Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 62 (1991)

Artikel: Lateral load behaviour of large panel precast buildings

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DOI: https://doi.org/10.5169/seals-47653

Lateral Load Behaviour of Large Panel Precast Buildings

Comportement sous charge latérale de bâtiments réalisés avec de grands panneaux préfabriqués

Verhalten von aus Stahlbetonscheiben vorgefertigter Hochbauten unter horizontaler Erdbebenbelastung

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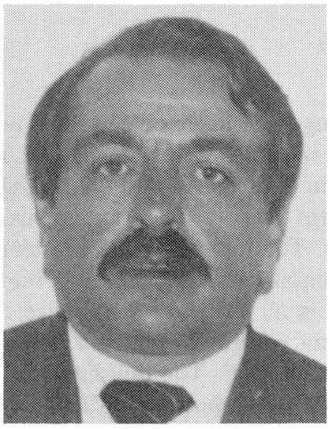

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SUMMARY

This paper describes the observed behaviour of a large panel concrete precast building under simulated seismic loads and its mathematical modeling. This mathematical model, calibrated by a series of static tests, was used to study the dynamic response of the structure for different types of earthquake motions.

RÉSUMÉ

Cet article décrit le comportement d'un édifice réalisé avec de grands panneaux préfabriqués soumis à un séisme simulé, ainsi qu'à sa modélisation mathématique; celle-ci étalonnée par une série d'essais statiques, a été exploitée pour étudier la réponse dynamique de la structure pour divers types de mouvements pouvant être provoqués par un tremblement de terre.

ZUSAMMENFASSUNG

Dieser Bericht beschreibt das Verhalten von grossen Stahlbetonscheiben vorgefertigten Hochbauten unter der Einwirkung von seitlichen Erdbebenlasten und das sich daraus ergebende mathematische Modell, das aus einem 1:1-Versuch hervorging. Dieses Modell wurde erweitert, um auch Antwortspektren verschiedener Erdbeben abzudecken.

1. INTRODUCTION

An insight into the behavior of large panel precast reinforced concrete buildings under lateral loading and its mathematical modeling has been obtained through a series of full scale lateral load tests on a five story large panel precast building. The static lateral loads simulated the distribution the inertial forces during an earthquake. Mathematical models were developed before the tests, which were subsequently calibrated using the actual test results. This models were extended to cover the behavior under seismic actions, which led to definition of behavior patterns under different types of earthquakes [1]. The building tested was very lightly reinforced and represented the type of precast construction used in the Republic of Colombia before the enactment of the Colombian Code [2]. In addition the building was not connected with tension reinforcement to the foundation.

2. EXPERIMENTAL PROGRAM

2.1 Precast Building Tested

The full scale precast building tested was a five-story low-rise apartment building with an area per floor of 157,64 m^2 as shown in Figure 1. The clear height of the precast panels was 2,20 m and the floor slabs had a thickness of 250 mm. The total height of the building was 12,25 m (Figure 2). The total mass of the building was 460 metric tons. All continuity reinforcement was located and welded in the gaps between precast elements. These gaps were filled later with grout. All the precast panels were 80 mm thick. The internal reinforcement of the panel did not extend from the borders of the panel. Each panel had horizontal bars that extended into the vertical connection. One 9,5-mm diameter vertical reinforcing bar was located in all vertical joints between panels. All 63 vertical joints between precast panels had vertical reinforcement; but only in 24 of them it was continuous from floor to floor. No vertical reinforcement was anchored to the foundation. The building was, therefore, very lightly reinforced having only 24 bars with a 9,5 mm diameter from top to bottom without any connection to the foundation. All floors, including the roof, were made up of plant precast steam cured elements. The edges the floor elements had connecting reinforcing bars that were welded to similar bars in the neighboring elements, achieving the continuity needed to obtain a floor diaphragm effect.

2.2 Test Setup and Procedure

Lateral loads were applied from the reaction frame to the precast building using high-strength steel cables attached to hydraulic rams. Two rams per floor were used. Applied loads were measured with twenty load cells, one at each ram. Lateral deflection measurements were obtained using extension bars that recorded the drift (relative lateral deflection) between floors. Each extension bar was provided with a 0.001-in mechanical dial gage. Horizontal and vertical base movements were obtained with additional mechanical dial gages. Ambient vibration records were made, before and after each series of lateral load applications.

