DAMAGE TO REINFORCED CONCRETE BUILDINGS DUE TO THE OITA EARTHQUAKE OF APRIL 21, 1975

by

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SYNOPSIS

Dynamic response of the main building of the K.L.Hotel, damaged beyond repair during the Oita earthquke, is analyzed and compared with the investigation after the earthquake, the principal cause of the severe damage is identified.

INTRODUCTION

The intensity of ground motion near the epicenter during the Oita earthquake was rather strong, resulting in damage to several structures.

The main building of the K.L.Hotel, a four-story reinforced concrete structure located about 2.5 kilometres from the epicenter was heavily damaged and collapsed partially. On the other hand, another three-story reinforced concrete hotel building, 350 metres apart from the former and having almost the same floor area, was slightly damaged. Both buildings had been designed according to the requirements of the same code and constructed in sequence. The contrast might have resulted from the difference in alignment and the amount of shear wall between the two buildings.

This paper concerns the details of their structural damage and the collapse process of the four-story building through the use of dynamic analysis.

STRUCTURAL SYSTEM

As the site slopes from the south to the north, the hotel gives the appearance of a five-story building from the north. The ground story is underground on the south and east sides. The plan of the upper four stories of this reinforced concrete building is composed of A, B and C blocks arranged in that order from the west (Fig. 1). The ground story and first story extend in plan beyond the upper stories. Each block is separated by a wooden form of 1.5 centimetres thickness (construction joint), but the foundation constructed as a single unit by raft foundation.

The relationship between soil conditions and structure is presented in Fig. 2. The structure consists primarily of a reinforced concrete frame. The columns used were reinforced with tied-bars. Shear walls belonging to A and C blocks were provided in the upper three stories and The ground story, but only to a limitted extend in the first story. The stiffness and strength of the first story were therefore smaller than the upper three stories and the ground story.

PRINCIPAL DAMAGE

The severe structural damage to the building was concentrated in the first story of each block. The upper stories and the ground story were slightly damaged. C block collapsed due to brittle failure of the column in the first story supporting the upper three stories. The earthquake caused A block large permanent deformations which consisted of translation

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towards the north-west combined with anti-clock wise rotation of the structure above the first floor. Permanent relative deformations between the first and the second story up to 3.0 centimetres, and sinking of a 7-frame column line with respect to a 2-frame column line of up to 4.0 centimetres were measured after the earthquake. Damage to the expansion-joint due to impact of each block was not detected. Similar failures in each block occurred in the tied columns in the first story supporting heavy permanent axial loads (Fig. 1, Fig. 3). Although columns and shear walls suffered considerable spalling and cracking, many of them except in C block, were able to develop significant ductility (Fig. 5). Since all of the spalling was inclined to the north side, the north-south component during the earthquake was considered to be more intensive than the east-west component.

GROUND MOTION

The K.L.Hotel was located less than 2.5 kilometres from surface faulting in the zone of most intense shaking. The ground motion at this site was, unfortunately, not recorded. The accelerogram obtained in the area of the strongest shaking was the one at Oita Steel Foundry of shin-nippon Steel Company, which had peak values of 0.06g in the horizontal motion and 0.02g in the vertical motion.

The peak acceleration during the earthquake is estimated to have a maximum value of 0.42g. Several after-shock accelerograms obtained at the site indicate the shocks had generated large vertical acceleration components as much as horizontals, and relatively short dominant periods on the site (Fig. 4).

For the dynamic analysis, four kinds of earthquake, El Centro, Managua. Tokachioki, and Oita aftershock were amplified to have the same peak of 0.4g.

