A FIVE-STORY PRECAST CONCRETE TEST BUILDING FOR SEISMIC CONDITIONS - AN OVERVIEW

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SUMMARY

In the final phase of the PRESSS (Precast Seismic Structural Systems) program, a large-scale five-story precast concrete building will be tested under simulated seismic loading. A brief summary of various structural features incorporated in the PRESSS building and test plans are presented in this paper while design details are provided in a companion paper. Construction and test preparations of this precast building have been already completed at UCSD Charles Lee Powell Structural Laboratory. All seismic testing of the building is scheduled to be completed by the end of August 1999.

INTRODUCTION

The PRESSS program has been on going for the past 10 years with an overall objective of developing seismic design recommendations for precast concrete systems [Priestley, 1991; Nakaki *et. al.*, 1999]. In the initial two phases of this program, experimental and analytical studies of ductile connection precast elements for frame and wall structures, as distinct from strong-connection precast structures which attempt to emulate monolithic reinforced concrete construction, were conducted. In the final phase, a large-scale precast five-story building utilizing different connection details is tested under simulated seismic loads [Nakaki *et. al.*, 1999].

STRUCTURAL FEATURES OF THE TEST BUILDING

Based on five-story prototype buildings with 100 x 200 sq. ft in plan (per floor), 12 ft 6 in. story height and 25 ft bay length, dimensions of the test building were established. It was first determined that, for seismic testing purposes, it would be only necessary to model 50 x 50 sq. ft plan area of the prototype buildings with 2 bays in each direction. The test building was then modeled at 60% scale of the resized prototype buildings in order to accommodate it inside the Charles Lee Powell Structural Laboratory at the University of California at San Diego (UCSD). This resulted in the test building having 30 x 30 sq. ft in plan, 7 ft 6 in. story height and 15 ft bay length and modeling all critical connections of a real building.

In one direction of the PRESSS building, seismic resistance is provided by precast frame systems, with a precast wall system and gravity frames in the orthogonal direction (Figs. 1 and 2). Four different beam-to-column connection details, based on the past PRESSS research program [Palmieri *et. al.*, 1997; Stanton *et. al.*, 1997], are modeled at different levels in the two parallel seismic frames. The prestressed frame shown in Fig. 3 models the hybrid and pretensioned connections while the TCY (Tension-Compression Yielding) frame in Fig. 4 models the TCY gap and TCY connections. In each frame, the connection type is identical in the first three floors and a different connection is used in the fourth and fifth floor levels.

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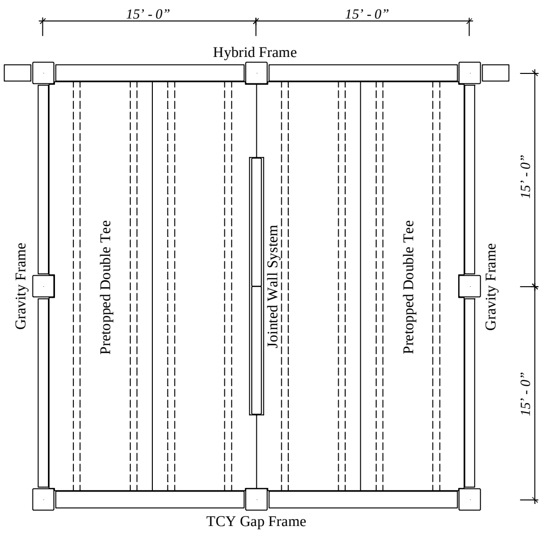

Fig. 1 Floor plan of the test building at Levels 1 - 3.

