RESEARCH PROGRESS ON EARTHQUAKE ENGNEERING IN THE FACULTY OF ENGINEERING, UNIVERSITY OF HIROSHIMA

by

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SUMMARY

The outline of the structural division of the faculty of Engineeing are briefly described with a focus on the earthquake engineering. Each contest of the five researches going on at present in the institute is presented; confinement effects of hoops on columns., The resistance of shear walls with and without opening, in-plane resistance of slabs with girders., earthquake resistance evaluations on the exisiting building and the fundamental studies on seismic design of the equipments on the building floors .

INTRODUCTION

Japan is a highly seismic land, belonging to the earthquake link area of the pacific ocean. Buildings have been designed mainly for earthquake forces. Until recently, the value of the seismic coefficient to structural design for buildings was the highest in the world. The Tokachioki earthquake of 1968, however damaged seriously the many reinforced concrete buildings which had been believed enough strong. From this earthquake, the philosophy on seismic design for reinforced concrete buildings have been considerably changed in Japan. Experimental studies with aim to clarify the true resistance of structural components for earthquake loading have been actively conducted in the various institutes, including the faculty of Engineering, University of Hiroshima in recent ten years or more. Dynamic analysis developed first for the seismic design of high-rise buildings have also been well utilized foı the earthquake responce analyses on low-rise buildings. The Miyagi-ohi earthquake which attacked the northern Japan in 1979, provided the new aspects of the damages, such as the overturning or destruction of the various equipments on building floors. The bounds on earthquake engineering will be still more widened. In the following, the five of the researches going on in our institute are explained. Two of them have been nearly completed with the conclusions while the others are still in progress.

OUTLINES OF THE STRUCTURAL DIVISION IN OUR INSTITUTE

Introduction

The faculty of Engineering of University of Hiroshima had as its beginning the Hiroshima Higher Technological School established in 1920. By the drastic reformation of the Japanese education system in 1949, this school was intergrated with seven other schools, becoming a faculty of the University of Hiroshima. Through several times reformations doctorate courses in engineering are at present opened among national universities. The faculty of Engineering is one of the eleven faculties belonging to the University of Hiroshima. On the other hand, the faculty of Engineering consists of seven divisions as research units or graduate level courses. The seven divisions are material engineering, systems engineering, engineering of transport phenomena, design engineering, industrial chemistry, structural engineering and environmental planning and its control. There are ten laboratories in the structural division, each laboratory having a professor, an assistant prof. one or two assistants and one technical official. The ten laboratories are structural mechanics (Prof. M. Hanai), concrete material engineering (Prof. M. Funakoshi), building structures (Prof. Y. Mukudai) structure in civil engineering (Prof. H.Omura), soil mechanics and foundation engineering (Prof. T.Aboshi), disaster

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prevention engineering (Prof. M.Matsuura), earthquake engineering (Prof. T.Shimazu), strength of ship structure (Prof. M.Kawakami), strength of welding (Prof. K.Nagai), and marine structure engineering (Prof. I. Nakamura). Researches on Earthquake engineering have been conducted mainly and consistently in the Earthquake engineering laboratory in cooperation with Disaster prevention engineering lab., although very temporarily conducted in Structural machanics or building structure laboratories.

The main facilities, commonly used by all the laboratories of structural division are 3000 tons capacity hydrauric testing machine as shown in Photo. 1, 400 tons capacity testing machine and 20 tons capacity servo-actuator, all accomodated by the Structural machine room. On the other hand, a reaction bed as well as shaking table as shown in Photo. 2. belongs to the earthquake engineering laboratory. The reaction bed is a reinforced concrete I section beam, 4m wide by 3m deep by 7m long–32holes, a hole having 20ton capacity with loading frames built up fixed to bed, using some of them to apply the reversals of horizontal loads for specimens by electrially controlled jacks. The shaking table is 1m cube with one degree of freedom in the horizontal direction, excited by servo-mechainism capable of generating a maximum force of 1ton and also used as a vibrator capable of exciting building with 3tons maximum force, by attaching to buildings.

CONFINEMENT EFFECTS OF HOOPS ON REINFORCED CONCRETE COLUMNS

Introduction

Deformation capabilities of structural components are one of the important factors in seismic design of buildings. It has been recognized that hoops have the significant effects on the deformation capabilities of reinforced concrete columns. Considerably numerous experiments on reinforced concrete columns have been conducted in Japan since the Tokachi-oki earthquake attacked northern Japan in 1968. The quantative understanding have, however, not yet been sufficient due to the complicated resistance characteristics of reinforced concrete columns. The aim of this study is to establish the relationship between the deformation capabilites of reinforced concrete columns and the hoop reinforcement ratios taking into considerations the effects of the other variables, longitudinal reinforcement ratios, shear span ratios and axial load levels applied, based on the test results mentioned above. In the 6 WCEE held at New dehli in 1977, a study on the hoop effects under both concentric and eccentric loading was presented. This study deals with the case of the columns subjected to reversals of both bending and shear under various axial load levels, corresponding with actual behavious of the columns under earthquake loadings. This study has been made by Prof. T.Shimazu, one of the authors of this paper.

Outline of studies

Numerous experimental works have been conducted in the various institutes in Japan, as the synthetic project with proposal of the Building Research Institute of the ministoy of construction. In these experimental works most of column specimens were subjected to about fifty times reversals of horizontal load until failure to obtain the hysteretic loop characteristics at each deflection amplitude as illustrated in Fig.1. Also in these project, the deformation capabilities have been defined as the deformation limitation at which the load maintained is eighty percentage of maximun load after maximum load point is passed. In this paper the deformation capabilites defined above are dealt with as an index representing confinement effects of hoops on reinforced concrete columns, including the test results of the invistigation other than those of the synthetic projects. Total number of the specimens used in this study was about three hundred.

The deformation capabilities are, as mentioned before, influenced by various parameters, shear span ratio, axial load level applied, the longitudinal reinforcement ratio etc. as well as hoop reinforcement ratio.

