LARGE-SIZE STRUCTURES TESTING LABORATORY AND LATERAL LOADING TEST OF A FIVE-STOREYED FULL-SIZE BUILDING STRUCTURE

by
Toshihiko Hisada
representing
Joint Committee on Housing Structures,
The Building Research Institute of the Ministry of Construction,
The Japan Housing Corporation

ABSTRACT

1. INTRODUCTION

The Large-Size Structures Testing Laboratory at the Building Research Institute was completed in 1967 to perform horizontal loading tests of full-size building structures. The results will provide us with valuable information on their structural behavior and ultimate load-carrying capacities under seismic forces and, at the same time, will serve in improving, rationalizing and economizing the structural design of housing construction.

As the first project in this Laboratory, static horizontal loading tests and supplementary dynamic tests were carried out on a full-scale five-storeyed housing structure of precast reinforced concrete panel construction.

2. OUTLINE OF THE TESTING LABORATORY

The Laboratory consists of an L-shaped reaction-wall of prestressed concrete, and a laboratory building of reinforced concrete as shown in Fig. 1.

As a loading equipment, twenty oil-jacks with the capacity of 50 tons each, both in pulling and pushing, are prepared. Those jacks are controllable by means of an operation panel. The overall capacity of the loading facilities is 1,000 tons in shearing force and 10,000 ton-meters in bending moment.

3. TEST RESULTS

The test building was a five storeyed precast reinforced concrete housing structure. Measurements were made with respect to strains in steel and concrete, horizontal drifts at each floor levels, rotations of each wall-panels, etc. Results of the tests can be briefly summarized as follows:

1) The housing structure of precast reinforced concrete panel construction designed according to present Japanese Structural Standard can resist the lateral force approximately seven times as large as the seismic force prescribed in the Japanese Building Code.

2) The load carrying capacity was reached by the shear failure in walls and excessive distortions at welding joints; however, the test building as a whole showed considerable ductility.

3) The fraction of critical damping derived from the load-deflection curves was about 7% at the loading stage equal to three times the design seismic forces.

LARGE-SIZE STRUCTURES TESTING LABORATORY AND LATERAL LOADING TEST OF A FIVE-STOREYED FULL-SIZE BUILDING STRUCTURE

by

Toshihiko Hisada*
representing
Joint Committee on Housing Structures,
The Building Research Institute of the Ministry of Construction,
The Japan Housing Corporation

SYNOPSIS

The Large-Size Structures Testing Laboratory with the capacity of 1,000 tons in shear and of 10,000 tm in bending moment was completed at the Building Research Institute in 1967.

A static horizontal loading test on a full-scale five storeyed housing structure of precast reinforced concrete panel construction was carried out in the Laboratory.

Structural behaviour up to failure were clarified and several proposals have been made for rationalizing the structural design of the housing structure.

I Outline of Loading Facilities

(1) Loading Facilities

The loading facilities in the Large-Size Structures Testing Laboratory have been designed so that it is possible to test a full-size five-storeyed reinforced concrete structure, consisting of ten dwelling units and a staircase as shown in Fig. 2.

The main parts of the loading facilities are essentially composed of a testing bed and a vertical reaction-wall of prestressed concrete, forming L-shape.

The horizontal loads can be applied both in pushing and pulling directions against a test building as shown in Fig. 4 by means of bilaterally operative oil jacks, and the maximum loading capacity is approximately equivalent to quasi-static seismic forces with an acceleration of 1.0 g.

---

* Director, Building Research Institute, Ministry of Construction, Japanese Government

To safely resist such high loadings, the testing bed and the vertical reaction-wall with a total thichness of 4.5 m and 3.0 m, respectively, are designed so as to develop resisting moment of about 10,000 tm at their perpendicular junction by means of prestressing cables of high tensile strength, the total number of which amounts to 340. The deflection at the top of the vertical reaction-wall does not exceed 1.4 cm under the maximum loading condition.

(2) Anchoring of a Test Building

In order to anchor the base of a test building, 660 sleeves with an internal diameter of 60.5 mm are provided at distances of 42 cm in the upper floor slab of the double-decked testing bed, through which the footing of a test building is fastened by means of screwed steel bars.

(3) Laboratory Building

The building accommodating the loading facilities is of ordinary reinforced concrete construction, the plan and cross section are outlined in Fig. 1a and Fig. 1b, respectively. The principal dimensions of the building are as follows:

Building area	476.9 sq·m
Total floor area	1,455.5 sq·m
Height	20.3 m

In the building, a travelling crane of 7.5 tons capacity, a hoist, dust-disposal equipment etc., are also installed as supplementary facilities.

II Behaviour of a Five Storeyed Precast Reinforced Concrete Housing Structure under Lateral Forces

(1) The Purpose of the Test

The principal purpose of the present test is to rationalize and to economize the structural design of the precast reinforced concrete housing structure actually used as a standard type of the Japan Housing Corporation, through the investigation of the behaviour against earthquake.

The above housing structure has been designed on the basis of simplified assumptions as mentioned below:

(i) Design seismic force coefficient is 0.2 at each floor level.
(ii) Each panel is subjected to shear force which is proportional to the length of each panel.
(iii) Panels with openning are subjected to bending moments corresponding to the shear forces, assuming that the point of inflection is at the mid-point of the height of each panel.

(2) General Description of a Test Building

(2-1) General

The specimen for this test is a full-size building structure of the standard type (65-5N-2DK-PC) of the Japan Housing Corporation.

The plan of the specimen is shown in Fig. 2. Elevations of each of the skeletons are shown in Fig. 3a and Fig. 3b. Vertical joints between panels are so-called wet-joints as shown in Fig. 5. In the wet-joints, shear-keys are provided, six for each storey, and a steel bar of 9 mm in diameter, projected from precast panels at the location of each shear-keys, is welded to vertical reinforcement in the joint. Concrete is cast in place in the vertical joints.

Horizontal joints are so-called dry joints (see Fig. 5). Steel plates anchored in each panel are welded to each other, and a mortar layer is provided beneath each panel.

Joints between floor-slabs and wall panels are made by a method similar to the above wet joint.

Joints among floor slabs are made by a method similar to the dry-joint.

(2-2) Properties of materials used.

