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SEISMIC PERFORMANCE OF MODEL STRUCTURES DESIGNED BY OLD AND NEW JAPANESE SEISMIC DESIGN CODES, AGAINST THE 1995 KOBE EARTHQUAKE GROUND MOTIONS

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ABSTRACT

On January 17, 1995, the Hyogo-Ken Nanbu Earthquake (Kobe-Earthquake) struck the Hanshin-Awaji region, Japan. Numbers of buildings were damaged or collapsed by this earthquake. In paticular, many old buildings designed by the old seismic provision were suffered serious structural damage. In this paper, the earthquake response analysis was conducted in order to perform following three objectives; 1) the differences in seismic performance of building structures designed by the old and new seismic provisions. 2) the features of the Kobe earthquake record in terms of damage potential for structures. 3) the influence of vertical earthquake ground motions on the overall response of building structures.

KEYWORDS

Kobe earthquake; earthquake response analysis; seismic performance; seismic provision; vertical earthquake motion.

INTRODUCTION

On January 17, 1995, the Hyogo-Ken Nanbu Earthquake (Kobe-Earthquake) of magnitude 7.2 struck the Hanshin-Awaji region, Japan. This earthquake resulted in severe damage and loss on a massive scale; numbers of buildings were also damaged or collapsed. In particular, it has been clearly found that most of the old buildings constructed before 1981 when the Japanese seismic design code was drastically amended, suffered serious structural damage, while almost all buildings newly built after 1981 performed very well. The reasons for such discrepancy in structural performance between old and new buildings should be more deeply investigated in order not only to further improve current seismic provision but also to rehabilitate a lot of existing buildings designed by the old code.

Thus, the objective of the paper is, firstly, to clarify the above reasons. That is, by setting appropriate model structures designed by the old and new seismic provisions, the differences in seismic performance were extracted by time history dynamic response analyses using the recorded earthquake ground motions at the Kobe Earthquake. The second objective is to identify the features of the Kobe earthquake record in terms of damage potential for structures. This was also done by dynamic response analysis using the same model structures and the Kobe and other major recorded earthquake ground motions that have been used in designing real buildings. The third objective is to investigate the influence of vertical earthquake ground motions on the overall response of building structures, since some investigators suggested that the severe damage was caused mainly by vertical ground motions.

ANALYZED FRAMES AND INPUT EARTHQUAKE MOTIONS

Design of model frames for analyses

As the model building structures for inelastic response analysis in this study, two-, four-, and eight-storied moment resisting steel frames were carefully chosen with basic configurations. Fig. 1 shows the eight-storied frame as an example for the model building structures in this paper. The story heights of all analyzed frames were assumed to be 3.75 m, but only 1-st story was set to be 4.0 m. Then, dimensions of members' sections of these frames were determined on the basis of requirements of the old and new seismic design provisions. Fig. 2 shows the design seismic force coefficients in order to determine the members' sections for each frame. Thus, we had six model structures for the inelastic response analysis as shown in Table 1.

Analytical conditions

A computer program for the dynamic response analysis is on the plane frame base with plastic hinges at the ends of each member. The restoring force characteristics of inelastic elements are assumed to have elasto-plastic bi-linear relationship. Further, the program was developed to be able to deal with inelastic behavior of panels of beam to column connections, P-delta effects and vertical ground motions. The inelastic response analysis in this paper was conducted on the basis of the following analytical conditions.
1) The second slopes of the bi-linear are assumed to be 2% of the first slope for beams and columns, and 1% for

