

EARTHQUAKE RESISTANCE OF THE "SCT" LARGE PANEL BUILDING SYSTEM

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SUMMARY

An analytical study of a large panel system to be used in high seismicity areas is presented in the paper. The analytical investigations are supported by experimental results. Special attention is given to inelastic response history analysis. New mathematical models of the hysteretic behaviour of joints, taking into account shear slip and gap opening, were developed and incorporated into the DRAIN-2D-2 program. Due to the system's suitable structural concept, a nearly monolithic response of SCT large panel buildings to strong earthquakes has been observed, which makes possible the construction of such buildings up to 10 stories high in zones with intensity IX.

INTRODUCTION

"The good performance of well designed shear walls has been demonstrated in several recent earthquakes" (6,4). The large strength and stiffness of shear wall buildings reduce the structural and especially the nonstructural damage significantly which leads to better behaviour in comparison with pure frame structures. An appropriate design can provide suitable ductility, too. "Combining the earthquake resistant characteristics of cast-in-situ structural walls with the speedy, economic manufacturing and erection process of precast elements is of course appealing" (4). However, to provide suitable ductile behaviour of large panel buildings, the appropriate structural solutions and details are needed. A proper balance between simplicity of structural details and seismic safety is decisive for the efficiency of a large panel structural system, and places the limits on its use in regions of high seismicity.

The meaning of "proper balance", "suitable solution", and "successful exploitation" depends a great deal on the local conditions. Systems with in-situ castellated joints and welded or looped reinforcement have proved to be the most successful in the Balkan region and Eastern Europe. The analysed system, developed by the SCT company, of Ljubljana, Yugoslavia, is of such a type, too. The region covered by the company lies mostly in zones of relatively high seismicity (zones VIII. and IX.) and moderately high buildings (up to 10 stories) are usually constructed there. To verify the seismic safety of the proposed structural system and to make possible improvements, an extensive research project has been started. The research efforts of SCT (technology and funding), Institute for Testing and Research in Materials and Structures - ZRMK (experimental research) and University E.K. in Ljubljana - Institute of Structural and Earthquake Engineering

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(analytical research) have been united. The results of the analytical work are reported in the paper.

BASIC PRINCIPLES OF THE ASEISMIC DESIGN OF LARGE PANEL BUILDINGS

The seismic forces in codes are significantly reduced, depending on the considerable inelastic deformation and energy dissipation capacity of buildings. The reduction factors are based on empirical observations of the behaviour of the common (usually cast-in-place) structural systems during recent earthquakes. However, if a system behaves specifically, the usual design procedures cannot be adopted without special considerations. This certainly holds true for large panel structures, whose behaviour is quite specific in the non-linear range.

It has to be kept in mind that "the design objective is not simply to provide lateral strengths for given lateral loads, but to provide a level of lateral strength, which ensures both that the inelastic deformations remain within the inelastic deformation capacity and that the structure does not become unstable under the combined effects of gravity loads, deflections and degraded stiffness". Any suitable combination of strength and ductility is possible, but a thorough knowledge of nonlinear behaviour under intense seismic loading is needed to define an economic and safe combination. Taking into account the typical ways of nonlinear behaviour of large panel buildings under seismic loadings, shown in Fig. 1 (10), the following design procedures can be used (1,3,4,10).

Elastic Design. The joints and panels of the structure are provided with sufficient strength (5 to 8 times larger than that prescribed by codes for ductile structures) to remain linear elastic during major earthquake. It seems reasonable to use this approach for low-rise buildings and/or for areas of modest seismicity, particularly for short period structures, in which inelastic energy dissipation is of little

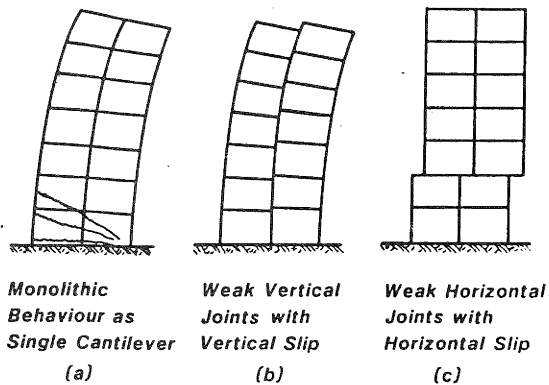


FIG. 1 - INELASTIC BEHAVIOUR OF LARGE PANEL BUILDINGS (10)

benefit (3). Any suitable combination of lower ductility (simpler details) and higher strength is possible.

In the case of high-rise buildings in zones of moderate to high seismicity, one has to depend on the dissipation of energy by inelastic deformations.

In the case of equivalent monolithic design (Fig. 1a), the strength of the joints is sufficient to ensure that the behaviour of large panel structures is similar to that expected for cast-in-place buildings. The usual design procedures for ductile RC buildings can be used in this case.