(a) Steel bars

Two kinds of steel bars were used as shown below:

	diameter (mm)	yield stress (kg/cm^2)	elongation (%)
Round bar	9 - 19	2,900 - 3,400	24 - 36
Deformed bar	19 - 29	3,900 - 4,200	21 - 26

(b) Concrete and mortar

The properties of concrete and mortar used are as shown below:

	Ultimate compressive strength F_c (kg/cm^2)	Young's moduls E_c (kg/cm^2)
For precast panels	230	250
For vertical joint	340	270
For foundation	370	280
For mortar layer	250	-

(3) Loading Devices and Method of Measurements

(3-1) Method of loading

Loading was performed by 15 jacks operated with equal pressure. The arrangement of the jacks are shown in Fig. 4.

The vertical distribution of lateral forces is the same as that assumed in the design, but the horizontal distribution has a slight eccentricity

Supplementary loadings such as twist loading, triangularly distributed loading in vertical direction were also performed.

(3-2) Method of measurements

The principal measuring points are as follows:

(i) Horizontal and vertical displacements by dial gauges and transits (178 points).
(ii) Rotation angles of joint-panels by clinometers (29 points).
(iii) Strains of concrete surfaces, and of steels by SR-4 strain gauges (356 points).

(4) Test Results and Analytical Considerations

(4-1) Load-deflection characteristics

The relation between total horizontal load and horizontal deflection of skeleton "B" measured at the level of the roof slab is shown in Fig. 6. In this figure, "line 1" shows the theoretical deflection calculated on the basis of elastic theory, assuming that the specimen is monolithic as a whole, and neglecting the effects of the end-walls perpendicular to loading direction, "curve 1" shows deflection due to over-all rotation caused by vertical displacement of horizontal joints between ground girder and the first storey's end-walls, and "curve 2" shows the sum of the deflection corresponding to "curve 1" and the deflection due to horizontal displacements (slips) at the horizontal joints.

The following facts may be pointed out from Fig. 6.

(i) Experimental elastic deflection is about 1.2 times the calculated value.
(ii) The deflection due to rotation and slip are practically negligible at "loading scale 1" which corresponds to the design horizontal load. The above deflections reach up to 30 - 40 % and 60 % of the total deflection, respectively, at the time of initial cracking (loading scale 4.5) and the vicinity of the maximum load (loading scale 7.2).
(iii) Angles of inclination calculated from the total deflection are as follows:

	R_t	R_1
At the time of initial cracking load ($P = 390^t$)	0.68×10^{-3} rad.	1.22×10^{-3} rad.
At the vicinity of max. load ($P = 645^t$)	3.7×10^{-3} rad.	7.6×10^{-3} rad.
At the time of final load ($P = 510^t$)	12.8×10^{-3} rad.	35.3×10^{-3} rad.

where: R_t is over-all angle of inclination
= (total deflection at roof slab level)/(total height).

R_1 is angle of inclination of 1st storey
= (total deflection at the 2nd floor level)/(the height of the 2nd floor).

(iv) The fraction of critical damping, corresponding to the loading scale up to 4.5 obtained from Fig. 6 is 5 to 8 %.

(4-2) Cooperation among panels

(i) Behaviour of wall-panels connected through a vertical joint.

Wall-panels connected through vertical wet-joint will behave practically as though a monolithic panel except in plastic range. The above fact is deduced from the test results, such as cracking patterns, and stress distribution in panels during the loading tests. Although the monolithic characteristic is lost in plastic range, it is worth while to note that the existence of the vertical joint does not become a direct cause of failure of the specimen.

(ii) Horizontal displacement (slip) in horizontal joint.

The slip in the horizontal joint is negligible up to the loading scale 1, and is fairly small up to the loading scale 3.

The slip increases rapidly afterwards, and even reaching 30 % of the total horizontal drift of the specimen in the vicinity of the maximum loads.

(iii) Three-dimensional effects of walls perpendicular to loading direction.

The transversal wall panels are not effective as to initial stiffness of the structure, but are effective in increasing considerably the load-carrying capacity of the specimen.

(iv) Composite effect between floor slab and beam (horizontal part of wall panels with opening). Rigidity of the floor composed of floor panels.

The composite effect between beam and floor slab is not recognized, judging from cracking patterns.

The floor is assumed to behave as a monolithic floor since horizontal drifts of each skeletons (skeleton "A", "B", and "C") are practically equal up to the final loading step.

(4-3) Rotation in horizontal joints

(i) The rotation in the horizontal joint is negligible up to "loading scale 1", and increases rapidly at "loading scale 2".

(ii) Vertical elongation in the joint is 2 to 10 times vertical shortening throughout the loading test. The quantity of vertical shortening remains relatively small even at the final loading step. The specimen shows the tendency of rocking as a whole.

(iii) Rotation in a composite panel, which consists of two panels connected through vertical (wet) joint, is practically the same as that in a monolithic panel.

(iv) Quantity of rotation is greater at the lower storey and in narrower panels, throughout the loading tests.

(4-4) Distribution of shearing strains

Distribution of the shearing stresses in all panels corresponding to several loading steps is shown in Fig. 7.

These shearing strains were measured by SR-4 strain gauges and calculated by assuming $G = 2.5 \times 10^5/2.5\ kg/cm^2$.

The following facts will be pointed out from the results of measurement.

(i) Most of the wall-panels which are connected through vertical joint behave monolithically.

(ii) The effect of walls perpendicular to the loading direction is not remarkable.

(iii) Relations between loads and shearing strains in wall panels are proportional up to the range of considerably high value of loading.

(iv) So-called truss-action of loading is recognized, as the intensity of shearing strain in wall panels is relatively higher in the wall panels which are located in the vicinity of the loading side.

(4-5) Distribution of strains due to bending moment

Measurement of the strains due to bending moment was performed in "skeleton A" and in a part of "skeleton B" as shown in Fig. 3.

The following facts may be pointed out from the above measurement.

(i) A composite wall panel which consists of two wall panels connected through a vertical wet-joint behaves as a monolithic panel up to "loading scale 3".

That is, strain distribution in most of the composite panels is recti-linear, then the vertical wet-joints seem to have functioned effectively.

(ii) On the other hand, a composite beam which consists of an upper-part of a wall-panel and a lower-part of another wall-panel joined to each other through horizontal (dry) joints behaves as a member composed of two beams, one of them is put on top of the other. That is, the strain distributions in the lower part and in the upper part of the composite beam are independent of the early loading steps.

