CUREe-Kajima Joint Research Program: Phase IV Assessment of the Seismic Performance of Reinforced Concrete Structures with Flat Plate Floor Systems

Quarterly Report: 10/1/00 – 12/31/00

John W. Wallace and Thomas H. Kang

Primary project goals for October 1 through December 31, 2000 involved completing specimen design, reviewing available analytical models, and conducting analytical studies to evaluate expected response for a variety of ground motions. Each of these tasks is discussed in more detail in the following paragraphs.

<u>Specimen Design:</u> Additional work has been conducted to determine appropriate dimensions and reinforcement for the specimens that will be tested. Preliminary design has been completed on a "reinforced concrete" slab-column specimen, with and without shear reinforcement at the slab-column connections. A scale factor of approximately one-half was selected for the reinforced concrete specimens, based on dimensional and reinforcing requirements. Geometry and reinforcing information is provided in Figure 1. Work was also conducted to design a post-tensioned slab-column specimen. Details are provided in Figure 2. A scale factor of approximately one-third is implied for this specimen based on the typical slab span-to-depth ratios used.

Two prototype buildings were selected to assist in design/literature review, as well as to assist in developing realistic specimen proportions and detailing. The prototype buildings consisted of a five-story flat-plate floor system with shear walls designed for UBC-97 zone 4 (high seismic region) and a three-story flat-plate floor system design for UBC-97 zone 2 (moderate seismic region, See Figure 3). Tentatively, a two-bay, two-story reinforced concrete specimen was selected with 2.75 m bay widths and a story height of 1.35 meters. Columns are approximately 200 mm square with 12.5 mm (US #4) longitudinal reinforcement. The floor system consists of 90 mm thick slab with 9.5 mm diameter bars (US #3).

Reinforced concrete and post-tensioned slab flexural reinforcement was selected to satisfy gravity load requirements based on using a direct design approach as described in ACI 318-99, Section 13.6 and using a equivalent frame approach in Section 13.7, 18.7-9, respectively. Design for combined gravity and lateral load is based on use of an effective beam width model.

Particular attention has been paid to the design of the connection region. In specific, several variations in the design have been studied to ensure that shear reinforcement is required within the connection region. Use of two types of shear reinforcement has been investigated – shear studs (Figure 4) and stirrups (use of shear reinforcement also impacted the selection of the specimen scale, as adequate slab depth is needed to accommodate the shear reinforcement). Design of the slab shear reinforcement was based on ACI 318-99 and ACI Committee 421-92 requirements (both requirements were checked). For shear reinforcement, use of bent stirrups and welded rebar grid are being considered.

Review of the designs is currently being conducted using the results of analytical studies that are described in the following section.

Model Review: Nonlinear models of two-story slab-column specimens were developed and used to assess expected behavior under various test conditions. The objectives of this aspect of the work were two-fold. First, it involved a review of options to model slab-column system behavior in available computer programs, as well as the need to develop improved models. Second, the computer models were subjected to ground motions (compressed to account for scale) to determine if the failure modes were consistent with our overall objectives of the project.