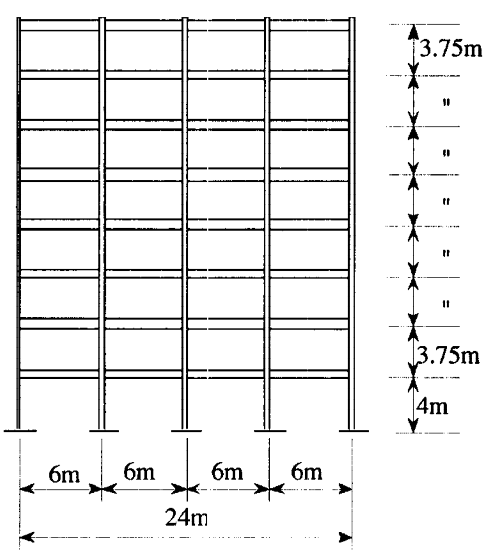
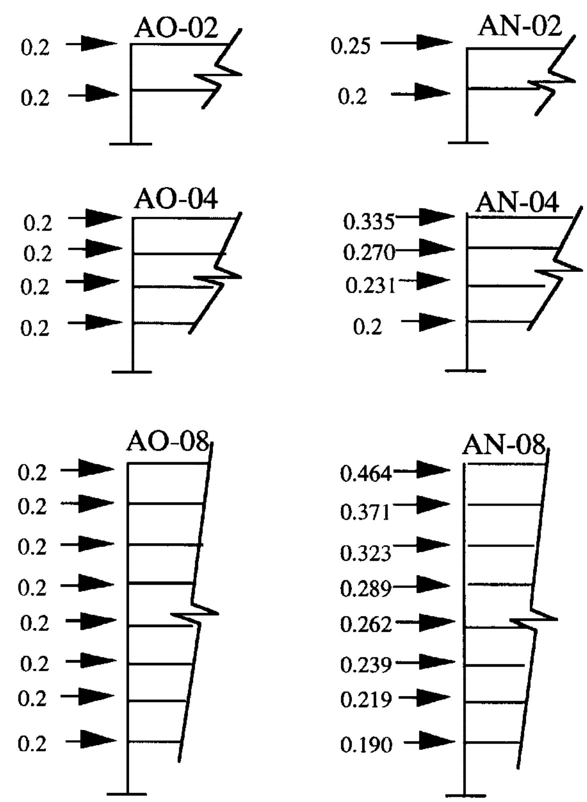


Fig.1 Analyzed frame (8-storied)

Table 1 Symbols of analyzed frames with total height

Story	Total height (m)	Old seismic design	New seismic design
2	7.75	AO-02	AN-02
4	15.25	AO-04	AN-04
8	30.25	AO-08	AN-08



old seismic design new seismic design

Fig. 2 Design sesimic force coefficients of analyzed frames

joint panels.
2) The column bases of the first story are assumed to be rigid.
3) Yield strength of all members for the inelastic analysis is set to be 1.1 times of nominal yield strength.
4) A half of mass on a beam is concentrated into the center of the beam, the other half mass is equally devided and distributed to the top of columns comected to the beams.
5) The damping coefficient is assumed to be 2% as the stiffness proportional damping.

Input earthquake ground motions

One of the input acceleration waves used for the analyses was, in the first place, the 3-D accelerometer record at the Kobe Observatory of Japan Meteorological Agency; the maximum accelerations were 818 Gal, 621 Gal and 332 Gal in the directions of N-S, E-W and U-D, respectively. Then, for the comparison, the acceleration records of the four past earthquakes, the 1940 El centro, 1968 Hachinohe, 1994 Northridge (Tarzana), 1994 Kushiro-Oki earthquakes, were used. Table 2 shows input earthquake motions using the dynamic response analysis. The accelerations of El centro NS and Hachinohe EW for this analysis are factored such that the maximum velocity of these earthquake ground motions are equivalent to 50cm/sec.. On the other hand, as the input earthquake ground motions of Northridge, Kushiro-Oki and JMA Kobe NS, these original accelerograms are used in the analysis. Fig.3 shows energy spectrum (Akiyama, 1985) of the above five input earthquake ground motions. From this figure, it was found that the values of input energy of Kobe NS record are considerably larger during the fundamental natural periods of 0.3-1.6 sec. than those of the other earthquake records.

Table 2 Input earthquake ground motions

Earthquake motions	Maximum velocities	Maximum accelerations	Duration of accelerograms
El centro NS	50 cm/sec.	511Gal	20 sec.
Hachinohe EW	50 cm/sec.	255Gal	20 sec.
Northridge	110 cm/sec.	1745Gal	30 sec.
Kushiro-Oki EW	33.5 cm/sec.	711Gal	50 sec.
JMA Kobe NS	87 cm/sec.	818Gal	30 sec.
JMA Kobe NS+UD	87 cm/sec.	818Gal,332Gal	30 sec.