(iii) The height of inflection point for □-shaped panels is in the range of 0.5 - 0.65H (where H is the height of the panels) through-out the loading tests. On the other hand, the height of inflection point for ⊓-shaped panels is 0.7 - 0.9H for low value of loadings, and gradually decreases with the increment of loadings.

(4-6) Cracking patterns and mode of failure

Progress of cracking pattern and mode of failure are summerized in the following list (see also Figs. 8 and 9).

Load (loading scale)	Cracking pattern and failure
90 tons (1)	Initial bending cracks at the corner of the large opening
180 tons (2)	Initial cracks in horizontal joint
390 tons (4.5)	Initial shear cracks in the 1st floor panel
651 tons (7.2)	Shear failure in the 1st floor panel
After the maximum load	Considerable numbers of shear failure of wall panel, including the shear failure of the beams

(4-7) Relations among initial cracking load, load-carrying capacity and mean shearing stress.

Those relations at several loading steps are summerized as follows:

	At the time of initial cracking load	At the time of maximum load
Load P(ton)	390	651 (503)*
Mean shear stress $\tau = P/Aw (kg/cm^2)$	10.3	17.1 (13.2)
$\tau / \tau d$	4.3	7.2 (5.5)
τ / Fc	1/22.2	1/13.4 (1/17.4)

* The value in the parenthesis indicates the corresponding value at the time of maximum load in opposite direction.

Where: Aw = total wall area = 37980 (cm^2)

τd = mean shear stress corresponding to the design seismic load (0.2g at the each floor levels) = 2.39 (kg/cm^2)

Fc = Ultimate compressive strength of panel concrete at the age of the loading tests = 230 (kg/cm^2)

(5) Concluding Remarks

As the results of this test, the following items may be recommended for the purpose of rationalizing structural design of the housing construction.

i) Unit length** and thickness of wall could be reduced considerably.

The minimum values of unit length and thickness of wall provided in AIJ standard are 15 cm/m^2 and 15 cm respectively.

Instead of the above values, the following revised provisions are recommended:

minimum unit length of wall = 12 cm/m^2
minimum thickness of wall = 12 cm
allowable maximum mean shearing stress = 6 kg/cm^2

** Unit length of wall is 'wall length (cm) divided by floor area (m^2)'.

ii) The following assumptions in the structural analysis for this kind of structure may be justified by the experimental results mentioned above:

a) The floor slabs are rigid.
b) Panels connected through vertical wet-joints behave as a monolithic panel.
c) Panels connected through horizontal dry-joints behave independently.
d) Mechanical property of horizontal dry-joints is represented by springs which rotate in proportion to bending moments which are transmitted through the joints.

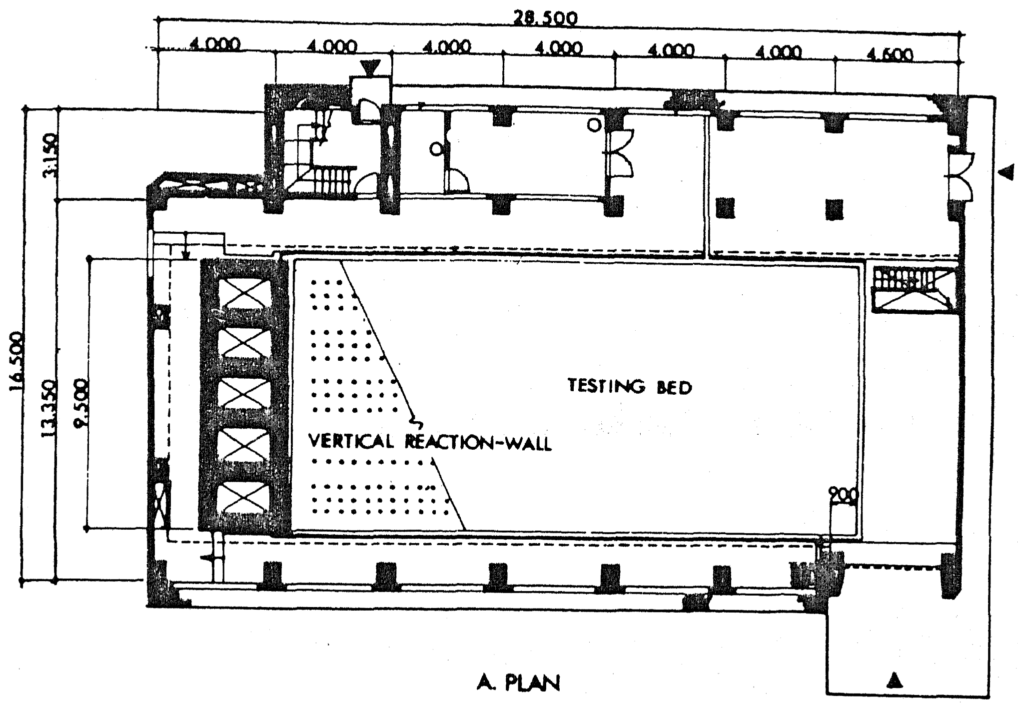
iii) A certain amount of reinforcements in panels and steels in joints may be reduced.