In Table I are shown the ranges of these paraments. It should be noted that columns are in general designed so that the flexural yielding preceeded before shean failure in calculation. According to the design formula of the Reinfoced concrete codes (1971) the calculated values for flexural yield are obtained as follows.

$$My = 0.8\, a_t \,.\, \sigma_y \,.\, D + 0.5\, \eta_0 (1 - \eta_0)\, FcBD^2 \quad \text{..... (1)}$$

in which a_t, σ_y, Fc, B, D and η_0 are the area and yield strength for longitudinal reinforcements, concrete strength, width and depth for concrete section and axial load levels applied (N/FcBD) respectively, as shown in Fig. 2. On the other hand calculated values for allowable shear capacity for short loads (long loads plus horizontal load) are obtained as follows.

$$Q = bj \left\{ fs + 0.5_w f_t\, (pw - 0.002) \right\} \quad \text{..... (2)}$$

in which b, j, f_s, $_w f_t$, p_w are section width, the distance between two force centers in section (effective depth times 7/8) allowable concrete strength for short loads (concrete strength times 0.5) hoop reinforcement allowable strength for short loads (about yield strength) and hoop reinforcement ratio. In Fig. 3–5 are plotted the hoop reinforcement ratio versus longitudinal reinforcement ratios used in the column specimens in the three groups of the same shear span ratios. In the tests the equations used to design for flexural strength for column specimens were mostly the Eq(1) but those for shear strength were not always the Eq(2). Some of specimens were designed for shear strength with the so called Ōno-Arakawa equation for ultimate shear strength taking into considerations the effects of axial loads as wall as the longitudinal reinforcements ratios. The lines shown in Fig. 3 – 5, indicating the lower bound for plotted values in each figure were obtained by equalling both the equations (1) and the equation (2) times shear span a, with η_0, σ_y, $_w f_t$ and F_c assumed as 0, 3t/cm^2, 3 t/cm^2 and 0.2 t/cm^2 respectively, and with the minimum hoop reinforcements as 0.2 percentage according to the requirements of the Reinforced concrete code of Japan. In Fig. 6–8 are plotted the deformation capabilities, the limit of deformation angle as defined above versus the axial load level applied, in the three groups of the same shear span ratio. These plotted points are classified according to the values of the ratios of hoop reinforcements to longitudinal (p_w/p_t) in each figure. It is found that the deformation capabilities, as a whole increase with less values of η_0 and with higher values of p_w/p_t. It should be commented herein that no systematic relationships have been found between deformation capabilities and hoop reinforcement ratios alone, as illustrated in Fig. 9. This seems due to the fact that the higher values of shear are developed for the columns with higher values of longitudinal reinforcements, indicating that higher hoop reinforcements are needed to obtain sufficient deformation capabilities for these columns. In each of Figs.6–8 is shown a line, indicating a lower bound for the plotted values. These lines are obtained based on the following three assumptions regarding ultimate conditions.

(1) The length of yielding region (critical region) equals to D, sectional concrete depth.

(2) The average curvatures in this region is $\varepsilon / \eta_0 D$

(3) The limiting concrete strain ε is 0.003

The first assumption was adapted from the fact that flexural shear cracks for column specemens are mostly observed to concentrate on the regions of the length equalling to the depth at ultimate. The second came from the same assumption with that used for the Eq. (1) mentioned before. The neutral axis depth ratio become to be the value of η_0 as shown in Fig. 2. The third is that commonly used. From these assumptions and referring to Fig. 10, the following can be derived as estimating deformation limit for the columns without hoop reinforcements.

$$Ru = \frac{0.003}{\eta_0} \left(1 - \frac{D}{2a}\right) \quad \text{..... (3)}$$

As mentioned before, this equation can be a good estimation as lower bounds for the tested values. According to this equation. the value of Ru however become infinite as the value η_0 approachs to zero. This can be explained by assuming that the coupled resistance by both the compression and tension reinforcements are extremely ductile unless the buckling or tension-broken occurrs for both the reinforcements.

The difference of horizontal distance between the tested values and the line of Eq. (3) in Figs. 6–8 are plotted against the values of p_w/p_t in Fig. 10. It is found that the effects of p_w/p_t increase nearly linearly depending upon the shear span ratio. The slopes are nearly the same for the columns with the shear span ratios of 2, 2.5 and 3 while those are a little different for the ratios of 1 and 1.5. For the high values of p_w/p_t, the effects seem to reach the peak. The effects, that is the vertical length Y in Fig. 11. can be expressed as follows

$$Y = 0.20\,(p_w/p_t) - 0.10 \qquad a/D = 1.0 \quad \text{.....} \quad (4)$$
$$= 0.34\,(p_w/p_t) - 0.10 \qquad a/D = 1.5$$
$$= 0.54\,(p_w/p_t) - 0.10 \qquad a/D \geqq 2.0$$

For intermediate values of shear span ratios, linear interpolation can be made for the slope in the Eq. (4). Backing to the Eq. (3) and taking into considerations that the effects of Eq. (4) are logarithmic values, the following can be obtained finally including the effects of p_w/p_t values.

$$R_u = \frac{0.003 \,.\, 10^Y}{\eta_0}\left(1 - \frac{D}{2a}\right) \quad \text{.....} \quad (5)$$

The test results and the calculated values obtained by Eq. (5) are compared in Fig. 11. with the reasonable agreements.

Conclusion remarks

Compared with strengths, deformations include much more uncertain factors, particularly in the range of post-yielding regarding deformation capabilities explained above. Neverthless a numerical formulation has been attained. Further researches are needed to improve the accuracy.

EARTHQUAKE RESISTANCE OF MULTI-STORY WALLS WITH AND WITHOUT OPENINGS INCLUDING RC TRUSS FRAMES

Introduction

There have been frequent damages due to earthquakes on outer frames, particularly south-side in the longitudinal direction of building. Less resistance capabilities against earthquakes have been also pointed out for these frames because little walls have been installed. The strength as well as stiffness for shear walls are by far greater than ordinary rahmen frames. The resistance of the outer frames can be considerably improved if one or two shear walls are installed. It must be, however taken into considerations that the installation of the walls without openings are in general unfavourable for outer frames. The utilization of walls with opening or braced frames are desirable.

The aim of this study is to establish the method of estimating the horizontal resistance of walls with openings including reinforced concrete truss frames. As the first step the test results for walls with and without openings conducted in many other institutes have been reviewed. There have been little tests on walls with openings, particularly for those with greater ratio of shear span corresponding to multi-storied walls. In the following, discussions are presented only on the overall trend of the past results on various tests. Expremental works are still in progress at present on the specimens of multistoried walls with and without openings including RC truss frames designed based on the review result. The test has the other aim to use for the analyses of the buildings damaged due to differential settlements as shown in the fourth section.

The study of this section has been made by Profs. M. Matsuura and T. Shimazu and N. Yoneya, assistant of earthquake engineering laboratory.