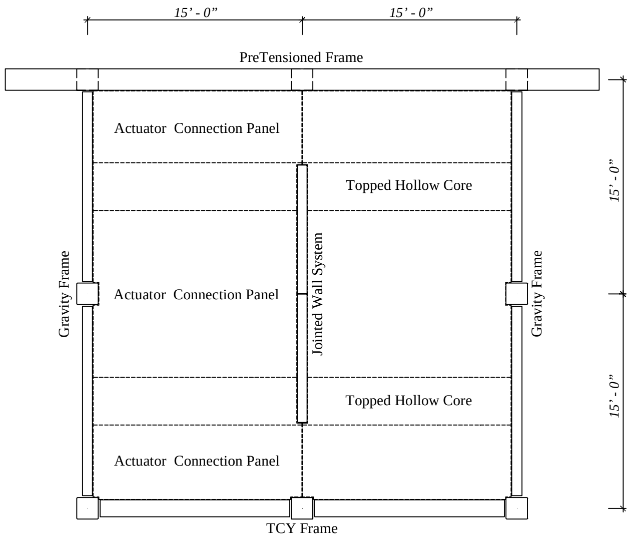

Fig. 2 Floor plan of the test building at Levels 4 & 5.

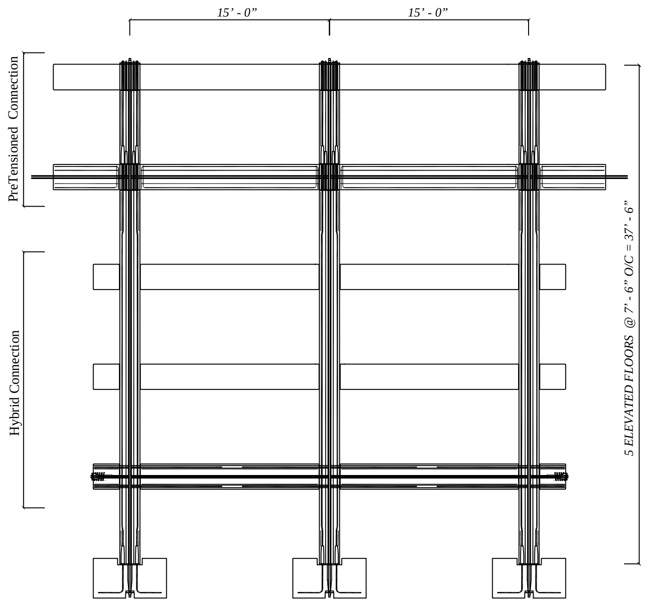

Fig. 3 Elevation of Prestressed seismic frame.

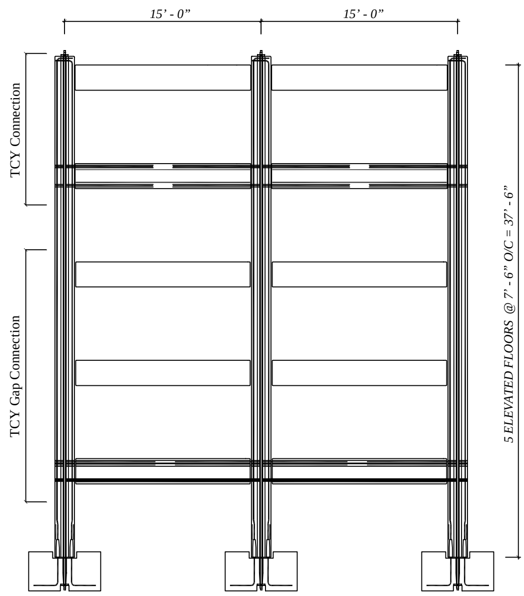

Fig. 4 Elevation of TCY seismic frame.

A brief description of each of the frame connections is as follows:

1. *Hybrid frame connection* (Fig. 5) – Beam-to-column frame connection is established with unbonded post-tensioning through the center of joint and field placement of mild steel reinforcement in ducts across the joint interface closer to the top and bottom beam surfaces. These ducts are grouted to ensure adequate bond for the reinforcement prior to post-tensioning.

