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MEASURED RESPONSE OF TILT-UP STRUCTURAL SYSTEMS

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ABSTRACT

An experimental program was developed to study the strength and deformation capacity of tilt-up structures. Two models of tilt-up systems were constructed and tested in the laboratory. The test structures were representative of one–story warehouse construction in the western United States: reinforced concrete wall panels were positioned around the perimeter of the buildings and connected by a plywood roof diaphragm. The overall dimensions of the specimens were the same; however, the number and distribution of the wall panels were varied. Each specimen was subjected to a series of lateral load reversals.

The measured response of the first test specimen is discussed in this paper. The overall structural response was controlled by the behavior of the flexible plywood roof diaphragm. Displacement histories measured along the length of the diaphragm revealed that the stiffness and strength of the structural system degraded as the amplitude of the imposed displacements and number of loading cycles were increased. The strength of the specimen was limited by buckling of the plywood panels.

KEYWORDS

Tilt-up structural systems; reinforced concrete wall panels; plywood roof; flexible diaphragms; seismic response; static load reversals; stiffness degradation.

INTRODUCTION

The term "tilt-up" was introduced more than 50 years ago to describe a method for constructing low–rise reinforced concrete buildings rapidly and economically. In tilt-up construction, reinforced concrete wall panels are cast horizontally at the site, rather than in place in vertical forms or in a prefabrication plant. These wall panels are then lifted (tilted) by a crane and set vertically on prepared foundations to form the exterior walls of the building. The wall panels are tied together with a roof diaphragm, which is typically flexible with respect to the panels. Tilt-up systems are used in approximately thirty percent of the existing low-rise industrial structures in the United States.

Although tilt-up systems are economical to build, this type of construction is susceptible to structural damage during earthquakes. Tilt-up structures are designed by assuming that the system will behave as a box structure when subjected to lateral loads. Inertial forces are intended to be carried through the roof diaphragm to the end walls and into the foundation. Failure of the connections between the wall panels and the roof diaphragm has often been cited as the cause of the seismic damage (Hall, 1994; Hamburger, 1988; and Jennings, 1971).

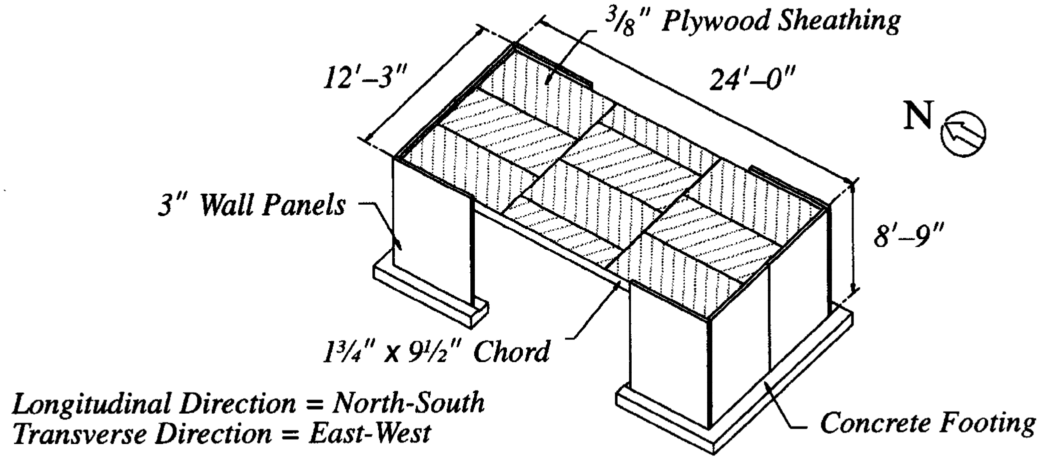

Fig. 1. Test Specimen TU1

The objective of an ongoing research project is to study the seismic response of tilt–up structures. The work has been divided into three phases: experimental tests of two complete tilt–up systems, development of nonlinear analytical models to represent the important features of the measured response, and development of evaluation techniques to assess the seismic vulnerability of existing tilt–up construction. This paper describes key aspects of the measured response of the first test specimen, TU1.