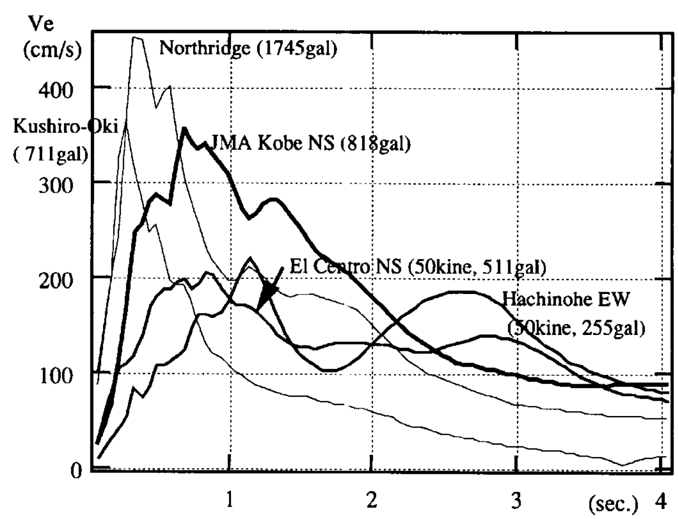


Fig. 3 Energy spectrum

RESULTS OF INELASTIC RESPONSE ANALYSIS

Natural periods of analyzed frames

Fig. 4 shows the fundamental natural periods of the six analyzed frames. The fundamental natural periods of the frames designed by the old seismic provisions in Japan are rather longer than that by the new seismic provisions.

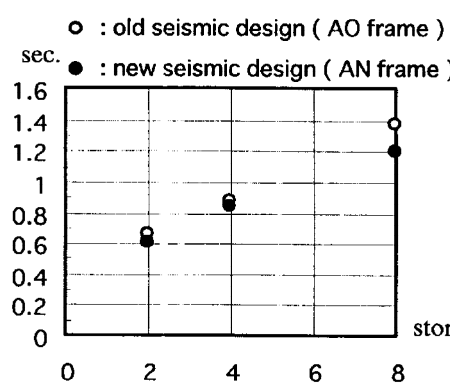


Fig. 4 Natural period of analyzed frames

Static inelastic analysis

Fig. 5 indicates the story strength - deformation relationships of the six frames obtained from incremental nonlinear analysis. Abscissa indicates story-shear (*Qi*) divided by total weight of the frame (*Wt*), ordinate indicates inter-story drift angle. It is found that the story-shear of the frames designed by the old seismic provision (AO frame) are smaller than that by the new seismic provisions (AN frame) . The maximum drift angle of the upper stories in the eight-storied frame designed by the old seismic provision (AO-08) are considerably larger than the other stories in this frame. This is because the horizontal load-carrying capacities of the upper stories in the AO-08 frame are designed to be smaller than those of the other stories.