2. *PreTensioned frame connection* (Fig. 6) – Continuous partially bonded pretensioned beams are connected to column segments extending from the top of beam at one floor level to the bottom of beam at the level above. The moment connection between the beam and column is established by extending the column mild steel reinforcement below the beam through sleeves located in the joint. The extended reinforcement is spliced to the column longitudinal reinforcement at the next level adjacent to the joint.

3. *TCY gap connection* (Fig. 7) – Mild steel reinforcing bars placed in grouted sleeves at the top of the beam and unbonded post-tensioning at the bottom of the beam provide the necessary moment resistance at beam ends. Beams and columns are separated by a small gap to avoid elongation of the beam due to seismic

action. This gap is partially grouted at the interface over 6" at the bottom of the beam with the post-tensioning force acting at the center of grout.

4. *TCY connection* (Fig. 8) - Behavior of monolithic reinforced concrete connections is emulated in this connection with top and bottom mild steel reinforcement in grouted sleeves across the beam-to-column interface.

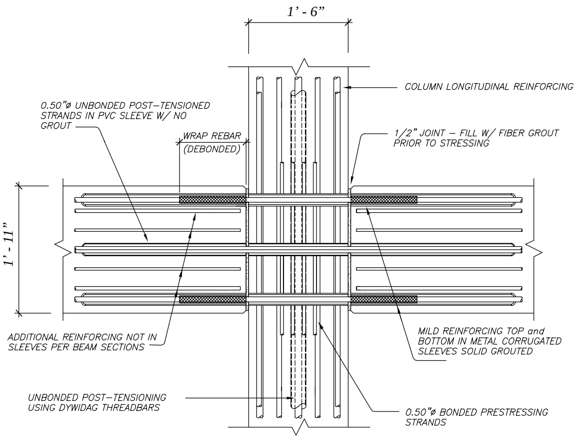

Fig. 5 Hybrid frame connection.

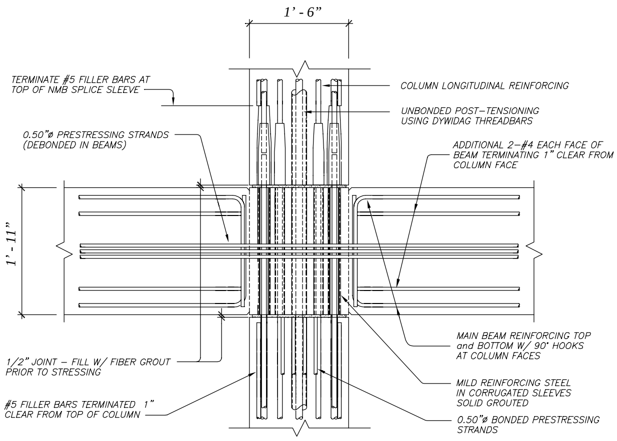

Fig. 6 PreTensioned frame connection.

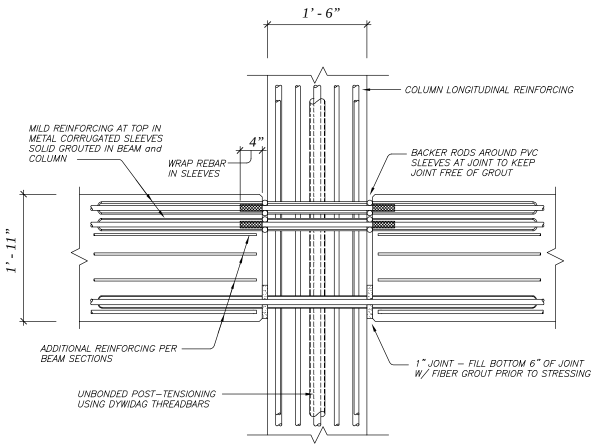

Fig. 7 TCY gap frame connection.