A schematic of the model is given in Figure 5. The bending stiffnesses for the beams that span between columns are based on an effective slab width as recommended in FEMA 274 and by Hwang (1989). Columns were modeled as elastic members with an effective stiffness of $0.7EI_g$. The beams are connected to the columns by a nonlinear rotational spring, or "connection element". The moment-rotation behavior of the connection element monitors the transfer of unbalanced moment at the connection, as well as the rotation capacity of the connection. Pure punching failure capacity of the connections is defined using the eccentric shear stress model of ACI 318-99 Section 11-12. Pure punching failure occurs if the connection punching capacity is reached prior to flexural yielding in the effective beam representing the slab. If the slab flexural reinforcement yields significantly at loads less then need to cause a pure punching failure at the connection, then the potential for punching failure depends on the inelastic rotations developed within the slab adjacent to the connection region. The relationship between the inelastic rotations developed within the slab and the potential for punching shear failure is based on experimental data, which strongly suggest that the inelastic rotations that can be developed within the slab decrease with increasing gravity shear stress demand on the slab-column critical section (e.g., see Moehle, 1989). Given a gravity shear ratio (V_g/V_o) of approximately 0.25, the rotation capacity of the effective beam representing the slab at an interior connection is assigned a value of 0.03 radians based on a review of experimental data. When the rotation demands reach 0.03 radians within the effective beam at an interior connection, a punching of the connection occurs. After punching, the connection region still resists vertical loads (due to bottom slab reinforcement providing protection against progressive collapse); however, the moment capacity drops to zero (or a true hinge). For an edge connection, no significant interaction is observed between shear and normal moment as discussed by Moehle (1988). Thus, ACI Committee 352 recommends that the effects of the shear stress on the slab critical section from the unbalanced moment be ignored. The inelastic rotation capacities of edge connections are determined using experimental data (if flexural yielding occurs prior to pure punching failure).

The DRAIN-2DX program was used to create models of the reinforced concrete and post-tensioned concrete specimens using elements 02 (beam-column element) and element 04 (nonlinear rotational spring). Since DRAIN-2DX modeling options do not incorporate features that account for punching type failures, multiple analyses were conducted to assess a range of variables.

Analytical Studies: Both nonlinear dynamic response-history analyses and nonlinear static (pushover) analyses were conducted to evaluate expected specimen behavior. Specimens with and without shear reinforcement were evaluated. Shear reinforcement was selected using two procedures, the one included in ACI 318-99 and the one included in ACI 421-92. Actual shear reinforcement used could consist of bent stirrups, welded rebar grid, or stud rails. In cases where shear reinforcement is used, punching failure is calculated to occur outside of the region strengthened with shear reinforcement.

Results for the static analyses are given in Figure 6 for the reinforced concrete specimens with and without shear reinforcement. Without shear reinforcement, punching failure is predicted to occur at a displacement of 1 cm, or an interstory drift ratio of 0.35 %. This is a relatively low drift ratio, which would be expected to occur within the essentially elastic range prior to significant yielding of flexural reinforcement within the slab. Use of shear reinforcement at the slab-column connection delays the punching failure at the connection; therefore, a displacement of 4 cm, or interstory drift ratio of 1.5 %, is achieved prior to a punching failure. Results of the post-tensioning specimen with shear reinforcement for the static analyses are given in Figure 7., which shows that the punching failure occurs at a displacement of 6 cm, or interstory drift ratio of 3 %.

The target roof displacement based on the nonlinear static analysis was obtained using procedures described in FEMA 273 (with time scaled). Nonlinear dynamic response-history analyses were performed for the various earthquake ground motion accelerations. As noted earlier, DRAIN-2DX program does not account directly for punching failures; therefore, a number of models were used to assess the degrading behavior that occurs due to punching failures. The response of the specimens for two levels of ground acceleration and the associated target displacement are given on Figure 8. Based on the results of these analyses, the specimen dimensions and reinforcement will be reviewed in greater detail and preliminary drawings prepared for distribution and comment.

In addition to comments from the project participants, comments are being solicited from several engineering firms on the west coast of the US that are very familiar with design and construction of slab-column systems.

Future Work: The immediate goals are to complete the analytical studies of specimen behavior so that specimen details can be completed and distributed for comments and discussion. While this review is underway, work will continue on developing an instrumentation plan, as well as a plan for testing. Several contractors also will be contacted to initiate the process of soliciting bids to fabricate the specimens.

