# PRELIMINARY CORRELATIONS FOR SHAKING TABLE TESTS OF PRECAST PANEL WALLS

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## SUMMARY

The seismic behavior of large precast panel structures has been under extensive investigation for the past few years, both in Europe and in the U.S. The most distinctive feature of such structures is the behavior of the joints between the panels.

This paper reports on the first phase of an ongoing research to verify the capability of existing models to simulate the true behavior of such structures based on the data obtained from the first shaking table tests of such structures. The validity of the computer program used in the correlation studies has been verified analytically. The first attempt to model the joint is presented and compared to quasi-static test results. Problems dealing with the realistic modeling of the test model are discussed.

## INTRODUCTION

The complex behavior of joints between precast wall panels and their potential role in the seismic response of large precast panel structures are the two main features that should be fully understood to develop a sound design procedure of these joints. These joints have been identified [1,2] to be the main source of nonlinear inelastic behavior in the structure. Analytical procedures were developed to simulate the expected response of such structures. [1,2] The development of these techniques was based on various mechanisms that are expected to be active in the joint modes of resistance. The isolated behavior of these mechanisms (coloumb friction, shear keys, shear friction, compression-tension) were modelled analytically and used simultaneously to simulate the joint behavior. The primary effort of these investigations focused on the behavior of the horizontal joints. Because the horizontal joint plays an important role in the stability and integrity of precast panel bearing walls. Still, the understanding of joint behavior and the verification of the existing analyitical models capability depends heavily on the experimental results of testing precast panel wall

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assemblages. Quasi-static tests on 3-storey 1/3 scale-models were performed at the University of Ziiz in Yugoslavia and shaking table test were conducted at EERC in Berkeley. The test models mid-height storeys in a high rise structure where shearing forces and overturning moment are expected to be critical. The joints between the panels are of the wet-reinforced type.

## THE COMPUTER PROGRAM

The computer program used in the correlation studies is a modified version of general purpose program developed at Berkeley [3] for the nonlinear dynamic analysis of 2-D structures.

The advantage of Drain-2D is that any mathematical model representing a certain behavior of a structural element can be incorporated within the program. The program has been used extensively in research dealing with dynamic behavior of various concrete structures. And most recently, it was used by Schricker [1] in his analytical research on the seismic behavior of simple precast panel walls.

The analytical correlation conducted in this study is using a modified version of the computer program used by Schricker [1]. This version has the following new additions which will allow more accuracy in the modelling process of the test model:

a) Substructuring of the wall panels similar to the techniques used by Llorente's [2]

b) Variable time stepping, which will improve the accuracy when sudden stiffness change is encountered

c) Event-to-event energy calculation, which might prove to be a significant issue with correlation studies

### The verification of the computer program

Before using the program in the analytical studies, it was realized that the program should be validated and verified because of three reasons. First, the program has been modified and not yet formally tested. Second, the program had undergone certain adjustment to run on a different machine (Harris 800). Third, since this study is basically concerned with verification of existing analytical algorithms, it is very significant to justify the incremental equilibrium equation which is used in the dynamic analysis.

The elastic stiffness of the panel elements was checked by comparing deflections and stresses from the computer analysis of a elastic cantilever shear wall to the analysis of the same wall using beam-theory with shear deformation included. The computer model was divided into three panels of equal height. And the interior nodes within each panel were condensed out, using the substructuring algorithm. Table (1) shows the results of the two methods. The agreement is very favorable.

All the hysteretic models in the computer program are piecewise linear as shown in Figs. (1a-d).

Each of these mathematical models has been verified by the analysis of S.D.O.F. mass-spring systems subjected to a harmonic sinusoidal loading. The analysis was performed by the computer program and then an analytical solution was obtained.

To demonstrate the analytical procedure developed, consider a S.D.O.F. mass-spring system. The spring has the stiffness characteristic for a simple friction element as shown in Fig. (1a). For the sake of simplicity, the system was assumed to have zero damping. This does not effect the validity of the comparison, but has the mere advantage of simplifying the analytical solution. The analysis is broken into phases. Each phase extends over a region in which the spring stiffness is constant. This means that a linear elastic solution of the dynamic equilibrium differential equation can be obtained for each phase. The initial conditions at the beginning of each phase were used to evaluate the constants. Table (2) shows the response of the system using the Drain program and using an analytical solution.

## PROBLEMS IN USING THE ANALYTICAL MODELS

The most significant problem in analyzing precast panel walls by the Drain program, lies in the extreme difficulty of obtaining numerical values for the existing analytical models. The development of the analytical models represents the first part of the process to produce a useful complete model. The second part deals with obtaining stiffness values for the models. The capability and effectiveness of these models are best realized once the second part is completed.

