

# Theme VIa: Safety and stability of elements and structures

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# **VI**

## **Constructions en béton préfabriqué**

### **Vorfabrizierte Bauwerke**

### **Precast Structures**

#### **VIa**

**Sécurité et stabilité des éléments et des  
constructions**

**Sicherheit und Stabilität von Elementen und  
Bauwerken**

**Safety and Stability of Elements and  
Structures**

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## Vla

### Structural Stability of Precast Buildings

Stabilité structurale des bâtiments préfabriqués

Stabilität vorgefertigter Hochbauten

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### INTRODUCTION

The great development in the industrialization of building and the vast proportion of constructions made of parts prepared in advance require both research and standardization for the various types of prefabricated building, and suitable constructive directives.

### GENERAL STABILITY

The general stability of a system consisting of precast panels is determined in a decisive way by the vertical joints between the prefabricated elements and the horizontal belts between the components prepared in advance.

The function of the horizontal and vertical joints from the construction point of view is to secure:

- a) the stability of the structure to horizontal forces, winds and earthquakes;
- b) the stability of the construction as a means for the creation of an alternative system for the passage of forces in case of defect or failure in one of the elements or in the connections, in order to prevent a general destruction as a result from a "progressive-collapse";
- c) the stability of the construction for the passage of the vertical loads.

### HORIZONTAL FORCES

In spite of their great rigidity, the precast constructions are sensitive to horizontal forces, winds and earthquakes, as a result from the sensitivity of the connections between elements prepared in advance.



In comparison with conventional constructions which are relatively elastic and light, the horizontal forces operated on a precast panel construction in an earthquake are relatively great, as they are proportional to the rigidity of the structure and its weight.

Seismology enables today a precise registration of vibrations, and accelograms registered during an earthquake serve as data for the dynamic calculation of structure in earthquakes.

Most of the standards include a standard spectral line for the evaluation of the dynamic forces of a planned construction in an expected earthquake:

$$F_z = \alpha \cdot \beta \cdot \gamma_z \cdot \delta \cdot \theta \cdot \eta \cdot W_z \quad (1)$$

$F_z$  = equivalent horizontal force equals to earthquake.

As the dynamic coefficient  $\beta$  is function of the dynamic properties of the structure, it is important to determine the fundamental period of vibration "T". In the simplest way of a console with one mass "m":

$$T = 2\pi \sqrt{\frac{m}{K}} \quad (2)$$

In a structure with a number of masses, the fundamental period of vibration T may be determined with the help of a computer, in a precise analytical way, or in an approximate way according to:

$$T = 2\pi \sqrt{\frac{\sum W_i \cdot A_i^2}{g \sum W_i \cdot A_i}} \quad (3)$$

(See Figure No. 1 ).

In various standards, there are empirical formulas for determining the fundamental period; in the French "Règles Parasismiques 1969" [3], there is, for example, a formula for the evaluation of a period suiting a structure with concrete walls, which, by its definition, is very close to the precast panel building:

$$T = 0.06 \frac{H}{\sqrt{L}} \sqrt{\frac{H}{2L + H}} \quad (4)$$

(See Figure No. 3 ).

On the other hand, in the American "Recommendation for Lateral Force" S.E.A.O.C. [2], after the changing of the formula into metric units, and in the French standard P.S.69 [3] for reinforced concrete skeleton, the period T is evaluated according to:

$$T = 0.09 \frac{H}{\sqrt{L}} \quad (5)$$

(See Figure No. 3 ).

A comparison was prepared between period formulas (4) and (5) and is obtained in Figure (3) for constructions of a different width  $L_x$ , beside uniform load calculation Figure (2).

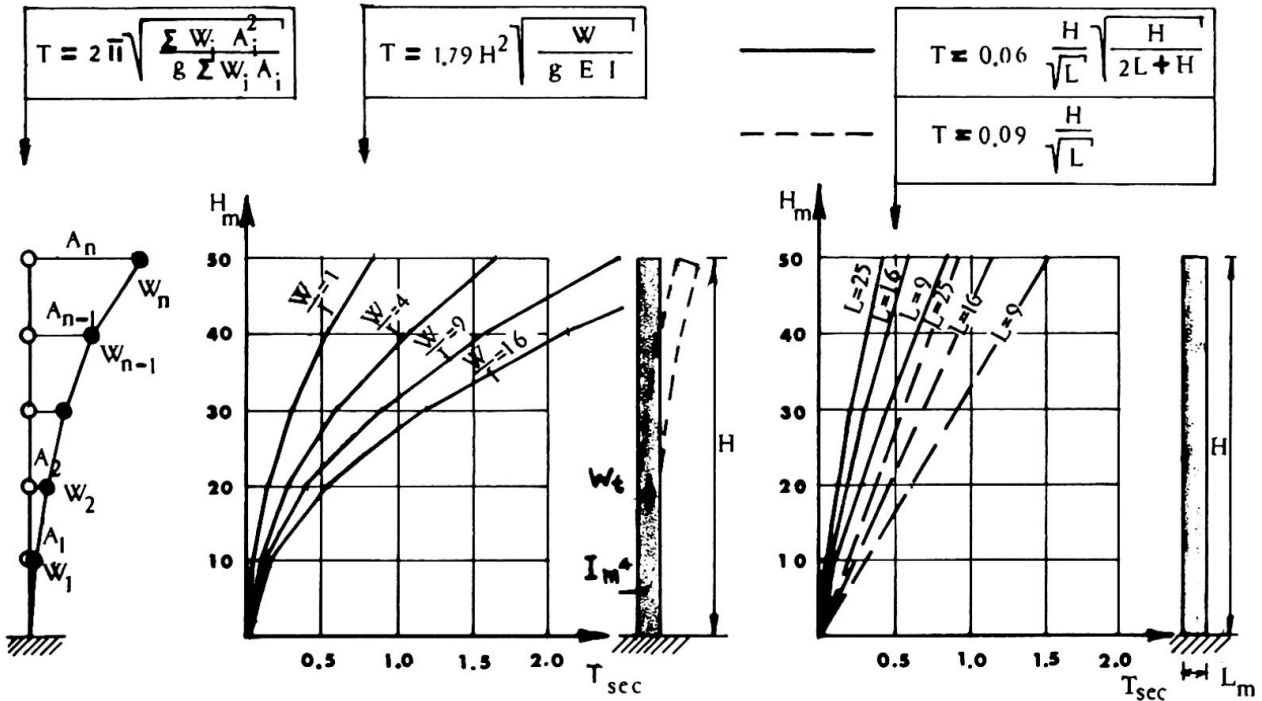


Figure (1)

Figure (2)  
Uniform load calculation

Figure (3)  
Comparison between  
code formulas

Figure (3) shows that the evaluation according to formula (4) gives a lower period than formula (5), the lateral forces therefore being greater in precast panel constructions than in reinforced skeleton.

Partial results of computer calculation for a number of prefabricated buildings, show a higher period than formula (4).

UNEXPECTED FORCES

In the last years, we have witnessed a number of failures in precast buildings, resulting from forces unforeseen in advance, such as an explosion of gas container, of hot water boiler, etc...

In picture No. 4, are seen the results of a hot water boiler explosion, on the 4th floor of an 8 storey building in Jerusalem in July 1975. The building consists of precast facades and ceilings, and gables cast in situ.

The comparison of the results of the relatively great damage caused by the known explosion of a gas container on the 18th floor of a 22 storey precast building at Ronan Point, London, in May 1968, and the relatively small damage in the explosion on the ground floor of a precast building in Algiers,



Figure (4)  
Results of boiler explosion  
in Jerusalem

stresses the importance of the horizontal and vertical belts cast between the precast parts, for the stability of the construction in the case of such failures. "Ronan Point building was not properly braced as was the one in Algiers or as the CEB/CIB Recommendations require". (Reference [4] ).

In order to prevent a progressive collapse in the case of failure in one of the construction components, there should be an alternative way for the passage of forces, as the precast parts are usually simply supported and the reserves in absence of fixing are small. There is a possibility of using a three dimensional system of belts cast around the precast parts, in order to ensure continuity, which may serve as substitute for the passage of forces, thus preventing a progressive collapse.

The efficiency of such a belt system depends to a great extent on the properties of the joints:

- a) Ability of the vertical joint between precast panels to transfer tangential forces on all the height of the wall, in a "joint organisé" with keys, etc., in order to secure a participation of nearby walls and to transfer them into one stiffening shear wall.
- b) use of the intersection between the horizontal and vertical belts to transfer the shearing forces as a bolt ("verrou") action.
- c) possibility of the joints to transfer tensile forces by the reinforcement in the belts, by locking bars or by steel welding.
- d) keeping infinite stiffness of the ceilings in their plane by belts between the ceilings and a suitable peripheral ring beam.

The reliability of the joints depends, inter alia, on their geometric shape and their execution. Fine sections, shrinking cracks and lack of precision in execution may cause failures which have not been taken into the account of stability.

VERTICAL LOADS

What characterizes precast constructions from the point of view of passage of vertical loads are the bearing walls with a big slenderness with a minimum reinforcement and sometimes with no reinforcement at all.

The load carrying capacity of such a bearing wall is influenced by the buckling following the big slenderness, by the eccentricity resulting from inaccuracies in execution and by eccentricity of the loads. In order to illustrate the buckling and eccentricity problems in precast constructions with their bearing walls, an example of curves was prepared [5] [6] on the basis of "CEB" recommendation [1], as follows:

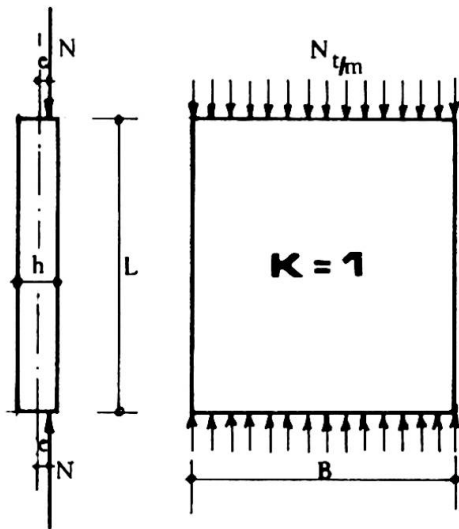


Figure (5)  
Assumption is that the vertical ends are free

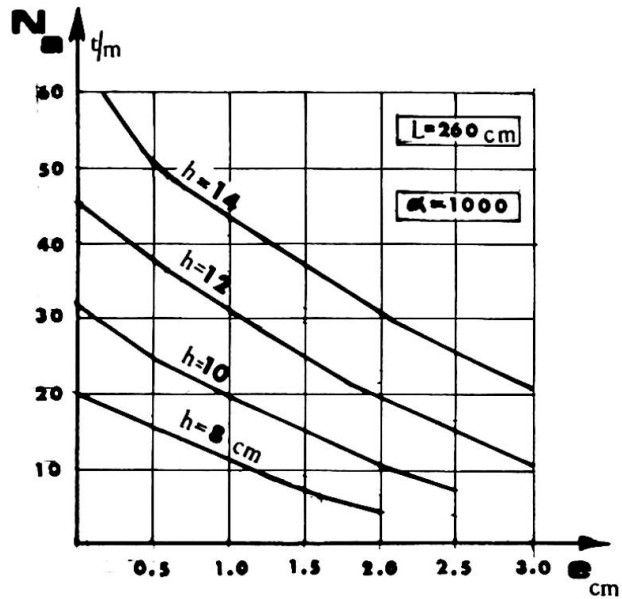


Figure (6)  
Allowed load on walls as function of eccentricity for height of L = 260cm. and  $\alpha = 1000$

$N_u$  - The load carrying capacity of the wall taking into consideration buckling.

$$N_u = \varphi N = \varphi \left( \frac{e^*}{h} ; \bar{\lambda} \right) \times \sigma_b \times A_b \tag{6}$$

$$\bar{\lambda} = \frac{L}{h \sqrt{\alpha}} \tag{7}$$

CONCLUSION

Precast buildings have specific stability problems both from the horizontal and vertical points of view. These subjects require the continuation of research and development.