TEST SPECIMEN TU1

The dimensions of the diaphragm in a typical tilt–up structure are on the order of hundreds of feet which precludes testing of a full-size tilt-up specimen in a laboratory. Therefore, a number of compromises were made in order to incorporate the essential features of tilt-up buildings into a reasonably–sized test specimen. The configuration of the test specimen was determined primarily by the desire to test a complete tilt-up system and to evaluate the interaction between the perimeter walls and the roof diaphragm.

A schematic drawing of test specimen TU1 is shown in Fig. 1. The plywood roof diaphragm is approximately 24 ft. long by 12 ft. wide and is supported 8 ft.–9 in. above the foundation girders by eight reinforced concrete wall panels. Two longitudinal and two transverse wall panels are positioned at the north and south ends of the structure.

A brief description of the primary structural components is presented below. In many cases, the structural components that form the test specimen were selected based on construction requirements, rather than stress levels calculated from the imposed loads.

Reinforced Concrete Wall Panels

All wall panels were 6 ft. wide by 9 ft.–6 in. high. The 3–in. thick panels were reinforced with a single layer of welded wire fabric located at mid–depth. No direct connection existed between the adjacent transverse wall panels (Fig. 2); however, the ledger beam in the roof was continuous across the panels. A steel angle was used to connect the transverse and longitudinal wall panels in the corners of the building. Two steel angles were used to connect each wall panel to the foundation. Through bolts were used for all structural connections to the wall panels.

Roof Framing Members

Four types of wood members were used to construct the roof framing system. Ledger beams formed the transverse ends of the diaphragm. The 2 x 10 beams were connected directly to the transverse wall panels with four ½–in. diameter bolts per panel. Two 1¾ x 9½ in. micro–lams were used for the diaphragm chords. The chords were connected directly to the longitudinal wall panels also with four bolts per panel. There was no physical connection between the ledger beams and the diaphragm chords in specimen TU1.

Two 2 x 6 purlins spanned between the diaphragm chords in the transverse direction of the building. The purlins were spaced at 8 ft. on center. Blocking was provided by 2 x 4 sub–purlins, which were spaced at 2 ft. on center throughout the roof. Standard metal hangers were used to connect all purlins and sub–purlins to their supporting members.

Plywood Roof Diaphragm

A $^3/_8$–in. thick, structural grade plywood (C-D Exterior) was used as sheathing for the diaphragm. Ten sheets of plywood were connected to the roof framing members using 8d common nails. Three nail spacings were selected: 4 in. on center along diaphragm boundaries and along continuous panel edges parallel to the applied load, 6 in. on center along all other plywood edges, and 12 in. on center along intermediate framing members.

Foundation

The foundation used in this study was not intended to be representative of actual construction, but was designed to provide a stable base for the panels. Each base girder was nominally rectangular in cross-section with a 2–in. deep step cast into the top to facilitate placement of the wall panels. The foundation girders were tied directly to the laboratory floor.

EXPERIMENTAL PROGRAM

Lateral forces were applied at the elevation of the roof in the east–west direction near the third points of the diaphragm by two hydraulic actuators. The master ram was driven under displacement control and the forces applied by the master and slave rams were set equal.

Specimen TU1 was subjected to five stages of loading (Fig. 3). Three complete load reversals to the same displacement level comprised each loading stage. The amplitude of the master ram displacements ranged from ±0.3 in. during the first stage to ±2.1 in. during the fifth stage. The response of the specimen was monitored during the test with 57 displacement transducers (LVDTs) and 20 strain gages. Most of the instruments were located in the north–east quarter of the structure.