It seems to be reasonable to take into account the specific possibilities of energy dissipation in large panel structures, particularly in their joints. Weak horizontal joint designs (Fig. 1c) provide dangerous solutions (similar to those based on the "soft story" concept) leading to unconstrained failure mechanisms which depends entirely on the characteristics of the earthquake loading. Weak vertical joint designs (Fig. 1b) offer more favourable solutions. It is a special case of capacity design, mostly advocated by authors in New Zealand, which has gradually spread to all modern aseismic codes. In this method member strength is provided such as to enforce the chosen sequence of plastification. "This alternative aseismic design philosophy appears particularly suited for structures consisting of both potentially ductile and brittle elements. Large panel constructions may rank among them" (4). Vertical joints, which are not primary gravity load bearing elements, are suitable to be chosen as primary energy-dissipating elements. Unfortunately, their energy dissipating capability is usually low. On the contrary, the coupling beams, which are a special case of vertical joints, can be designed as excellent dissipating elements (5,7).

Which method is to be used depends on the chosen structural system and its details, the expected inelastic behaviour, the chosen height of the buildings and the

seismicity of the region.

CHARACTERISTICS OF THE SCT LARGE PANEL SYSTEM

It was the goal of the research project to develop a large panel system which could be used for moderately high buildings in zones of relatively high seismicity.

Since no sophisticated analysis can save a badly conceived structure, careful architectural design is needed first. The typical lay-out of SCT large panel buildings is given in Fig. 2. Although the spans are relatively large (6m), the ratio of the total cross section of the seismic resistant shear walls to the total area of the lay-out is very favourable (3.56% in the longitudinal X-direction and 4.44% in the transverse Y-direction). The maximum foreseen span is 7.2m. The exterior walls form part of the seismic resistant system which increases in strength and torsional stability of the building. Coupling beams (diagonally reinforced) are used wherever possible. The long wall Y11 should be cut to avoid unfavourable torsional effects.

The axiometry of the structural system is given in Fig. 3. In-situ castellated joints are used. Typical vertical and horizontal joints are shown in Figs. 4 and 5, respectively. In both, the vertical and the horizontal joints, distributed reinforcement loops are provided. Additional continuous vertical reinforcement is concentrated in the vertical joints and in special channels at the ends of the panels (Fig. 6). It has not yet been decided how to achieve the continuity of this concentrated reinforcement. Mechanical connectors will probably be used. Horizontal ties are provided along the horizontal edges of panels, and the slabs' reinforcement is anchored in them.

It is expected that sound structural integrity will be achieved by the proposed structural concept and details.

A SHORT REVIEW OF THE EXPERIMENTAL WORK

The experimental study of the joints and structural assemblages of the SCT large panel system was performed at the Institute for Research and Testing in Materials and Structures, Ljubljana (9). The mechanism behaviour of both vertical and the horizontal joints was first identified and the parameters for mathematical modelling were given. Then test on three-story structural assemblages, built to a scale of 1:3, were performed. They indicated "that the joints' capacity will enable the 10 to 12 storeyed buildings to behave monolithically, although subjected to the strongest expected earthquakes" (9).

Tests on the precast diaphragms were performed separately at University E.K. in Ljubljana. It was proved that the slabs act like rigid diaphragms in the range of the expected loads. The decisive influence of the horizontal ties was pointed out.