Outline of studies

In Fig. 12 are plotted maximum strength versus shear opan ratio for walls without openings. Maximum strength was obtained by dividing with sectional area, wall width times span length, the tested values of maximum loads which have been taken from Refs. 4. The ranges of each variable used are shown in Table 2. It is found from Fig. 12 that the minimum value of maximum strength is about $2.0\sqrt{F_c}$ (F_c: concrete strength kg/cm^2) for the shear span ratio less than 1.5 and decreases with higher values of shear span ratio. In Fig. 12 are also plotted the values obtained by subtracting the effects of shear reinforcements assumed as $p_w\ \sigma_y$ (reinforcement ratio times yield strength kg/cm^2). It is interesting that minimum value without the effects of shear reinforcements is about $1.0\ \sqrt{F_c}$ being consistent with that suggested in Ref. 5 although the former and the latter are tested results for specimens with I section having two side columns and rectangular section respectively. The scatter of the maximum values can be attributed to the difference of longitudinal reinforcements in side columns and the axial load levels applied. On the other hand, the values of maximum strength from the test results for walls with openings and truss frames are plotted in Fig. 13. The maximum strength was obtained by dividing maximum load with the values of wall width times span length for walls and equivalent area for truss frames. It is found that the strength can be, as a whole, estimated about half of that for walls without openings although the shape and the ratio of openings influence the strength as shown in Fig. 16. In Figs. 14 and 15 are shown deformation angle at maximum, deflection at top level divided by height, versus shear span ratio for walls without and with opening including truss frames respectively. It is seen that the minimum values of deformation angle at maximum is about 0.4×10^{-2} rad. for both walls with and without openings and increases with higher values of shear span ratio particularly for walls with openings. The truss frames are more ductile.

Remarks

High resistance capabilities can be expected for walls even with openings or truss frames. These elements should be more utilized for the outer frames of buildings, taking into considerations that sufficient hoops must be provided for the side columns.

IN–PLANE RESISTANCE OF REINFORCED CONCRETE SLABS WITH GIRDERS

Introduction

There have been little literatures regarding the in-plane resistance of reinforced concrete slabs, in spite of fact that slabs with girders are indispensable earthquake-resistance elements connecting the vertical elements cooperatively, with the effects of determining the shear shares for each vertical elements and that there have been serveral observations on the damages of the slabs due to earthquakes. It is also necessary to clarify the in-plane resistance of reinforced concrete slabs as a basis to evaluate the resistance of the various type of slabs recently developed, such as prefabricated slabs of reinforced concrete, the steel braced slabs, or wooden slabs whose stiffnesses are assumed to be much smaller than reinforced concrete slabs. Slabs with girders are similar to shear walls with columns in the point of plate but the presence of out-plane loading as well as the reinforcement ways in the plates is quite different. Very recently two reports[6),7)] have been published on the behaviours of the reinforced concrete plates subjected to both in-and-out-plane loading but these report dealt with the simply reinforced plates without girders, subjected to pure bending. This paper deals

with the actually reinforced slabs with girders subjected to both bending and shear. These studies have been made by Prof. T. Shimazu, T. Fujinami and Y. Nakano, graduate students, Univ. of Hiroshima.

Test program

Test Specimens as shown in Fig. 17 were one-fifth scale model of the design example shown in the slab item of Reinforced Concrete Code of Japan. A specimen consists of two slabs connected each other with six-side and one-middle girders. The one side slab with three side girders was tested part subjected to horizontal load at the free end as a cantilever, while the rest part was used to fix the specimen and planned to use for another type test (for pure flexural test not yet carried out). In total eight specimens were tested and three variables were considered as shown in Table 3. The first variable was width-length ratios which was 2/3 (L type) and 3/2 (S type). The second one was the level of vertical load. The three levels of 0.4t, 1.0t and 3.0t were applied; 0.4=only dead load, taking into consideration the scale effect of one-fifth, 1.0t= dead plus live loads and 3.0t=ultimate load calculated. The third one was the amount of the flexural reinforcements in the girders, in which 6ϕ and 4.4 ϕ were used. Mechanical properties of concrete and reinforcement material are shown in Table 3. The ordinary mixed mortar was used as concrete, with the slump being 15cm on the average. Mortar was cast in horizontal position, first steam-cured and later aircured. The reinforcements used in the slabs and girders were 2.6 ϕ and 6 ϕ (4.4 ϕ). Test set-ups are shown in Fig. 18. Vertical loads were applied using thin steel plate of 50mm X 60mm at four one-four points of specimen surface and their reactions were holded by the four corness of the specimen, corresponding to columns through ropes, pulleries and steel beams. On the other hand, horizonted loads were applied at the steel channel covering the free side girders of the specimen (cantilever) with bolts, so that horizontal load can be applied distributebly along the girder through the bolts. First, vertical loads were increased at several stages up to planned level and then reversing horizontal loads were applied under constant level of vertical loading, with inplane deflection at free end controlled, loading at the complete one er two cyles of deformation angle R, deflection divided by span lengh (120cm or 80cm), = 1/2000, 1/1000 (two) 1/500, 1/250 (two) and 1/62.5. Furthermore vertical loads were again increased till failure after this horizontal loading. Horizontal and vertical displacements with the inplane deflection at free end of horizontal loading point as well as the out-plane deflection at the middle point being focused on, were measured by using electric dial gauges and the strains of reinforcements and concrete were also measured by using wire-stain gauges.

Test results

Test results are summarized in Table 3. Crack patterns at the both last stages of vertical loading only and horizontal loading under constant vertical loading are shown in Fig. 19 – 21 for the specimen of L-6-3 and S-6-3 which had the 3ton vertical load. Cracks were not observed clearly at the both top and bottom surfaces under vertical loads except L-6-3 and S-6-3 specimens, both having 3ton constant vertical loading. For horizontal loadings maximum loads for L type were 5.2 – 7.8ton and their failure modes were flexual-failure type failing at the part near the root of the fixed side with various shear cracks on the slabs observed except the L-6-3 specimen failing in the shear compression along the yelding line due to vertical loading. In the 6 ϕ main bar girder type main bars yielded but not broken with slab bars broken at the last stage of horizontal loading while in the 4.4 main bar girder type main bars were also broken. On the other hand, maximum loads for S type were 9.7 – 14.2 tons and their failure modes were shear-failure type failing at the part near the free end and with the level of vertical loads being higher, the failure mode became the combined feature of yield-line and shear crack. In 4.4 ϕ main bar girder type, the shear-failure near the free end, particularly at the side of horizontal loading was considerable and at the last stage of horizontal loading

the seperation occurred along this shear-failures line (diagonal direction). The bearing capacities under re-vertical loading after horizontal loading were 2.2 – 2.8tons for the L-type specimen except the L-6-3 specimen failing with punching shear along the yield-line of shear compression and 1.0 – 1.4tons for the S-type specimen except the S-6-1 Specimen These values can be explained as follows. In the L type, the failure due to horizontal loading was concentrated on the root of cantilever so the resistance of the short span was still active while in the S type, the failure due to horizontal loading was concentrated on the part near the free end so the resistance of long span was still active except the S-L-1 specimen which was active in both directions because of little failure of the part near the free end due to pre-vertical loading. Horizontal load-deflection curves are illustrated for the Specimens of L-6-2 and S-6-2, in Fig. 22, 23. In the S type specimens, two different curves of horizontal load versus in-plane diflection were obtained at one end and the other along the free end, assumed to be due to the distributed loading. In Fig. 24, 25 are shown the envelope curves obtained from horizontal load-inplane deflection curves, with larger deflection of the further point from direct loading point of the two ends along free end, being adapted in S type specimen.