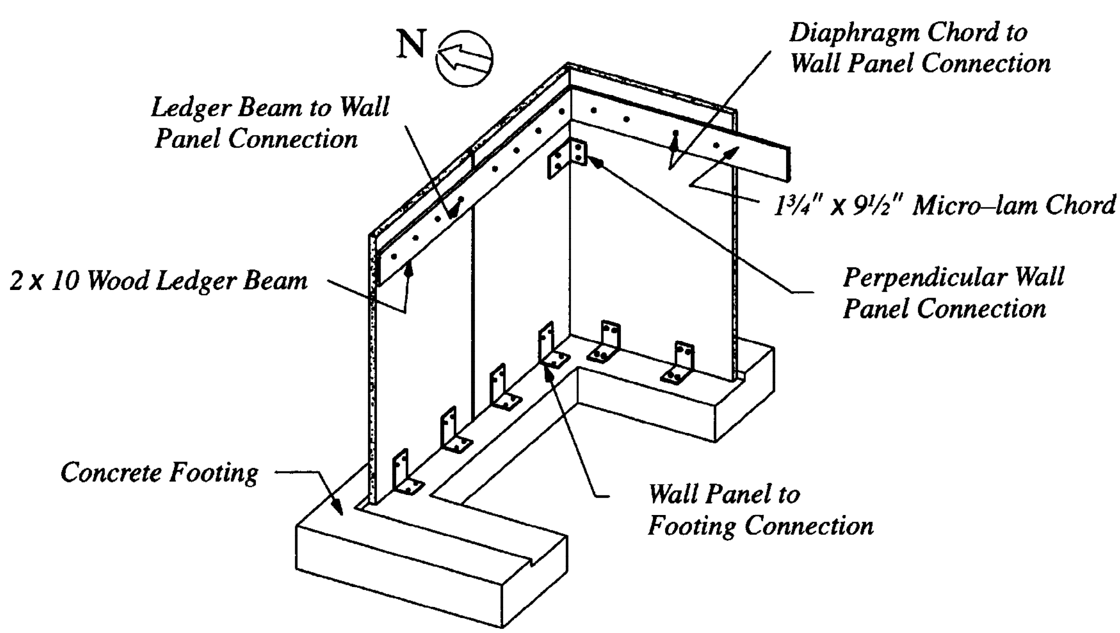

Fig. 2. Connections between Primary Structural Components in Specimen TU1

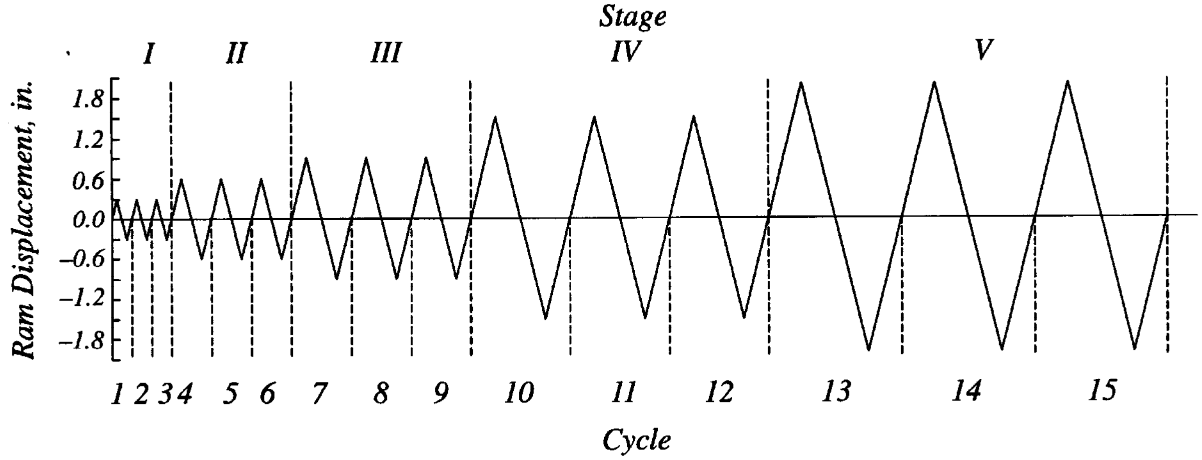

Fig. 3. Ram Displacement History for Specimen TU1

MEASURED RESPONSE OF THE ROOF DIAPHRAGM

The transverse diaphragm displacements were measured at five locations along the east side of the north half of the specimen (Fig. 4). Hysteresis curves for the complete loading history indicate the significance of the flexible diaphragm. The transverse displacements measured at the center of the diaphragm (TDD7) were approximately 1.5 times those measured at the south end of the longitudinal wall panel (TDD3).