Postscript

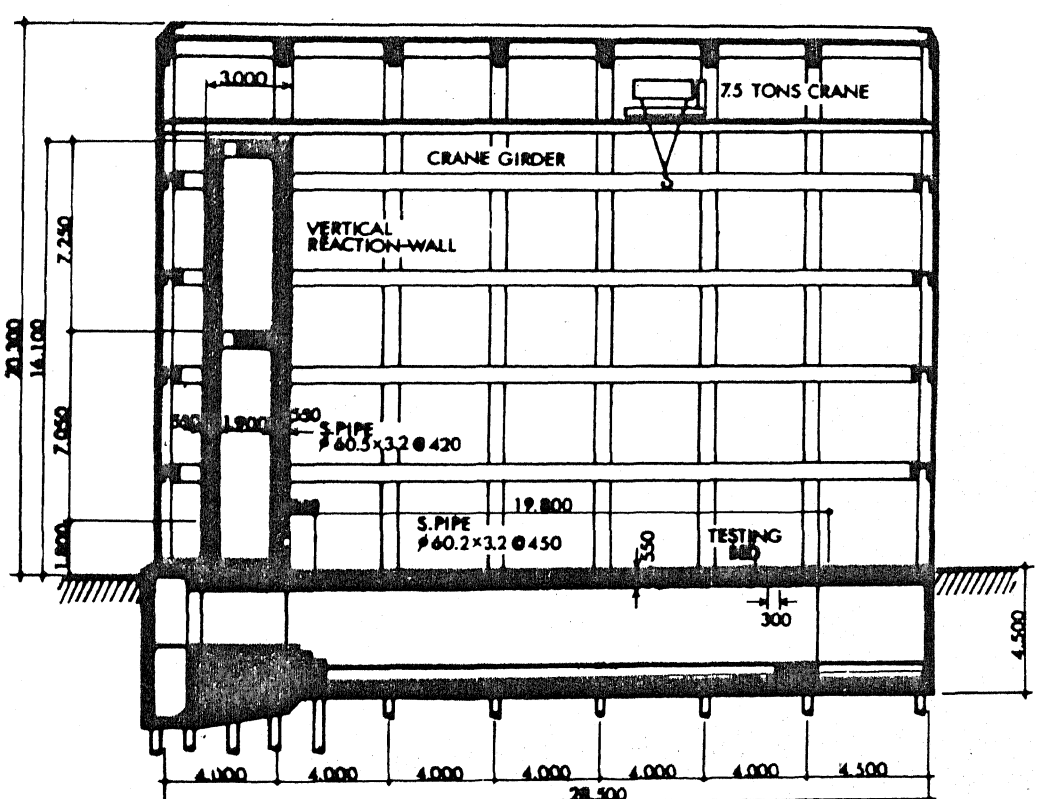
The project concerning the testing laboratory and the loading test was performed by the Committee members mentioned below:

Messrs. T. Hisada, Y. Ohsaki, Y. Kameda, K. Nakagawa, K. Nakano, T. Shinagawa, K. Kimura, M. Hirosawa, K. Kawashima, Y. Murata, Y. Kobayashi of the Building Research Institute, and Messrs. M. Shima, T. Higashi, M. Tsugawa, T. Miwa, S. Tsukushi, T. Araki, K. Oda of the Japan Housing Corporation.

The present paper was compiled by Messrs. K. Nakano and M. Hirosawa.