Discussion of test results

In this section, discussion are focused on the resistance of specimens against horizontal loading. Flexural crack loads at which flexural cracks were at first observed were 1.0–1.5tons for L type specimen and 2.0–2.8tons for S type specimen, respectively each nearly constant, regardless of the level of vertical loads applied. The deformation angle at flexual cracks strength was 0.16×10^{-3} rad. on the average for L type specimen and 0.09×10^{-3} rad. on the average for S type specimen respectively. Thus the secant modulas stiffness calculated from these values of flexural crack loads, and their deflection were obtained to be 65 t/cm for L type specimen and 333 t/cm for S type specimen, each nearly agreeing with the elastic values of cantilever theory. Returning to flexural crack strength, the comparison between test values and the calculated values obtained by assuming the fiber strees as $1.8\sqrt{Fc}$ as shown in Ref. 3 were made and shown in the Table 3. It should be noted that the test values are lower, considerably for S type specimen. It seems due to the section's being of the channel shape instead of I shape.

Shear crack strength observed were 1.6–2.0tons for L type and 2.4–3.2tons S type, nearly constant, regardless of the level of vertical loads and the reinforcement amount of girders. On the other hand, the deformation angle at shear cracking was 0.28×10^{-3} rad. for L type and about 0.20×10^{-3} rad. for S type respectively. These values are a little smaller than that reported in Ref.8 in which the shear deformation at shear cracking is 0.25×10^{-3} rad. Returning to shear crack strength the mean stress at shear cracking was 7.5 kg/cm^2 (0.031 Fc) for L type and 7.8 kg/cm^2 (0.034 Fc) for S type. These values are lower than those of shear walls, reported. It seems due to the same reason with that of flexural cracking strength.

In Fig. 26 are shown shear stiffness reduction ($k - \frac{Pl}{GA} - / (\delta - \frac{Pl^3}{3EI}$)) up to, large deflection with the abscissa being the deformation angle. It is found that the trend is as a whole similar to that of Tomii equation established for shear wall except a few specimen, in spite of the fact that shear crack strength are lower than that of shear wall, mentioned above.

Maximum loads were 5.2–7.8 tons for L type and 9.7–14.2 tons for S type, as shown in Table 3. The L type specimens exceeded the calculated flexural yield strength except the L-6-3, having 3 ton vertical load, while the S type specimens did not exceed the calculated flexural yield strength except the S-4.4-1, having the 4.4 ϕ reinfocement in girders. These calculated values were obtained by the equation shown in the note of Table 3. L-6-3 specimen and S type three specimens failed in shear compression near the fixed end and near the free end respectively. The average shear strees at maximum loads was 31.6 kg/cm^2 (0.132 Fc)

for L-6-1, L-6-2 and 22.9 kg/cm^2 0.096 Fc) for L-6-3, L-4-1 respectively, while the stress was 36.4 kg/cm^2 (0.158 Fc) for three of S type and 26.9 kg/cm^2 (0.117 Fc) for S-6-3 respectively. In Fig. 27 are plotted the tested to calculated ratios, with the abscissa being thr ratios of vertical load applied to ultimate load calculated (3.3 ton) and compared with the results shown in Ref. 7. Both types are in right downward trend ,considerably different from that of Ref. 7, which dealt with simple plate subjected to pure bending.

The deformation angle at maximum load was 1/250 rad. for two 3 ton vertical load specimens, about 1/125 rad. for three of S type and L-4-1, and 1/62.5 rad. for the rest of L type. In Fig. 28 are shown the vertical deflection at the middle point of slab at the last stages of both vertical loading and horizontal loading with the abscissa being the vertical load applied.

Strength degradation at the amplitude of deformation angle given, due to one cycle reversing loads varied from 0.6 to 1.0, being much less for S-6-3 and L-4-1. In Fig. 29 are shown equivalent viscous damping factors with the abscissa being the deformation angle. The trends of both types are nearly the same and similar to that of shear wall repoted in Ref. 9, being at first about 0.30, gradually decreasing and being about 0.16, constant, regardless of deformation angle after 1/500 rad.

Concluding remarks

Based on the experiments reported herein, the following conclusions can be drawn.

The initial stiffiness of the slabs with girders is fairly large and can be estimated by using elastic theory, regardless of the level of vertical loads. The average stress at shear cracking was 7.5–7.8 kg/cm2 (0.031–0.034 Fc), and the average shear stress at maximum loads was 31.6–36.4 kg/cm^2 (0.132–0.158 Fc), both the values being little influenced by the level of vertical loads. The deformation angle at maximum load was 1/250 rad. for shear compression failure type and 1/62.5–1/125 for the rest respectively.

EARTHQUAKE RESISTANCE EVALUATIONS ON THE EXISTING BUILDING STRUCTURES DAMAGED DUE TO DIFFERENTIAL SETTLEMENTS, INCLUDING THE BUILDING DAMAGED BY THE ATOMIC BOMBING OF THE AUG, 6, 1945

Introduction

Hiroshima city is one of the two central cities in the southen Japan, with a population of about one million. The main old part of this city rely on a reclaimed land, as shown in Fig. 30. The considerable number of buildings without pile supports have been damaged due to differential settlements. The inspections have been conducted on the actual situations of differential settlements for existing bulidings, apartments and schools, etc., by the members of the building foundation committee in the chugoku branch of Architectural Institute of Japan in 1963. The inspected results are summarized in Fig. 31. About 30 percent buildings suffered the differential settlements more than 30mm as relative values between both the ends of the buildings. Usually the slope of settlements become maximum at the end span with several cracks found on the walls of buildings. The partial yielding of girders in the buildings are assumed to occur in the extreme case of the buildings suffering the slope of settlements more than 10^{-2}rad. at the end span. Furthermore the differential settlement values change a little along with time as shown in Fig. 32, being small in general.

Very recently earthquake resistance evaluations have been actively conducted, particularly on the public buildings such as schools in various cities, in responce with the damages of buildings due to recent earthquakes. In Hiroshima city the earthquake resistance of the public buildings, particularly with differential settlements have attracted much attention, recently initiated to study by the authors of this paper. The horizontal resistance of reinforced concrete frames after yielding due to vertical loading was once investi-

gated experimentally by Prof. Aoyama, University of Tokyo, as shown in Ref.11. According to this, ultimate strength against horizontal loading on reinforced concrete frames with the yielding of the parts due to vertical loading is the same with that on frames without vertical loadings, but the stiffness becomes considerably low as shown in Fig. 33. Based on these results, it can be said as the conclusions for the effects of differential settlements on horizontal resistance that the buildings with differential settlements have nearly the same strength with those without differential settlements but that the natural periods become quite longer. It should be, however, noted that the test results are those for frames alone without any walls. The buildings with many cracks observed were mostly walled frames. Thus experimental studies are being planned on the horizontal resistance of walled frames with yielding due to vertical loadings as a part of the project mentioned in the third section of this paper.