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## SUMMARY

In spite of their great rigidity, the precast buildings have specific stability problems, determined in a decisive way by the sensitivity of the connections between elements. The function of the joints is to secure the stability of the structure to horizontal and vertical forces, and to prevent a general destruction as a result from a "progressive collapse".

## RESUME

En dépit de leur grande rigidité, les bâtiments préfabriqués ont des problèmes spécifiques de stabilité, déterminés par la sensibilité des joints entre les éléments. La fonction de ces joints est d'assurer la stabilité de la structure soumise à des forces horizontales et verticales, et d'empêcher une destruction générale découlant d'un effondrement de proche en proche.

## ZUSAMMENFASSUNG

Vorgefertigte Hochbauten zwingen eine grosse Steifigkeit. Trotzdem existieren Stabilitätsprobleme, die vorwiegend auf die Verbindungen zwischen den einzelnen Elementen zurückzuführen sind. Die Verbindungen müssen die Stabilität der Tragwerke unter vertikalen und horizontalen Lasten sicherstellen und fortschreitendes Versagen verhindern.

Vla

**Tests of Cantilever Action in Damaged Large Panel Structures**

Essais sur l'action en porte-à-faux de structures endommagées,  
composées de grands panneaux

Untersuchung des Kragverhaltens in beschädigten Konstruktionen der  
Grossplattenbauweise

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1. HIGHLIGHTS

This report describes initial tests being conducted in a major investigation to develop criteria for design and construction of large panel concrete structures. The tests were carried out to determine the stability of large-panel structures in the event of partial loss of support caused by the ineffectiveness of a lower panel. Details of the test structures are intended to permit panels above the one removed, to act as cantilevers from the adjacent undamaged structure.

The tests were carried out to determine if the cantilever portion acted as a single unit, as individual story-high cantilevers or as some intermediate structure. In addition, data needed to design a comprehensive test series were obtained.

A 3/8-scale model representing an end wall of a 6-story large panel building was assembled using details common to North American construction. To represent the ineffective support, one-half of the first story end wall was omitted. As load was applied at each floor level, stresses in the connecting tension ties at each floor level were measured. Tests were conducted for cantilevers of 2 stories, 3 stories, 4 stories and 5 stories. Finally, the 5-story cantilever was loaded to destruction.

Partial movement of joints between panels and floor elements prevented the cantilever from acting as a single unit and complicated the structural behavior. Measured story load versus deflection relationships indicate that performance of the 5-story cantilever was similar to that calculated for a 2-story cantilever.

Measured stresses in the tension ties at each floor level were consistently less than the calculated values.



## 2. TEST STRUCTURE

Construction and testing started with a 2-story cantilever as shown in Fig. 1.

Wall panels were 3-in. (76 mm) thick, 36-in. (914 mm) high and 11-ft. 3-in. (3.4 m) long. Two shear connectors were used at the vertical joint between panels at each story. Hollow core slabs spanning between wall panels were represented in the test structure by short stub sections. The floor stub elements were 3-in. (76 mm) thick to represent 8-in. (203 mm) thick slabs in the prototype structure. Spaces between the ends of the slabs were provided by a 1-in. (25 mm) wide slot through the length of the element.

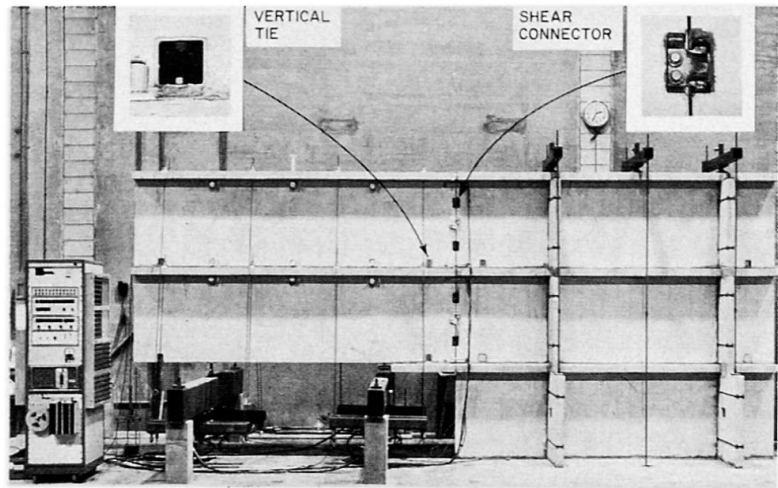


Fig. 1 - Two Story Cantilever

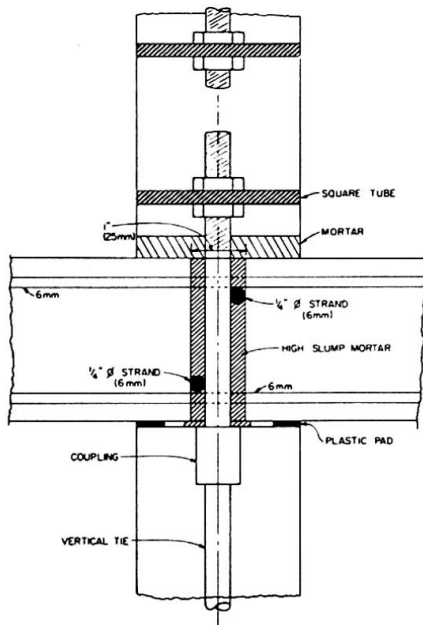


Fig. 2 - Joint Details

In Fig. 2, a detail of the joint at a vertical tie shows the horizontal tie consisting of 1/4-in. (6.4 mm) strand. This tie was placed in the slot between ends of the floor stubs and was continuous the length of the specimen. A fluid mortar was placed in the joint shown in Fig. 2.

After the floor slab stubs were placed, a wall panel was positioned over the joint. Dry mortar was then packed between the floor element and the wall panel above. Continuous vertical ties located 11 1/4-in. (285 mm) from each end of each wall panel were used to tie the newly placed panel to the structure.

Deflections, applied loads, strains in the ties, and joint movements were measured.<sup>(1)</sup> Strain gaged load cells were used to sense applied loads. A linear potentiometer was used to measure cantilever deflection. Strains in ties were sensed with bonded electrical resistance strain gages. Dial gages were used to sense horizontal and vertical joint slip. Vertical joint opening was monitored with potentiometers.

## 3. TEST PROGRAM

The cantilever structure was tested using hydraulic jacks to apply force through loading rods to 8 points on each floor. The load was applied in increments to a maximum representing full dead load and one-third of design live load. This combination was selected as representative of ordinary residential loading. (2,3)

At completion of the test on the 2-story cantilever, the free end was jacked back to its original position. A new story was added to the top of the test specimen to form a 3-story cantilever. The test load was then applied to the new structure. This process was repeated until a 5-story cantilever had been constructed and tested.

After dead load plus one-third of design live load had been applied, the complete 5-story cantilever structure was loaded to destruction. As load was increased, the vertical ties connecting the bottom panel to the structure fractured, as shown in Fig. 3. Continued loading produced additional tie fractures until only the top panel remained. Further application of load resulted in slip between the panel and the floor above.

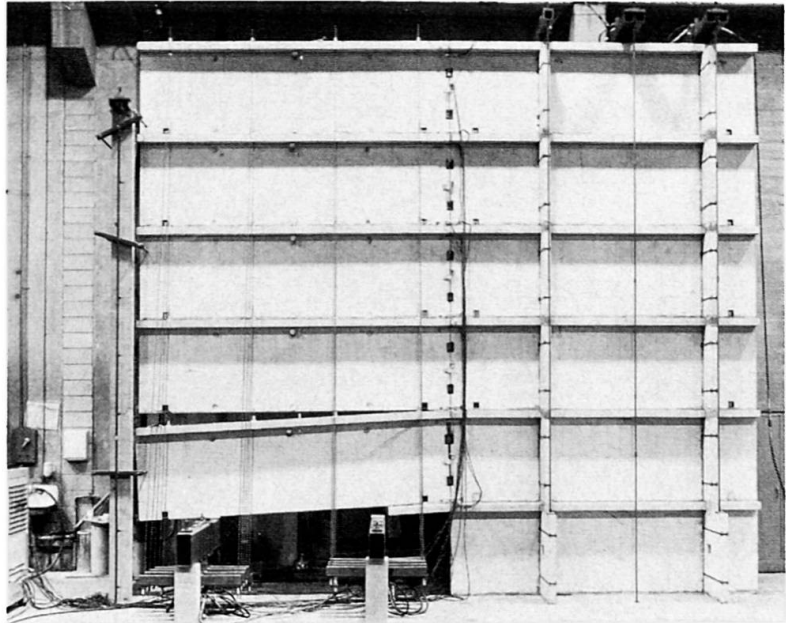


Fig. 3 - Test Structure After Fracture of Vertical Tie at Floor Level 2

4. TEST RESULTS

Measured story load versus deflection relationships for each service load test are shown in Fig. 4. The test load of 8.2 kips (36.4 kN) per story is marked on the load axis.

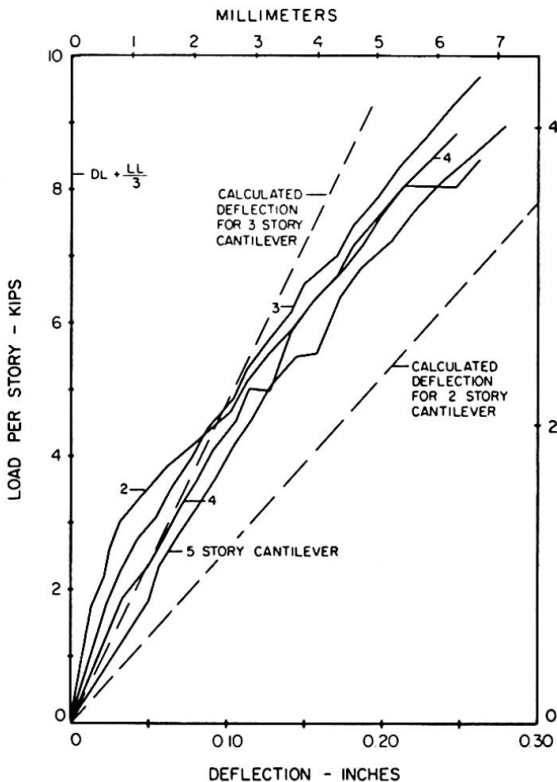


Fig. 4 - Measured Deflections at Low Loads

Measured deflections of the 5-story cantilever as it was loaded to destruction are shown in Fig. 5. Comparison of measured

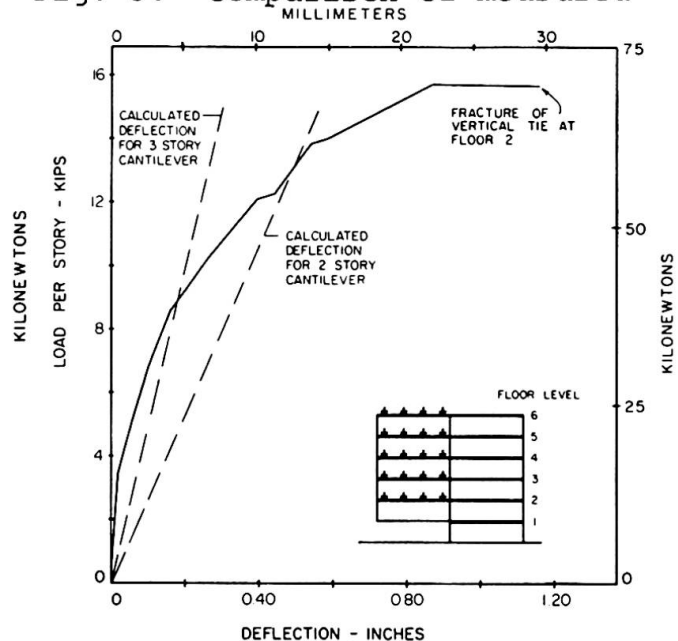


Fig. 5 - Deflection Measured During Test to Destruction



deflections with those calculated suggests that the upper two stories did not act compositely at higher loads.