Little evidence of damage was observed during the first three stages of loading. A slight reduction in stiffness with increasing levels of deformation may be seen at all locations in Fig. 4. The first indication of appreciable damage was observed during stage IV. Near the maximum positive displacement during cycle 10, the applied force dropped slightly when the longitudinal wall panel at the north-east corner of the specimen (WP4) cracked (Point A in Fig. 4). During subsequent cycles to this displacement level, modest degradation of both stiffness and strength was observed. Cracking of the other three longitudinal wall panels was also observed during stage IV; however, no indication of the formation of the cracks may be seen in the transverse displacement histories for the diaphragm.

During the final stage of loading, the specimen experienced significant pinching of the hysteresis curves and appreciable degradation of strength and stiffness. The reduction of strength was primarily due to the excessive buckling of the plywood panels at the south end of the diaphragm.

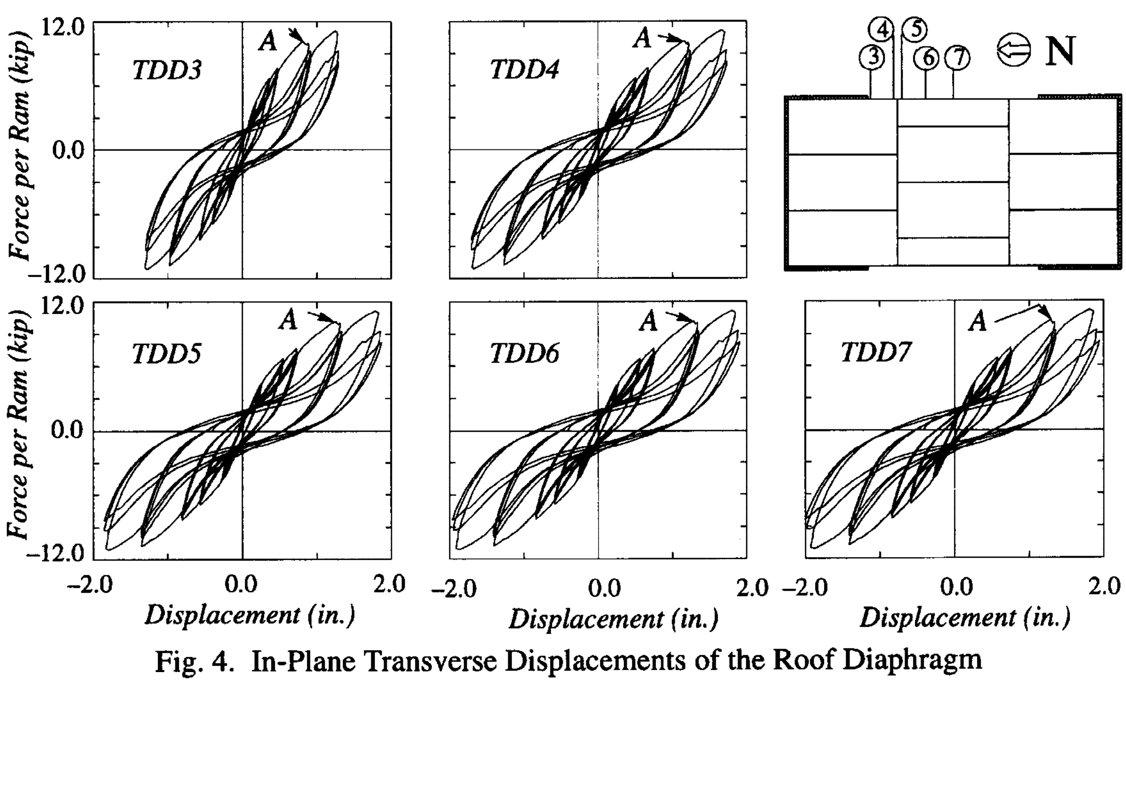

Fig. 4. In-Plane Transverse Displacements of the Roof Diaphragm

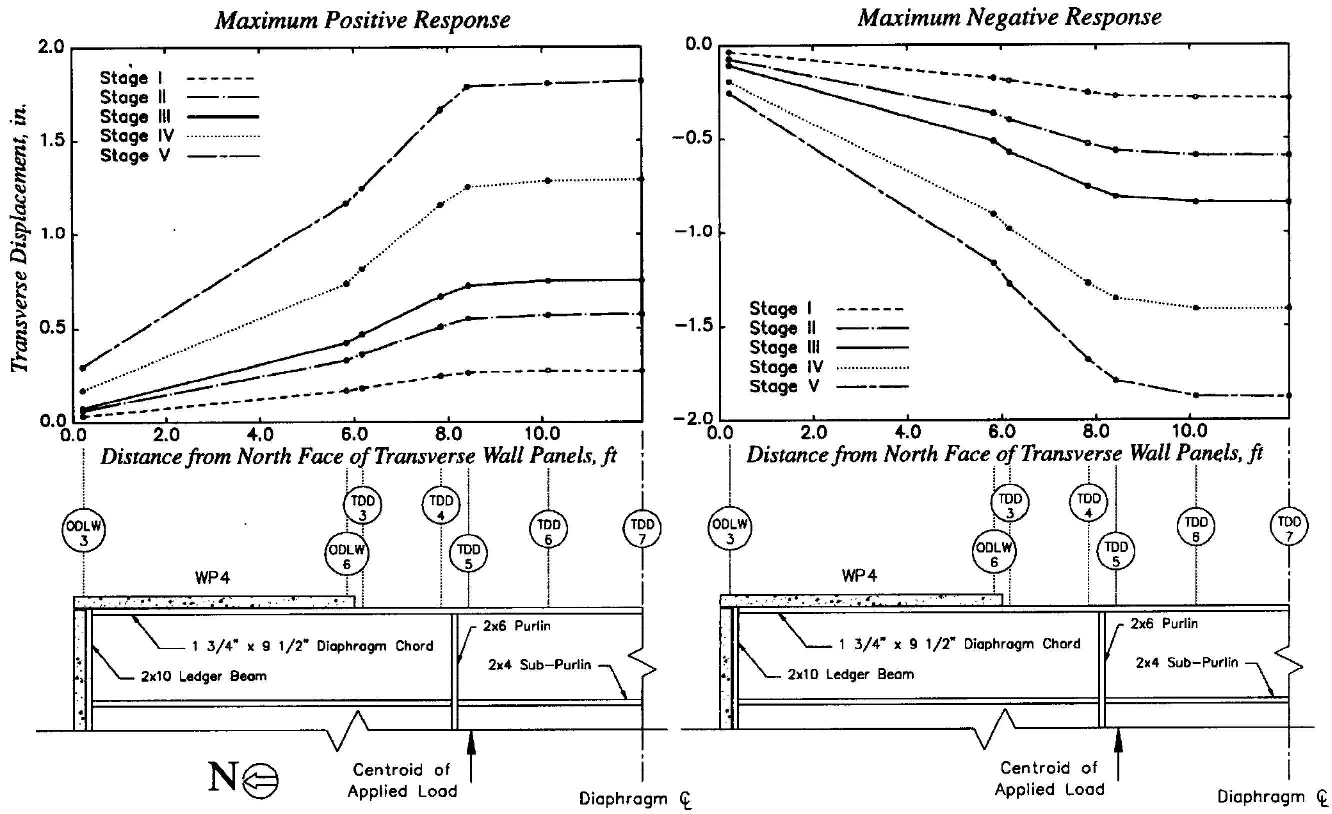

Fig. 5. Distribution of Displacements Measured Along the East Edge of the Diaphragm

Distributions of maximum positive and negative diaphragm displacements during each stage of loading are shown in Fig. 5. Data measured at the top of longitudinal wall panel WP4 are also plotted. As noted previously, the amplitude of the displacements increased toward the center of the diaphragm. The amplitudes of the displacements at the center of the diaphragm were approximately six times those measured at the top of the longitudinal wall panel near the end of the diaphragm. The general nature of the response was similar in the two directions of loading.