2.3 Earthquake Simulator Friction Tests

A series of static and dynamic tests [3], with the dynamic tests including two different types of earthquakes, were performed as part of the research project on the earthquake simulator of the Católica University in Lima, Peru. The prototypes were designed to have normal contact stresses in the same range of those found in the horizontal joints of the precast building. The observed static friction coefficient ranged from 0,65 to 0,68. The dynamic friction coefficient observed in the harmonic tests varied from 0,71 to 0,73. In the seismic tests a value of 0,78 was obtained. Based on this, for the evaluation of the experimental tests performed on the precast building a value of 0,66 was used.

3. EXPERIMENTAL RESULTS

3.1 Base Shear

Seven lateral load tests were performed on the structure from July to September of 1989. These tests are named 0, A, B, C, D, E and F. Table 1 gives a summary of the direction of loading of each test, the maximum static base shear reached, V_{max}, in kN and the value of the seismic coefficient, C_s, defined as $C_s = V_{max}/W$, where W is the active lateral load mass (W = 437 metric tons). Test 0 corresponds to first cracking of the building in the E-W direction. Test B corresponds to the first time the lateral load strength of the building was reached in the E-W direction, obtaining a seismic coefficient of 37,9% of the mass of the building, value that was surprisingly high for a building so lightly reinforced and disconnected from the foundation. Test C correspond to the only test that was performed in the N-S direction and it did not reach the strength of the building although a base shear of 37,3% of W was measured. With Test E the lateral strength of the building was reached in the opposite direction (W-E) of Test B. A value of C_s of 41% of W was measured. Test F was performed to obtain the reduction in stiffness of the building after having reached twice the lateral load strength.

3.2 Measured Deformations

A large decrease of stiffness was observed for the two tests that reached the strength level in the E-W direction (Test B) and then in the W-E direction (Test E) as the roof deflection reached in Test B was 16,5 mm for a base shear of 37,9% of the mass of the building, compared with 90,6 mm for a base shear of 41% of the mass of the building. It is note worthy that only for Test E the story drift in two of the floors was greater than 1% of the story clear height. For Test E the graph of story shear vs. story drift is presented in Figure 3. From these graphs it is evident that the structure went well into the inelastic range.

3.3 Observed Cracking

During Tests 0 and A, no cracking due to the applied lateral loads was observed. During Test B the first cracks in the vertical joints between precast panels were observed. These cracks extended as the lateral load was increased. Then the first cracks in the top horizontal joint of the panels appeared in the north and south exterior walls of the western sector of the building. These horizontal cracks had extended to all the building before the strength level was reached. No internal cracks in the precast panels were observed, with the exception of panels with window openings. During Test C, the only one in the N-S direction, no cracking was observed, different form that caused from the previous tests. The same is true for Test D. For Test E there was a notorious increment of the width of the vertical cracks in the vertical joint of the precast panels. There were interior cracks in some of the panels. Notorious work in the horizontal joints with vertical continuous reinforcement caused by sliding of the joint was observed.

4. INTERPRETATION OF OBSERVED BEHAVIOR

The cracking sequence was: (a) Fine cracking was observed in the vertical joints between panels (C_s=25,6%). (b) The cracks extended and increased in width as more lateral load was applied. (c) The first cracks at the top horizontal joint of the panels appeared at the third floor of the West side of the precast building at a C_s of 32,1% of W. (d) The cracks at the top joint of the panel had appeared in all of them when the C_s reached 35,7% of W. The load was applied from East to West and the first horizontal cracks appeared in the West zone of the building, where the overturning moment introduces compression. These means that the cracks were not caused by flexural induced tension; they were caused by roll-over (overturning) of the precast panels. This interpretation was confirmed by the mathematical modeling.

5. ANALYTICAL MODEL

The analytical evaluation of a large panel structure has been performed traditionally assuming that the structure is monolithic. The internal stresses are then obtained from a linear elastic analysis and strength for the elements and their connections are provided for these stresses. A "box effect" is present due to the non planar connection between precast elements thus forming a three dimensional structure. Large differences between the theoretical behavior and the one encountered in tests due to the use of planar analytical models.