A. PLAN



B. CROSS-SECTION

Fig. 1 Testing Laboratory

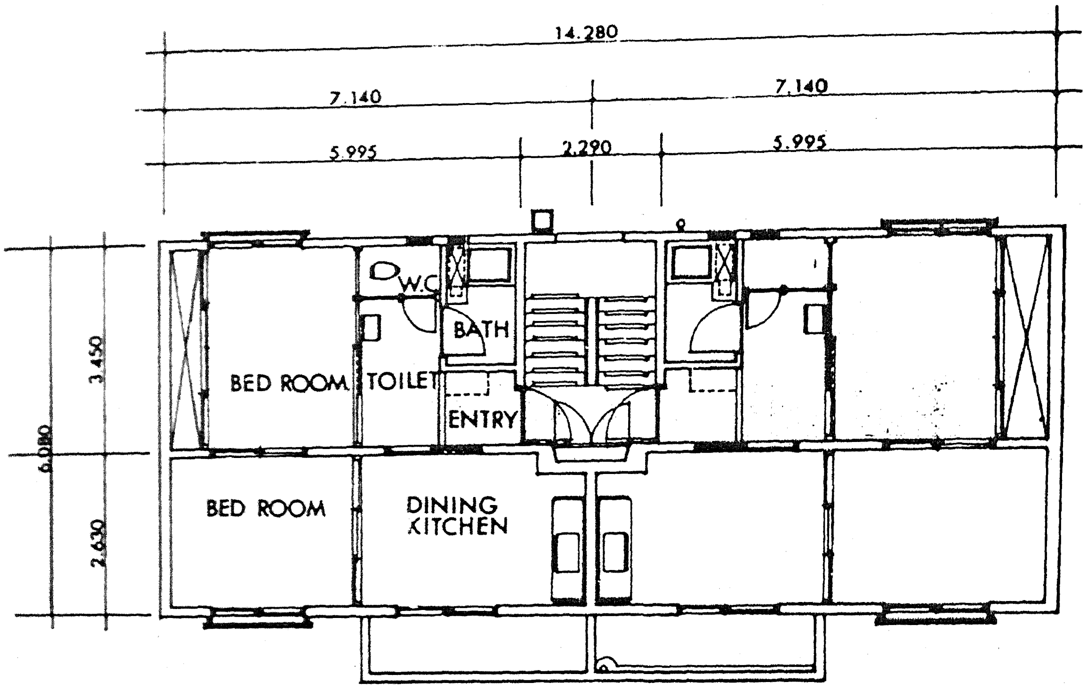


Fig. 2 Plan of Specimen

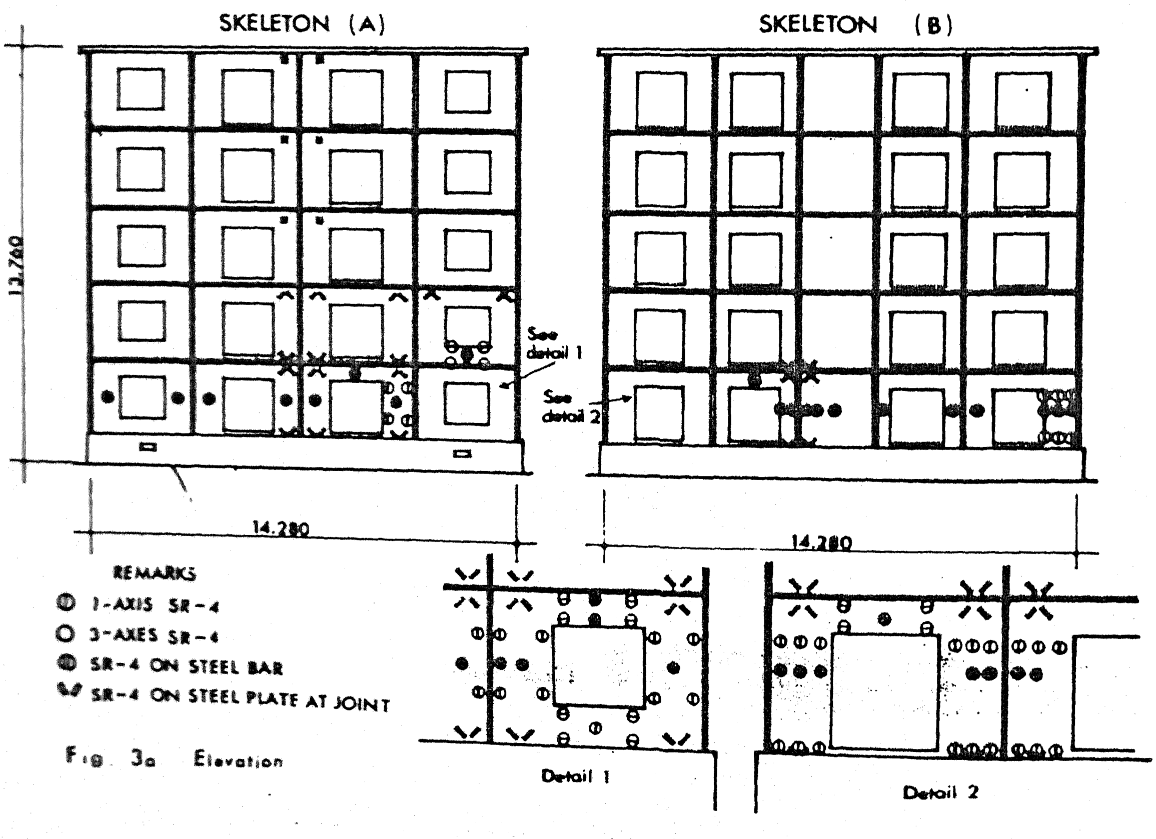
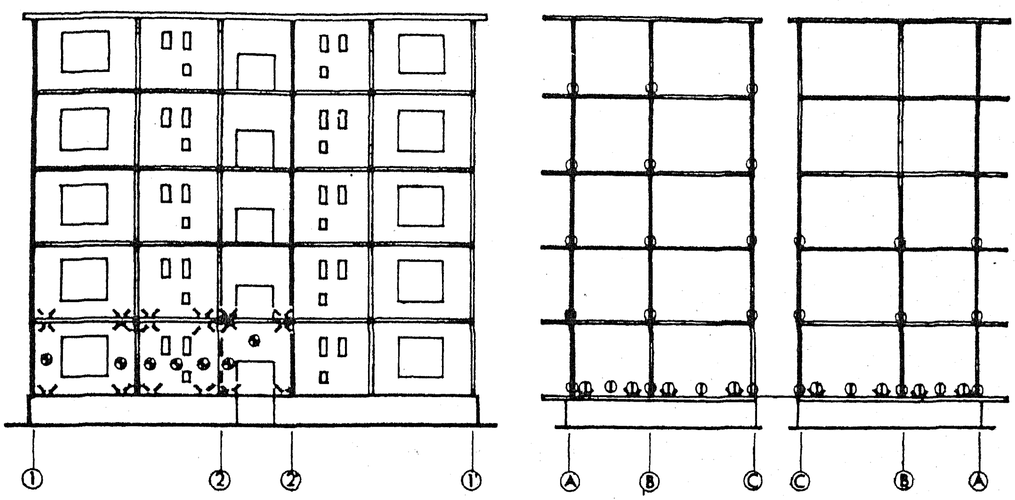


Fig. 3a Elevation



SKELETON (C) SKELETON (1') SKELETON (1)

Fig. 3b Elevation (2)