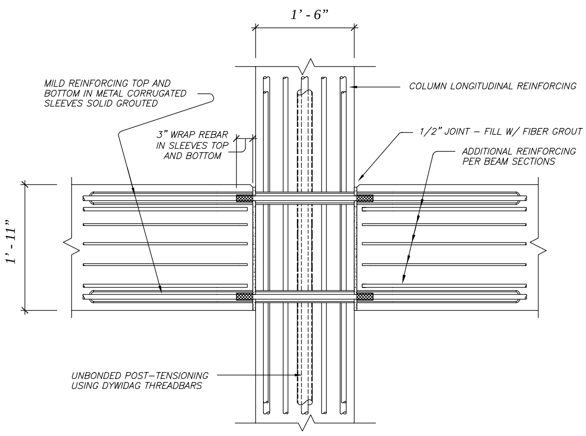

Fig. 8 TCY frame connection.

The walls in the orthogonal direction contain unbonded vertical prestressing, with special energy dissipating connectors located in a vertical construction joint between wall elements (see Fig. 9a). U-shaped stainless steel flexural plates (UFP), as shown in Fig. 9b, are used as connectors based on an earlier phase of the PRESSS investigation [Schultz and Magana, 1996]. Design details of the wall system and the fame connections are presented in a companion paper [Stanton *et. al.*, 2000]. Two precast flooring systems are also included in the test building. The first three floors are constructed using pretopped double tees (Fig. 1) while hollow-core panels are used in the upper floors (Fig. 2), with an in-situ topping.

Combinations of two building systems and four ductile frame connections adopted in the PRESSS test building effectively provide experimental verifications of seismic behavior of five different precast prototype buildings. In addition, application of the two most popular precast flooring systems to different seismic resistant building systems is also examined.