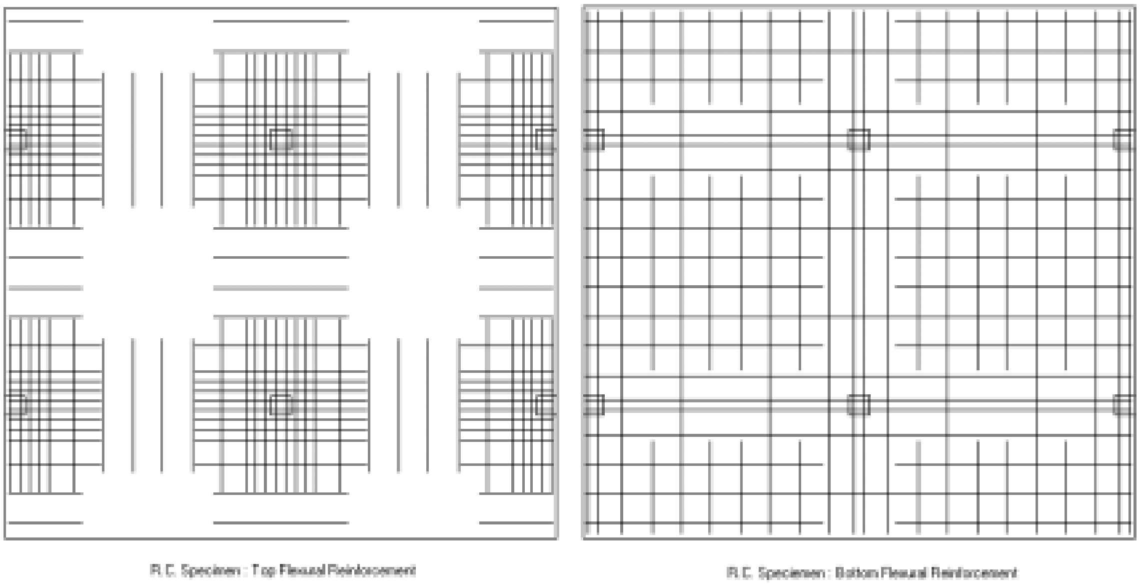

Fig.1. R.C. Specimen : Flexural Reinforcement

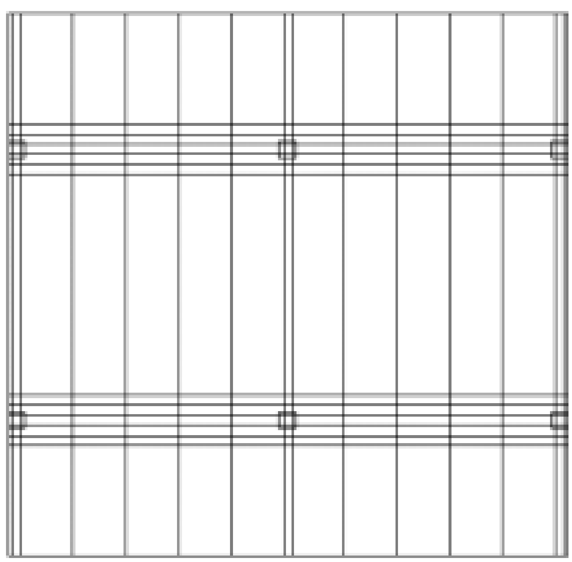