The experimental results will be

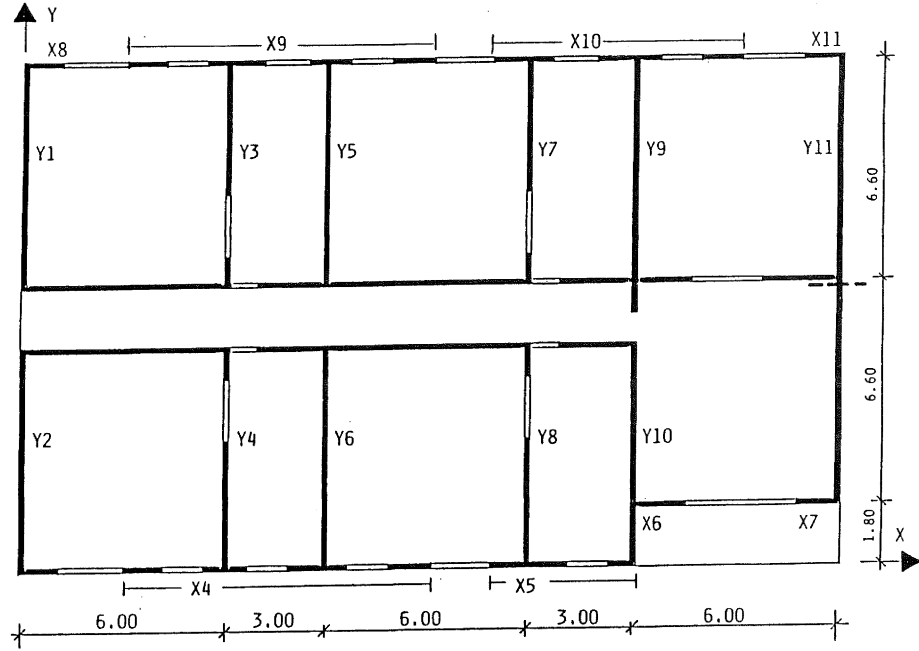


FIG. 2 - TYPICAL LAY-OUT OF SCT LARGE PANEL BUILDINGS

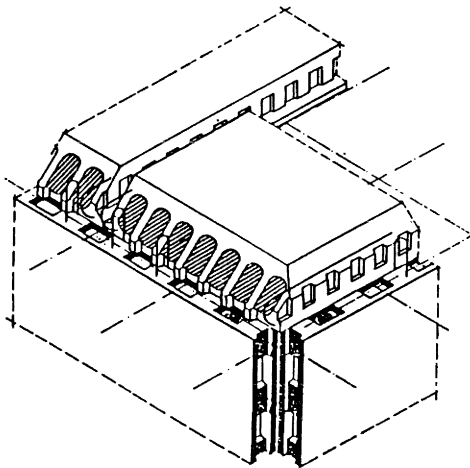


FIG. 3 - AXIOMETRY OF THE SCT STRUCTURAL SYSTEM

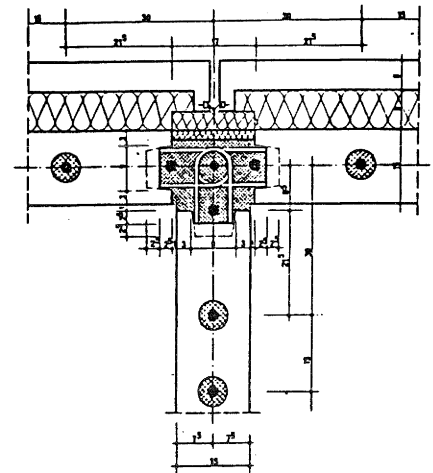


FIG. 4 - TYPICAL VERTICAL JOINT

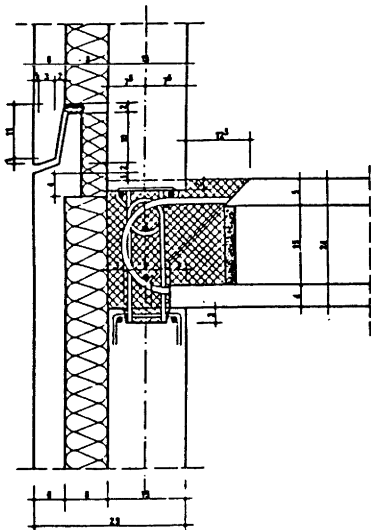


FIG. 5 - TYPICAL HORIZONTAL JOINT

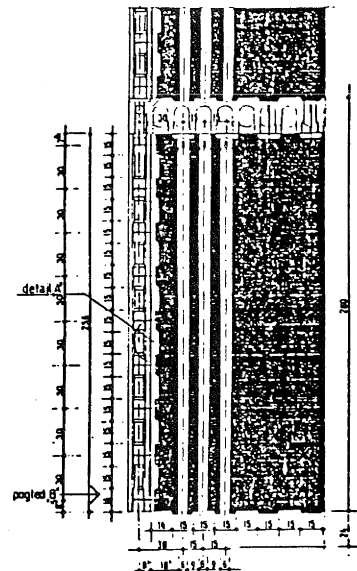


FIG. 6 - WALL PANEL AND CONNECTIONS

discussed in more detail within the chapter on the mathematical modelling of inelastic response.

LINEAR ELASTIC ANALYSIS

Elastic code spectrum analysis, as well as the elastic response history analysis of the typical 10-story building, shown in Fig. 2, was performed. Monolithic behaviour with rigid joints was assumed. Based on the experimental results, it was also assumed that the slabs act like rigid diaphragms in their own plane.

Free Vibration Analysis

The building proved to be quite rigid. The fundamental periods were about $N/30$ in both directions, where N denotes the number of stories ($T_{1x} = 0.31$ s and $T = 0.36$ s).

Elastic Code Response Spectrum Analysis

According to the assumption of monolithic behaviour, the elastic seismic forces for ductile RC structures defined by the Yugoslav aseismic code were used $S = K G$, with a base shear coefficient of $K = 0.1$. This assumption was later justified by the results of the inelastic analysis and experiments. Due to favourable lay-out and structural concept, in most cases minimum flexural reinforcement satisfied the requirements.