On the other hand, Hiroshima city has been very famous for the Atomic bombing of the Aug. 6, 1945. Reinforced concrete buildings mostly survived. One of these is a municipal office building being presently still used. This building was within several hundred meters from the center of explosion. Concrete coring tests were once conducted on this building. The strength was 120–140 kg/cm^2 for the cylinder cored from the walls directly exposed to the bomb blast (Photo. 3), about 60–70% of the strength of cylinders from non-exposed walls. Horizontal resistance of this building is under evaluation. The studies of this section have been conducted by Profs. M. Matuura and T. Shimazu, being still in progress.

Remarks

It has been found through evaluation procedures going on at present that the earthquake resistance of existing buildings are considerably varied with factors, such as, building type, other than those mentioned above.

ON SEISMIC DESIGN OF BUILDING — EQUIPMENTS SYSTEMS

Introduction

The Miyagioki earthquake of June. 1979 damaged the number of reinforced concrete buildings at Sendai, in the northern Japan. The engineering feature no less impotant than building damages were the overturning or destruction of various equipments on building floors, bookselves, chemical equipments such as gas cylinders, etc. having the possibility of causing much confusions.

Hiroshima University is as a whole moving to the other city within two years with fairly concrete plans, due to the crowdness of the city center. On the stage of the planning of new buildings, the problems occurred concerning the total number of stories for the buildings, strongly influenced by the confusing damages of the equipments in the Tofoku University by the Miyagioki earthquake. The buildings of the faculties of science or engineering usually contains various equipments on several floors. Forced vibration tests were planned for the existing buildings of both the science and engineering faculties of the University of Hiroshima in order to obtain fundamental information on the dynamic characteristics between actual buildings and their equipments on the various floors. However the tests by using the vibrators explained in the first section of this paper have not yet been conducted being still in plan. Preceding this experimental works, dynamic analyses have been made on the responce of new buildings floors to earthquake excitations. In the fllowing the summary of this analyses are presentaed.

In Fig. 34, 35 are shown the plan and the elevations of new buildings under planning. The examinations have been made on which number of total stories are preferable, four, six or eight with regards to magnitudes of accelerations on the floors, from which judgements can be made for equipments installations.

In the examinations by dynamic anlayses, fundamental natural periods were varied for each storied

building, changing, in many ways, the values for stiffness or rocking coefficients on shear walls, under considerations of uncertainities of these values. In Fig. 36 are shown the results on periods, compared with the distribution of measured values for actual buildings which has been obtained by Prof. Shiga, University of Tohoku. On the other hand the earthquake excitations used were four records shown in Table 5, each being magnified to 330 gal at maximum accelerations. The responce results on the range of maximum accelerations on each floor are shown in Fig. 37 for the daming ratio of 5%. It is found that the ranges of maximum accelerations at the same level become smaller for higher-storied buildings, besides the case of the Miyagi-oki earth quake showing a little different trends. Both the distribution of ranges on maximum displacements, velocities and shear coefficients are illustrated in Figs. 38,39. Judging from the results, higher-storied buildings, eight-storied buildings were proposed to be preferable for new buildings, with regards to earthquake resistance of equipments as well as other factors such as the efficiency in use of buildings. This studies have been conducted by Prof. T. Shimazu and H. Araki, graduate student, Univ. of Hiroshima.

Remarks

Forced vibrations tests are still in progress. The analyses mentional above, prior to the tests, has been found to be important from the view points of practical seismic design on safety for equipments installations involved in highly uncertain factors.

CONCLUSIONS

Each content of the five researches on earthquake engineering at present going on in our institute were presented togther with the outline of the structured division. The conclusions drawn are as follows.

(1) Deformation capabilities of reinforced concrete columns representing measures of the confinement effects of hoops were numerically formulated, based on the results of a large number of column tests conducted in Japan.

(2) Earthquake resistance of walls with and without openings has been examined by using the past test results.

(3) In-plane resistance of reinforced concrete slabs with girders has been clarified through the experimental works. It was found that the resistance characteristics for slabs as a whole are similar to those for shear walls, little influenced by vertical loadings.

(4) Measured results on differential setllements of actual buildings as well as on buildings damaged by the Atomic bombing were explained as a factor influencing earthquake resistance of buildings.

(5) Results of dynamic analyses made preceding forced vibration tests to obtain basic information on the installation of equipments on building floars, were presented.

REFERENCES

!' Architectural Institute of Japan, "Seismic loads and earthquake resistance of building structures", 1976, pp. 167–169.

2. Building Research Institute, et al. and other investigators, "Studies on reinforced concrete columns," "Proc. of the annual meeting of A.I.J 1973 pp.1389–1428 1974 pp1305–42, 1975 pp.1087–1120, 1976 pp.1391–1420, 1977 pp.1495–1502, 1978 pp.1747–75, 1979 pp.1237–54.

3. Architectural Institute of Japan, "Reinforced concrete buildings codes" 1979 pp.69–74.

4. Building Research Institute, et al. and other investigators "Studies on reinforced concrete walls" Proc. of the annual meeting of A.I.J. 1973 pp.1303–5, 1975 pp.1175–92, 1976 pp.1595–1608, 1977 pp.1633–86, 1978 pp.1609–70, 1979 pp.1279–82, 1469–1536.

5. A. E. Cardenas, et al. "Design provisions for shear walls" Journal of A. C. I. Vol. 70. No.3, March 1973 pp.221–230.

6. W. F. Research Group "Experimental Study on Reinforced Concrete Wall Subjected to Shear Force

both in Plane and out of Plane (Part I)" Report of Technical Research Institute Ohbayashi-gumi Ltd., No.813 (1976) pp.34–38.

7. H. Umemura, H. Aoyama, et al. "Experimental Study on Reinforced Concrete Shear Wall Subjected to Shear Force out of Plane (Part 1)" Proc. of Annual meeting of A. I. J. Oct. 1977. pp. 1651 ~ 52
8. M. Tomii, Y. Osaki "Study of Shear Resistance on Wall with Girder" Trans. of A. I. J. Sept. 1955 pp.1~9
9. M. Yamada, et al. "Study of Elasto-Plastic Behaviours on Reinforced Concrete Shear Wall (Part XVI)" Proc. of Annual meeting of A. I. J. Oct. 1977. pp.1627–28.
10. S. Kasai "Measurements of differential settlements on the buildings of Hiroshima city" Report to A. I. J. 1964. pp.38–89.
11. H. Aoyama "Plastic strength of reinforced concrete frames (in case of frames having Self-balanced studies)," Annual report of Architectural Structural Lab. Tokyo University, Dec. 1959. pp.8–10.
12. T. Shimazu "Earthquake resistance of the municipal city office building," Report to Hiroshima city 1979. pp.7–8.
13. T. Shimazu and H. Araki "On seismic design for secondary structures" Proc. of Chugoku branch symposium of A. I. J. 1979. pp.17–20.