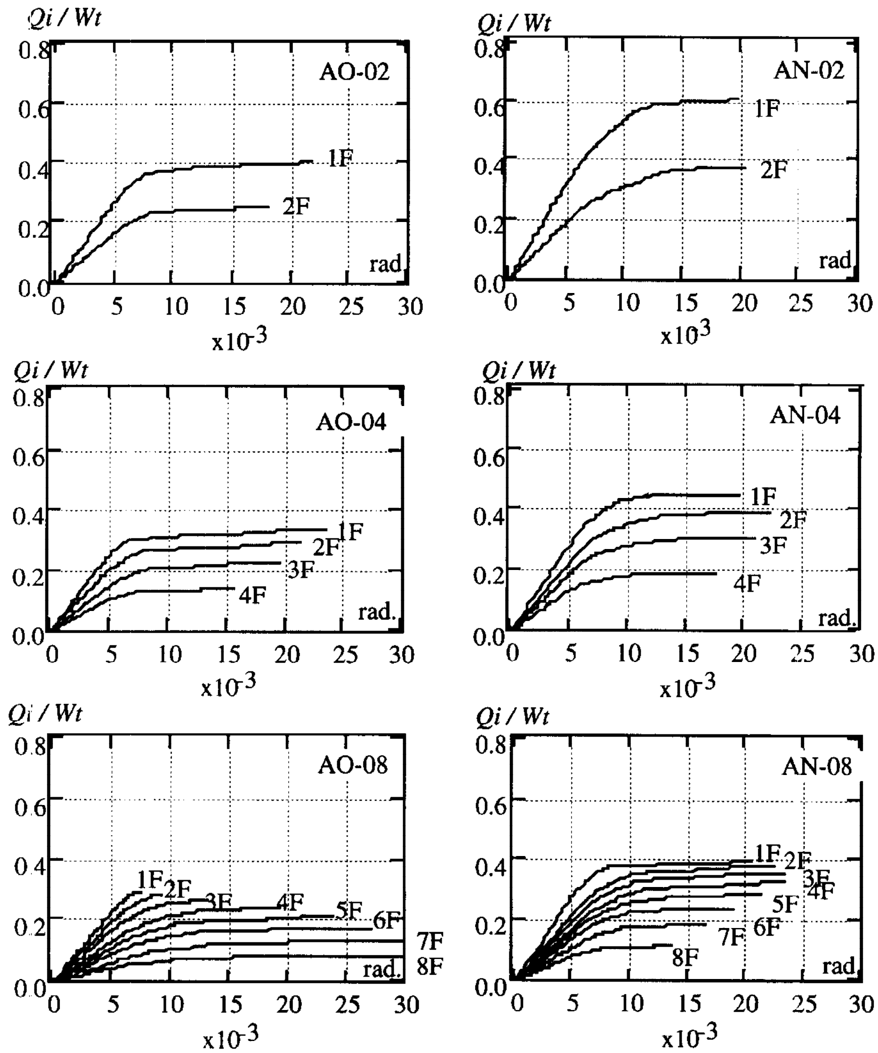


Fig. 5 Story strength - deformation relationships

Dynamic response analysis

Fig. 6 shows the maximum inter-story drift angle of the six frames against each five input earthquake ground motions. The maximum inter-story drift angles against the Kobe NS and Northridge earthquake motions are larger than those against other earthquake motions. In particular, the maximum inter-story drift angles of two-storied frames against the Kobe NS are more than 0.04rad.. Comparing the response of each frame designed by the old and new seismic provisions, it is found that the response of the tow- and four-storied frames are almost similar in terms of the maximum inter-story drift angles; on the other hand, the maximum inter-story drift angles of the upper stories in the eight-storied frame designed by old seismic provisions (AO-08) are larger than those of the AN-08 frame designed by the old seismic provision.

Fig. 7 shows the damage distribution along the height of the four-storied and eight-storied frames which were designed by both old and new seismic provisions against the Kobe NS earthquake ground motions. The damage index of each story (η_f: cumulative ductility) is defined by the following equation.

$$\eta_f = \frac{W_f}{Q_p \cdot \delta_p} \qquad (1)$$

where, Wf : cumulative energy absorption of each story, Qp : horizontal load carrying capacity of the story obtained from Fig. 5, and δp : elastic inter-story drift of the story corresponding to Qp. From this figure, it is found that damage concentration is occurred into the upper stories of the eight-storied frame designed by the old