Measured load versus strand stress in the 2-story cantilever is shown in Fig. 6. Figure 7 shows load versus strand stress for the test to destruction. In all cases the observed strand stress was substantially lower than values calculated assuming a cracked composite section. For example, the calculated maximum stress on the 3-story cantilever at a load of 8.2 kips (36.4 kN) per story was 105 ksi (724 MPa). Stress obtained from measured strain was about 55 ksi (380 MPa).

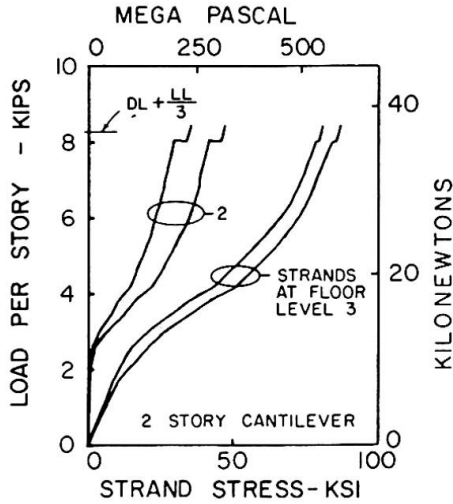


Fig. 6 - Measured Strand Stress

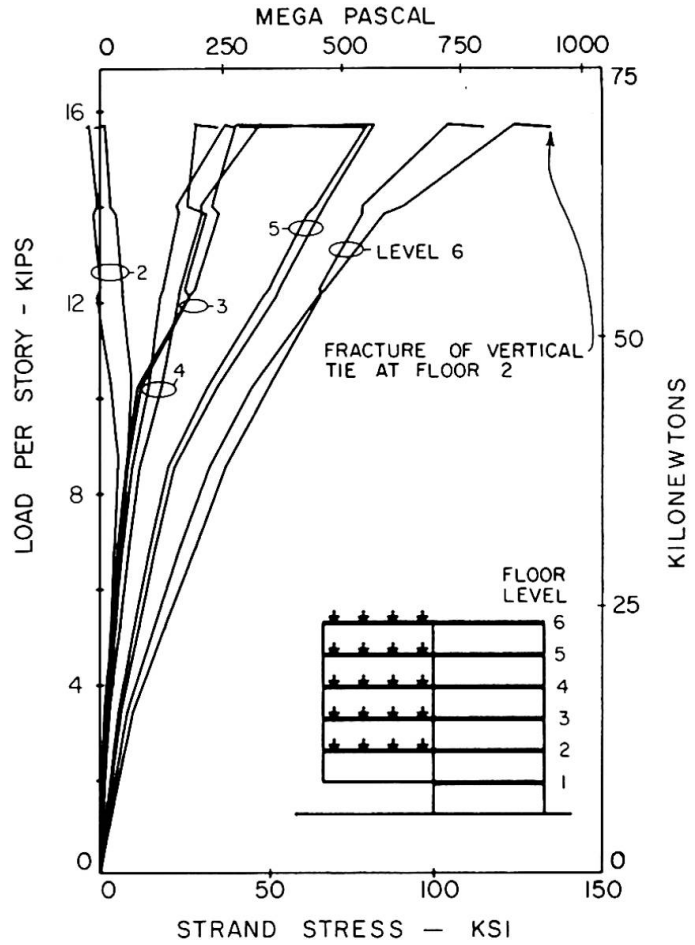


Fig. 7 - Strand Stress During Test to Destruction

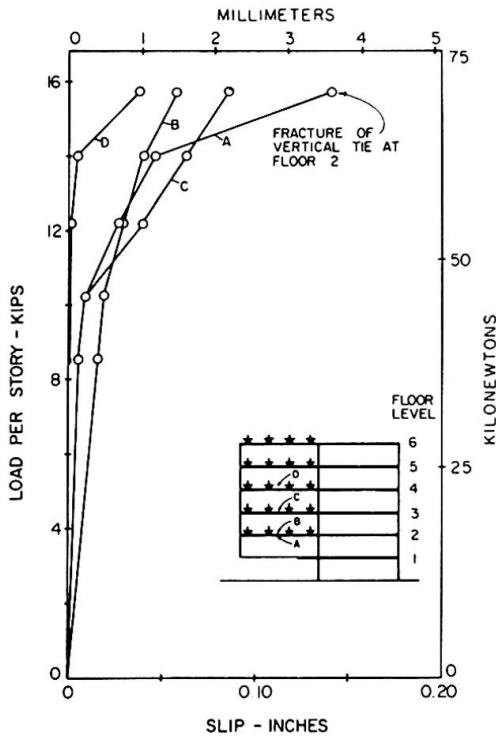


Fig. 8 - Slip During Test to Destruction

The differences between measured and calculated deflections and stress are attributed to slip at the horizontal joints in the lower stories. Slip measured during the test to destruction is shown in Fig. 8.

## 5. CONCLUSIONS

These tests indicate that large panel structures as built in North America can be constructed so that progressive collapse is avoided even when large portions of lower story supports become ineffective. Further testing and analysis will be directed toward determining minimum tie requirements to ensure self-support of the wall panels in the event of catastrophic loss of a lower panel.

## 6. ACKNOWLEDGMENT

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## SUMMARY

This report describes a part of a major study to develop design criteria to minimize the possibility of progressive collapse. Large panel precast concrete structures as built in North America were considered. A 3/8-scale structure representing an end wall of a 6-story building was constructed and tested. The tests were used to determine the stability of large panel structures in the event of partial loss of support by accidental removal of a lower panel. Results of these tests indicate that if suitable details are provided, progressive collapse can be avoided even when large portions of lower stories become ineffective.

## RESUME

Une étude majeure a été entreprise afin de développer des règles de dimensionnement pour minimiser la possibilité d'effondrement de proche en proche, dans le cas de grands panneaux préfabriqués pour structures en béton construites en Amérique du Nord. Une structure à l'échelle 3:8, représentant la paroi extrême d'un bâtiment de six étages fut construite et soumise à l'épreuve. Les essais devaient déterminer la stabilité de la structure composée de grands panneaux en cas de perte partielle d'appui, par suite du déplacement accidentel d'un panneau inférieur. Les résultats de ces essais ont montré que des détails constructifs bien conçus peuvent éviter l'effondrement de proche en proche, même lorsqu'une partie importante des étages inférieurs disparaît.

## ZUSAMMENFASSUNG

Um den fortschreitenden Einsturz der in Nordamerika aus grossen Betonfertigteil-Platten hergestellten mehrstöckigen Bauten auszuschliessen, wird zur Zeit eine grosse Untersuchung durchgeführt mit dem Ziel, entsprechende Bemessungsregeln zu entwickeln. Im Rahmen der Untersuchung wurde ein Modell der äusseren Wand eines 6-stöckigen Gebäudes im Massstab 3/8 hergestellt und unter der Belastung geprüft. Die Versuche dienten zur Bestimmung der Stabilität solcher Bauten unter der Voraussetzung, dass die untere Platte zufälligerweise ausfällt. Die Versuchsergebnisse haben gezeigt, dass bei Anordnung geeigneter Massnahmen ein fortschreitender Zusammenbruch vermeidbar ist, selbst wenn grosse Teile der unteren Stockwerke nicht mehr wirksam sind.

**Structural Safety of Precast Concrete Apartment Houses against Earthquake**

La sécurité structurale de bâtiments d'habitation en béton précontraint vis-à-vis des tremblements de terre

Die konstruktive Sicherheit von Wohnhäusern aus Betonfertigteilen gegen Erdbeben

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1. Introduction

This report discusses the structural safety of publicly operated precast concrete apartment house developed in Japan, based upon the tests on full-size housing structures, carried out in the Large Size Structures Testing Laboratory.

This laboratory has a double decked testing bed of 9.5 m x 20 m in plan, and a vertical reaction wall of 16.1 m in height as shown in Fig. 1. Horizontal loads can be applied both in pushing and pulling direction against the reaction wall by means of 20 oil jacks of 50 ton capacity each. Three dynamic exciters of the maximum capacity of 10 ton power each were also furnished to perform forced vibration tests.

2. Construction systems and test specimens

Four tests have been carried out in the above laboratory since 1967 to provide the basic data in regard to the rationalization and the economization of the earthquake resistant design of precast concrete structures.

The outline of the test specimens and some connection details are summarized in Fig. 3 to 7.

The first test was concerned with the 5-story precast concrete large panel construction. Horizontal connections between concrete panels were made by welding of steel plates anchored in these panels. Vertical connections were cast-in-place connections. Each panel had six shear keys and reinforcing bars of 9 mm diameter anchored in the center of each shear key were welded to the vertical reinforcing bars placed through vertical joints. The test specimen consisted of ten housing units as shown in Fig. 7-1.

The second test was carried out in order to realize an 8-story apartment buildings with precast concrete large panel constructions. In this case, the splice sleeves filled with non-shrink mortar were used to connect vertical reinforcing bars cast in precast wall panels instead of welding connections. The test specimen shown in Fig. 7-2 represented the transversal shear wall structure of real 8-story apartment building.

The third test was concerned with the precast concrete structure assembled with post-tensioning method. This housing structure consisted of following

precast reinforced concrete elements: L, +, T, I shaped wall columns, shaped girders and slabs. Prestressing bars anchored in underground girders cast-in-place were extended through wall columns and girders and tensioned and grouted after completion of assembly works of every five stories. The outline of this construction system is summarized in Fig. 5. Specimen 3-1 was the 4-story wall frame structure consisting of full size structural elements almost same as those of the lower 4-story of a real 8-story apartment building. Specimen 3-2 corresponded to transversal wall frame structure full-filled with bearing wall.

The last test was intended to realize an 11-story precast concrete apartment building which had 14-bay rigid frame structures in the longitudinal direction and shear wall structures in the transversal direction. The typical precast reinforced concrete member in the rigid frame structure had double cross shape as shown in Fig. 6-a). These members were connected each other at the ends of the each member, that is at the mid-span of beams and columns of the frame structure. The longitudinal reinforcing bars were connected with screws in the beams and with splice sleeves filled with non-shrink mortar in the columns. The transversal shear wall consisted of precast wall panels with top beam. Connection details between wall panels and columns are shown in Fig. 6-d). The specimen 4-1 was of a 4-story full size model representing the rigid frame of an 11-story building, and specimen 4-2 was of an 8-story half size model representing the transversal shear wall structure.

### 3. Test procedure

In each test, a specimen was fixed to the testing bed and horizontal loads were applied to the specimen using oil jacks fixed to the vertical reaction wall. The distribution of horizontal loads in each test is shown in Fig. 8. Except the first test, vertical axial loads representing the dead and live loads of the upper portion of an actually designed building were also applied during horizontal loading as shown in Fig. 2.