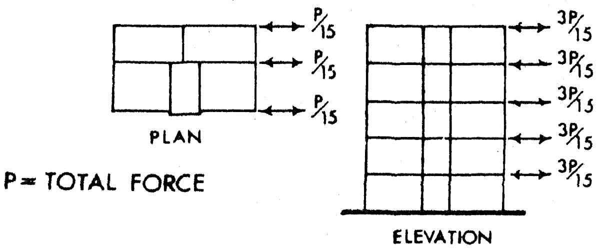


Fig. 4 Arrangement of loading jacks

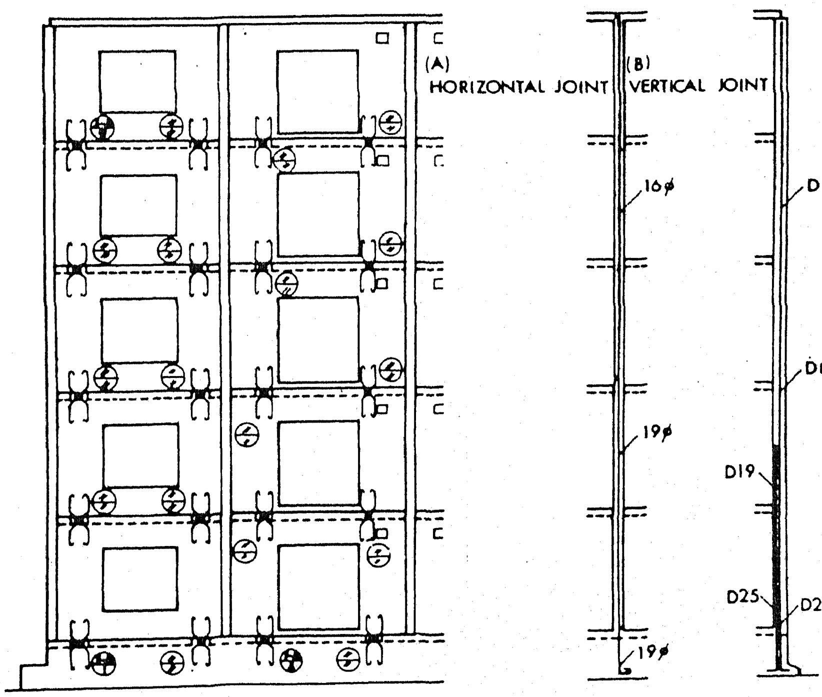


Fig. 5 Horizontal and vertical joints

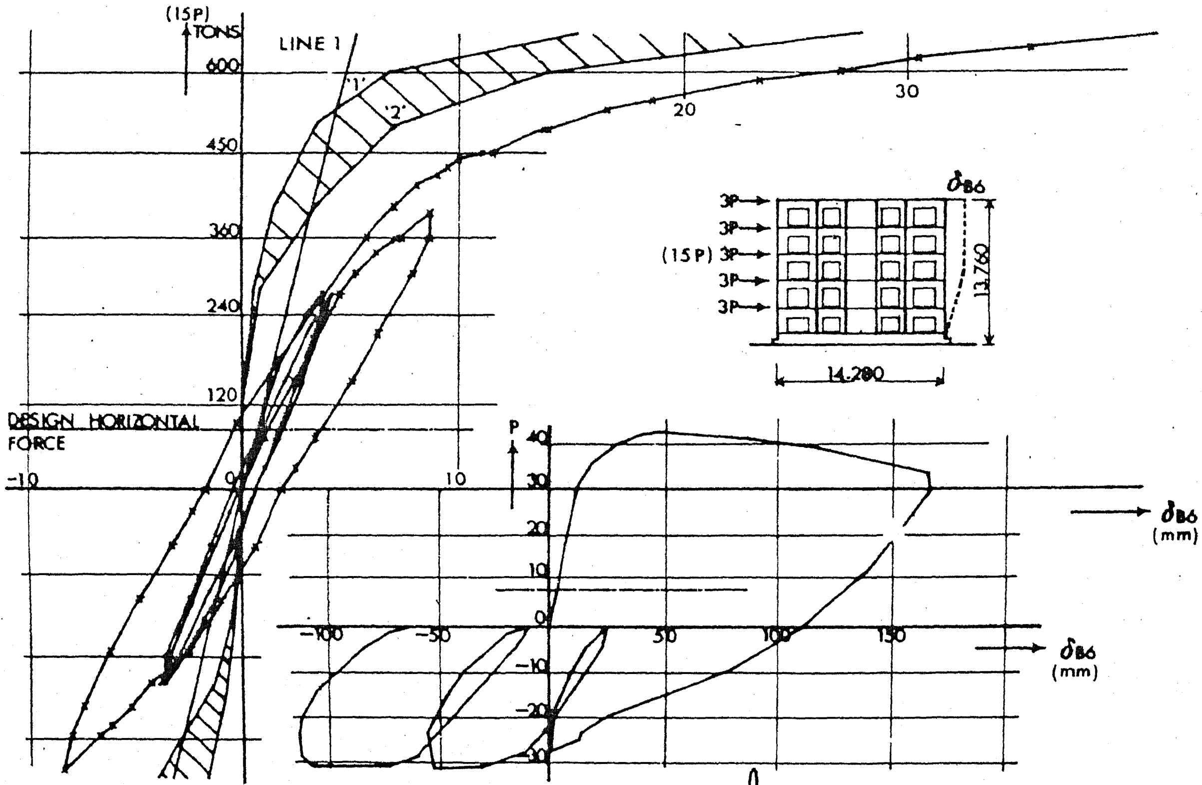


Fig. 6 Load-deflexion curves

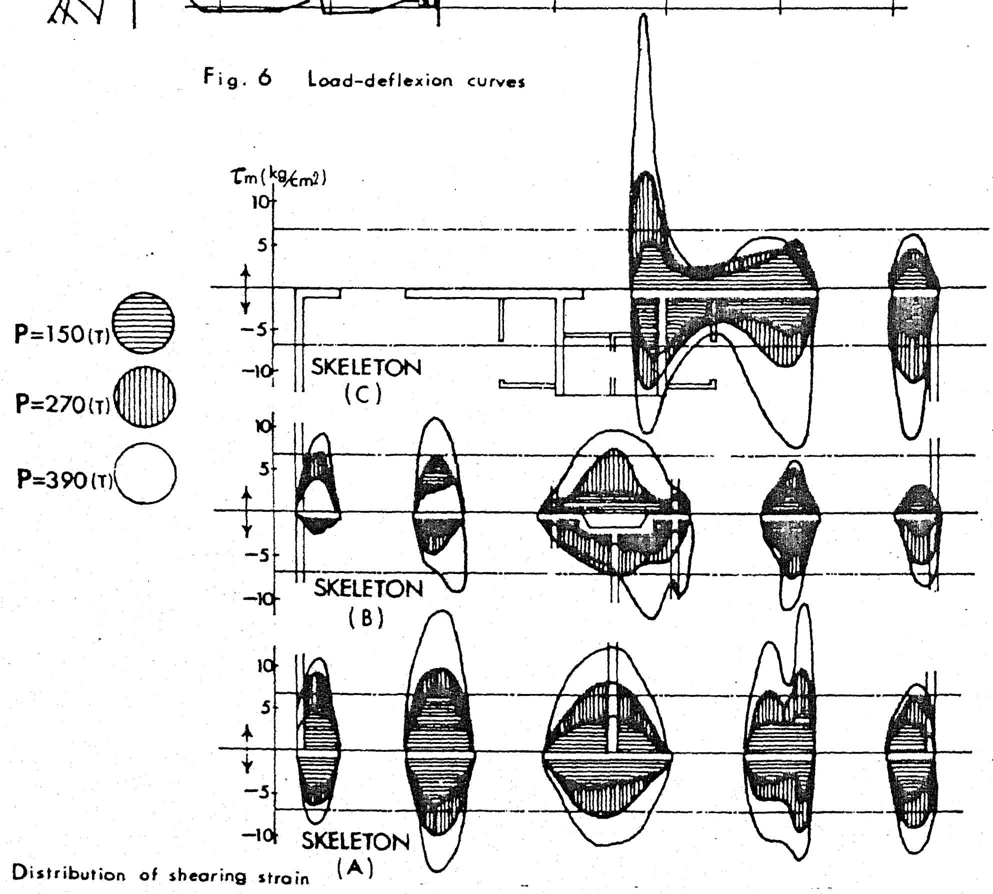


Fig. 7 Distribution of shearing strain

REMARKS

- P.P.W. Rupture of welding between setting plates.
- P.R.W. Rupture of welding between setting plate and anchored bar.
- P.D. Deformation of setting plate.
- P.L.D. Severe deformation of setting plate.
- R.B. Rupture of anchored bar.
- W.R.B. Rupture of reinforcement in panel.

