal of Structural and Construction Engineering, Architectural Institute of Japan*, **454**, 51-60 (in Japanese with English abstract).

Dan, K. (1995). Simulation of soil and structure interaction observed in the earthquake motions at the Kushiro District Meteorological Observatory. *Journal of Structural and Construction Engineering, Architectural Institute of Japan*, **470**, 75-84 (in Japanese with English abstract).

Fukutake, K., A. Ohtsuki, M. Sato, and Y. Shamoto (1990). Analysis of Saturated dense sand-structure system and comparison with results from shaking table test. *Earthquake Engineering and Structural Dynamics*, **19**, 977-992.

Giberson, M.F. (1969). Two nonlinear beams with definitions of ductility. *Journal of the Structural Division, Proceedings of the ASCE*, **95**, **ST2**, 137-157.

Hokkaido Architect Association, Kushiro Branch (1982). Soil in the city of Kushiro (in Japanese), [Hokkaido Kenchikushi-Kai, Kushiro-Shibu (1982). Kushiro no jiban]

Kabeyasawa, T., H. Shiohara, S. Otani, and H. Aoyama (1983). Analysis of the full-scale seven-story reinforced concrete test structure. *Journal of the Faculty of Engineering, the University of Tokyo (B)*, **XXXVII**, **2**, 431-478.

Takeda, T., M.A. Sozen, and N.N. Nielsen (1970). Reinforced concrete response to simulated earthquakes. *Journal of the Structural Division, Proceedings of the ASCE*, **96**, **ST2**, 2557-2573.

Watanabe, K., N. Fukuwa, K. Ohsawa, Y. Hayashi, M. Kaneko, K. Tamura, and Y. Koyanagi (1989). Development of dynamic analysis code for 3-dimensional nonlinear problems (DAC3N). *Proceedings of the 11th Symposium on Computer Technology of Information, Systems and Applications, Architectural Institute of Japan*, 181-186 (in Japanese with English abstract).

Yoshida, K. (1994). A comparative study on dynamic impedance functions for surface foundations on an elastic half space. *Journal of Structural and Construction Engineering, Architectural Institute of Japan*, **457**, 19-28 (in Japanese with English abstract).