Figs. (1a-d) show some of the elements developed by Schricker [1] to model the expected joint characteristics. These elements, which are represented by discretized springs, can be placed anywhere along the joint. The hysteretic shapes of these elements were defined on the basis of various experimental results that dealt with each mechanism separately. The philosophy was that if such a mechanism is active in the joint modes of resistance, the contribution of this mechanism is represented by the shown analytical model. The major question at this point is whether these mechanisms when acting together would behave in the same manner as they would when acting separately and whether the sequence of activating these mechanisms can be properly simulated with the proposed models. So, it is clear that the user of these analytical models is obligated to make many assumptions, some of which might be incorrect and might lead to deviation of the computer analysis from the true behavior of the joint. However, regardless of how ambigious many of the characteristics are, assumptions should be made to set forth a justified expected behavior of the joint. These assumptions should then be modified until a correlation with test results is achieved. However, the problem remains whether these modified values can be justified physically.

The other part of the problem deals with fitting these analytical models to different types of joints. Fig. (2) shows the details of the horizontal joint used in the test models. This joint has two large keys at the corners. It is reinforced only at the two edges.

The development of the shear resistance of this type of joints requires three basic elements; key elements, simple friction elements and shear friction elements. The shear friction mechanism is associated with distributed reinforcement. Therefore, the shear friction as a separate element will be discarded and the contribution of the concentrated rebars will be accounted for in the simple friction element. The second problem to face is associated with estimating the yield level in the simple friction mechanism. This yield level is governed by two factors; the coefficient of friction at the interface, and the normal force (variable). Also the strain hardening in the simple friction, which is associated with the reinforcement is difficult to estimate. The analytical key model was developed on the basis that the keys are small and distributed along the interface. The problem here is whether the model can be used to represent the behavior of large keys that are experiencing, in addition to the shearing force, a variable compressive force that can approach critical limits in terms of concrete strength. Assuming that all the above mentioned problems were solved, the last issue associated with shear mechanisms deals with figuring out how to control the participation of the various models at different load levels. From the physical point of view these large keys are not expected to be active until slippage occurs. Also, there are different possible modes of failure for these keys. Each would define a different rate of softening.

The second mode to model is the flexural mechanism. Two basic models are needed to develop this resistance; a gap model to produce the compression resistance of concrete and steel and a truss model to produce the tension resistance of the rebars. Two alternatives to develop stiffness values can be used. The first alternative is based on the specific material properties and the specific joint geometry. Dividing the cross section of the joint into segments allows the development of stiffness values for each segment associated with each gap element. The problem in this method is that in the actual situation, the compatability of joint deformation is imposed by the gradual opening and closing along the joint length which in turn is defined by the deformed shape of the panel edge. Therefore, the response of the elements that are developed by the first method are expected to deviate from the actual deformation pattern. These elements are not constrained so as to produce a specific moment resistance for a specific straining pattern across the joint. The second alternative to obtain stiffness values for the flexural mechanism models is based on obtaining a moment curvature relation for the joint cross section. By using a computer program, different material

material properties for different parts of the cross section can be defined. This method assumes plane section remain plane, which violates the expected gradual opening and closing at the joint-panel interface. The advantage of this method is that compatability and equilibrium when obtaining stiffness values for the models can be accounted for.

## PRELIMINARY MODELLING OF TEST MODEL

The first attempt to model the test simple wall (Fig. (3)) was based on the observed behavior of the 3-story wall during the quasi-static testing. It was observed that slip along the bottom joint had a minor contribution to the top wall deflection. The magnitude of the gap opening showed that the rotation in the joint region had a pronounced effect on the magnitude of the top wall deflection. Therefore, it was decided to place a very stiff key element in the joint region of the computer model. This key with high stiffness will inhibit any slipping in the joint region.

In modelling the flexural mechanism in the joint region. The test wall was thought of being made of two elements in series as shown in Fig. (4a). One element is linear elastic representing the wall panels and the other one is the nonlinear inelastic element representing the bottom joint. The other two joints are assumed to behave in a linear elastic manner, because the moments experienced by these joints are far below the ultimate that will be imposed on the bottom joint. Figs. (5b-d) show that the total deformation at the top of the wall is due to flexure and shear deformation in the wall panels and due to considerable rotation in the bottom joint region. Similar to quasi-static testing, a horizontal force P is applied at the top of the wall.