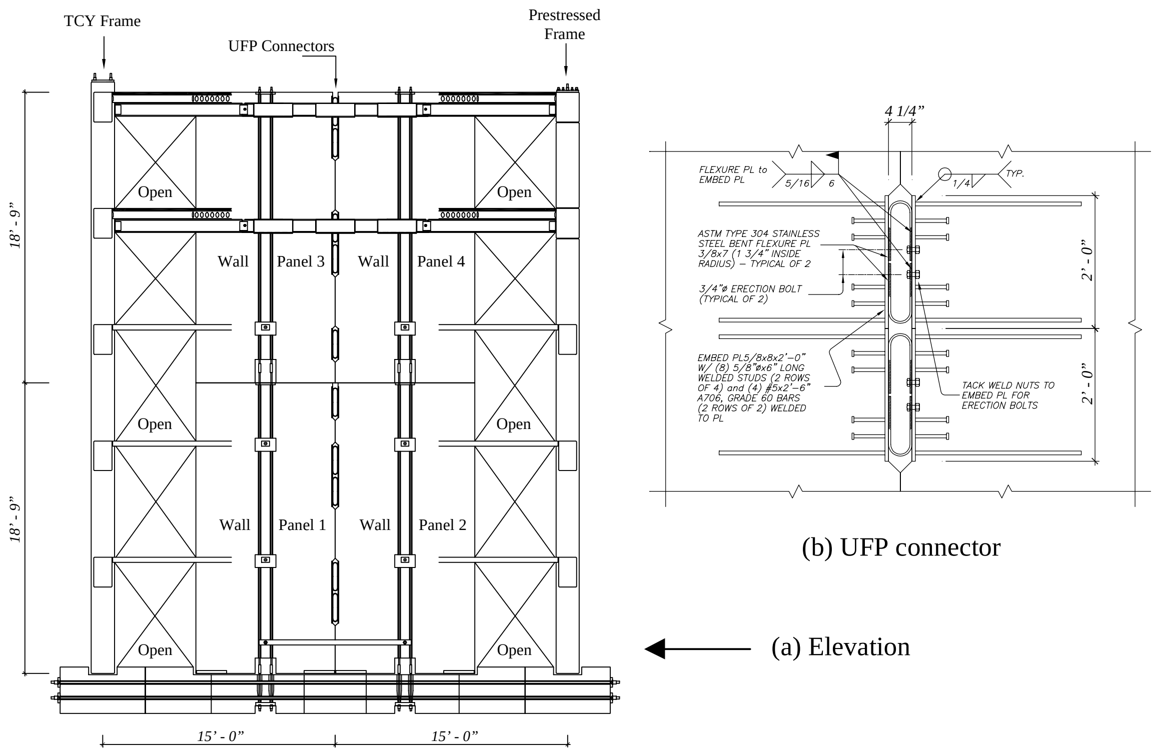

Fig. 9 Jointed precast wall system.

DESIGN METHODOLOGY

The PRESSS building was designed using the direct-displacement based approach (DBD) [Priestley, 1998] to sustain a maximum drift of 2% under a design level earthquake that represents the 1997 UBC Zone 4, Soil type Sc acceleration spectrum [UBC, 1997].

The UBC provisions do not include design displacement spectra that are required in the DBD approach. Hence, the 5% damped displacement response spectra included in Appendix G of the draft SEAOC Bluebook [PBSE-SEAOC, 1998] were used in the PRESSS building design. As detailed later in this paper, acceleration design spectra in UBC and SEAOC Bluebook are comparable, and thus utilizing SEAOC displacement spectra in the design of the test building was considered satisfactory.

The reason for using the DBD approach to designing the PRESSS building was that force based design does not sufficiently account for behavior of jointed precast systems. Furthermore, the R factors given in design codes as part of the force based design are also intended for precast systems emulating monolithic concrete connections, rather than for some of the connections incorporated in the PRESSS building. As a result of applying the DBD procedure, less conservative design base shears were obtained in both the frame and wall directions of the test building when compared to those obtained from the force based design method [Nakaki *et. al.*, 1999]. Reduced base shear results in significant cost savings in addition to improving performance over traditional precast systems.

SEISMIC TEST PLAN

Seismic testing of the PRESSS building will be independently conducted in the two orthogonal directions. In each direction, three different test schemes, namely the stiffness measurement test, pseudodynamic test and inverse triangular load test, will be used.

The stiffness measurement test is a quasi-static loading test through which the stiffness matrix of the test building is formulated. The stiffness matrix, which is updated following increased intensity of the lateral seismic loading, is useful in (a) determining the appropriate integration time step when explicit schemes are used in solving the equation of motion in the pseudodynamic testing procedure, (b) improving convergence of implicit integration schemes in the pseudodynamic testing procedure, (c) characterizing structural behavior consistent with the direct-displacement based approach, and (d) monitoring damage levels using stiffness as a damage indicator.

Significant portions of the testing in the two orthogonal directions will be performed using a pseudodynamic testing procedure. In this procedure, the external dynamic load is applied to the structure quasi-statically through ten on-line controlled hydraulic actuators. Combining numerical computation and experimental measurements, the pseudodynamic test is carried out using the concept outlined in Fig. 10. To ensure speedy convergence of solutions and minimize error propagation in this test procedure, algorithms developed as part of the TCCMAR masonry building test [Igarashi *et. al.*, 1994] will be used with some modifications. Several segments of earthquake time histories, including one with intensity exceeding that of the design level earthquake, will be used. Details of the input acceleration motions are given in the following section.

In the inverse triangular load test, the test building is subjected to a full load reversal using a set of lateral forces distributed in an inverse triangular fashion, causing the structure to deform approximately to its first mode shape. The purpose of the inverse triangular load test is twofold. When designing structures either using force-based or displacement-based method, member forces are determined by assuming approximately an inverse triangular acceleration pattern. Hence, response of the building from inverse triangular load tests can be directly compared to the response assumed in the design procedure. The other benefit of the inverse triangular load test is that equivalent viscous damping of the building can be quantified. In the DBD approach, the design base shear is determined using theoretically estimated equivalent viscous damping of the structure as a whole at the design drift level. Since this equivalent damping represents hysteretic energy dissipation occurring during reverse cyclic loads, results from inverse triangular load tests can be also used to experimentally quantify this critical design parameter at different drift levels.