REMARKS

- S.A. Shear crack (less than 1 mm)
- S.B. Shear crack (1 to 5 mm)
- S.C. Shear crack (5 to 10mm)
- S.D. Shear crack (larger than 10 mm)
- B.A. Bending crack (0.5 to 1 mm)
- B.B. Bending crack (1 to 3 mm)
- B.C. Bending crack (larger than 3mm)
- ☐ Crack accompanied by crush of concrete
- ( ) Crack accompanied by spalling-off of concrete

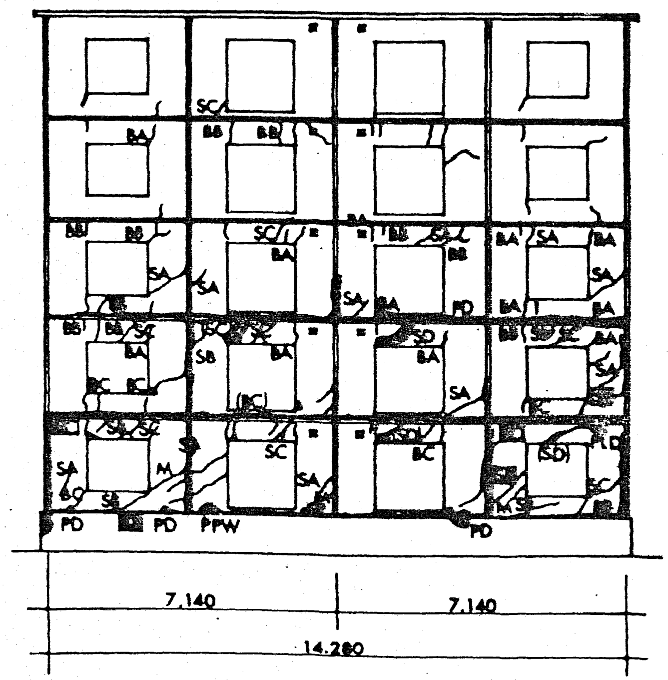


Fig. 8a Final cracking pattern of "Skeleton A."

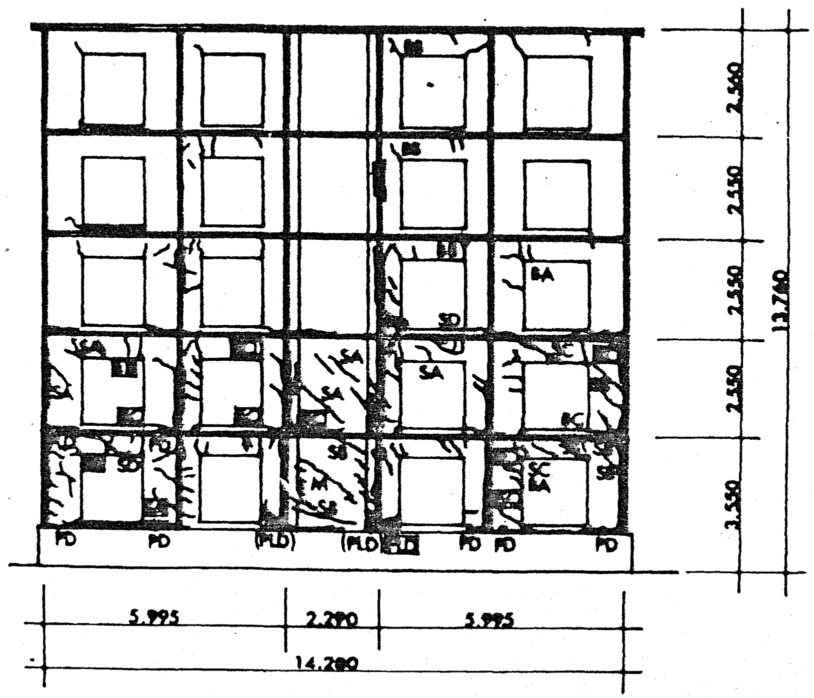


Fig. 8b Final cracking pattern of "Skeleton B."