Fig.2. P.T. Specimen : Post-Tensioning Tendon Arrangement

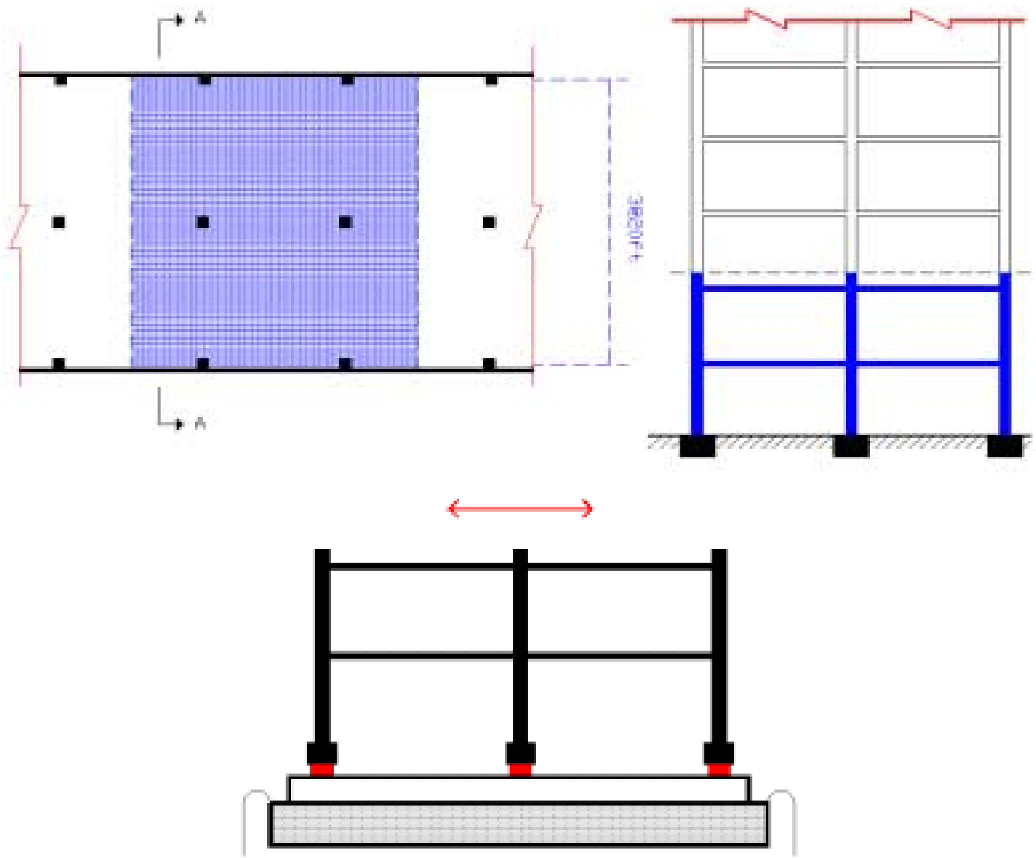

Fig.3. Prototype Structure and Test Structure

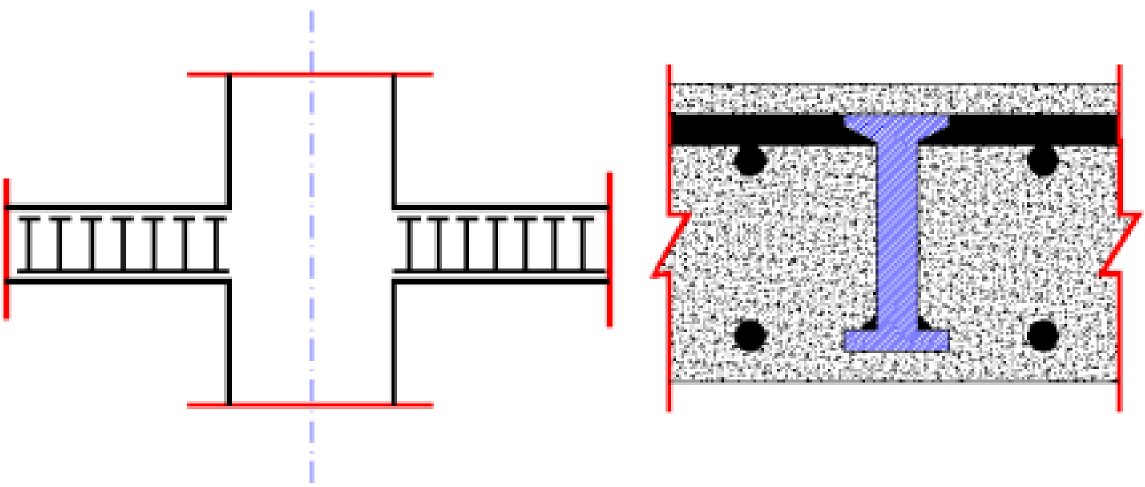