Table 1 Ranges of each variables in column specimens

Shear span ratio	1.0	1.5	2.0	2.5	3.0	Total	Axial load σ_0 (N/bD)	0~0.1	0.1~0.2	0.2~0.3	0.3~0.4	Total
Numb. of speci.	37	37	141	29	39	283	Numb. of speci.	47	152	76	8	283
Hoop rein. ratio (%)	0~0.5	0.5~1.0	1.0~1.5	1.5~2.0	2.0~2.5		Concrete strength (kg/cm²)	124 ~ 344				
Numb. of speci.	106	85	63	18	11	283	Steel strength (kg/cm²)	1050 ~ 6380				
Long. rein. ratio (%)	0~0.5	0.5~1.0	1.0~1.5	1.5~2.0	2.0~2.5		Section (cm)	7.0 x 7.0 ~ 50.0 x 50.0				
Numb. of speci.	72	186	25	0	0	283	Length (cm)	20.0 ~ 200.0				

Table 2 Ranges of each variables in wall specimens

	No opening	Opening 1)
Shear span ratio	0.5 ~ 1.8	0.5 ~ 1.1
Axial load σ_0 (kg/cm²)	0 ~ 120	0 ~ 120.3
Hoop rein. ratio (p_h · σ_y kg/cm²)	2.4 ~ 60.9	4.0 ~ 36.5
Long. rein. ratio (p_g %)	0.29 ~ 1.92	0.56 ~ 1.70
Cocrete strength (kg/cm²)	155.0 ~ 406.0	193.7 ~ 435.0

Note.
1) Opening ratio
l_o/l = 0.33 ~ 1.0
h_o/h = 0.33 ~ 1.0
$\sqrt{\frac{l_o h_o}{h l}}$ = 0.33 ~ 0.87

Table 3 Properties of materials and test results

Symbol of Speci.	Out-pl-force	Span leng Lx (mm)	Span leng Ly (mm)	Slab Dep. (mm)	Slab rein. (%)	Girder rein. (%)	Concrete σ_B (kg/cm²)	Concrete E (×10⁴ kg/cm²)	Girder rein. σ_y (kg/cm²)	Flex crack load (t) Exp.	Flex crack load (t) Calc.	Shear crack load (t) Exp.	Shear crack load (t) Calc.	Max lo. (Hori.) (t) Exp.	Max lo. (Hori.) (t) Fl. cal	Ulti (t) load Calc.
L-6-1	0.4 (t)	1200	800	30	2.6	8-6φ 3.14	234	2.27	4316	1.5	2.17	1.7	2.27	7.8	6.90	2.2
L-6-2	1.0						282	2.49		1.0	2.32	1.8	2.40	7.4	6.90	2.8
L-6-3	3.0				0.40		217	2.19		1.3	2.08	2.0	2.23	5.7	6.90	0.4
L-4-1	1.0					8-4φ 1.69	226	2.23	4453	1.5	1.95	1.6	2.25	5.2	4.33	2.4
S-6-1	0.4	800	1200		2.6	8-6φ 3.14	271	2.64	4316	2.8	6.16	3.2	6.14	14.2	18.45	3.2
S-6-2	1.0						205	2.13		2.0	5.47	2.8	5.94	12.3	18.45	1.4
S-6-3	3.0				0.59		217	2.19		2.4	5.60	3.0	5.98	9.7	18.45	1.3
S-4-1	1.0					8-4φ 1.69	226	2.23	4453	2.0	5.33	2.4	6.00	12.8	12.68	1.0

Note.
1) $1.8\sqrt{F_c} \cdot z_e$
2) $1.8\sqrt{F_c} \cdot A_e$
3) $(0.9\, a_t \cdot \sigma_y + 0.4\, a_w \cdot \sigma_{wy}) \cdot l/h$

Table 4 Cylinder test results

	Cored specimens Location	dia (cm)	ℓ (cm)	Strength (kg/cm²)	Young (kg/cm²) Modulus
1	Exposed to Blast (Roof walls)	9.8	21	127	1.0
2	〃	10.0	17	118	0.8
3	Not exposed to Blast (Roof walls)	9.7	17.5	142	1.3
4	〃 (Not Roof walls)	10.1	19.5	219	2.0

Table 5 Earthquake excitations used

Earthquake	Year	Max. Acc. (gal)
Elcentro NS	1940	330
Miyagi oki NS	1978	258
Tokyo 101 NS	1956	70
Sendai 501 NS	1962	50

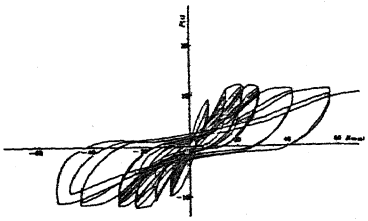


Fig 1 Load-deflection curves illustrated (Ref. 1)

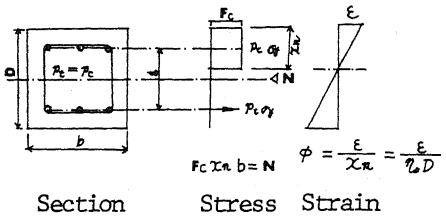


Fig. 2 Equibrium in section at ultimate (Ref. 3)

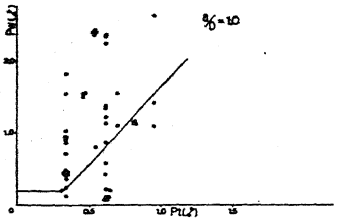


Fig. 3 Hoop ratios versus longitudind reinforcements used. (a/D=1.0)

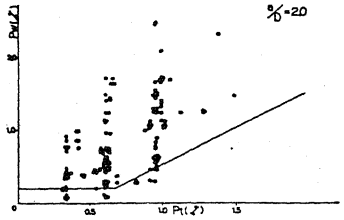


Fig. 4 Hoop ratios versus longitudinal reinforcements used. (a/D=2.0)

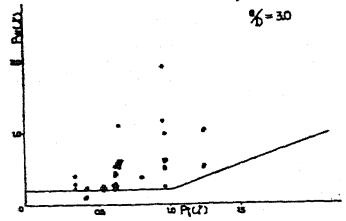


Fig. 5 Hoop ratios versus longitidinal reinforcements used. (a/D=3.0)