Elastic Response to Actual Earthquakes

Earthquake loading is without doubt the most uncertain parameter in aseismic analysis, particularly when a structural system for a variety of locations and buildings is being developed. Therefore,

the elastic response to a broad spectrum of 36 different accelerograms, which have been recorded in the U.S.A., Italy, and Yugoslavia was studied (for details refer to the paper "Parametric Study of Inelastic Response Spectra" prepared by the first two authors, (11)). The "Petrovac N-S" record (Fig. 7), obtained during the 1979 Montenegro earthquake proved to be critical for typical SCT large panel buildings with relatively short fundamental periods of between 0.1 and 0.5 s. Assuming 5% of critical damping, it can be concluded from the acceleration response spectrum (Fig. 8) that the elastic forces are nearly 10 times greater than the code specified seismic forces, for the typical analysed buildings in zone IX. It is obvious that inelastic behaviour of moderate to high rise SCT buildings is expected. However, since the bending moments exceed the actually supplied strength far more than the shear forces do, favourable flexural behaviour is expected.

INELASTIC ANALYSIS

Computer Programs and Numerical Procedures

The current available computer software for the inelastic analysis of large panel buildings is very limited, and it has been usually developed for specified structural systems and research projects. The most promising program for general large panel systems seems to be DRAIN-2D-2 (8), which has been developed at the University of California, Berkeley. Unfortunately, it was not officially released at the time when the project started. An earlier version of the program was obtained from the University of California. Some errors in the program were corrected, and some

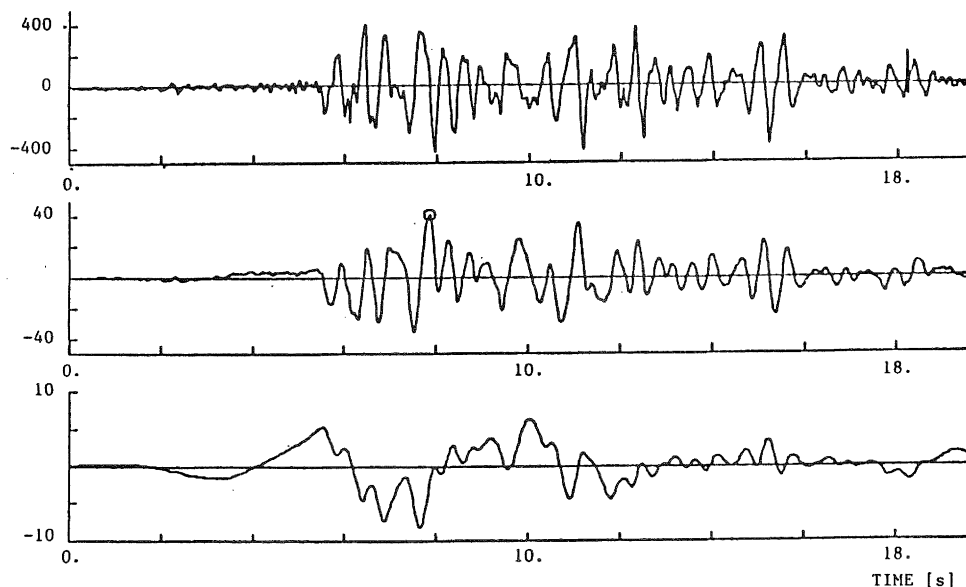


FIG. 7 - "PETROVAC N-S" RECORD

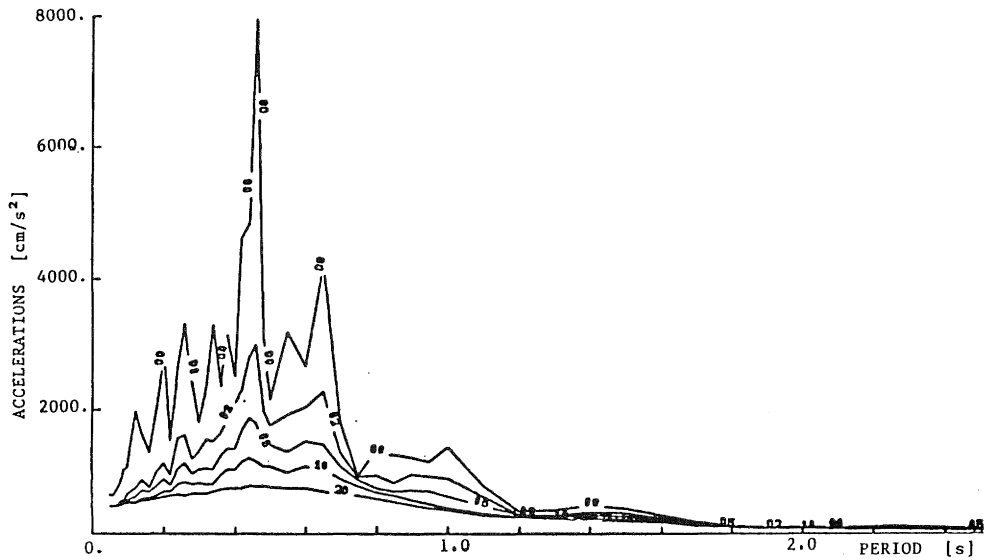


FIG. 8 - ELASTIC RESPONSE SPECTRUM OF "PETROVAC N-S"

improvements were made (for example, the possibility of the simulation of static cyclic tests was added); two new elements to stimulate joint behaviour were added.