SELECTION OF EARTHQUAKE INPUT MOTIONS

In the pseudodynamic test, performance of the structure is assessed for a given earthquake input motion. It was felt desirable to subject the test building to several input motions with progressively increasing intensity, thus allowing building performance to be examined at different limit states. However, it was not feasible to compile a suite of suitable acceleration time histories recorded from past earthquakes. Therefore, it was decided that appropriate input motions for pseudodynamic testing be established by modifying recorded earthquake motions on soil type S_c.

The first step in establishing suitable input motions for pseudodynamic test was to choose a set of target spectra. For this purpose, four levels of performance based spectra as recommended by PBSE Ad-Hoc Committee of SEAOC [1998] were used. Using the design spectra in the 1997 NEHRP Provisions [Building Seismic Safety Council, 1997] as the basis, the PBSE Ad-Hoc Committee recommends acceleration spectra for earthquake hazard ranging from EQ-I to EQ-IV. These four levels represent, respectively, frequent, occasional, rare, and maximum credible earthquakes. In Fig.11, the four level earthquake hazard spectra are shown for soil type S_c, which is one of five soil types classified in the NEHRP provisions. It is noted that the EQ-III level spectrum, which represents design level earthquakes, is identical to the design spectrum included in NEHRP and 1997 UBC codes up to 4.0 s period. At periods beyond 4.0 s, these codes adopt 1/T decay in the spectral values while the spectrum in Fig. 11 reduces spectral accelerations in proportion to 1/T2 to maintain constant spectral displacements at longer periods. Accelerations corresponding to the EQ-IV spectrum are intended to be 50% stronger than those of EQ-III (see Fig. 11).

The procedure adopted for obtaining suitable input motions is described here by deriving an EQ-III level input motion from the El Centro record obtained from the 1940 Imperial Valley earthquake (see Table 1). The duration of El Centro record is 53.7 s. As discussed subsequently, a seven-second segment of this record containing the peak acceleration cycle was considered sufficient for pseudodynamic testing. The starting time of all segments, except for EQ-I motion, was decided such that the first peak of each segment is the first peak in the record exceeding 0.1g ground acceleration. For EQ-I level input motion, the same criterion was used with the first peak exceeding 0.05g. Duration of each segment was kept in the range of 4 s for lower intensity motions to 9 s for higher intensity earthquake records with long strong duration

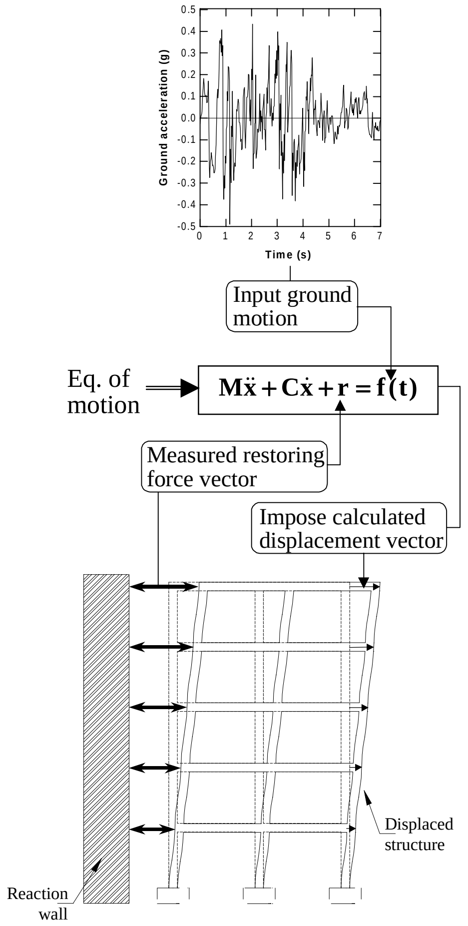

Fig. 10 Pseudodynamic test concept.