The test specimen was loaded statically up to collapse in the horizontal direction. In the test No.1 and No.3 some supplemental forced vibration tests were carried out to clarify the change of dynamic properties such as natural period and damping, according with the progressing of failure.

### 4. Test result

The principal test results on static horizontal loading are summarized in Table 1 and Fig. 8. The ordinate and abscissa of Fig. 8 show the ratio of total applied loads in each test to the sum of dead weight and additional live loads supposed to be beared in the corresponding portion of the actually designed building and the translation angle defined as the ratio of total horizontal displacements measured at the top of the specimen to the total height of it, respectively. The design load was defined, hereafter, as the loads corresponding to 20% of the above mentioned gravity loads.

Specimen 1 : Flexure cracks appeared in beams just at the design load. As the load increased, flexure cracks in beams and shear cracks in walls progressed successively. The maximum load carrying capacity attained to 7.6 times design load with the translation angle of  $13.5 \times 10^{-3}$  at the second floor and of  $2.7 \times 10^{-3}$  at the top. Considerable decreasing of load carrying capacity due to shear failure of wall panels was observed after the maximum load. The natural period and damping obtained from the forced vibration tests were 0.1 sec. and 6.3%, respectively before static loading test and 0.17 sec. and 10.6%, respectively after maximum load attained.

Specimen 2 : Initial shear cracks appeared in the coupling beams at 2.5 times design load. The maximum load carrying capacity was attained at the 4.8 times design load with the translation angle of  $5.1 \times 10^{-3}$  at the top. The coupling beams had heavily failed in shear before reached to the maximum load.



The final translation angle at the top was  $21.7 \times 10^{-3}$ .

Specimen 3-1 : The load displacement curves indicate a fairly linear relation up to 1.2 times design load. At this stage, no column cracks but slight cracks of girders were observed. The initially linear stiffness decreased gradually beyond this loading stage with gradual increase of horizontal slippage and progressing of bending cracks of wall columns and girders. The maximum load carrying capacity attained to 3.9 times design load, with the translation angle of  $12.2 \times 10^{-3}$  at the 2nd floor and of  $9.7 \times 10^{-3}$  at the top. The final failure mechanism was the shear failure of 2nd and 3rd floor beams and further repeating of load did not cause significant decrease of load carrying capacity up to the translation angle of  $24.4 \times 10^{-3}$  at the 2nd floor and of  $16.1 \times 10^{-3}$  at the top. The natural period and damping obtained from forced vibration tests were 0.12 sec. and 2% before applying any static load and 0.28 sec. and 7 to 9% after failure, respectively.

Specimen 3-2 : The load-displacement curve show linear relation up to 2 times design load. Beyond this loading stage, the vertical slippage between wall columns and bearing walls occurred. The stiffness of the structure decreased rapidly after the applied load reached to 3.5 times design load due to occurrence of shear cracks of wall columns and progressing of shear cracks of beams just above the vertical joint between wall columns and bearing walls. The maximum load carrying capacity was about 4 times design load and the translation angle at this loading stage was  $5.3 \times 10^{-3}$  at the 2nd floor and  $3.8 \times 10^{-3}$  at the top. The decreasing of load carrying capacity was about 36% of the maximum value at the translation angle of  $10.2 \times 10^{-3}$  at the top.

Specimen 4-1 : Before reached to the design load, bending cracks of beams and columns had progressed extensively. Diagonal shear cracks in the panel zone of the beam-column connection occurred almost at the design load. At 1.7 times design load yielding of the main reinforcing bars at the ends of columns and beams and compression failure of concrete at the top of the 4-th story column and at the bottom of the 1st story column started. The maximum load carrying capacity was 1.8 time design load. The final displacement reached to 4.5 times the displacement at the yield load not showing any decreasing of load carrying capacity. This frame structure showed very ductile behaviour under horizontal loading, because the flexural failure at beams and columns was dominative instead of shear failure of the members in the other specimens.

Specimen 4-2 : The bending cracks at the 1st story column-column connection appeared at 1.3 times design load and considerable increase of crack width at this portion was observed in successive loading. The maximum load carrying capacity was 2.1 times design load at the translation angle of  $4 \times 10^{-4}$  at the top. Beyond the maximum load, significant decrease of load carrying capacity was observed, because of the slipping out of longitudinal reinforcement at the 1st story column-column connection.

## 5. Concluding remarks

In case of precast concrete structures, it is not always easy to estimate the structural behaviour as a whole up to failure with analytical procedures, because of not only the difficulty of making model representing the behaviour of connection, but also the difficulty of considering the soundness of the structure affected by construction works. Tests on full scale structure as mentioned above give us the direct informations for estimating the soundness of structures as a whole under the earthquake loading. Further accumulation of such test datum will make us possible to estimate the extent of damage caused by earthquake more precisely and bring us the economization and the rationalization of earthquake resistant design of such the structures.