Two distinct discontinuities in the transverse displacement profiles may be observed in Fig. 5. The first discontinuity occurred near the centroid of the applied load, dividing the displacement profiles into two regions: the shear span and the center third of the diaphragm. Within the center third of the diaphragm, the measured displacements were nearly independent of location, while the displacements within the shear span increased with increasing distance from the transverse wall panels. The second discontinuity was located within the shear span at the south end of the longitudinal wall panel WP4. Measured displacements increased abruptly immediately south of the wall panel.

As indicated in the hysteresis plots (Fig. 3), the stiffness of the diaphragm decreased as the amplitude of the imposed displacements increased. The degradation of diaphragm stiffness may be quantified by comparing the effective stiffness during each stage of loading with the initial stiffness at a given location. The effective stiffness may be defined as the absolute sum of the maximum applied forces in the positive and negative directions of loading divided by the absolute sum of the maximum measured displacements in the two directions.

The effective stiffnesses determined from the initial loading cycle during each stage of loading were normalized with respect to the effective stiffness during stage I. The variations of the normalized effective stiffnesses are shown in Fig. 6 for each instrument located along the east edge of the diaphragm. The maximum transverse displacement of the diaphragm is used as the horizontal axis for all plots. The data indicate that the degradation in stiffness along the roof diaphragm was nearly independent of the location of the instrument. The average effective stiffness at the end of the test was approximately 35% of the initial effective stiffness.