Fig.4. Shear Reinforcement : Shear Studs

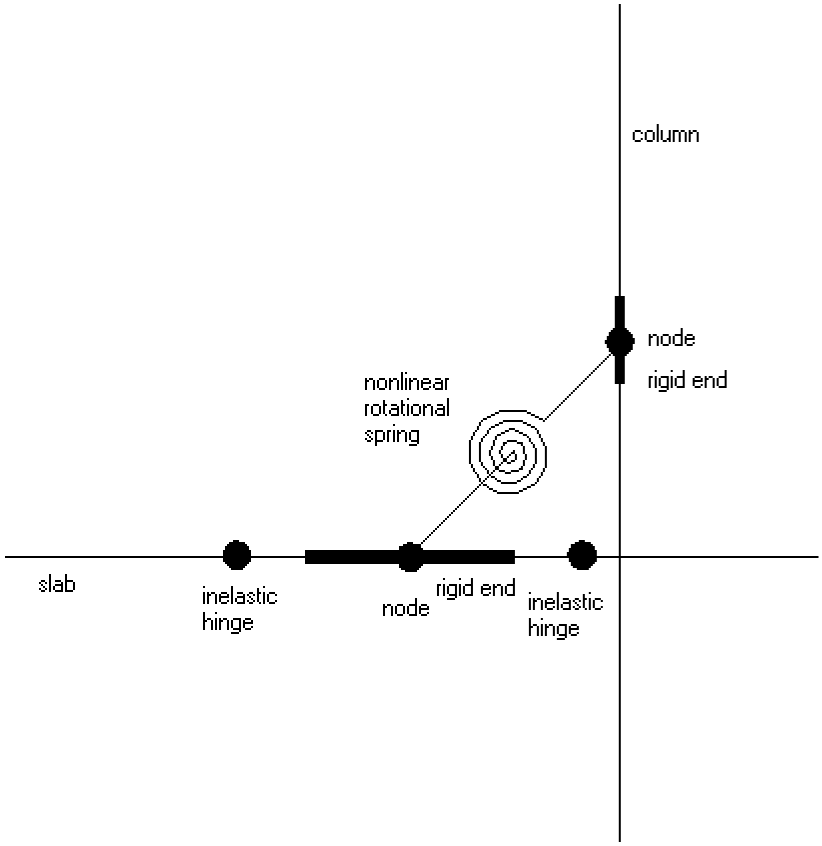

Fig. 5. Nonlinear Rotational Spring