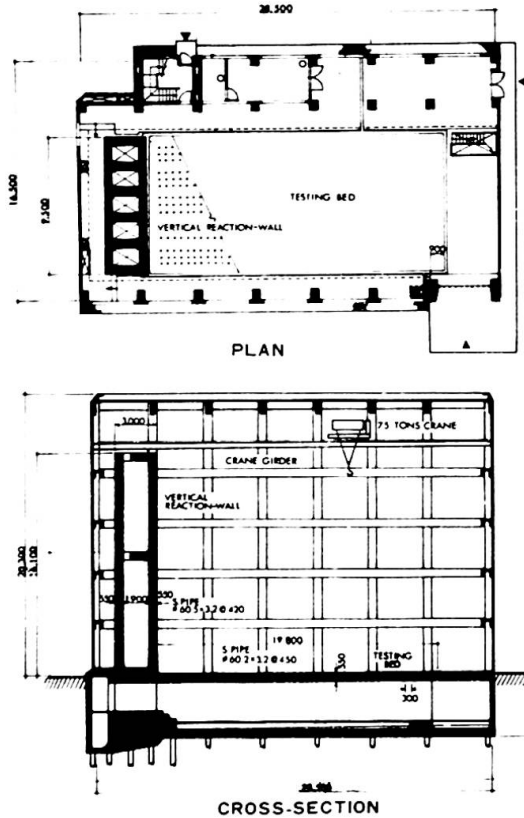


Fig. 1 Outline of Large Size Structure Testing Laboratory

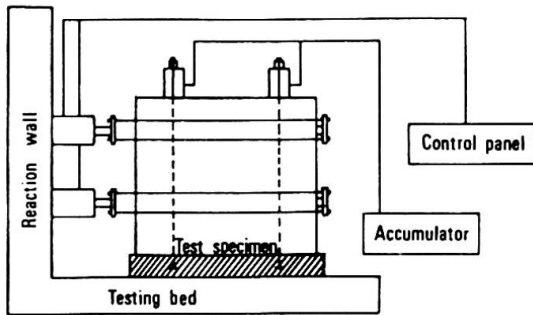


Fig. 2 Block Diagram of Loading Facilities

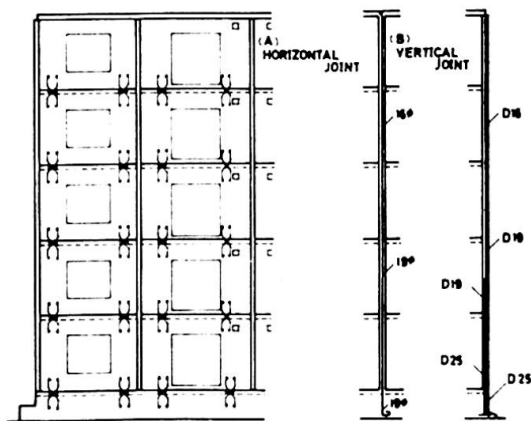


Fig. 3 Detail of Construction System of Specimen 1

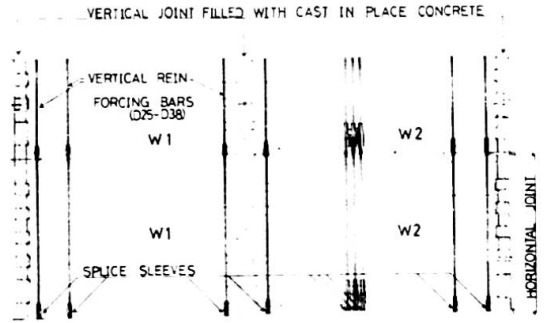


Fig. 4 Detail of Construction System of Specimen 2

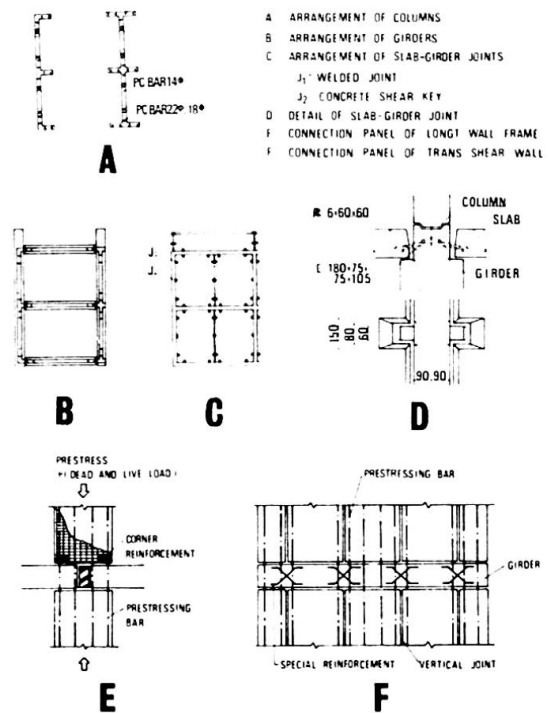


Fig. 5 Detail of Construction System of Specimen 3

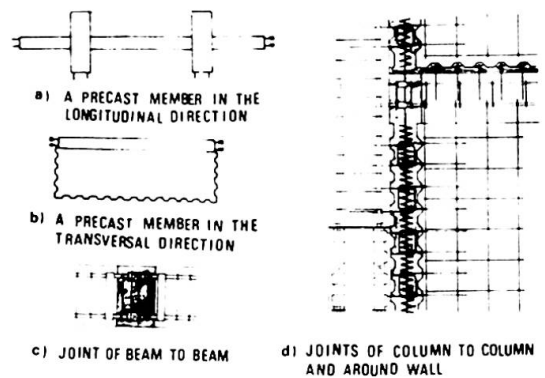


Fig. 6 Detail of Construction System of Specimen 4

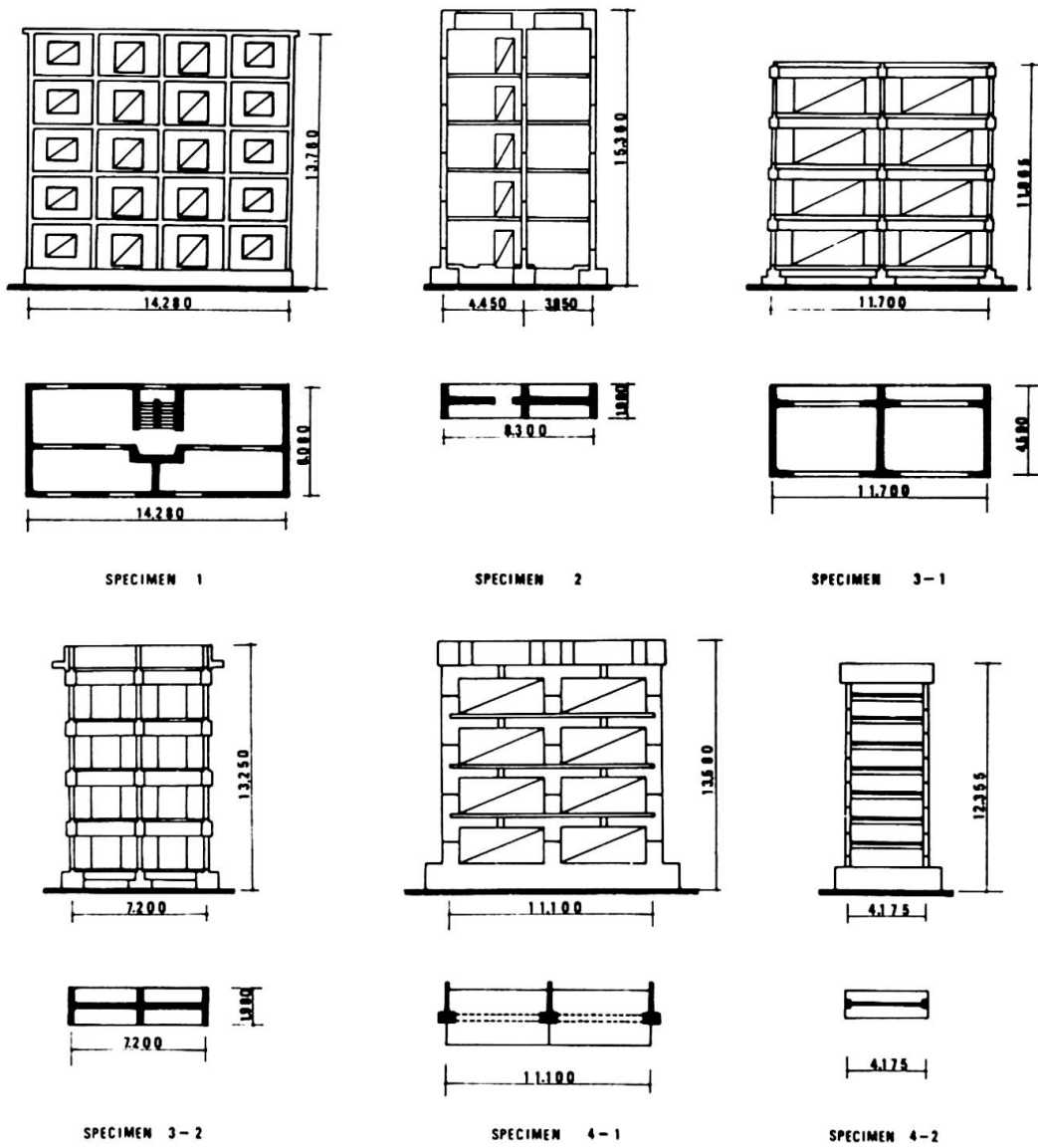


Fig. 7 Outline of Test Specimen

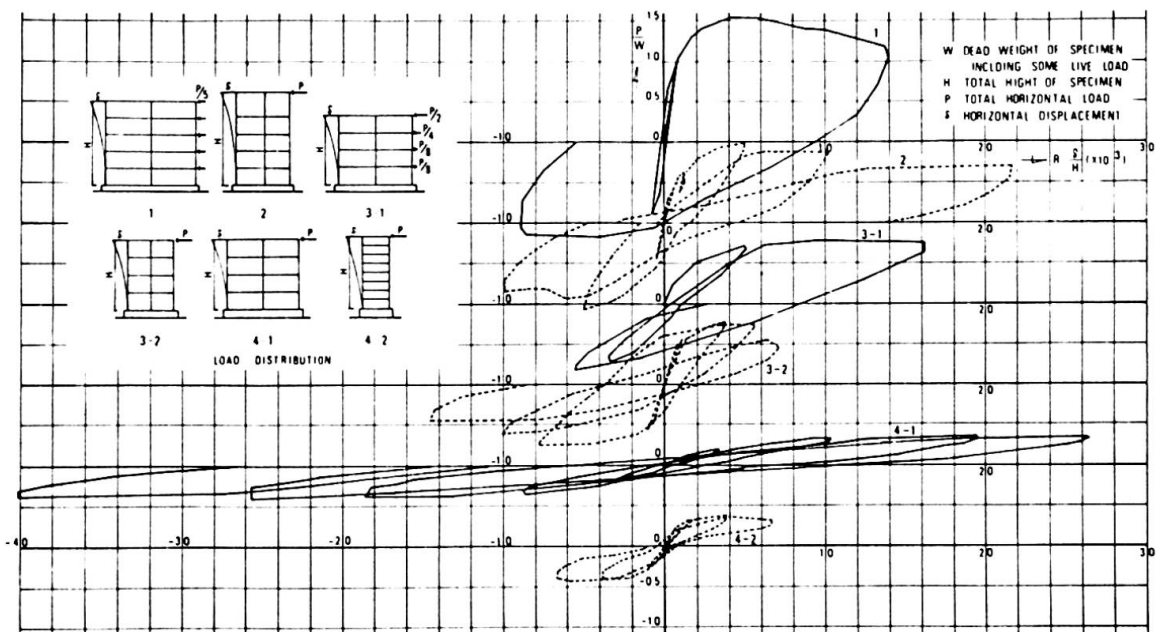


Fig. 8 Dimensionless Load-displacement Curves



Table 1 Result of Test

NO.	1	2	3		4	
			3-1	3-2	4-1	4-1
CONSTRUCTION SYSTEM	R. C. LARGE PANEL CONSTRUCTION	R. C. LARGE PANEL CONSTRUCTION	P. C. CONSTRUCTION		R. C. FRAME CONSTRUCTION	
STRUCTURAL TYPE OF SPECIMEN	SHEAR WALL	SHEAR WALL	WALL FRAME	SHEAR WALL	RIGID FRAME	SHEAR WALL
NUMBER OF FLOORS OF TEST SPECIMEN (OF REAL BUILDING)	5	5 (8)	4 (8)	4 (10)	4 (11)	8 (11) (HALF SCALE MODEL)
THICKNESS OF WALL (cm)	15	18.25	18	18	-	7.5
SECTION (cm)	BEAM	-	30 × 74	30 × 74	40 × 55	20 × 27.5
	COLUMN	-	-	-	55 × 87	27.5 × 43.5
SECTIONAL AREA OF WALL OR COLUMN PER UNIT FLOOR AREA (cm <sup>2</sup> /m <sup>2</sup> )	425	317	247	244	207	421
LENGTH OF WALL PER UNIT FLOOR AREA (cm <sup>2</sup> /m <sup>2</sup> )	28.3	16.7	13.7	13.6	-	18.0
REINFORCEMENT RATIO FOR SHEAR (%)	0.34	0.44	0.63	0.64	1.27	0.56
CONCRETE STRENGTH (kg/cm <sup>2</sup> )	230	335	380	414	400	320
MEAN SHEAR STRESS AT THE DESIGN LOAD BASED ON $k=0.2$ (kg/cm <sup>2</sup> )	2.4	3.8	8.1	7.8	9.1	4.7
MEAN SHEAR STRESS OF WALL AT THE MAXIMUM LOAD (kg/cm <sup>2</sup> )	17.1	18.8	31.5	30.3	18.3	9.5
$T_u/T_d$	7.2	3.3	3.9	3.9	1.8	2.0
COLLAPSE MECHANISM	SHEAR FAILURE OF WALL COLUMN	SHEAR FAILURE OF COUPLING BEAMS	SHEAR FAILURE OF BEAM	SHEAR FAILURE OF BEAM	FLEXURAL FAILURE OF COLUMN AND BEAM	FAILURE CAUSED BY SLIPPING OUT OF REINFORCEMENT AT JOINT OF COLUMN
TRANSLATION ANGLE OF THE FIRST STORY AT MAXIMUM LOAD (10 <sup>-3</sup> )	7.6	4.2	20.8	22.2	20.0	5.9

\* R.C.; Reinforced concrete, P.C.; Prestressed concrete

## SUMMARY

This report discusses the structural safety against earthquake of precast concrete apartment houses developed in Japan. The evaluation is based upon tests on full-size model structures. The results show that the construction system of precast elements have a sufficient load carrying capacity and ductility to resist severe earthquakes.

## RESUME

La sécurité structurale des bâtiments d'habitation préfabriqués en béton, développée au Japon vis-à-vis des tremblements de terre, a été testée sur des essais en vraie grandeur. Les résultats montrent que les capacités portante et de déformation de ce type de construction sont suffisantes pour résister à de sévères tremblements de terre.

## ZUSAMMENFASSUNG

Die Abhandlung beschreibt die konstruktive Sicherheit von in Japan entwickelten Wohnbauten aus Betonfertigteilen. Die Untersuchung gründet sich auf Versuche im Massstab 1:1. Die Resultate zeigen, dass Konstruktionen aus Betonfertigteilen ausreichende Tragfähigkeit und genügendes Verformungsvermögen aufweisen, um auch schweren Erdbeben zu widerstehen.

## Trag- und Verformungsverhalten aus einzelnen Fertigteilen zusammengesetzter Wand- und Deckenscheiben

Strength and Deformation Behaviour of Walls and Floors composed of Precast Elements

Résistance et déformation des dalles et murs composés d'éléments préfabriqués

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### 1. Problemstellung

Für die Aussteifung zahlreicher Gebäude ist das Zusammenwirken von einzelnen Fertigteilen als großflächige Scheibe ausschlaggebend. Das Trag- und Verformungsverhalten dieser Scheiben wird im wesentlichen von dem der Fugen beeinflusst. Da der Spannungszustand der Fugen in Abhängigkeit von ihrer Lage innerhalb der Scheibe sehr unterschiedlich sein kann, sind reine Scher- oder Druckversuche an Fugen nicht ausreichend für die Beurteilung des Verhaltens von Scheiben. Versuche an Scheiben im Maßstab 1:1 sind sehr aufschlußreich; ihre Ergebnisse sind jedoch nur anwendbar, wenn gleiche Fertigteiltafeln, Verbindungsmittel und Auflagerbedingungen vorhanden sind. Es liegt daher nahe, ein allgemeingültiges Rechenverfahren zu entwickeln, in dem Scheiben mit beliebigen Fugen- und Auflagerbedingungen berechnet werden können. Aus diesem Grunde wurde aufbauend auf [1] ein Rechenprogramm nach der Methode der finiten Elemente entwickelt [2], [3], in dem das Trag- und Verformungsverhalten der Fugen, in Erweiterung eines Grundgedankens von Franklin [4], durch orthogonale Federn simuliert wird. In diesem Beitrag werden die wichtigsten Ergebnisse der Auswertung von Belastungsversuchen, die für die Steuerung des Fugentrag- und Verformungsverhaltens im Rechenprogramm Anwendung finden, sowie wesentliche Ergebnisse der rechnerischen Untersuchungen mitgeteilt.

### 2. Trag- und Verformungsverhalten von Fugen

In [1] bis [3] sowie [5] und [6] werden Auswertungen zahlreicher Versuchsergebnisse über Fugen in Fertigteilwänden und -Decken angegeben. In [7] werden Ergebnisse numerischer Parameteruntersuchungen an Wand-Decken-Knoten mitgeteilt.