The program is based on the well known standard DRAIN-2D program (2), except for a new numerical technique which permits solutions for elements with large stiffness changes. In the equilibrium correction strategy used in DRAIN-2D (2), the unbalanced load at the end of each load increment is calculated, and added to the load increment for the following time step (Fig. 9a, taken from (8)). Thus, violation of equilibrium is permitted for a short time within the current step. Unless very small load increments are used, considerable errors can occur. In the event-to-event strategy of DRAIN-2D-2 (8) the load increments are subdivided into subincrements, so that the solution proceeds from "event"

(i.e. stiffness change) to "event" (Fig. 9b, taken from (8)).

To illustrate the efficiency of both strategies, a structure consisting of a single, highly nonlinear contact element, described in the following section, was chosen. The period of the structure was $T = 0.33$ s. The dynamic response of this structure to the "Petrovac N-S" earthquake was calculated (only the strong part of the record, from $t = 5.0$ to $t = 10.0$ seconds was used). The accelerogram had been digitalized with a time step of 0.02 s. The following time increments were used in the dynamic solutions: 0.02, 0.01, 0.005, 0.001, and 0.0005 s. The results for maximum gap opening are illustrated in Fig. 10. Practically exact results were obtained by event-to-event strategy, regardless of how long the time step was. On the contrary, very short time increments are needed to

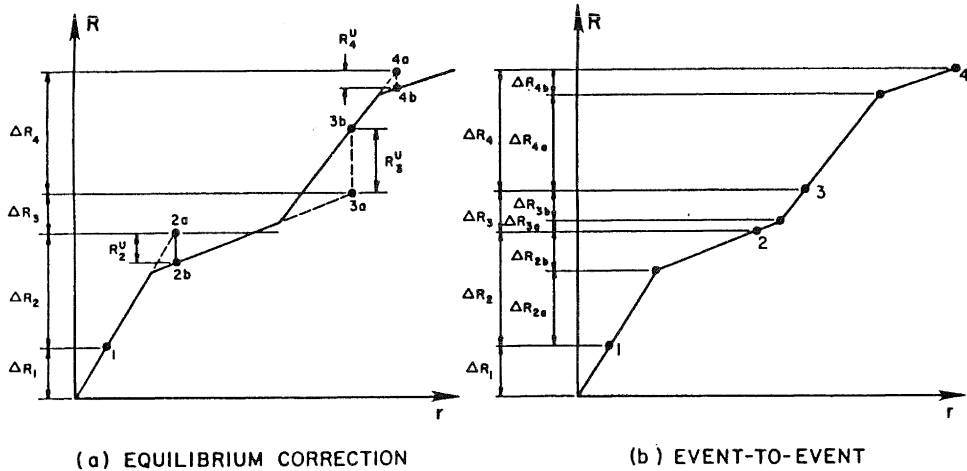


FIG. 9 - DYNAMIC RESPONSE SOLUTION STRATEGIES (8)