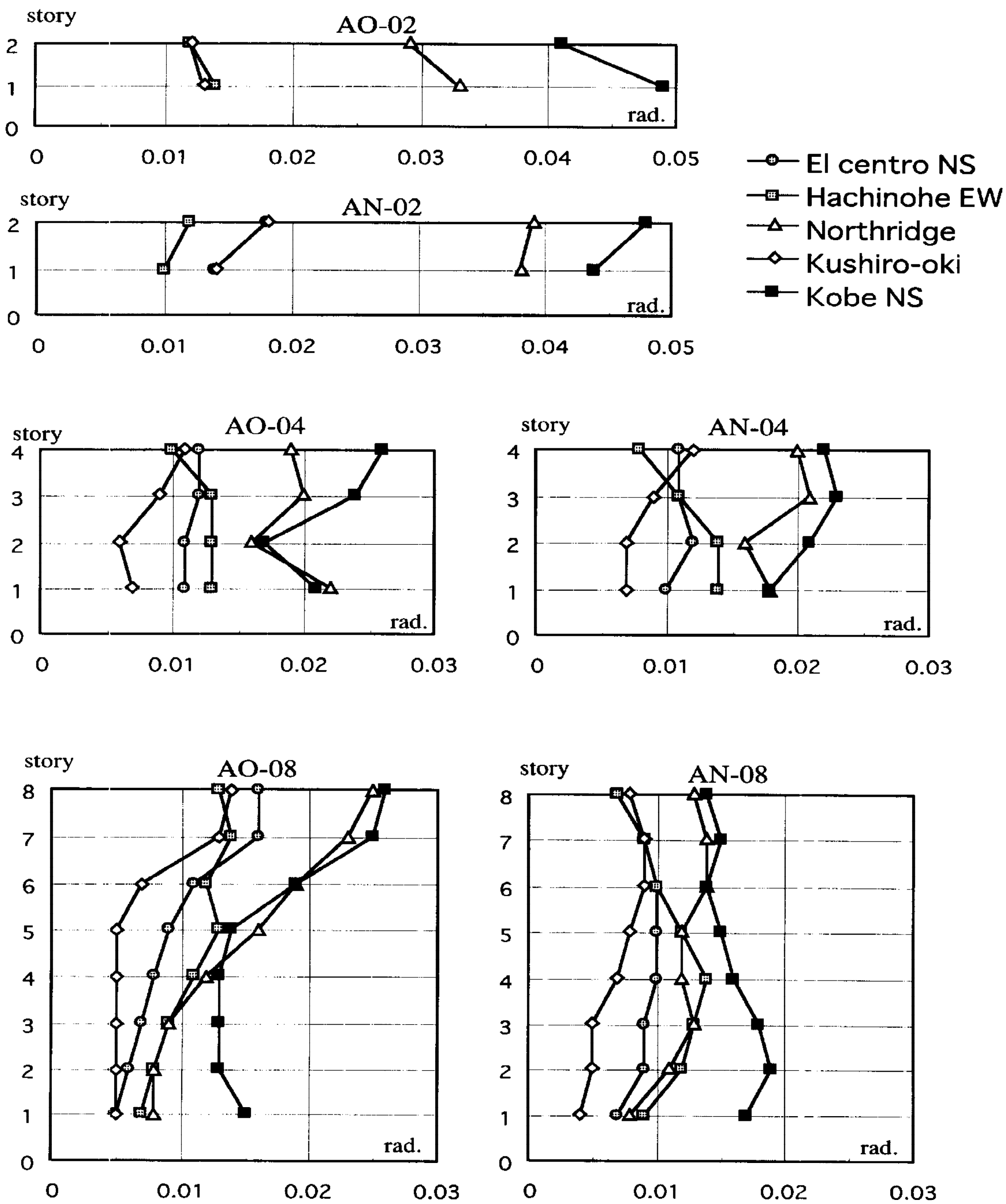


Fig. 6 Maximum inter-story drift angles

seismic provisions (AO-08). This is owing to the difference in seismic design forces along the height of buildings, required by the old and new codes.

Fig. 8 shows the maximum damage of beam-end in each story of the four- and eight-storied frames (AN-04, AN-08) against the Kobe NS and Northridge earthquake ground motions. Here, the damage index of the beam (η_b : cumulative ductility) is defined by the following equation.

$$\eta_b = \frac{W_b}{M_p \cdot \theta_p} \tag{2}$$

where, *Wb* : cumulative energy absorption of a hinge of the beam, *Mp* : full plastic moment of the beam, *and* θ_p : elastic rotation angle of the beam under untisymmetric moment (*Mp*). From this figure, it is found that the energy absorption capacity (η_b) of the beam-end in order to prevent the fracture of beam flange needs about 20 against the Northridge earthquake, and more than 40 against the Kobe earthquake in case of the AN-04.