Die Schubtragfähigkeit von Fugen hängt im wesentlichen von den Parametern Fugengeometrie, (Randprofilierung), Bewehrung (Bewehrungsgrad, Anordnung der Bewehrung) und der Betongüte ab. Für die in Bild 1 definierte Fugenausbildung beträgt der Mittelwert der Bruchschubspannung für Fugen mit

a) glatten Fugenträndern

$$\tau_u = 0.047 \cdot (\mu_{B_s} + \sigma_N) \cdot \sqrt{f_{Bp}} \quad (2.01)$$

b) verzahnten Fugenrändern mit verteilter Bewehrung (Schlaufen)

$$\tau_u = \sqrt{\beta_p \cdot \frac{B}{F_u} (0,4 + 0,44 (\mu \beta_S + \sigma_N))} \quad (2.02)$$

Bei extremen Verhältnissen  $\mu \beta_S / \sigma_N$  ist die additive Wirkung der beiden Einflüsse mit Vorsicht zu handhaben, da die Bewehrung erst nach einer Rißbildung die Fließgrenze erreichen kann. Für unterschiedliche Randprofilierungen, definiert durch die bezogene Zahnfläche  $B/F_u$  sind in Bild 1 die Auswertung der Gleichungen (2.01) und (2.02) dargestellt. Nach den in [6] beschriebenen Versuchen ergibt sich bei sonst gleicher Ausbildung der Fugen (Fugenrandausbildung, Bewehrungsgrad, Betongüte) für eine am Fugenrand konzentrierte Bewehrung (Abstand 2,5 m) eine um 20 - 30 % geringere Bruchlast als für eine über die Fugenlänge verteilte Bewehrung.

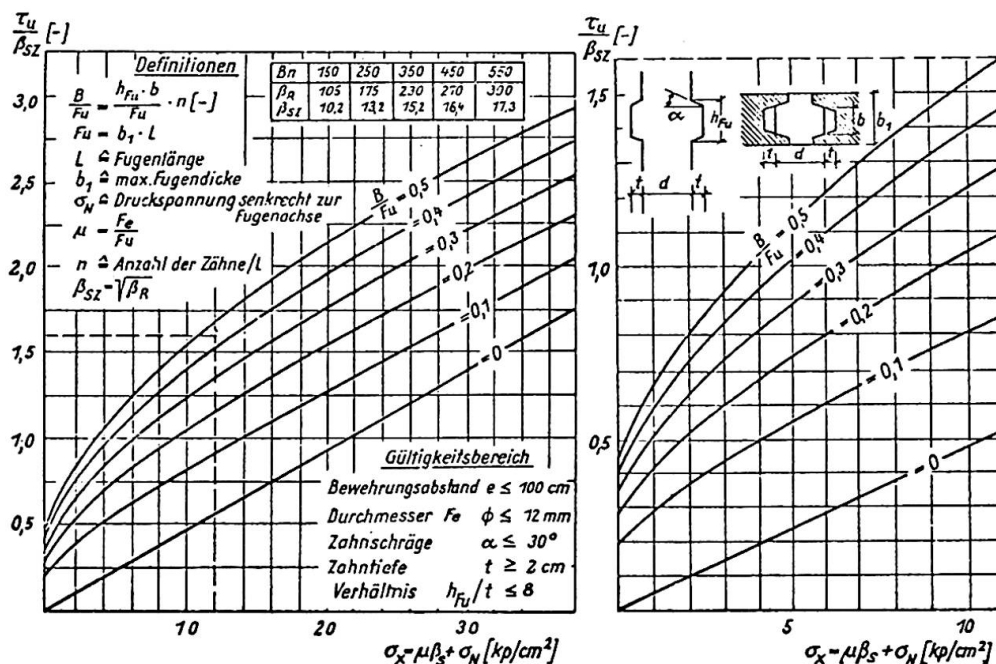


Bild 1: Kurventafel zur Bestimmung des erforderlichen Fugenbewehrungsgrades  $\mu = F_e / F_u$  bei vorgegebener Schubspannung  $\tau_u$

Ein Spektrum von  $\tau - \sigma$ -Beziehungen für den gesamten Schub- und Druckbruchbereich ist aus Bild 2 zu ersehen. Im Schubbruchbereich entspricht die untere Begrenzungsgerade der Charakteristik einer Fuge mit glatten Rändern, die obere Begrenzungskurve der einer verzahnten Fuge.

Typische Schubspannungs-Verschiebungsbeziehungen aus eigenen Versuchen [6] sind in Bild 3 dargestellt. Die Fugensteifigkeit, definiert als Sekantenmodul  $K = \tau / \Delta$  ergibt sich für den Bruch nach [2] zu:

$$K_u = 6,8 \mu \beta_S + 454 \cdot B/F_u \quad [kp/cm^2] \quad (2.03)$$

Die oben angegebenen Fugencharakteristiken können direkt als Bemessungshilfen für die Ingenieurpraxis oder aber auch als Eingabewerte für das Rechenprogramm [2], [3] Anwendung finden.

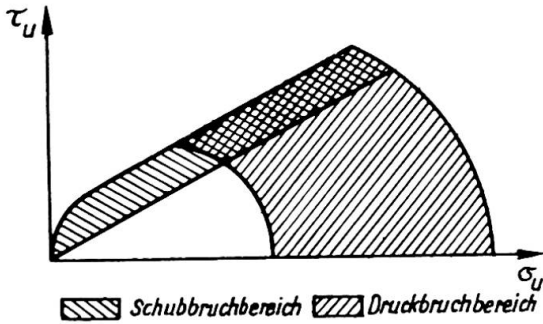


Bild 2: Spektrum von  $\tau - \sigma$  - Beziehungen

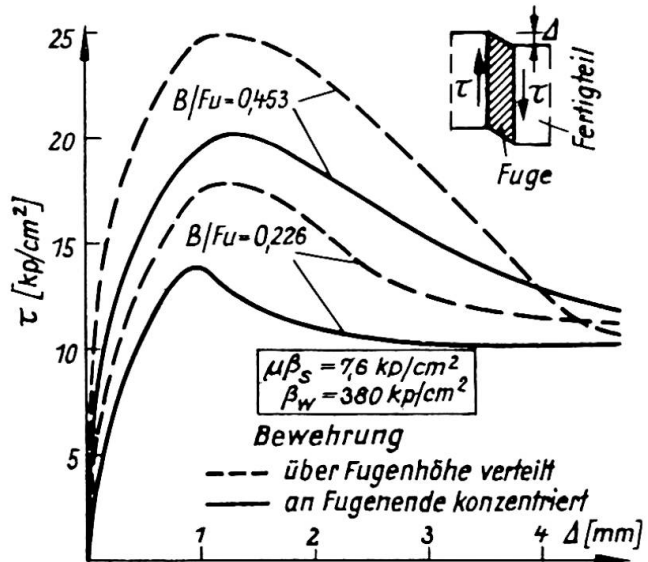


Bild 3: Schubspannungs-Verschiebungsbeziehungen

### 3. Tragverhalten von Wandscheiben aus Fertigteilen

Vertikalfugen von Wandscheiben übertragen im wesentlichen Schubkräfte. Üblicherweise haben diese Fugen geringere Breiten als die Wandtafeln. Infolge Zwangsbeanspruchungen der Wände sind die Kontaktflächen zwischen Fugenverguß und Fertigteiltafeln meistens gerissen. Dadurch ist eine reduzierte Schubübertragung der Fugen bedingt. Die Schubsteifigkeit derselben kann versuchsmäßig ermittelt werden, siehe Gleichung (2.03) und im Rechenprogramm zur Steuerung des Verformungsverhaltens der Fugen berücksichtigt werden.

Für die Horizontalfugen wird die Schubübertragung nicht als kritisch erachtet, da sie im allgemeinen durch Reibung gewährleistet ist. Demgegenüber kann die Drucktragfähigkeit der Fugen infolge der heterogenen Zusammensetzung des Wand-Decken-Knotens gegenüber monolithisch hergestellten Wänden stark reduziert sein, Beispiele siehe z.B. in [2], [7].

Unter der Annahme, daß alle Horizontalfugen überdrückt sind, und ihre Schubverformung vernachlässigt werden kann, ergeben sich die in Bild 4 [8] dargestellten Abminderungsfaktoren für die Trägheitsmomente der Wand bei Berücksichtigung der Schubverformung der Vertikalfugen. Diese reduzierten Trägheitsmomente sind bei der Verteilung der Horizontalkräfte anzusetzen. In [9] sind darüberhinaus Kriterien angegeben, bei deren Erfüllung der Nachweis der Horizontalkräfte von Großtafelbauten vernachlässigt werden kann.

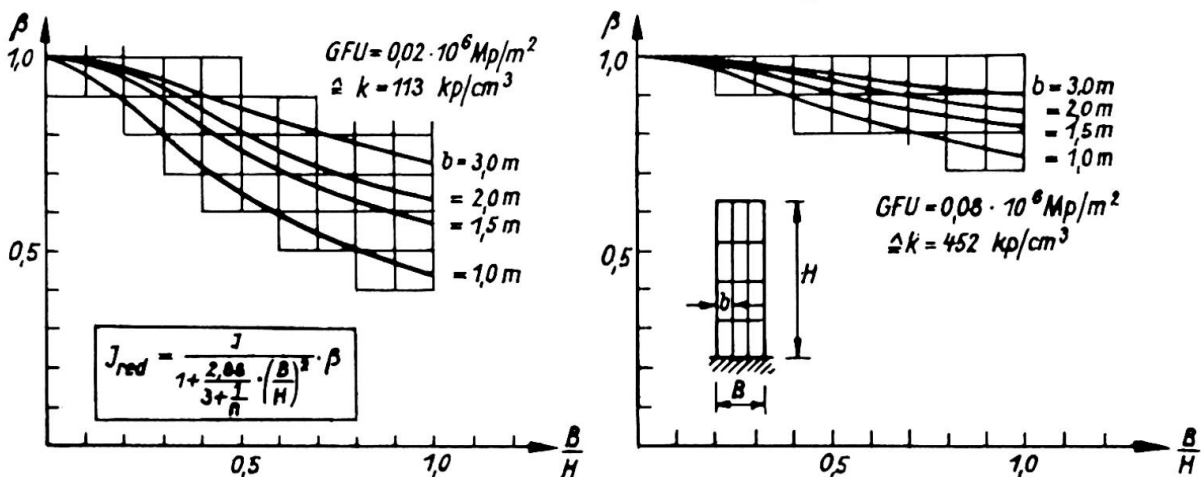


Bild 4: Abminderungsfaktor  $\beta$  für das Trägheitsmoment der Wand

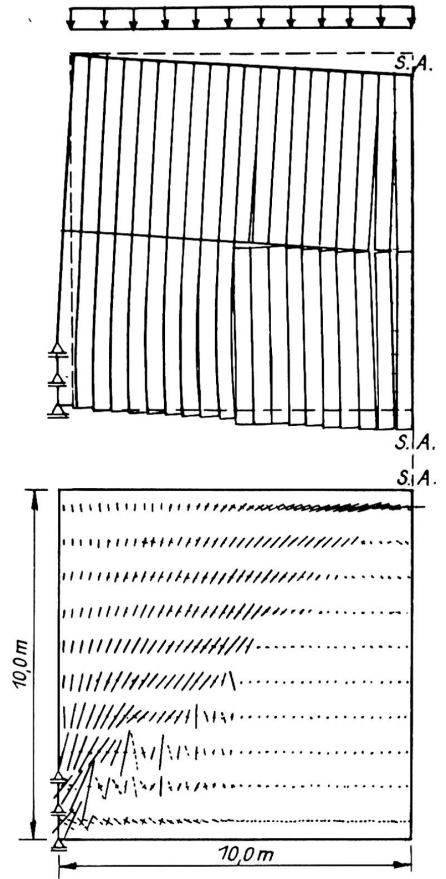
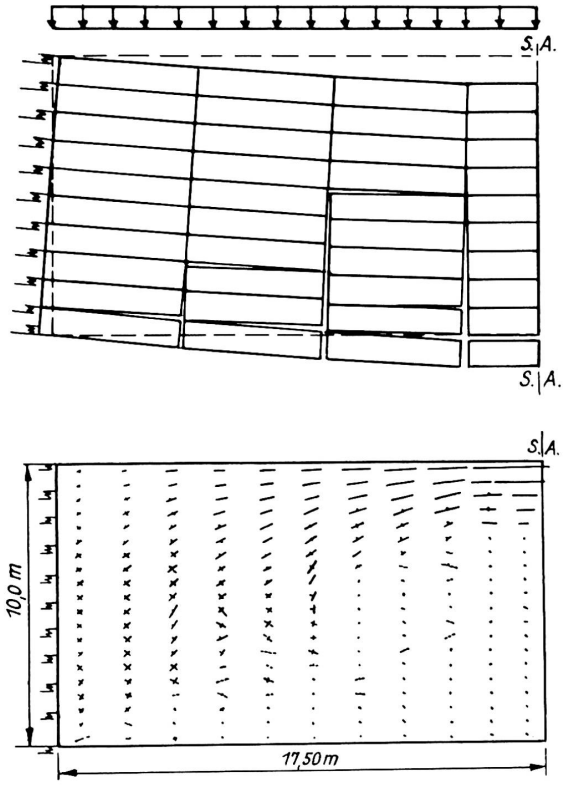