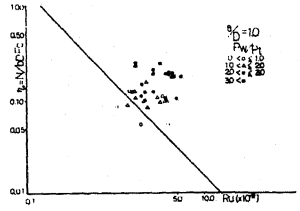


Fig. 6 Axial load level versus deformation capabilities (a/D=1.0)

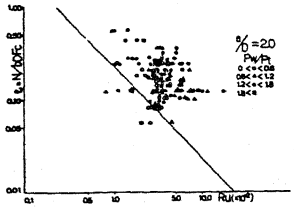


Fig. 7 Axial load level versus deformation capabilities (a/D=2.0)

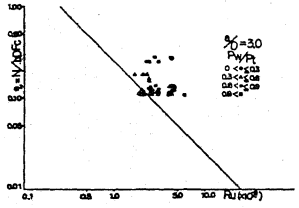


Fig. 8 Axial load level versus deformation capabilities (a/D=3.0)

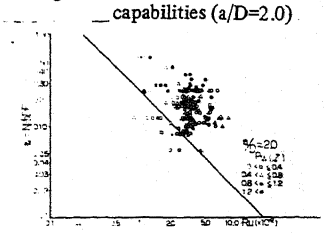


Fig. 9 Axial load level versus deformation capabilities (a/D=2.0 classified with p_w values alone)

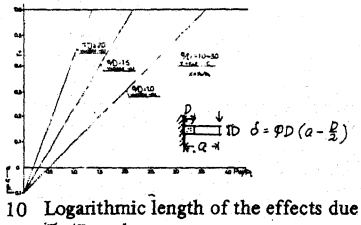


Fig. 10 Logarithmic length of the effects due to p_w/p_t values

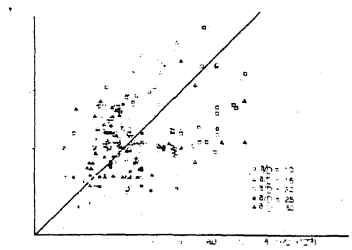


Fig. 11 Tested versus calculated values

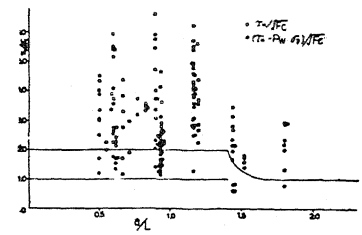


Fig. 12 Maximum strength for walls without openings

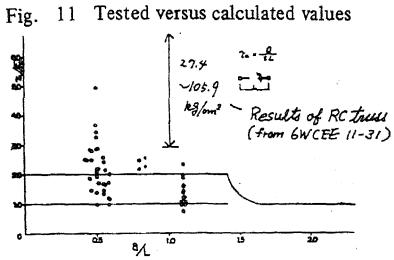


Fig. 13 Maximum strength for walls with openings

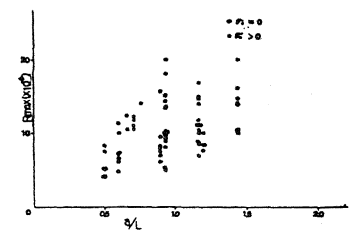


Fig. 14 Deformation angle at maximum for walls without openings

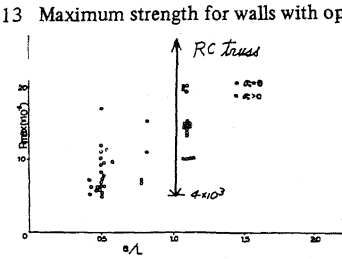


Fig. 15. Deformation angle at maximum for walls with openings

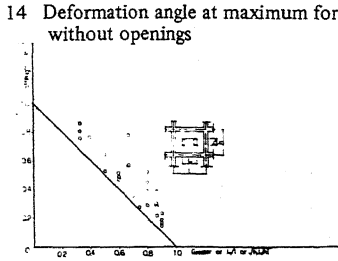


Fig. 16 Effects of opening ratios on maximum strength

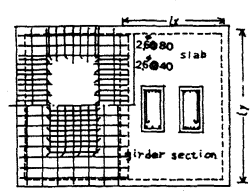


Fig. 17 Details of specimens

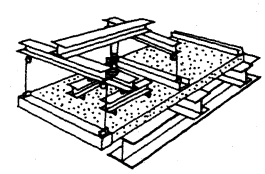


Fig. 18 Test set up

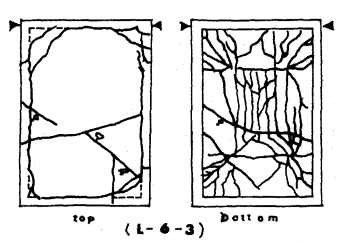


Fig. 19 Crack patterns under constant vertical load

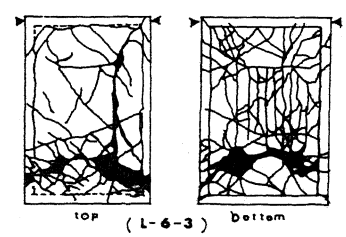
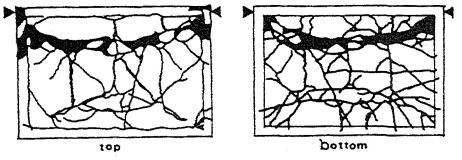


Fig. 20 Failure patterns due to horizontal loadings



(S - 6 - 3)