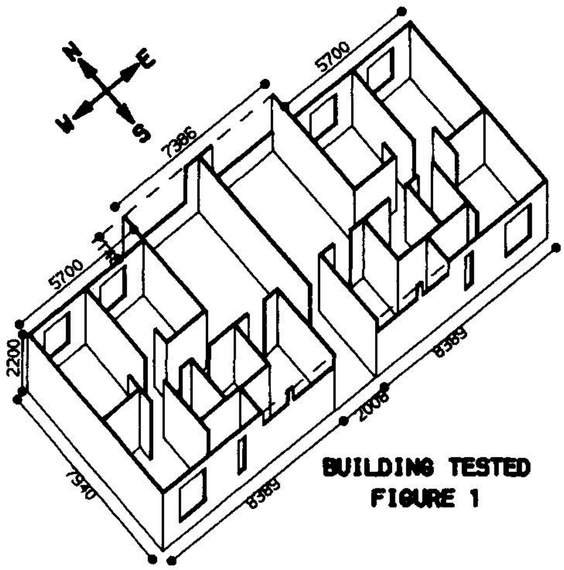
An analytical model appropriate for describing the behavior observed during the experimental tests of the building was developed before the tests were carried out and was subsequently calibrated using the experimental results. The model takes into account the non-linear response of the structure by modifying the stiffness of the different elements as they are stressed by the lateral load. A procedure of successive linear stages was adopted to describe the general non linear behavior. The analysis is initiated with a monolithic structure. With the results of this analysis the possible modes of failure of the elements and their connections are evaluated and the one that occurs at a lower lateral load level is taken as critical. The properties of the critical element or connection involved are then modified. The level of lateral load and deflection obtained are recorded and a new linear elastic analysis is performed where a new evaluation is performed, giving a new critical element or connection. This procedure is repeated until loss of equilibrium is detected. The modes of failure that are verified correspond to: (a) sliding of the horizontal connection, (b) overturning of the panel, (c) cracking and sliding of the vertical connection between panels, (d) internal shear failure of the panel, and (e) yielding of the vertical continuity reinforcement. The results obtained with this mathematical model correspond to the first lateral loading of the building. Figure 4 shows, for the E-W direction, the results obtained for the third floor as compared with the measured experimental results of the same floor for Tests 0 and B. The agreement of the results is significant and the same is true for all the other floors.

6. DYNAMICAL RESPONSE IN THE INELASTIC RANGE

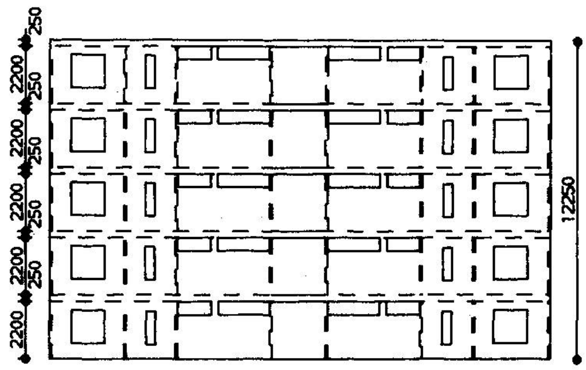
Using the program presented in [4] and linear idealizations of the relations between story shear and story drift obtained from the experimental lateral load tests of the precast building, the response of the building to base excitation representing strong ground motion corresponding to different soil conditions was obtained. A parametric study of the dynamic response was performed for cases that included: (a) variations of the hysteretic characteristics, (b) different levels of lateral stiffness of the structure, (c) earthquake records with different types of soil conditions, (d) peak ground acceleration of the earthquake records, (e) effects of damage accumulation caused by the structure being subjected to several successive earthquakes, and (f) an earthquake acting in a direction at an angle with the principal plan directions. Records obtained in El Centro (1940 Imperial Valley Earthquake), Castaic (1971 San Fernando Earthquake), Wilshire (1971 San Fernando Earthquake), Viña del Mar (1985 Chile Earthquake), Miyagi (1978 Japan), and Mexico City (1985 Mexico Earthquake) were used.

From the observed damage during the static tests of the precast building, bounds for the response were established. These bounds were a story drift of 4 mm for minimal damage and a story drift of 24 mm for the threshold of life threatening damage.

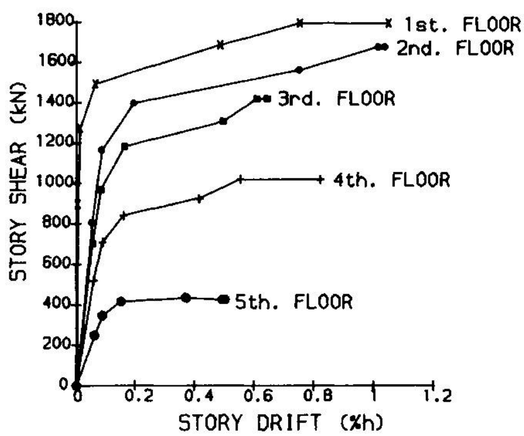
The effect of the peak ground acceleration is shown in Figure 5, where the maximum story drifts are plotted against peak ground acceleration of the earthquake records. Only those cases where the stiffness of the structure was reduced to one half and one quarter of the initial stiffness have story drift beyond the 24 mm limit. With the soft soil records (Mexico and Miyagi) the maximum story drift obtained was 1,2 mm, well into the minimal damage zone. For damage accumulation, three successive application of the full El Centro record, showed for the third application a maximum story drift of 24,3 mm.