Bild 5: Verformungs- und Hauptspannungsbilder von Fertigteil-Deckenscheiben

#### 4. Tragverhalten von Deckenscheiben aus Fertigteilen

Zum Studium des Tragverhaltens von Deckenscheiben wurden zahlreiche numerische Untersuchungen durchgeführt, in denen die rechnerische Bruchlast der Scheiben ermittelt wurde. Die Lasten wurden dabei stufenweise aufgebracht und iterativ die zugehörigen Steifigkeiten der Fugenfedern und des Verbundes zwischen Stahl und Beton bestimmt. Die wesentlichsten Parameter für das Tragverhalten sind: die Fugenausbildung (Fugenrandprofilierung → Schubtragfähigkeit), die Lagerungsbedingungen der Scheibe (eine konzentrierte Lagerung am unteren Rand der Scheibe ergibt günstigere Schubübertragung in Auflagernähe und eine größere Druckzone in Scheibenmitte, eine über die Scheibenhöhe verteilte Lagerung führt zu kleinerer Druckzone und kann zu ungünstigeren Verhältnissen bezüglich der Schubübertragung in Auflagernähe führen). Schließlich ist die Querkraftübertragung in Scheiben, bei denen die Plattenspannungsrichtung senkrecht zur Stützrichtung der Scheibe verläuft, ungünstiger als bei Plattenspannungsrichtung parallel zur Stützrichtung der Scheibe. Rechenergebnisse sowie ein Bemessungsvorschlag sind in [6] und [10] angegeben. Die aus der Rißbildung entstehende Spannungsumlagerung ist exemplarisch in Bild 5 dargestellt.

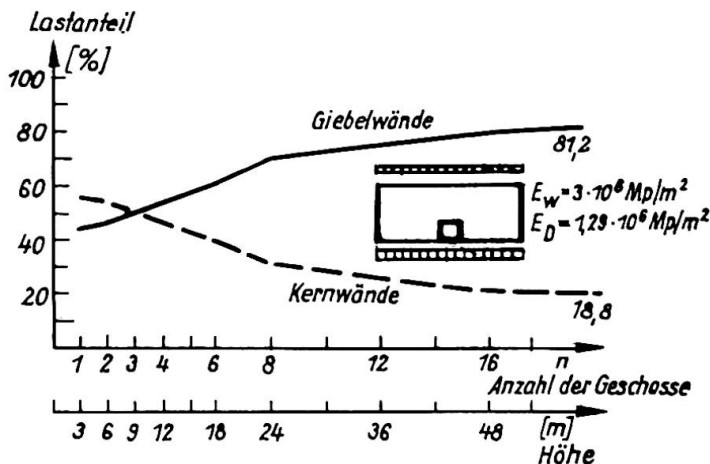


Bild 6: Verteilung der Horizontalkräfte bei Annahme von  $E_D \neq \infty$

Die Steifigkeit der Deckenscheiben aus Fertigteilen ist geringer als die von Ortbetonscheiben. Wird von der Annahme starrer Deckenscheiben bei der Verteilung der Horizontalkräfte auf die aussteifenden Wände abgegangen, was bei Fertigteilenscheiben oft zu berücksichtigen ist, so ergeben sich vor allem in den unteren Geschossen wesentliche Abweichungen gegenüber den Ergebnissen mit starren Scheiben.

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#### ZUSAMMENFASSUNG

Auf der Grundlage numerischer Untersuchungen mit einem Programm nach der Methode der finiten Elemente, in dem das Trag- und Verformungsverhalten von Fugen realistisch erfasst werden kann, wird der Einfluss mehrerer Parameter auf das Trag- und Verformungsverhalten aus einzelnen Fertigteilen zusammengesetzter Scheiben aufgezeigt. Zur Bestimmung des Fugenverhaltens werden empirische Formeln angegeben.

#### SUMMARY

The influence of several parameters on the loadbearing and deformation behaviour of precast floor and wall diaphragms is shown by a finite-element-analysis taking into account the realistic loadbearing and deformation behaviour of the joints between the different precast elements. Empirical formulas are given for the determination of the joint behaviour.

#### RESUME

L'influence de plusieurs paramètres sur la résistance et la déformation des dalles et murs de contreventement composés d'éléments préfabriqués est donnée par un calcul et un programme sur ordinateur, basé sur la méthode des éléments finis, dans lequel le comportement des joints peut être pris en considération. Des formules empiriques pour la détermination du comportement des joints sont données.

## Vla

### Assemblages en béton armé sous charges répétées diverses

Stahlbetonfugen unter verschiedenen mehrfach wiederholten Belastungen

Reinforced Concrete Connections under Different Repeated Loadings

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1. Les assemblages étudiés sont réalisés par bétonnage entre deux éléments plats en béton armé, des armatures en attente dépassant dans l'assemblage.

Les études expérimentales présentées ont porté d'une part sur le cisaillement longitudinal d'assemblages crantés et armés, du type utilisé en construction entre deux panneaux préfabriqués, et d'autre part sur le cisaillement et la flexion simultanés d'assemblages armés mais non crantés. Dans les deux cas, il a été procédé à des répétitions de charges.

2. Une étude systématique du comportement au cisaillement d'assemblages crantés et armés a été faite, de 1966 à 1972, par M. Michel POMMERET. Elle a permis de déterminer le comportement de ce type d'assemblage, de montrer la possibilité d'un fonctionnement plastique avant épuisement de sa capacité de résistance, de donner par une formule empirique la valeur de cette résistance ultime. (Annales de l'I.T.B.T.P. de Mars 1974).

Plusieurs essais de cisaillement longitudinal ont été faits ensuite (POMMERET, ASTRUC) sous charges alternées successives, dans les conditions suivantes :

Les assemblages de 18 par 18 cm de dimension transversale et 200 cm de long, étaient d'un type connu, utilisé pour la fonction verticale de panneaux préfabriqués, ayant déjà fait l'objet d'essais statiques (figure 1). Chacun comportait deux bordures, crantées, d'où dépassaient des aciers en attente, en forme de boucles. L'assemblage était coulé une dizaine de jours, après les bordures, avec un béton de résistance inférieure à celle du béton des bordures.

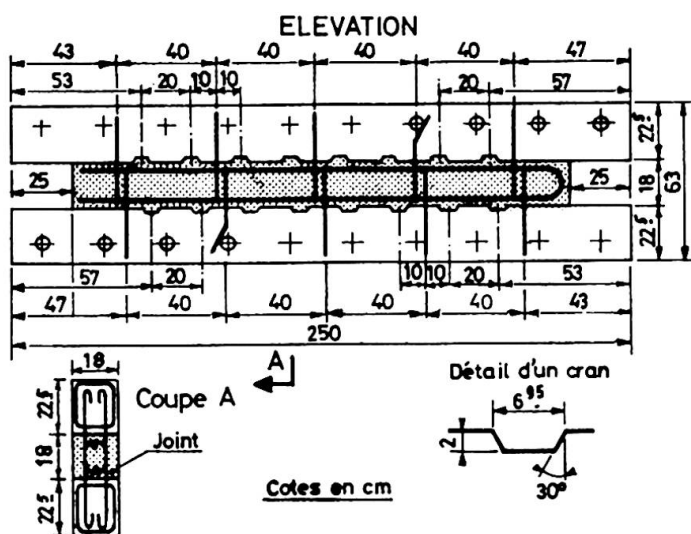


Fig. 1 - Modèle essayé



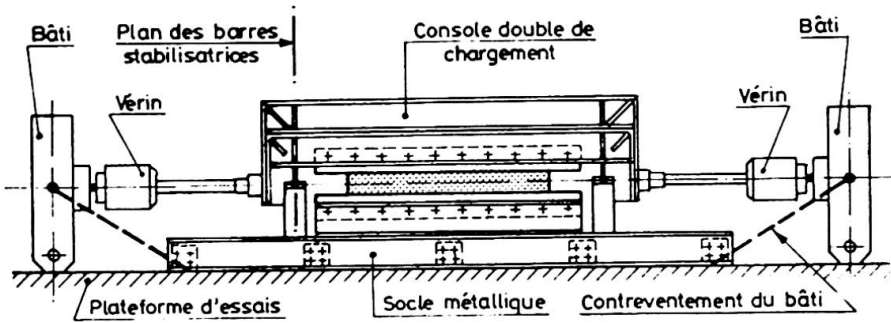


Fig. 2 - Montage d'essai

Chaque assemblage était soumis alternativement à des forces exercées suivant son axe dans une direction puis dans l'autre (figure 2). Un essai statique a été exécuté pour chaque série, à titre de référence sur un corps d'épreuve semblable et a donné une valeur de référence. Avant tout essai, chaque assemblage est fissuré aux plans de reprise par extension transversale par des vérins pour éviter un collage.

Les sollicitations alternées ont été exercées de deux manières différentes :

a - par déformation alternée répétée, avec une amplitude de  $T_u$  0,1 mm, assez rapidement, la courbe force-déformation a présenté une boucle d'hystérésis très dissymétrique (figure 3) (force appliquée variant de 0,65 à 0,70  $T_u$  dans un sens et pratiquement zéro de l'autre). Après 1000 cycles dans un cas et 20 000 dans l'autre, l'essai a ensuite été poussé à rupture et la résistance ultime obtenue est restée voisine de la résistance statique de référence  $T_u$ .