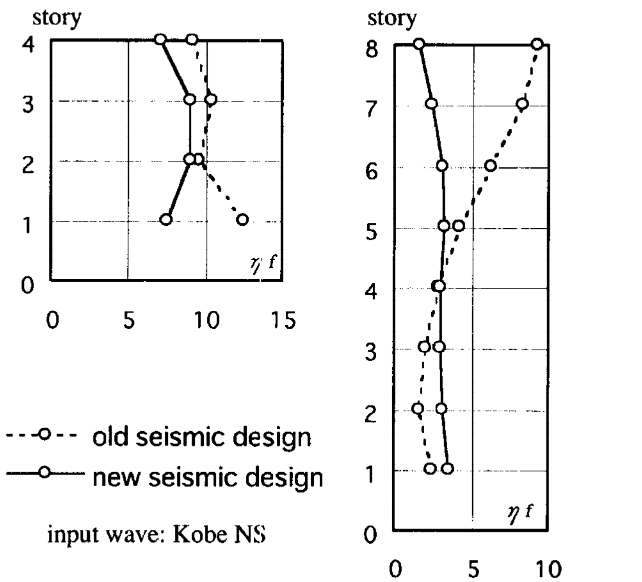


Fig. 7 Damage distribution along height of frames

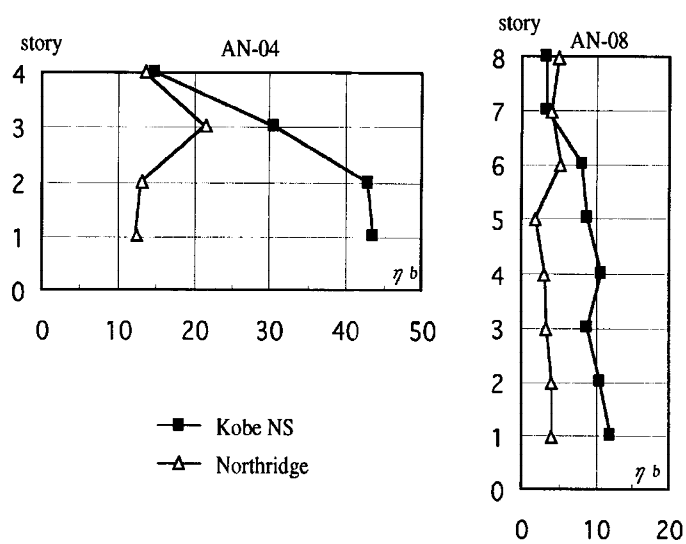
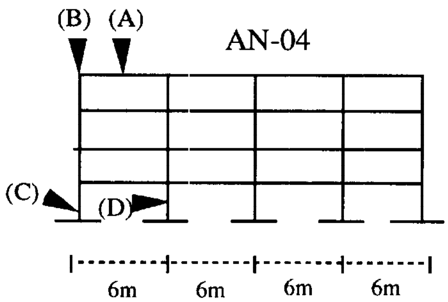