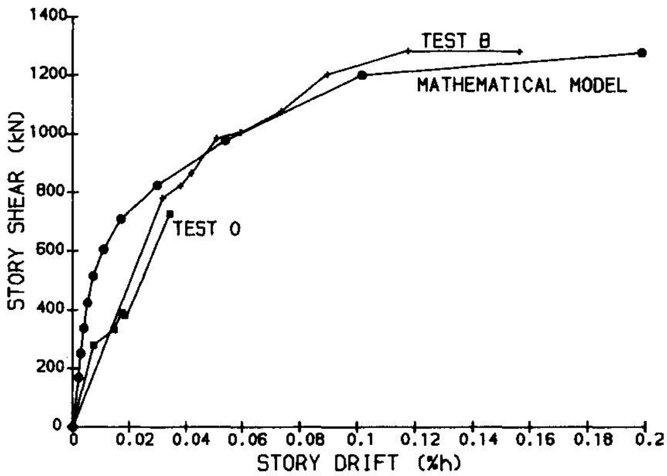
BUILDING TESTED
FIGURE 1



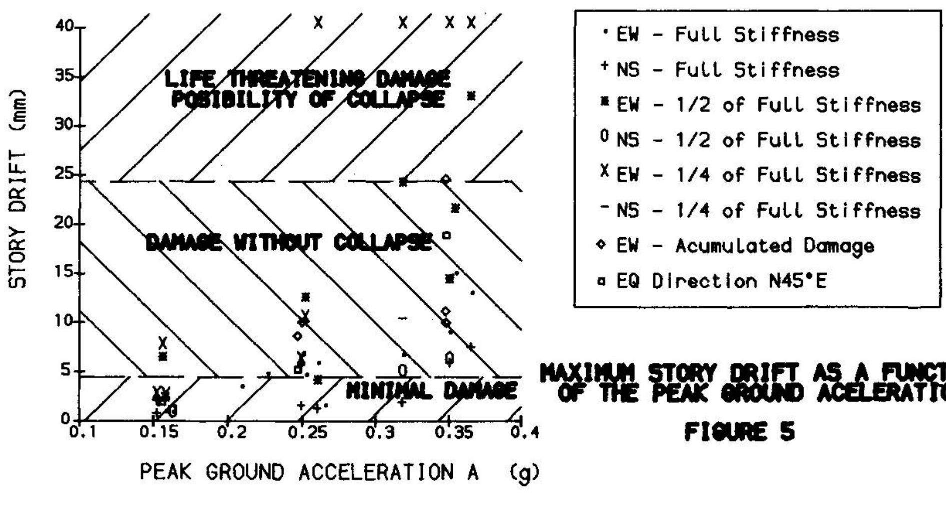
ELEVATION
FIGURE 2



MEASURED STORY DRIFT
TEST E
FIGURE 3



MATHEMATICAL MODEL RESULTS
THIRD FLOOR
FIGURE 4



MAXIMUM STORY DRIFT AS A FUNCTION OF THE PEAK GROUND ACELERATION
FIGURE 5

7. CONCLUSIONS

A full scale very lightly reinforced large panel precast building was subjected to seven lateral load tests obtaining base shear strengths of the order of 3/8 of the mass of the building and load deflection responses that exhibited a non fragile capacity. Even though the building was not connected to its foundation this did not affect the static lateral load strength of the building. It was possible to explain the failure mechanisms of the building and to establish analytical bounds of response under lateral loads. The static and dynamic friction tests of concrete surfaces cast at different moments gave an insight of the friction phenomena and permitted the development of an analytical model for the response of this type of structures to base excitations representing strong ground motion. Using the dynamic analytical model it was possible to confirm that ground motions rich in short vibration periods tended to affect the building more than motions rich in long periods as encountered in earthquake records from soft soil sites.

The observed behavior in the tests and the results obtained from the analytical models could serve as a basis for future performance requirements [5] and guidelines for analysis methodologies [6] for seismic loading for large-panel precast buildings.

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TABLE 1
SUMMARY OF THE STATIC EXPERIMENTAL LATERAL LOAD TEST

TEST	-O-	-A-	-B-	-C-	-D-	-E-	-F-
Direct.	E-W	E-W	E-W	N-S	W-E	W-E	W-E
C_s	0,209	0,151	0,379	0,373	0,210	0,410	0,284
V_{max}*	913	660	1655	1630	918	1793	1241

* V_{max} in kN