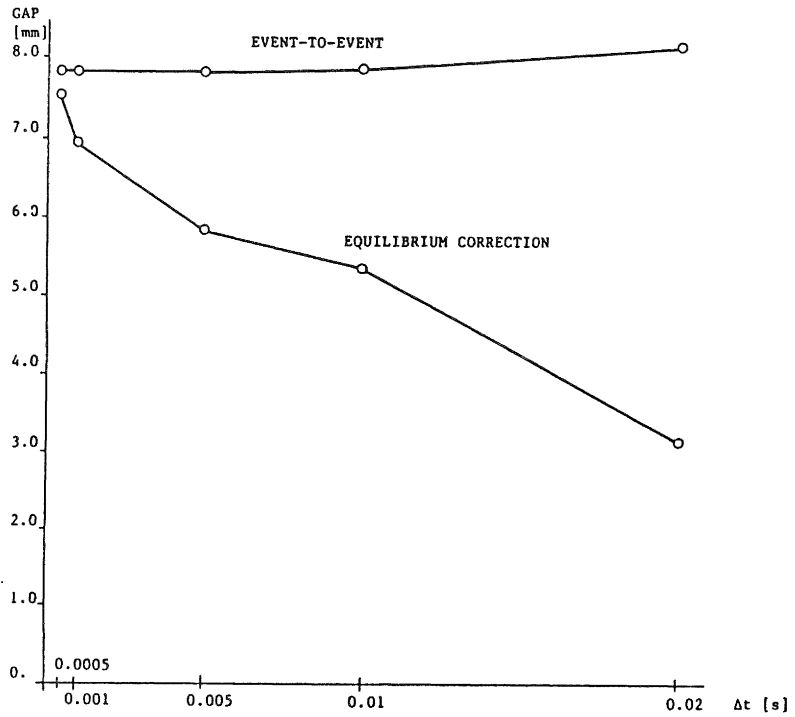


FIG. 10 - MAXIMUM GAP OPENING CALCULATED BY DIFFERENT SOLUTION STRATEGIES AND DIFFERENT TIME INTERVALS

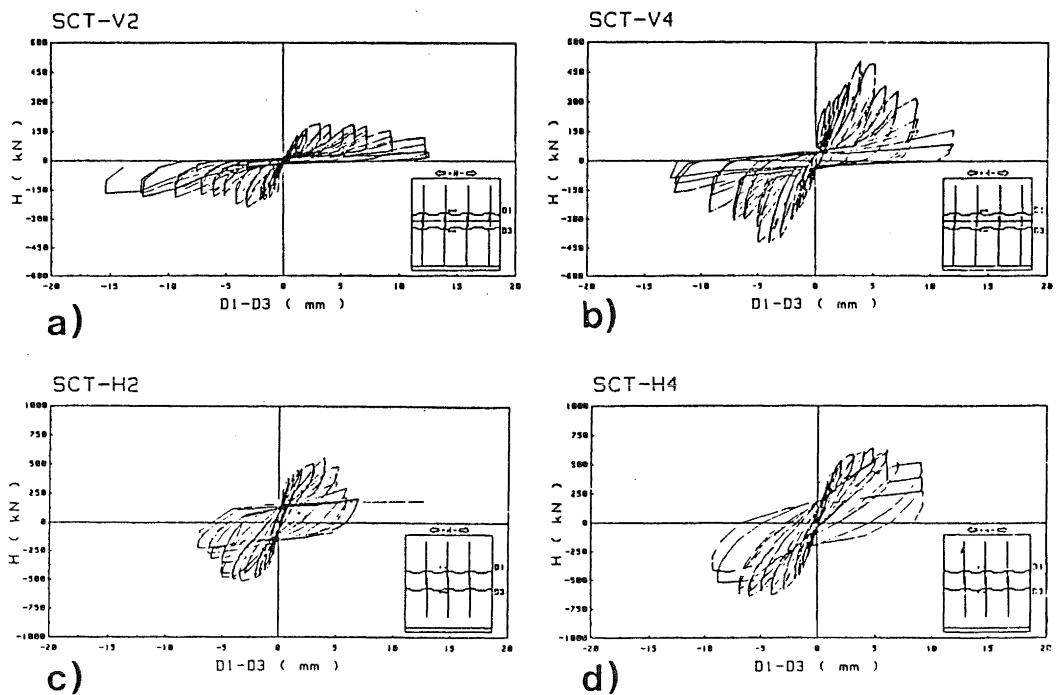


FIG. 11 - EXPERIMENTAL SHEAR FORCE-SLIP HYSTERETIC RELATIONSHIP (9)
 (a) VERTICAL JOINT WITHOUT AN AXIAL FORCE, (b) VERTICAL JOINT WITH A MODEST AXIAL FORCE, (c) HORIZONTAL JOINT WITH A MODERATE AXIAL FORCE, (d) HORIZONTAL JOINT WITH A LARGER AXIAL FORCE

get satisfactory solution by the "Classic" equilibrium correction strategy.

Mathematical MOdelling of Joints' Behaviour

Two predominant modes of the inelastic deformation of joints, shear slip and gap opening, were identified by experiments (9). Typical shear force-slip hysteretic relationships are shown in Figs. 11 a-d in the case of (a) a vertical joint without an axial force - V2, (b) a vertical joint with a modest compressive stress (1 MPa) - V4, (c) a horizontal joint with a moderate compressive stress (2 MPa) - H2, and (d) a horizontal joint with a larger compressive stress (3 MPa) - H4. The following can be observed: (a) the pinching effect, (b) the degradation of strength and stiffness, (c) after a number of cycles, a clearly visible plateau with nearly zero stiffness increases again due to transition from the damaged to the undamaged region and the champing action of the reinforcement, (e) the characteristic multilinear strength envelope can be defined, (f) the behaviour of the vertical and horizontal joints in qualitatively similar.

Opening of the horizontal joints follows nearly elasto-plastic behaviour in tension. Nevertheless, special unloading characteristics can be noticed. Unloading stiffness is, at first, very large. Then due to the degradation of the contact area, the stiffness degrades noticeably, and the joint closes with little inelastic deformation at zero load. Elastic behaviour in compression was observed up to the crushing of concrete and buckling of the reinforcement.

The influence of the gap opening on the shear-slip behaviour is taken into account implicitly, since the joints were subjected to a combination of shear force and bending moments during the tests.