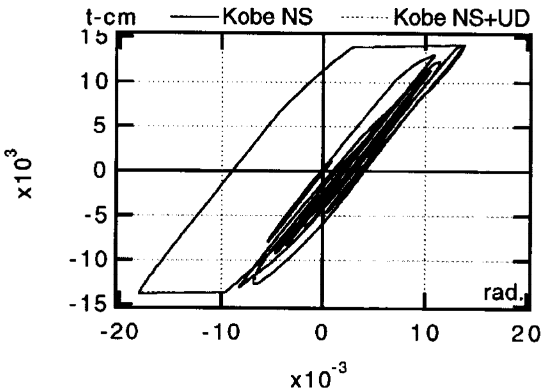
Fig. 8 Maxumum damage of beam-end

Influence of vertical earthquake motion

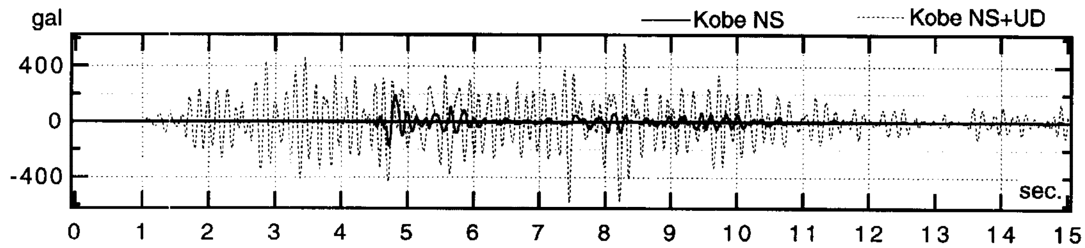
The influence of vertical earthquake ground motions on the overall response of building structures was investigated using the Kobe earthquake motions. Fig. 9 shows both responses of the four-storied frame(AN-04) against the Kobe NS earthquake motions and against the Kobe NS+UD earthquake motions. The maximum vertical acceleration at the center (A) of the top floor beam against the Kobe NS+UD earthquake is more than 500 Gal. On the other hand, the vertical earthquake motion input has a little influence (less than 100 Gal) both on the vertical acceleration response at the column top (B) and on the axial force response in the out-side column (C). This is because, the axial force response in the exterior column (C) is susceptible to be affected by the horizontal earthquake motions. The axial force response in interior column (D) is affected by the vertical earthquake motion input, but the values of the maximum acceleration is less than 200 Gal. The vertical earthquake motion input has also few effect on *M*- θ relationship of the column (C).

Fig. 10 shows the maximum inter-story drift of the three frames (AN-02,-04,-08) against the Kobe NS motions and NS+UD motions. The maximum inter-story drifts of the all frames against the NS+UD are almost similar

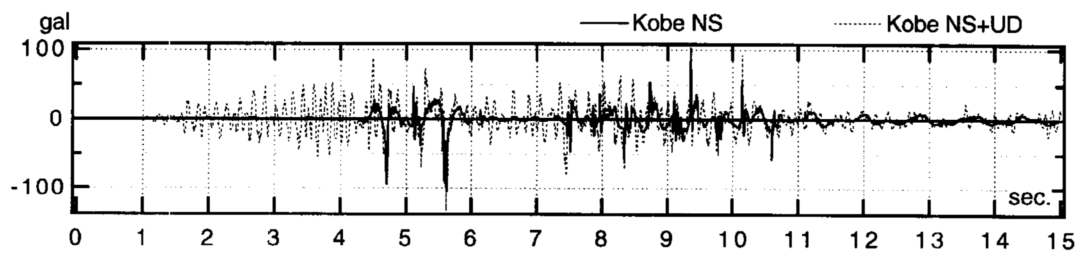




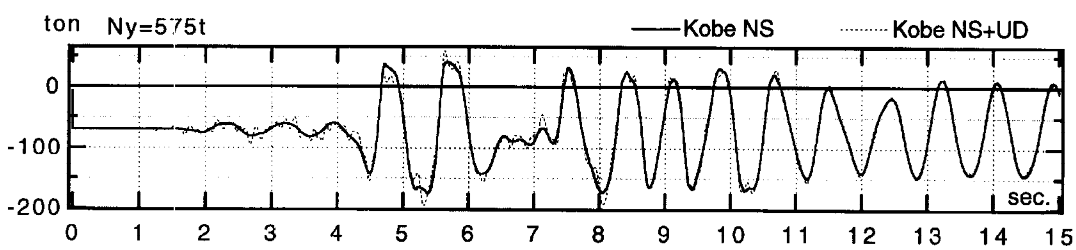
Moment-rotation relation in the column (C)



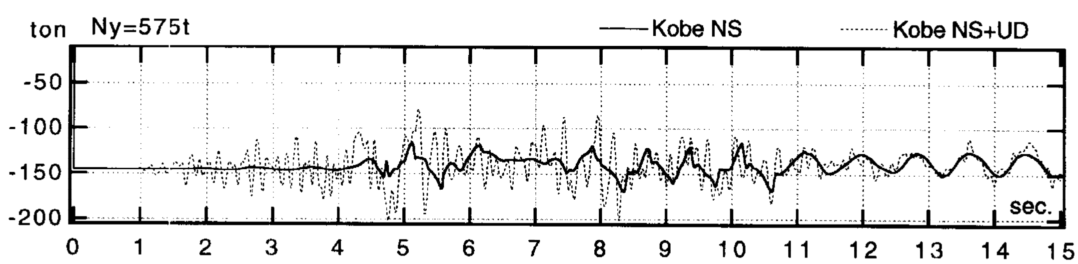
Vertical acceleration response at the center of the beam (A)