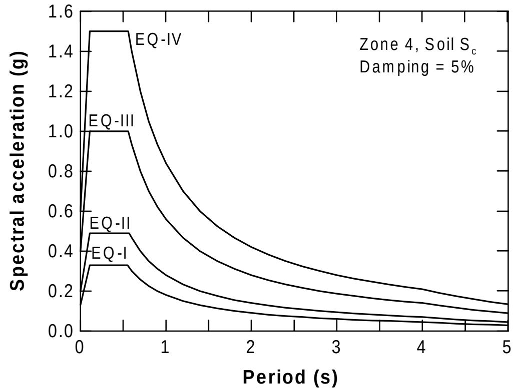

Fig. 11 EQ-I to EQ-IV earthquake hazard spectra.

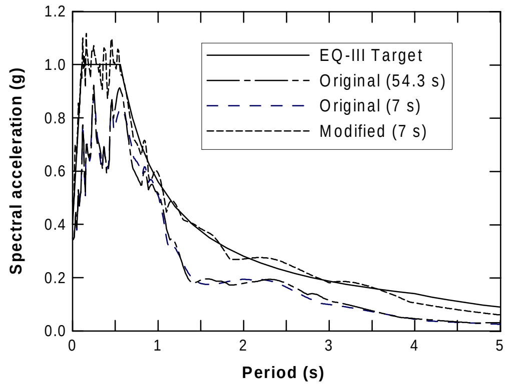

Fig. 12 EQ-III spectrum and 5% damped El Centro acceleration response spectra.

In Fig. 12, acceleration response spectra obtained for 53.7 s and 7 s duration of the El Centro record are compared with the EQ-III spectrum. Close agreement of these two El Centro spectra validates the choice of a reduced 7 s duration for the test simulation. The 7 s segment is then modified such that it provides an acceleration spectrum comparable to the EQ-III spectrum. In Fig. 12, it can be seen that the spectrum of the modified motion satisfactorily matches the EQIII spectrum. The necessary modification to the earthquake segment was performed using the program "SHAPE" [Earth Mechanics, 1998], in which the changes are made iteratively by multiplying Fourier amplitudes of the original motion by spectral ratios established between target acceleration response spectrum and spectrum of the input motion. The original and modified segments of the El Centro records are shown in Fig. 13.

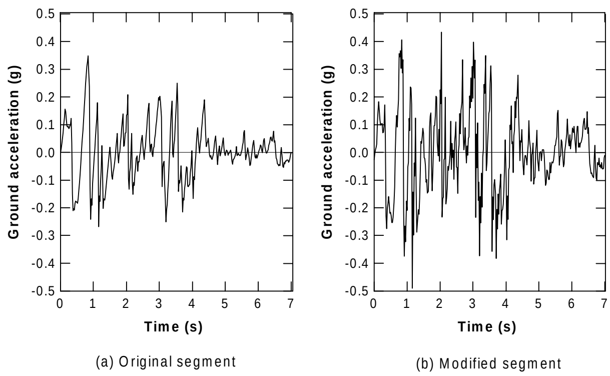

Fig. 13 Seven-second segment of the El Centro record.

Table 1: Details of the original input records.