EARTHQUAKE RESPONSE

Inelastic Response   A complete analyses, which would include coupled two-dimensional horizontal and torsional vibration, realistic inelastic material behavior and three-dimensional ground motion of this complex structure, is impractical. Consequently, a part of the three blocks of the building which appears to have vibrated primarily in the north-south direction is selected. For the purpose of inelastic dynamic analysis, the shear-force coefficient-deformation relationship are idealized as bilinear with yielding stiffness 10% of the initial elastic stiffness on shear walls and short columns, 50% on long columns. The unloading and reloading is assumed to follow a bilinear hysteresis loop considering degradation in stiffness and strength that may arise due to cracking and spalling of concrete. In considering brittle failures of shear walls or short columns, their stiffness is extinguished after failure. The mass of structure is lumped at each of the five floor levels. The damping is taken to be 2% in the first mode shape of vibration. The validity of this model is established by comparing its period and model shape of vibration with these results of vibration test for A block after the shock (Fig. 5). The agreement is satisfactory. The computed displacements are smaller than those caused by the earthquake, and are larger in proportion to decreasing amount of shear wall (Fig. 6). The drift in the second story which is inconsistent with observed damage is similar in magnitude. The ground motion at the site during the earthquake is not known. The response depends strongly on the characteristics of ground motion. Although it lacks in a number of ways, the response to the ground motions identifies some of the features of damage that occurred to each block. Drs. Shiga and Aoyama indicated after the Tokachioki earthquake that the damage is closely related to the ratio of sectional area of the shear wall to the total floor area (ratio of shear wall) and the average shear stress for gravity load of upper stories(average shear stress). Also they suggested that there is a fair chance of the building being included in the hatched zone of damage suffered during the earthquake. The relationships obtained from the data of buildings near the K.L.Hotel are plotted (Fig. 7). A and C blocks of the Hotel is included in the hatched zone. The other building out of the zone was slightly damaged.

Coupled Response As previously mentioned, the long columns suffered considerable spalling and cracking, for which two main reasons may be considered. Firstly, the elements had low shear capacities. The yielding and large deformation are concentrated in the first story. Secondly, judged from the after-shock accelerograms, axial stress was changed to a large extent. Obtaining the variance of axial stress due to coupled horizontal and vertical vibration, the six-lumped mass model of A block is translated into a one-lumped mass model. The variance of axial stress for the permanent axial stress is presented in Eq. (1).

$$V = Av \pm \chi \cdot Ah \qquad (1)$$

where: V=variance of axial stress, Av=vertical response acceleration of the model having first dominat period of 0.054 sec. to vertical component of the ground motion; Ah=horizontal response accleration of the model having translational period of 0.162 sec (Fig. 5); and χ=the ratio of the axial stress. Although the χ-value is dependent on the rigidity of the frame, and whether shear wall exists or not, D and G column line are considered here to resist the overturning moment by the translational response shear. In this work χ is 0.74, decided by the responses of the six-lumped mass model. The maximum variance is 2.7, obtained from responses to the Oita aftershock, it happens two seconds before the translational response leads to the maximum value (Fig.8). These results indicate that axial stress of the outer column during the earthquake had suffered 3.7 times compressive stress as large as permanent axial stress (nearly equals to the compressive strength of concrete), and instantly suffered large tensile stress before yielding (Fig. 1).

CONCLUSION REMARK

The preliminary dynamic response analysis presented in this work demonstrate that the severe damage to the K.L.Hotel building was due primarily to the severity of ground motion, and the large change in stiffness and strength across the second floor level, thus imposing extremely large ductility requirements the elements of the first story, which had apparently not been anticipated.

To protect buildings from damage such as that sustained by C block, it is very important to consider the average shear stress of columns and shear walls, and to consider the varriance of axial stress in the column during the earthquakes, especially for buildings near the Fault.