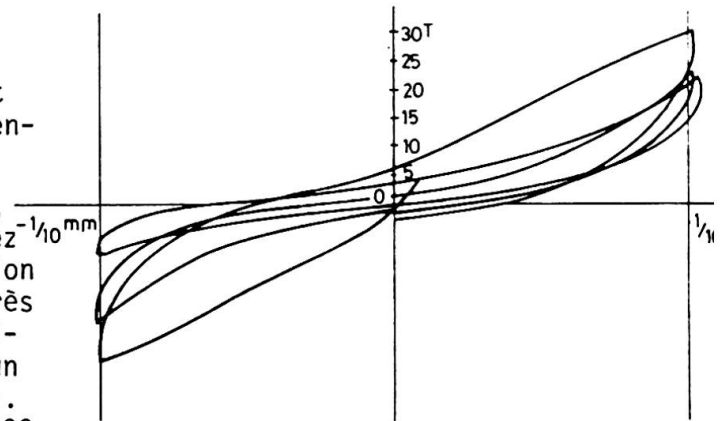


Fig. 3- Cycles d'hystérésis à variation de déformation imposée

b - par forces alternées, égales et opposées, en appliquant au corps d'épreuve une force  $K T_u$ , et en notant le nombre  $N$  de cycles d'alternances que la pièce pouvait supporter avant rupture. La boucle d'hystérésis se déplace progressivement jusqu'à rupture. Les résultats ont été les suivants : pour des corps d'épreuve âgés de 32 à 45 jours (résistance du béton de l'assemblage d'environ 16 N/mm<sup>2</sup>) essayés en 1972, on a :

$K=0,91$   $N=6$     $K=0,82$   $N=16$     $K=0,73$   $N=979$

Pour des corps d'épreuve âgés de 635 à 670 jours, essayés en 1974, (résistance du béton de l'assemblage env. 27 N/mm<sup>2</sup>), on a obtenu avec une rupture d'ailleurs brutale :

$K=0,77$   $N=9$     $K=0,69$   $N=37$     $K=0,62$   $N=7100$

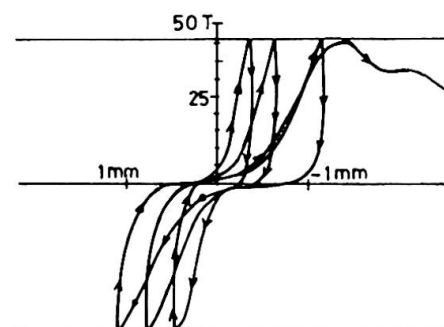


Fig. 4 - Cycles d'hystérésis à variation de charge imposée

L'augmentation de résistance du béton de  $16 \text{ N/mm}^2$  à  $27 \text{ N/mm}^2$  (due au vieillissement et probablement aussi à une différence de qualité) aurait dû, d'après la formule expérimentale de POMMERET, faire passer la résistance statique par n de l'assemblage de 14800 daN à 18500 daN; en réalité, la résistance statique est passée de 22000 daN à 27350 daN, c'est-à-dire dans la proportion prévue, mais la résistance aux charges alternées n'a pratiquement pas augmenté puisque l'on trouve, par interpolation, pour 1000 alternances, 16000 daN de charge limite en 1972 et 17000 daN en 1974. A titre de repère, notons que la résistance à rupture de l'assemblage, calculée d'après la règle des coutures qui suppose des bielles de béton inclinées à  $45^\circ$  est de 4150 daN/ml, donc bien inférieure.

3. Dans une autre recherche (SOUBRET), nous nous sommes proposés d'examiner le comportement d'une série d'assemblages sollicités en flexion et soumis à un cisaillement longitudinal.

Nos 6 corps d'épreuve ont les dimensions transversales réelles d'un assemblage exécuté sur un pont à deux tabliers à caissons construits successivement.

Chaque modèle est constitué de deux panneaux en béton armé préfabriqués, qui comportent des aciers en attente droits, et de l'assemblage en béton armé qui est coulé horizontalement. (figure 3).

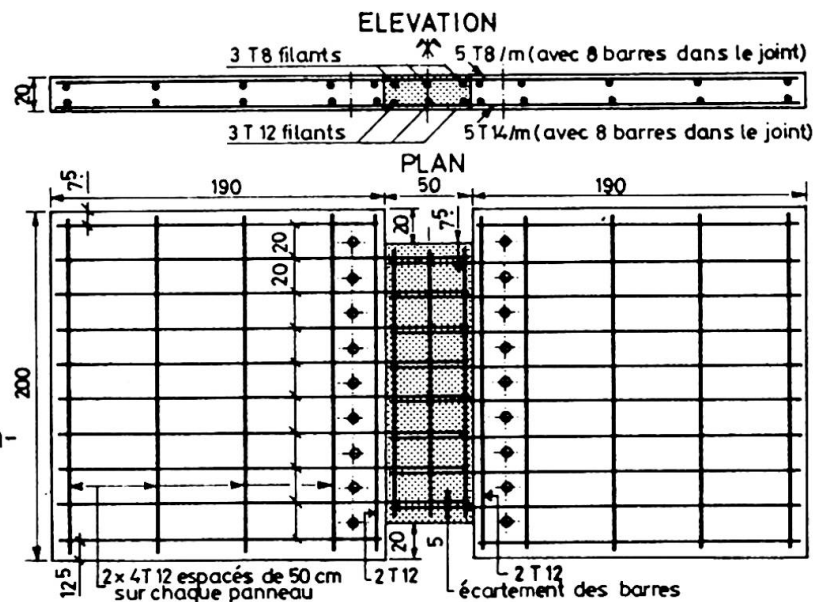


Fig. 5 - Corps d'épreuve soumis à flexion et cisaillement

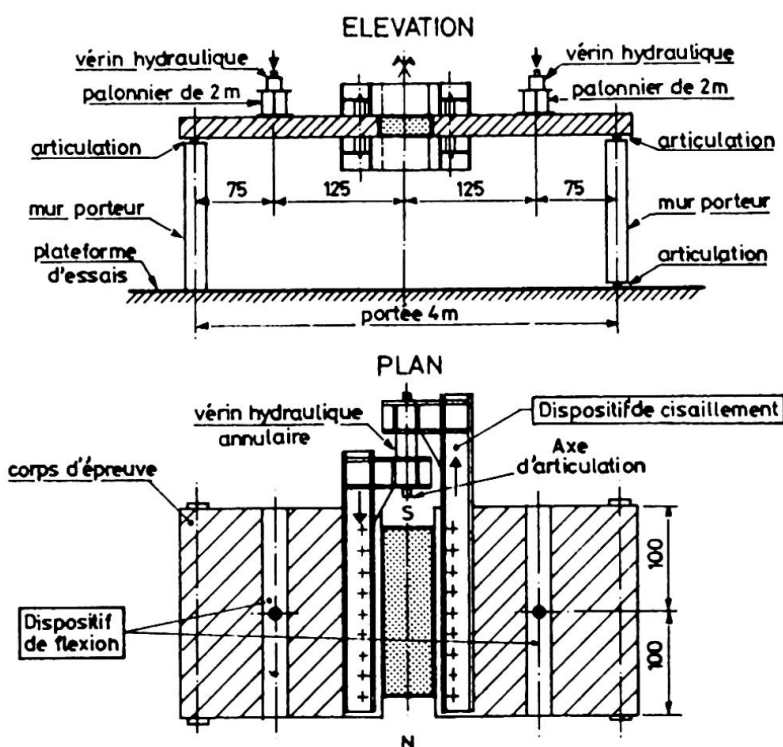


Fig. 6 - Montage d'essai

Les surfaces de reprise sont lisses et pour éviter tout "collage", un badigeon d'huile de décoffrage a été passé sur le bord de chaque panneau avant bétonnage.

Une série de tubes a été placée sur chaque panneau près du joint pour assurer par boulonnage la mise en place du dispositif de cisaillement, et le montage d'essai a été réalisé conformément à la figure 6.

Repère des corps d'épreuve	Sollicitation de l'assemblage		But recherché
	En flexion (vérin P)	Au cisaillement (vérin T)	
FV 1	par paliers statiques jusqu'à rupture	pas de chargement	Détermination expérimentale du moment de rupture $M_u$
FV 2	pas de chargement par vérins poids propre uniquement	par paliers statiques jusqu'à rupture	Détermination expérimentale de la force de cisaillement ultime $T_u$ et du cisaillement $\tau_u = \frac{T_u}{S}$
FV 3	application dynamique du moment $M_u$ (contrainte calculée des aciers 280 N/mm <sup>2</sup> )	applications successives des taux de cisaillement : 0,3 $\tau_u$ 0,6 $\tau_u$ 0,9 $\tau_u$	étude de l'influence du cisaillement sous charge de service
FV 4	application dynamique du moment $M_e$ (contrainte calculée des aciers 420 N/mm <sup>2</sup> )	applications successives des taux de cisaillement : 0,3 $\tau_u$ 0,6 $\tau_u$ 0,9 $\tau_u$	étude de l'influence du cisaillement lorsque la limite d'élasticité de calcul est atteinte en traction dans les aciers
FV 5	application dynamique du moment : 0,9 $M_u$	applications successives des taux de cisaillement : 0,3 $\tau_u$ 0,6 $\tau_u$ 0,9 $\tau_u$	étude de l'influence du cisaillement au voisinage de la rupture par flexion
FV 6	application constante du Moment $M_s$	par paliers dynamiques jusqu'à rupture	détermination d'une contrainte de cisaillement admissible sous charge de flexion en service

Lorsque les modalités de chargement comportent des phases "dynamiques" les charges hydrauliques correspondantes sont appliquées 10 000 fois pour P et 1000 fois pour T entre la valeur du palier considéré et environ le tiers de celle-ci.

En flexion les assemblages se sont comportés comme des poutres en béton armé monolithes et le calcul donne une bonne approximation des charges de rupture sur la largeur du joint essayé.

La contrainte de cisaillement limite sous charge de service en flexion trouvée sur FV 6 (voir le tableau) est de 1,13 N/mm<sup>2</sup>.

La combinaison du cisaillement longitudinal et de la flexion n'a révélé un effet d'interaction qu'au voisinage des états ultimes, c'est-à-dire bien au-delà des états d'utilisation.

L'influence d'une répétition de charges, en flexion comme au cisaillement, se manifeste seulement pour des sollicitations proches de sollicitations ultimes.

L'ensemble de ce travail expérimental montre qu'un assemblage en béton armé à partir d'aciers droits laissés en attente, peut assurer une liaison efficace entre deux poutres principales, sous des efforts tangents combinés à une flexion.

En conclusion, l'exécution d'essais sur des modèles reproduisant les dimensions réelles d'assemblages a permis de mieux comprendre le comportement des assemblages entre éléments en béton armé et de proposer des méthodes de calcul.

#### RESUME

Des études expérimentales ont été menées sur le comportement d'assemblages en béton soumis au cisaillement. Dans une première étude, des assemblages crantés et armés du type employé pour la liaison verticale de panneaux préfabriqués ont été essayés sous efforts tangents alternés, à deux âges différents; la résistance aux efforts alternés est importante, mais augmente moins vite avec l'âge que la résistance statique. Dans une deuxième étude, des assemblages à faces planes, armés par des barres droites en attente, ont été essayés en flexion et cisaillement longitudinal combiné; l'effet de la répétition des charges n'apparaît pas en service.

**ZUSAMMENFASSUNG**

Das Verhalten schubbeanspruchter Stahlbetonfugen wurde untersucht. Eine erste Untersuchung betrifft verzahnte und bewehrte Fugen, wie sie normalerweise bei der Verbindung von vorfabrizierten Wänden verwendet werden. Mehrfach wiederholte Schubbeanspruchung wurde auf zwei verschieden alte Prüfkörper aufgebracht. Der ermittelte Widerstand ist beträchtlich, steigt jedoch mit dem Alter weniger rasch an als der Widerstand gegenüber statischer Schubbeanspruchung. Der zweite Versuch wurde mit glatten Fugen und normalen geraden Anschlusseisen durchgeführt. Die Belastung bestand aus einer Kombination von Biegung und Längsschub. Der Einfluss einer wiederholten Belastung war im Gebrauchszustand nicht bemerkbar.

**SUMMARY**

Experimental studies have been made on the behaviour of reinforced concrete connections under shear. In a first study, reinforced and keyed connections of the type used for jointing vertically precast panels have been tested under alternating shear, at two different ages; the strength under alternating forces is important, but grows with age more slowly than static strength. In a second study, connections with plane interfaces, reinforced by straight bars have been tested under flexural and tangential longitudinal combined action; the effect of repetition of loading does not appear in service state.