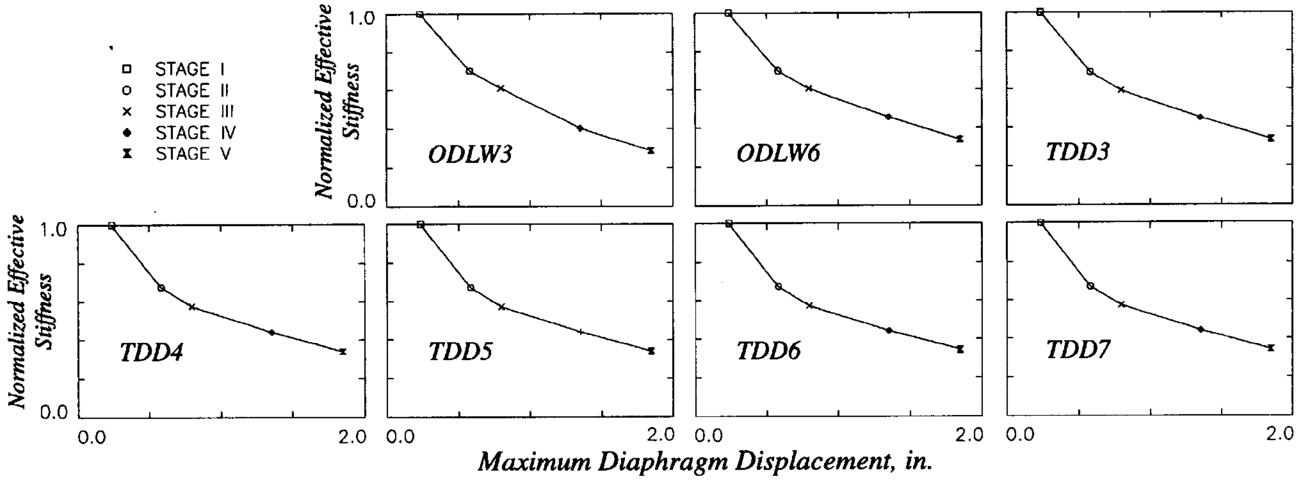

Fig. 6. Stiffness Degradation along Roof Diaphragm

MEASURED RESPONSE OF LONGITUDINAL WALL PANELS

The transverse (out-of-plane) displacements of longitudinal wall panel WP4 were monitored at three elevations over the height. Hysteresis curves for the complete loading history are shown in Fig. 7 for the instruments at mid-height and at the top of the wall panel. As expected, the amplitudes of the displacements increased with height above the base, and the amplitudes of the displacements measured along the south edge of the wall the panel exceeded those measured along the north edge of the wall panel.

Instrument ODLW5 was located immediately below the crack that developed in wall panel WP4 during stage IV. The wall displacement at this elevation decreased suddenly by approximately 0.11 in. after the crack formed (Point A in Fig. 7). The largest displacement experienced by the wall at this elevation occurred immediately before the panel cracked. In contrast, the displacements measured in the upper south corner of the wall panel (ODLW6) experienced an increase in displacement of approximately 0.05 in. when the crack formed (Point A in Fig. 7). The measured response at the top of the wall panel (ODLW6) was essentially the same as the transverse displacement of the diaphragm immediately adjacent to the panel (TDD3) as shown in Fig. 4. The shapes of the two hysteresis curves were similar and the amplitudes of the wall displacements were approximately 85% of the corresponding diaphragm displacements.