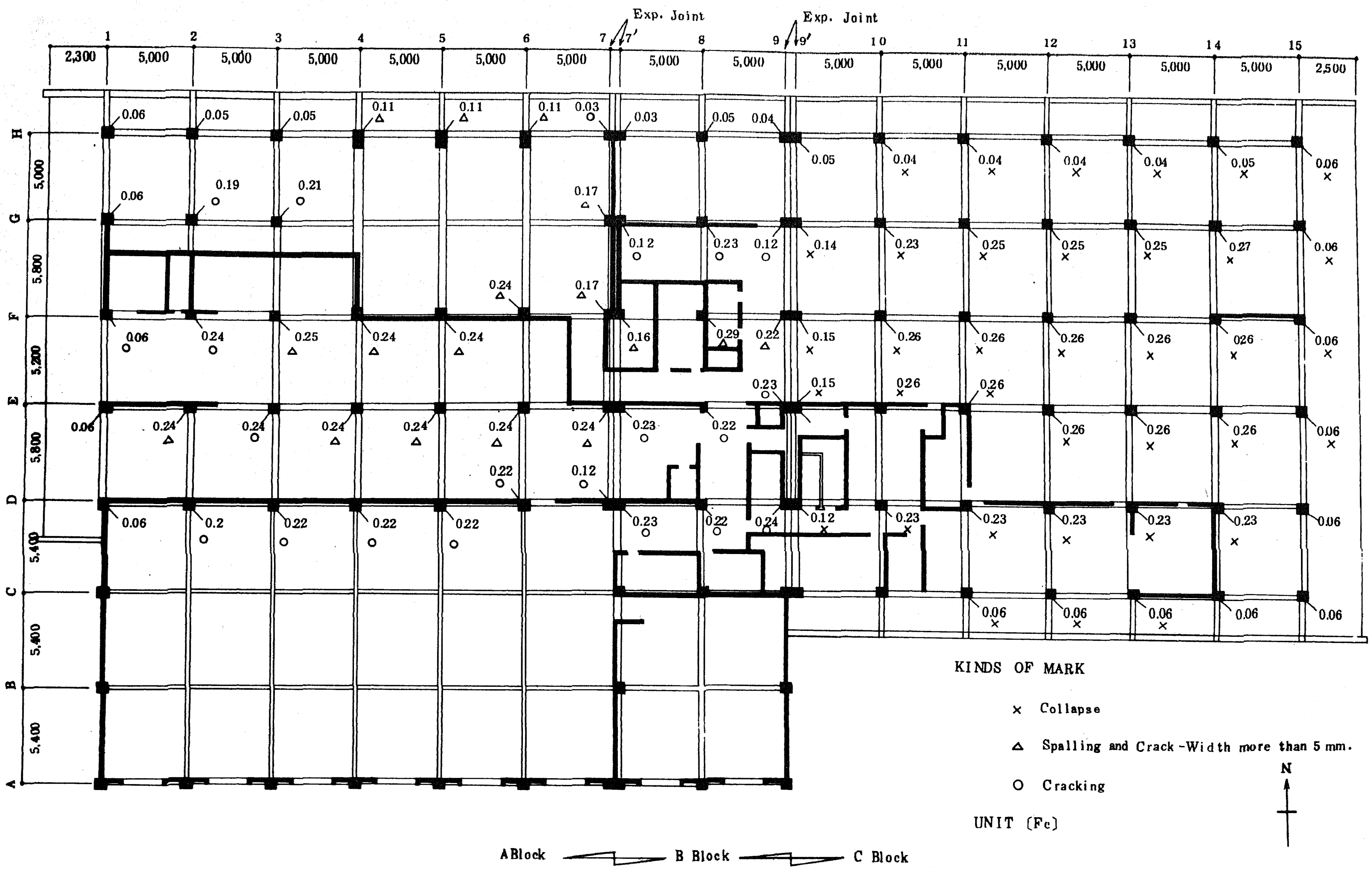

FIG.1 : K.L. HOTEL FRAMING PLAN OF FIRSTSTORY AND DEGREE OF DAMAGED COLUMNS

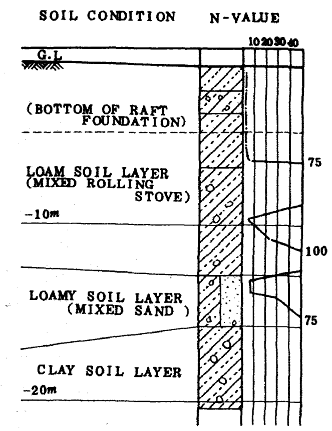

FIG.2 : SOIL CONDITION

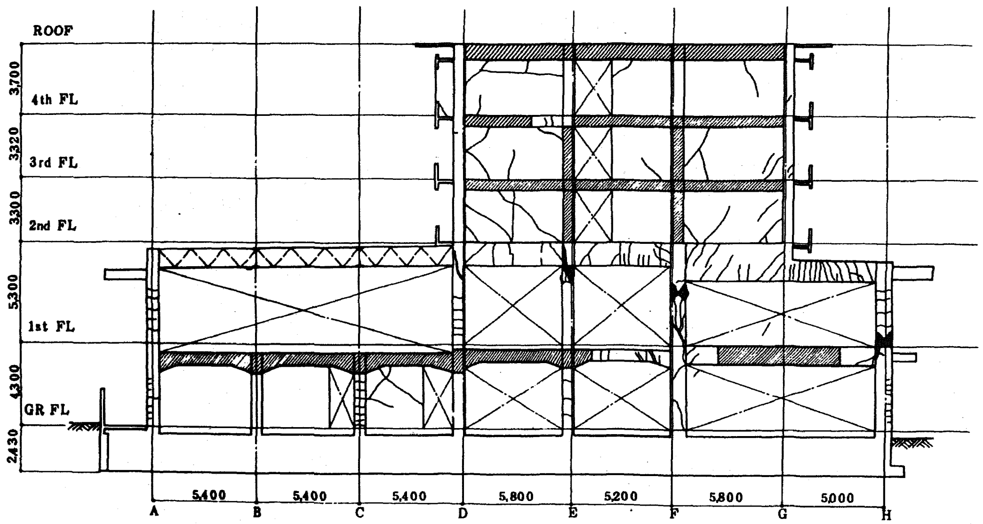

FIG.3 : SPALLING AND CRACKING IN 6 COLUMN LINE