Vertical acceleration response at the column top (B)



Axial force response of the column (C) in 1st story



Axial force response of the column (D) in 1st story

Fig.9 Influence of vertical earthquake ground motions on response of the frame to the response against the Kobe NS. From these results, the vertical earthquake ground motions has a influence on the vertical acceleration response at the center of beam, but almost no influence on the maximum inter-story drifts and the damage of columns.

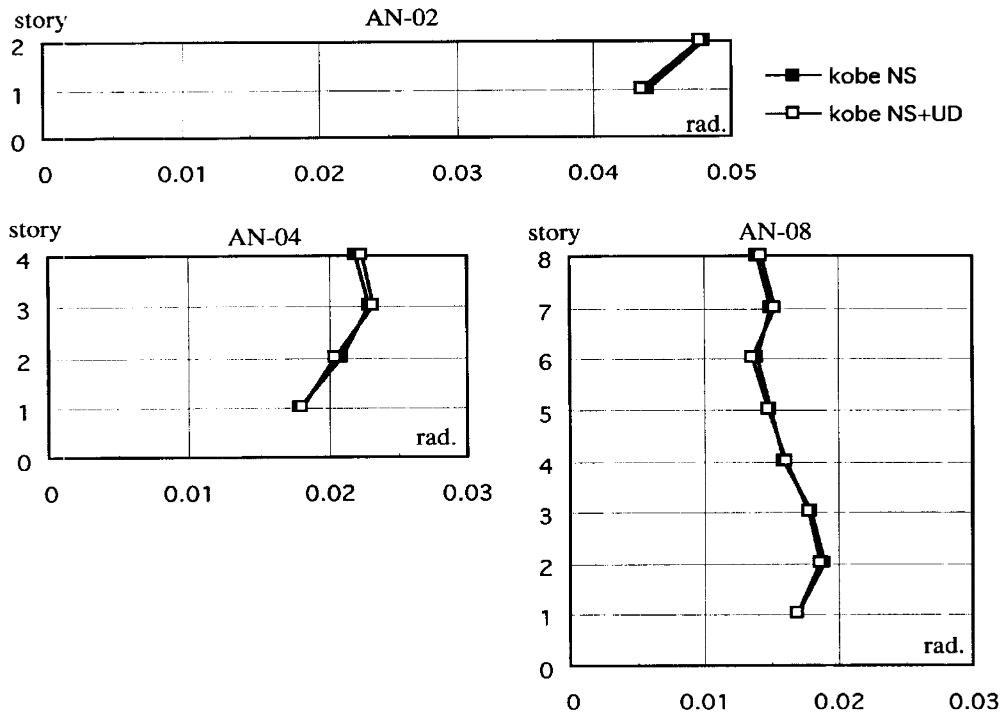


Fig. 10 Maximum inter-story drift

CONCLUSIONS

In this paper, a set of the earthquake response analyses was conducted on the six frames designed by the old and new seismic provisions in order to investigate the difference of structural performance against the 1995 Kobe earthquake ground motions. From the results of analysis, the following conclusions can be obtained;

1) The eight-storied frame designed by the old seismic provisions exhibited concentric damage at the upper stories. This is owing to the difference in seismic design forces along the height of buildings, required by the old and new code.

2) The damage potential of the Kobe earthquake for building structures was found to be considerably larger than that of the other past major earthquakes, in particular for low- and medium-rise buildings.

3) Vertical earthquake ground motions has a influence on the vertical acceleration response at the center of beam, but no significant influence on the overall frame responses.

REFERENCES

Akiyama, H. (1985). Earthquake-resistant limit -state design for building. University of Tokyo Press.