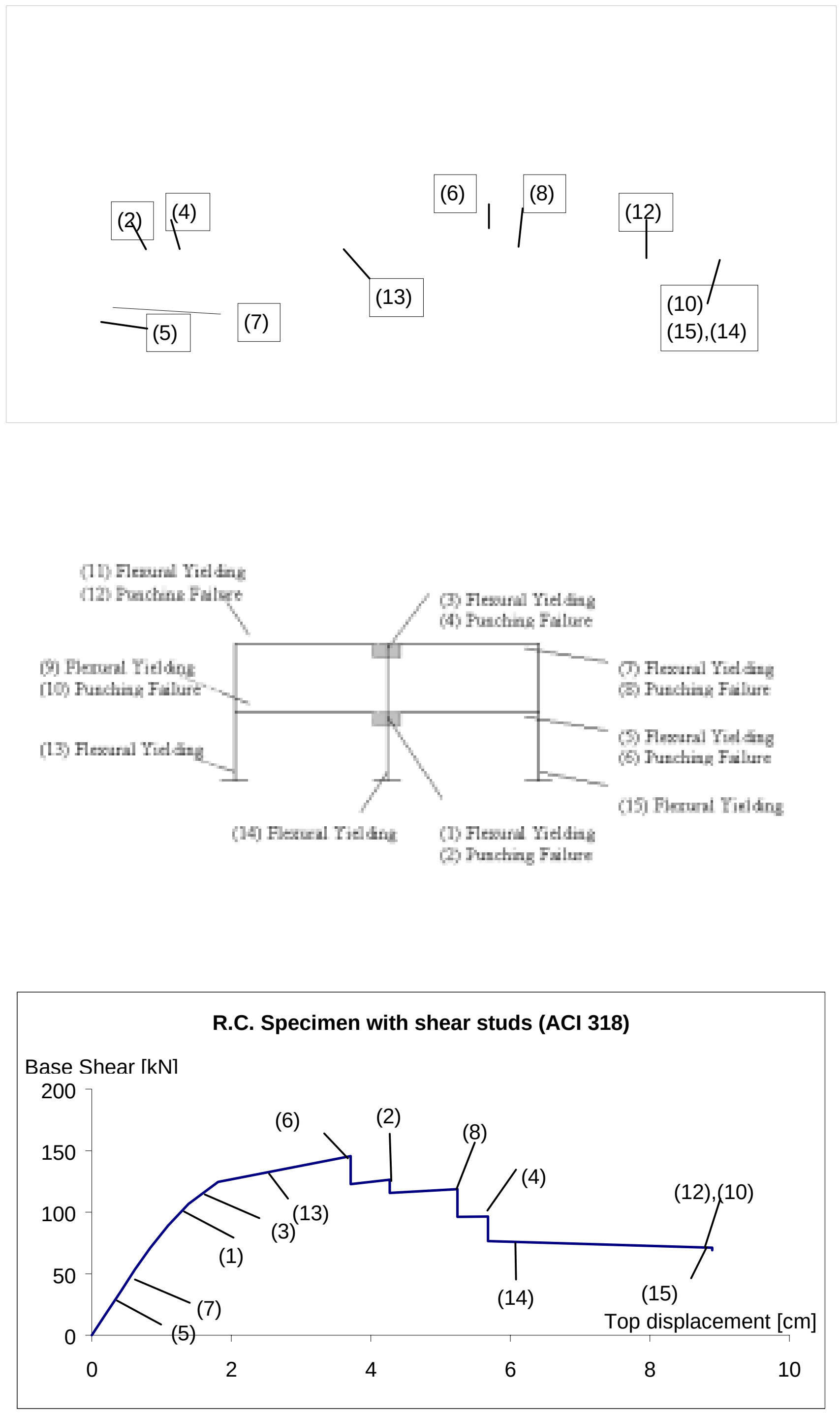

Fig. 6. Nonlinear Static Analysis (DRAIN-2DX) – R.C. Specimen

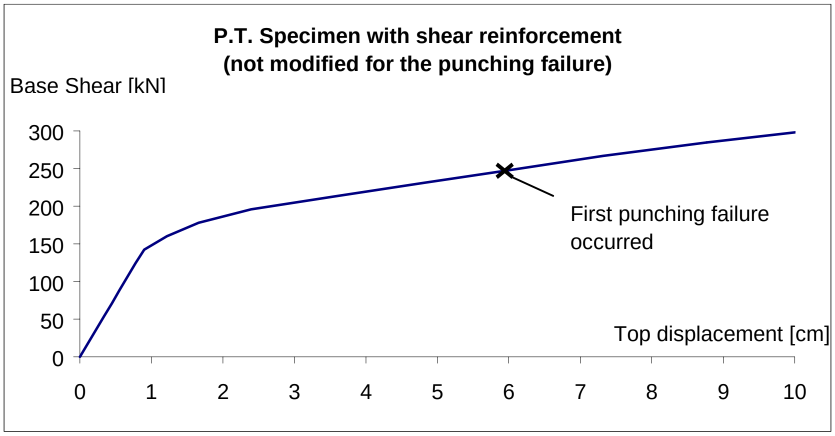

Fig. 7. Nonlinear Push-Over