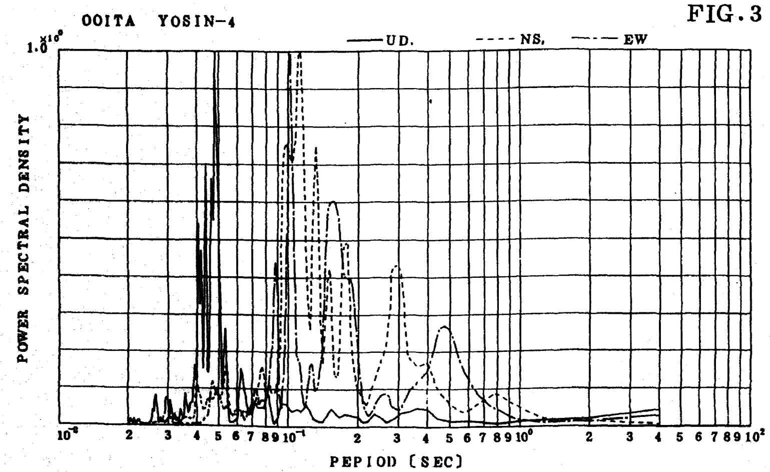

FIG.4 : POWER SPECTRUM OF OITA AFTERSHOCK

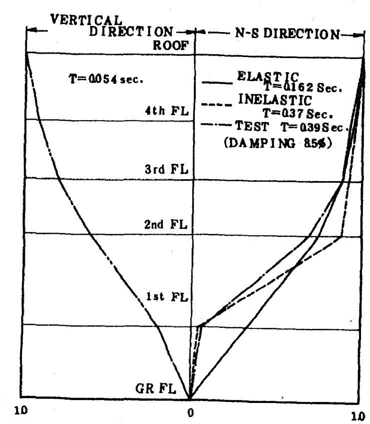

FIG.5 : FIRST MODE SHAPES AND DOMINANT PERIODS

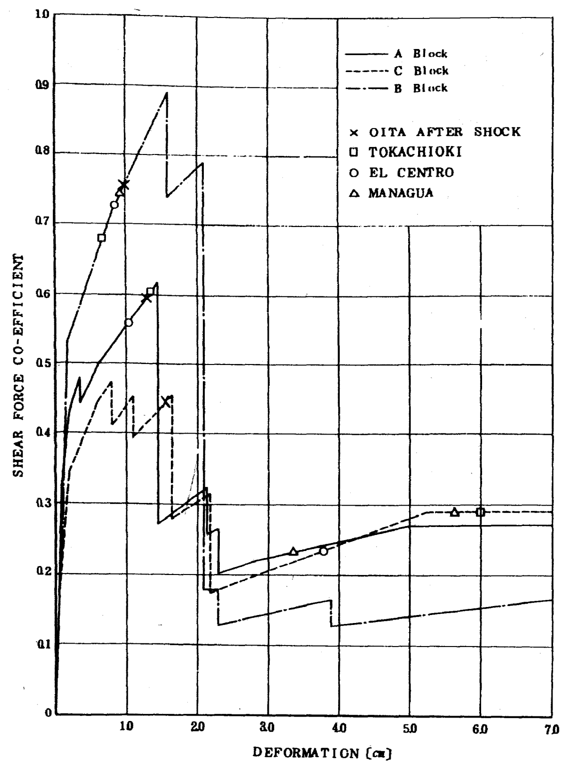