To simulate the observed behaviour, two mathematical models were developed. The hysteretic rules of the shear-slip and the contact element are illustrated in Figs. 12a and 12b, respectively. When the elements were subjected to the strong part of the "Petrovac N-S" loading, the typical hysteresses, shown in Figs. 13a and 13b, were obtained.

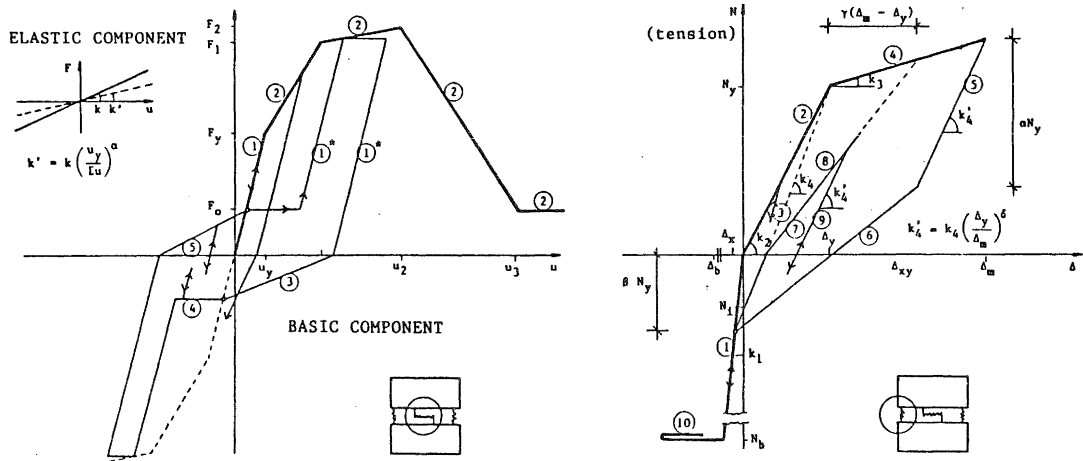


FIG. 12 - HYSTERETIC RULES
 (a) SHEAR SLIP ELEMENT
 (b) CONTACT ELEMENT

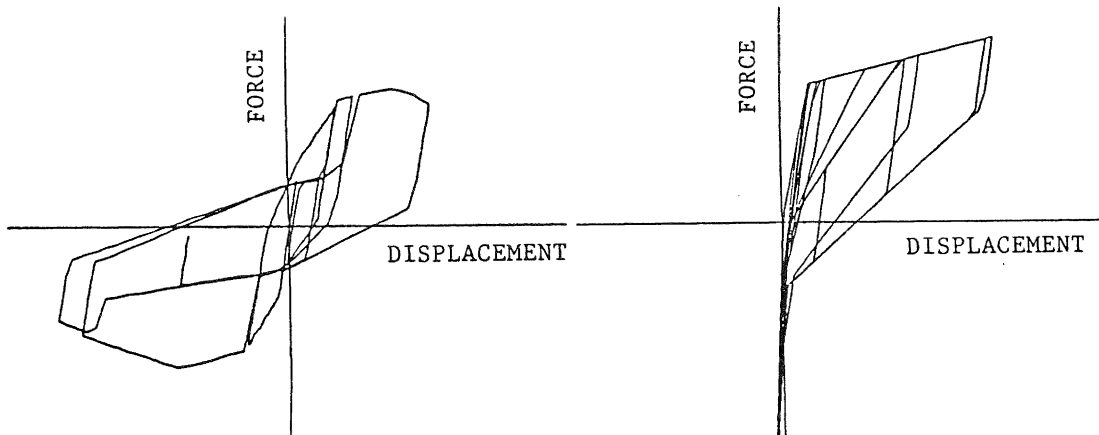
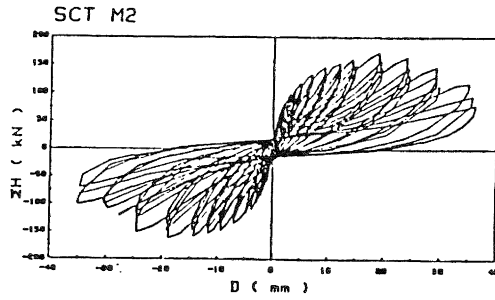
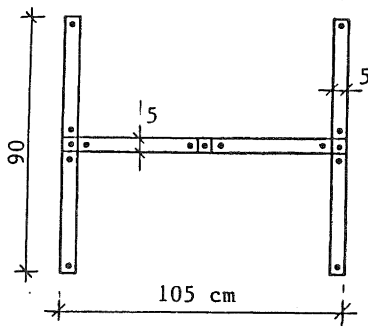