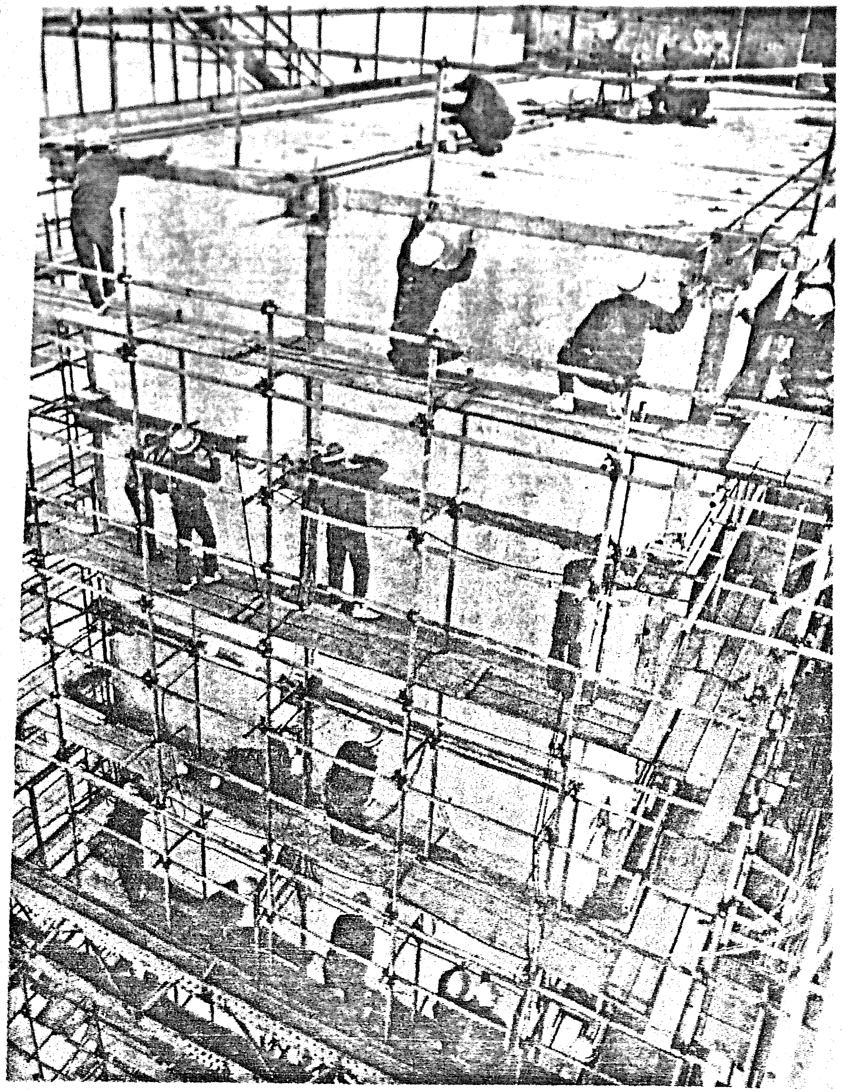


Photo. 1 General View of the Specimen

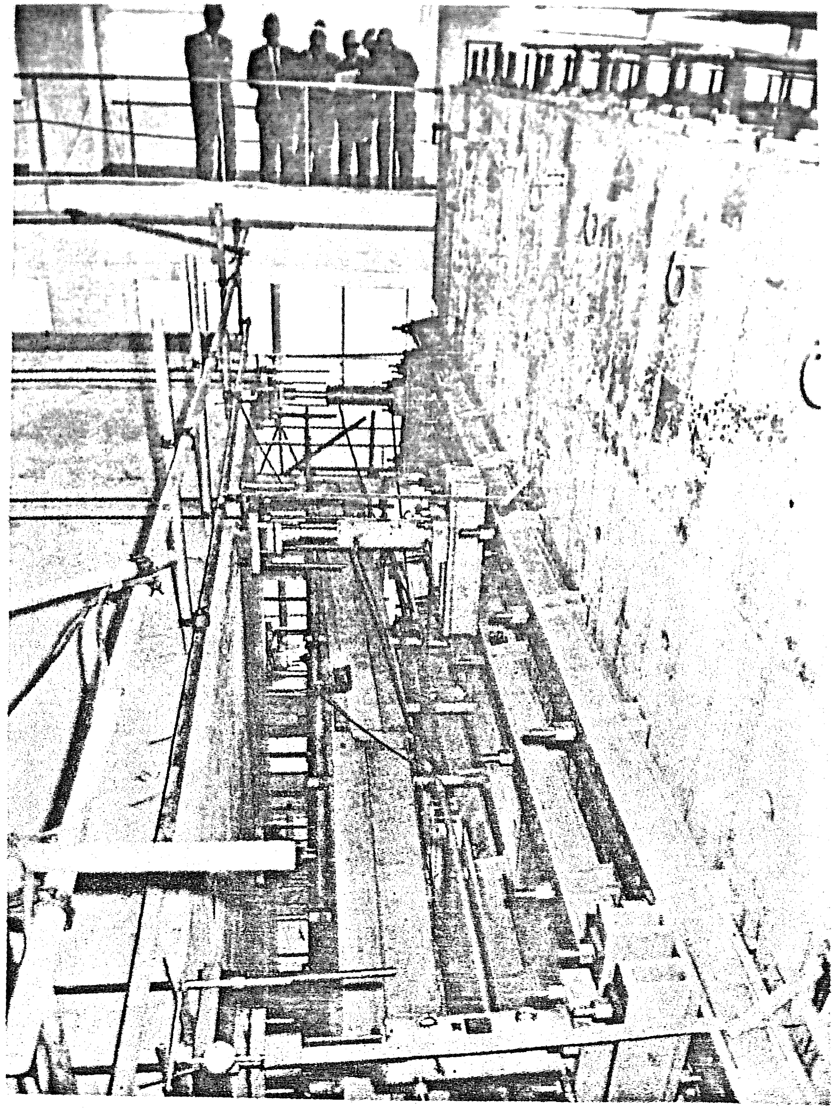


Photo. 2 Loading Jacks

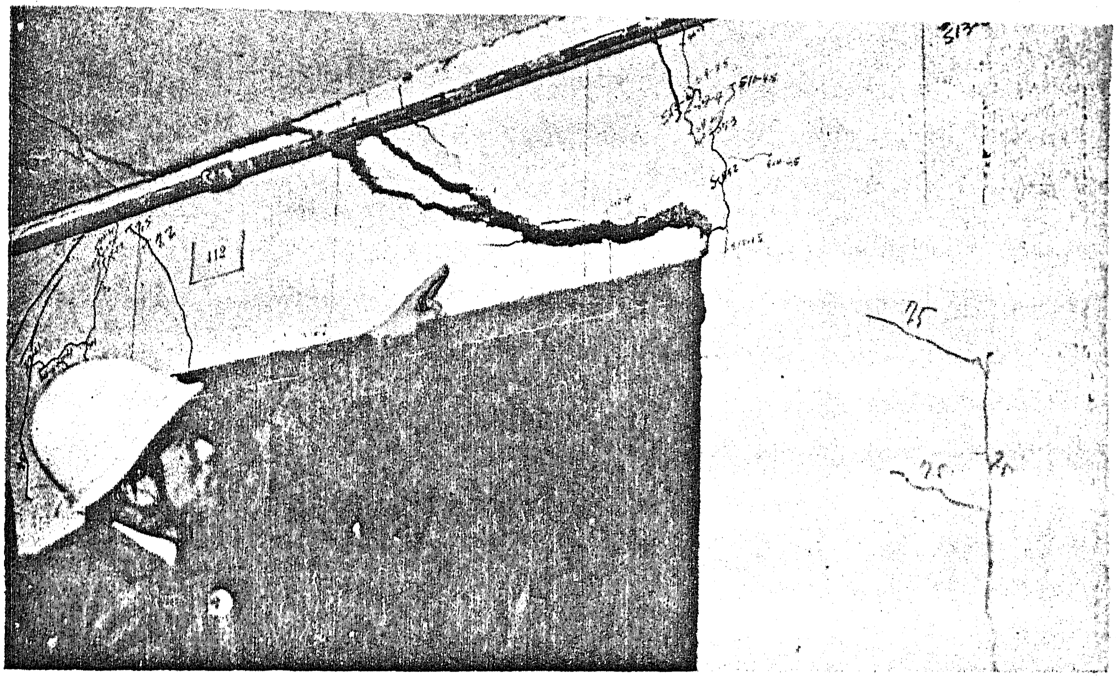


Photo. 3 Typical Cracking Patterns