FIG.6 : RESULTS OF INELASTIC RESPONSE AND SHEAR FORCE CO-EFFICIENT-DEFORMATION RELATIONSHIP

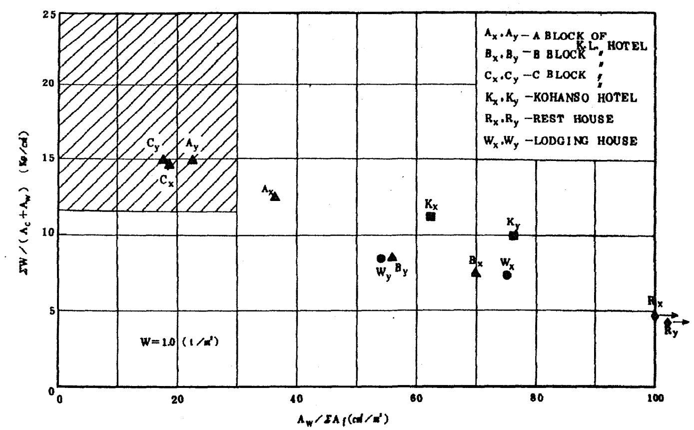

FIG.7 : DAMAGE AND RELATIONSHIP BETWEEN RATIO OF SHEAR WALL AND AVERAGE SHEAR STRESS

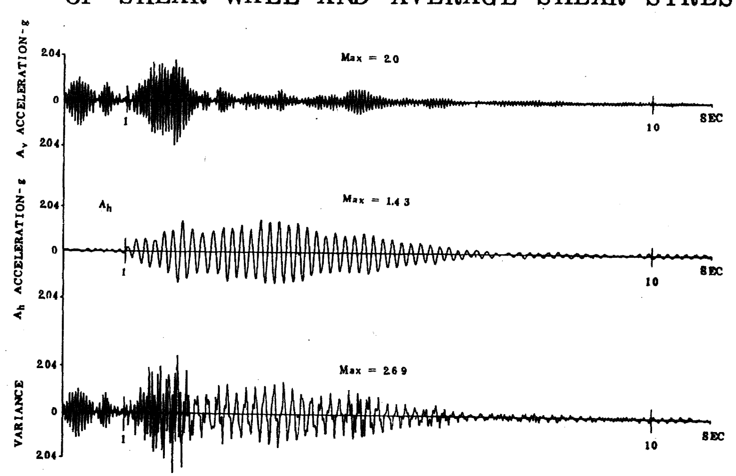

FIG.8 : RESULTS OF COUPLED RESPONSE