Analysis (DRAIN-2DX) – P.T. Specimen

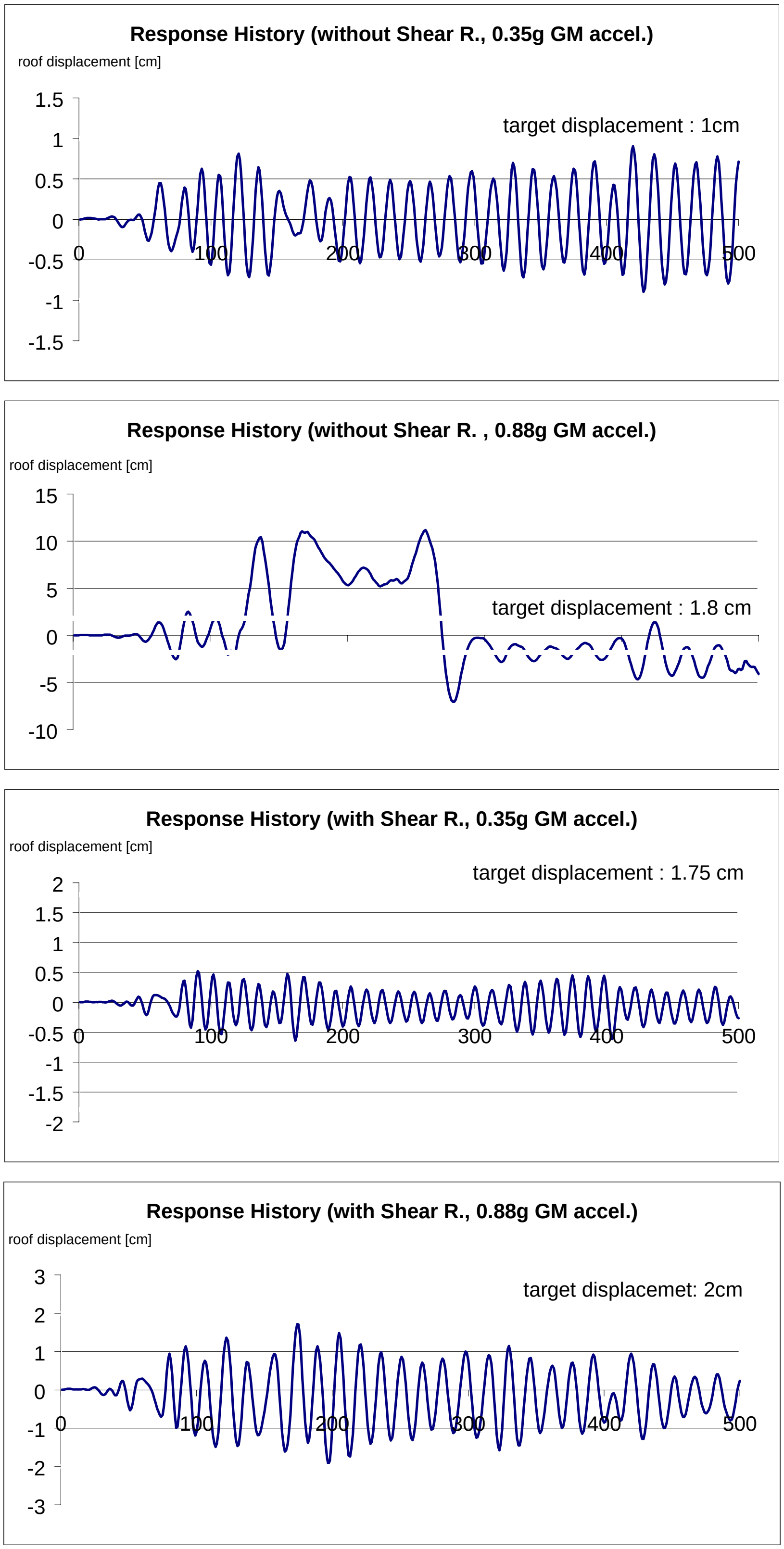

Fig. 8. Response History and Target Roof Displacement for R.C. Specimen