Fig. 21 Failure patterns due to horizontal loadings

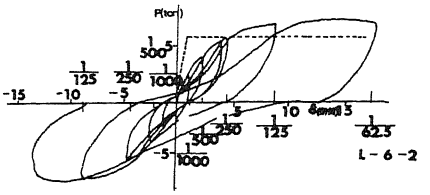


Fig. 22 Horizontal load-deflection curves

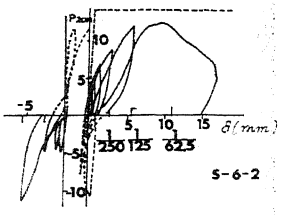


Fig. 23 Horizontal load-deflection curves

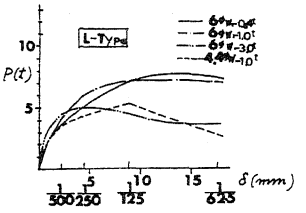


Fig. 24 Envelope curves

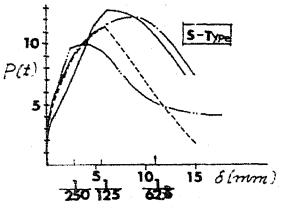


Fig. 25 Envelope curves

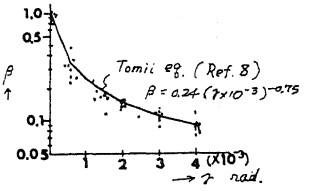


Fig. 26 Shear stiffness reduction ratios

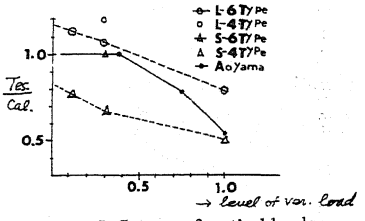


Fig. 27 Influence of vertical loads

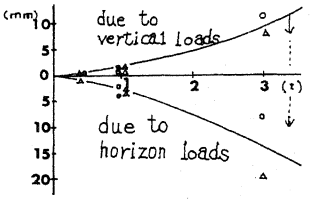


Fig. 28 Deflection characteristics

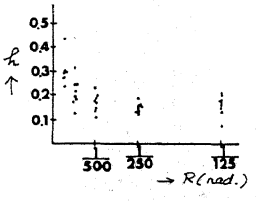


Fig. 29 Equivalent viscous damping ratios

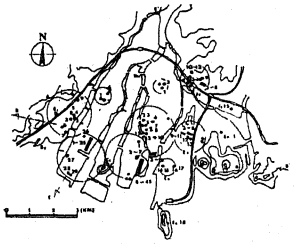


Fig. 30 Map of the center of Hiroshima city (Ref. 10)

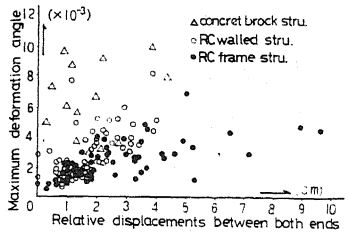


Fig. 31 Diagram on measured results of differential settlements (Ref. 10)

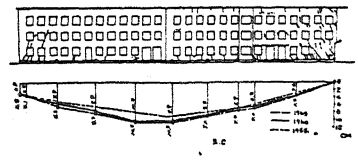


Fig. 32 Illustration of differential setlements (Ref. 10)

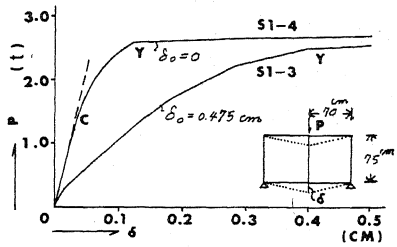


Fig. 33 Influence of self-balanced forces (Ref. 11)

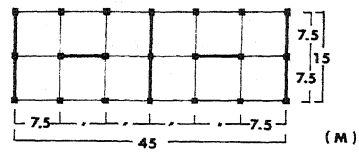


Fig. 34 Floor plan under planning

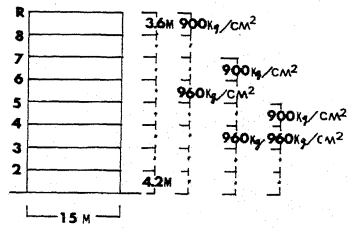


Fig. 35 Vertical section under planning

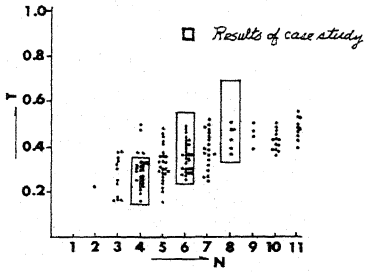


Fig. 36 Natural period distribution for actual buildings

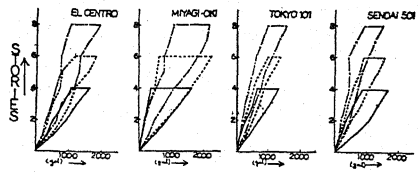


Fig. 37 Distributions of maximum accelerations

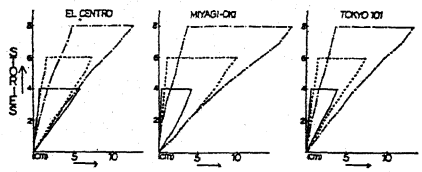


Fig. 38 Distributions of maximum displacements

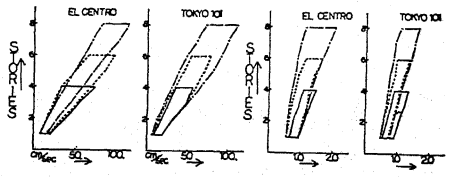


Fig. 39. Distribution of maximum velocities and shear coefficients

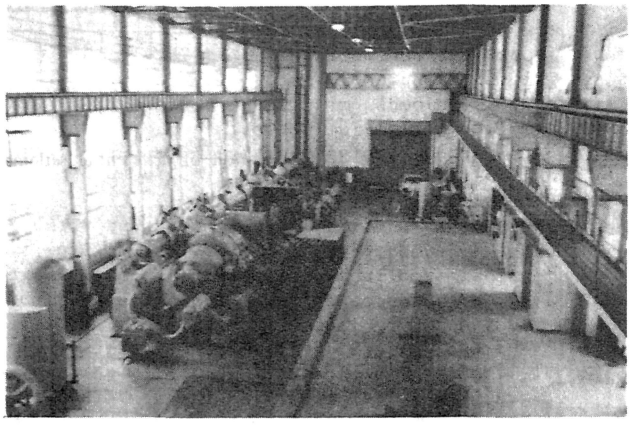


Photo 1 3000 tons capacity testing machine

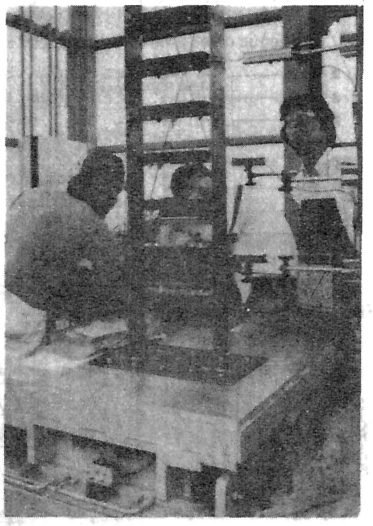
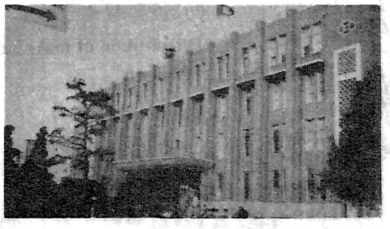


Photo 2 Reation bed and Shaking Table



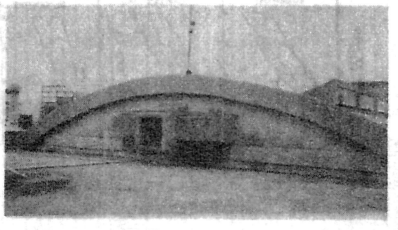


Photo 3 Roof wall damaged by the Atomic bomb (Municipal office)