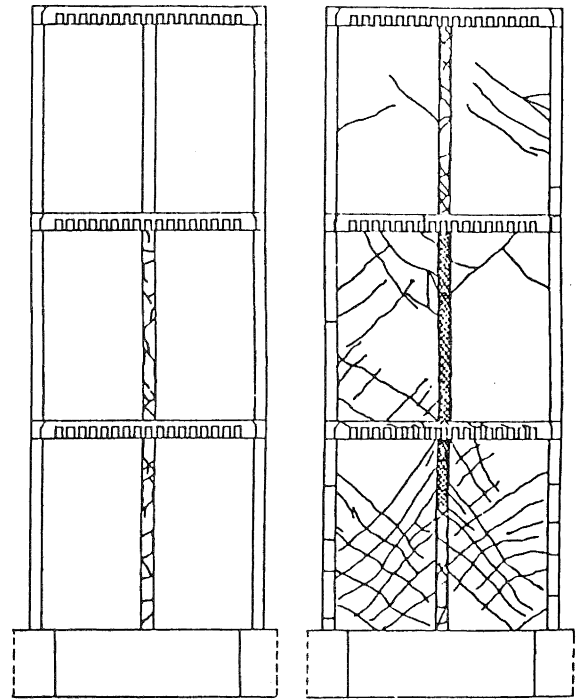
FIG. 13 - TYPICAL RESPONSE TO "PETROVAC N-S"
 (a) SHEAR-SLIP ELEMENT
 (b) CONTACT ELEMENT



b) Base shear - top displacement relationship



a) Lay-out of the model



Early stage

Late stage

c) Typical damage

FIG. 14 - TEST RESULTS (9)

Mathematical Modelling of Structural Subassemblages

A cyclic test of a three-story H-shaped model structure was performed at the Institute for Research and Testing in Materials and Structures (9). The lay-out of the model is given in Fig. 14a, the obtained base shear-top deflection relationship is given in Fig. 14b and typical damage at two different stages of the test is shown in Fig. 14c.

In the early phases of the test, monolithic behaviour was observed. Later, considerable damage occurred in the vertical connection in the middle of the wall, followed by a considerable decrease in the strength and stiffness of the model. Slips along the horizontal joint were small.

The mathematical model, shown in Fig. 15, was used in the analysis. Linear elastic behaviour of the panels was assumed, and all inelastic behaviour was concentrated in the joints. The joints followed the hysteresis rules shown in Fig. 12. Preliminary analysis have shown acceptable correlation with the test results.

Inelastic Response of SCT Buildings

Inelastic response history analysis of ten-story SCT buildings has not yet been completed. First results have indicated

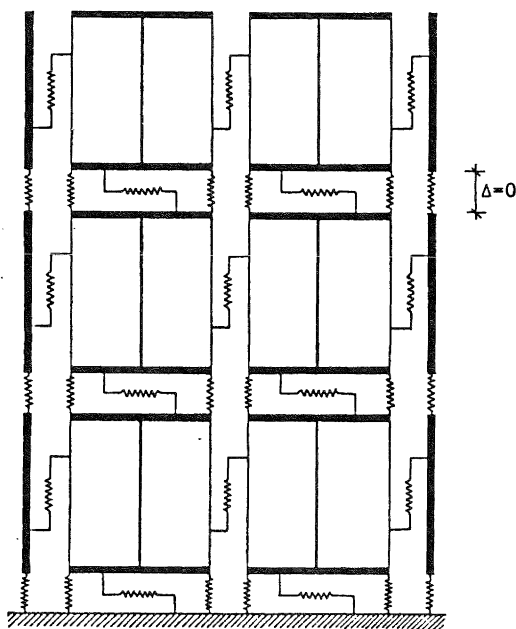


FIG. 15 - MATHEMATICAL MODEL OF THE TEST STRUCTURE

that these buildings have a large lateral resistance. This compensates somewhat for their lower ductility. Yielding of vertical reinforcement is predominant, and slips along the vertical joints have been observed. However, slips along the horizontal joints have been negligible.

Since an extremely unfavourable strong ground motion (the Petrovac N-S record) was used in the analysis, the results indicate acceptable behaviour of the building system during severe earthquakes.

CONCLUSIONS

The specific characteristics of large panel systems require specific design procedures, which are based on a thorough knowledge of the inelastic behaviour and possible failure mechanisms of these buildings.

Special computer programs are needed. DRAIN-2D-2, developed at the University of California, Berkeley, was modified and used in this study. The program incorporates a modified step-by-step technique, which permits solutions for elements with large stiffness changes. This event-to-event strategy proved to be superior in comparison with the standard equilibrium correction strategy used in the basic DRAIN-2D program.

The two predominant modes of deformation of joints, shear slip and gap opening, as well as the behaviour of structural assemblages, were modelled and compared with test results. The developed mathematical models and computer routines proved to be capable of the realistic simulation of the inelastic response of large panel buildings to severe earthquakes and of the estimation of their safety.

A favourable, nearly monolithic, response to strong earthquakes was observed during the preliminary inelastic analysis of SCT large panel buildings.

ACKNOWLEDGEMENTS

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