The elastic element deformation is obtained by the analysis of a cantilever shear wall assuming uncracked section. The relation is:

$$\Delta_{S+F} = \left(\frac{1}{GA/1.2L} + \frac{1}{3EI/L^2}\right)P$$

This relation considers flexural and shear deformation. The joint rotation will be obtained by first getting a moment-curvature relation for the joint cross section. This relation transformed to a moment rotation relation which in turn is transformed to a force-deformation relation. An overall P-$\Delta$ relation will be obtained by adding the two deformation at each level of P.

This P-$\Delta$ relation is compared to the strength envelope of the P-$\Delta$ hysteretic strength envelope obtained the quasi-static testing, Figure (5). It can be seen here that the correlation between the two is very favorable. So it can be concluded at this point that the moment curvature analysis of the joint cross section can be used to obtain information to develop stiffness values for compression and tension elements in the joint region.

The moment curvature analysis was performed by the RCCOLA computer program [4]. The analysis allows dividing the cross-section into areas with different properties. In the analysis for the simple wall only small portion of the concrete was defined to be confined.

In the simple wall analysis, the elastic stiffness and the yield level matched favorably between the two P-$\Delta$'s. So, it was decided to use RCCOLA results to obtain numerical values for analytical models simulating the rocking mechanism. Using location of the neutral axis, the location of the compression force and the strain at that location, the force deformation relation for the gap element was obtained at different curvature levels which represents the compression-resistance of the cross section.

The reinforcement bars were modeled using the truss element. The stres-strain relation for the rebars was used to obtain numerical values for the model.

Figure (6) shows the computer model that was analyzed by the Drain program. The panels were substructured, leaving only the nodes at the joint levels to be active. The masses at the top nodes represent the central mass that was used in the shaking table tests. The earthquake record used in the dynamic analysis is the Petrovac (1/3 time scale).

Fig. (7) shows P-$\Delta$ hysteretic results from the analysis. The strength envelope of the quasi-static test correlates favorably with the analysis. But the shapes of the loops show that a more complex modelling process is needed before a correlation between the loops shape is obtained.

References

(1) Schricker, V., Powell, G., "Inelastic Seismic Analysis of Large Panel Building," Report No. UCB/EERC-80/38, Sept. 1980, Berkeley, California.

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(3) Kanaan, A.E., Powell, G.H., "General Purpose Computer Program for Inelastic Dynamic Response of Plane Structures," Report No. EERC 73-6, Berkley, Calfironia, April 1973.

(4) Mahin, S., Bertero, V., "RCCOLA A Comptuer Program for Reinforced Concrete Column Analysis", Dept. of Civil Engng., University of California, Berkeley, 1977.

Table 1

| [illegible] | | [illegible] | |
|---|---|---|---|
| computer | analytical | computer | analytical |
| 0.01345 | 0.01336 | 0.987 | 1.040 |

Table 2

| TIME (sec.) | DISPLACEMENT(in.) | |
|---|---|---|
| | COMPUTER | ANALYTICAL |
| 0.1 | 0.092 | 0.092 |
| 0.2 | 0.632 | 0.632 |
| 0.3 | 1.636 | 1.633 |
| 0.4 | 2.783 | 2.785 |
| 0.5 | 3.669 | 3.671 |
| 0.6 | 3.719 | 3.718 |
| 0.7 | 2.500 | 2.496 |