EQ Level	Event	Magnitude	Record	Component	PGA
EQ-I	1974 Hollister earthquake	M_L = 5.2	Gilroy Array #1	S67W	0.14g
EQ-II	1971San Fernando earthquake	M_W = 6.6	Hollywood Storage	N90E	0.21g
EQ-III	1940 Imperial Valley earthquake	M_W = 6.9	El Centro	S00E	0.35g
EQ-IVa	1994 Northridge earthquake	M_W = 6.7	Sylmar	N00E	0.84g
EQ-IVb	1978 Tabas earthquake	M_W = 7.4	Tabas	N16W	0.94g

Four other input motions derived for possible application in the pseudodynamic test of the PRESSS building are shown in Fig. 14, with details of the original records in Table 1. It is noted that in all modified motions, some high frequency content uncharacteristic of natural records is apparent, which elevated the peak ground acceleration (PGA) of the modified records by as much as 50% higher than the target PGA. Low-pass filtering of these records would eliminate the high frequency content and reduce the PGA closer to the target values. However, such filtering was considered unnecessary because the response of the test building will not be sensitive to such high frequency content. Using input motions in the pseudodynamic testing, which closely match the required spectrum but with higher PGAs, will also demonstrate that structural response is not highly influenced by PGA.

TEST SEQUENCE

The first step in seismic testing of the PRESSS building is to formulate the stiffness matrix in the uncracked state through a stiffness measurement test. This will be followed by a test sequence consisting of a pseudodynamic test, an inverse triangular load test, and a stiffness measurement test. This sequence is repeated several times with intensity of the input motion for the pseudodynamic test increasing from EQ-I to EQ-IV. At each level of input motion, the inverse triangular load test is performed for one cycle with full reversal such that the resulting maximum positive and negative roof drifts equal the maximum recorded drift in the preceding pseudodynamic test.

Using the above procedure, the PRESSS building will be first tested parallel to the jointed wall system and then in the orthogonal direction to examine the behavior of seismic frames.

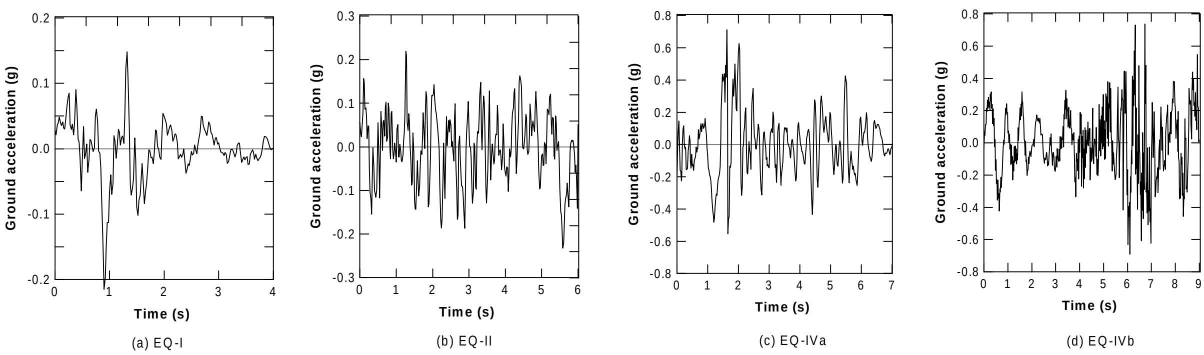

Fig. 14 Four other possible input motions for the PRESSS building test.

CONCLUDING REMARKS

A brief description of structural features of the PRESSS five-story precast test building and its seismic test design are presented in this paper. To sufficiently characterize the behavior of the PRESSS building modeling five prototype building systems, three different tests, namely stiffness measurement test, pseudodynamic test and inverse triangular load test, are considered. Significant portions of the test in the two directions are pseudodynamic, in which different level input motions with progressively increasing intensity are used. It is expected that the different levels of pseudodynamic testing together with stiffness measurement and inverse triangular tests will sufficiently quantify the performance of the PRESSS building at different limit states.

Construction and test setup of the PRESSS test building have been completed in the Charles Lee Powell Structural Laboratory of the University of California at San Diego. Low amplitude shakedown testing of the building is currently underway. Wall direction of testing and the frame direction of testing are scheduled to be completed by the end of August 1999. Preliminary results from seismic testing of the PRESSS building will be presented at the conference.

ACKNOWLEDGEMENTS

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