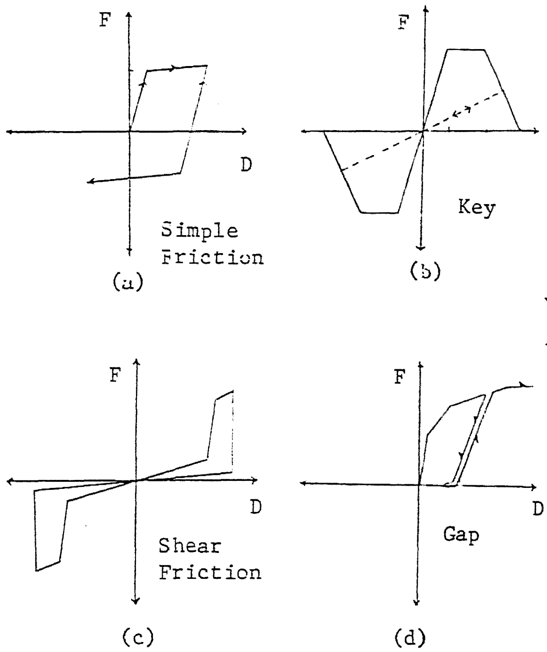


Figure 1: Analytical Models

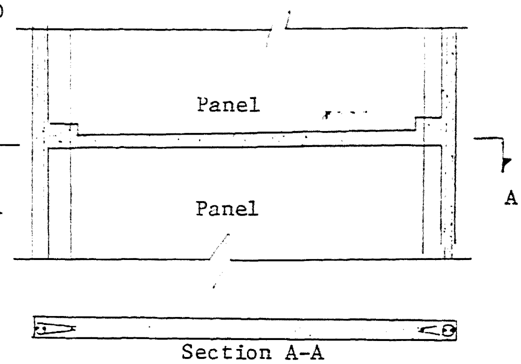


Figure 2: Joint Detail

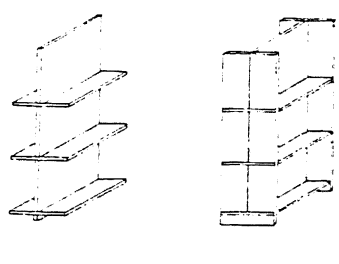


Figure 3: Test Model

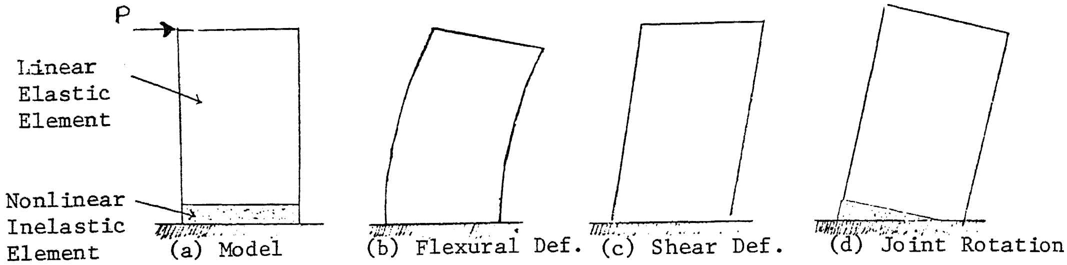


Figure 4

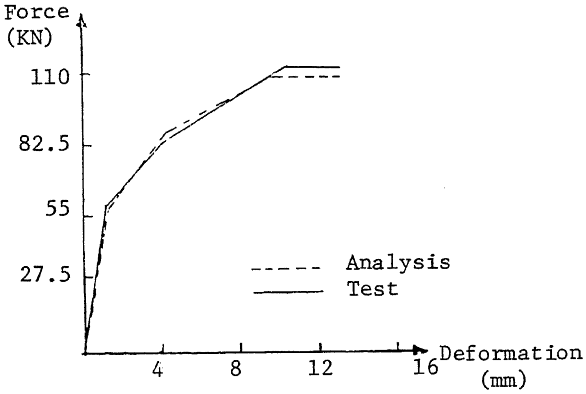


Figure 5: P-Δ relation

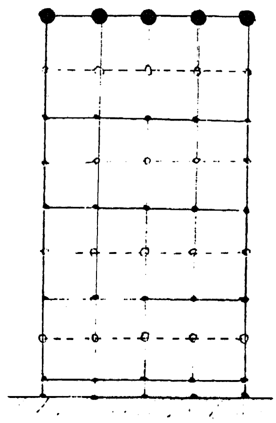

Figure 6: Computer Model

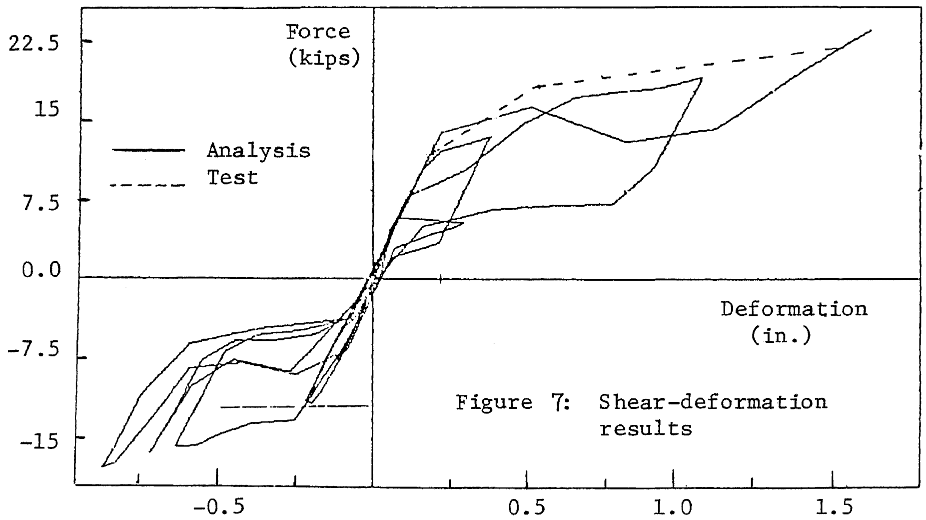


Figure 7: Shear-deformation results