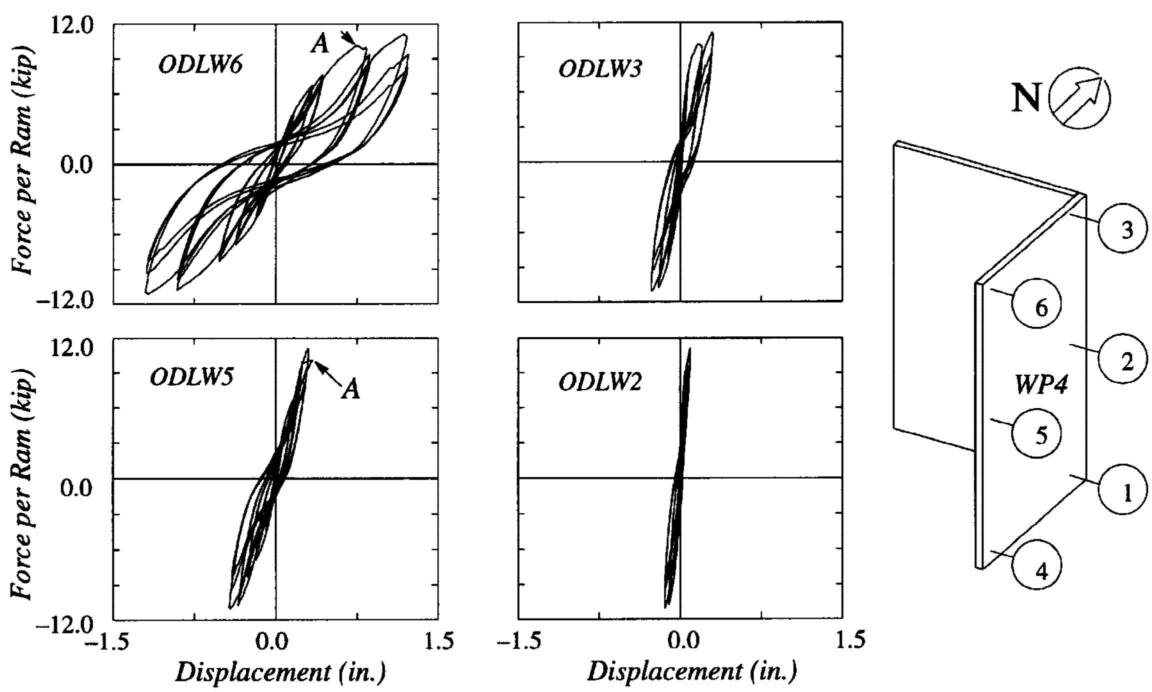

Fig. 7. Out-of-Plane Displacements of Longitudinal Wall Panel WP4

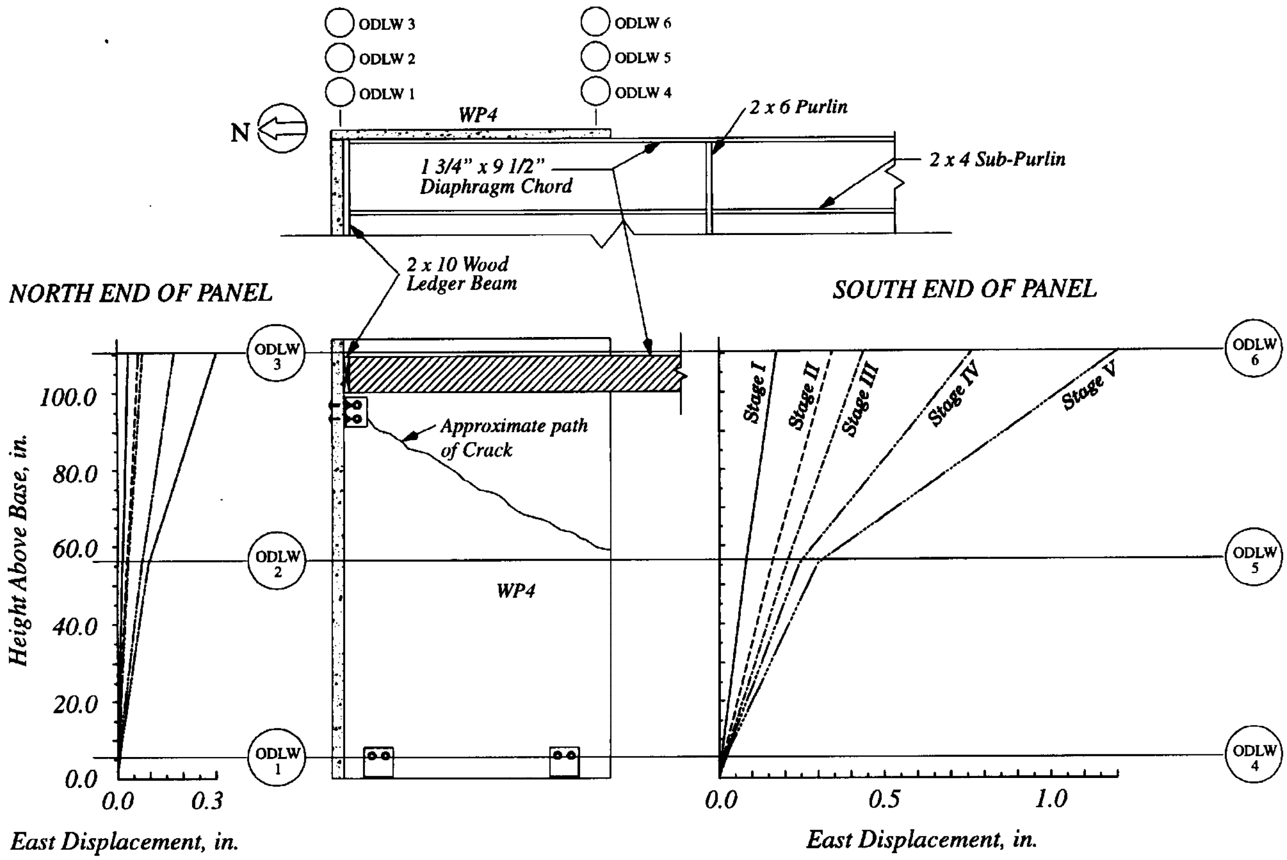

Fig. 8. Distribution of Transverse Displacement Measured along the Height of Wall Panel WP4

Distributions of the maximum positive displacements measured over the height of wall panel WP4 are shown in Fig. 8. An elevation of the wall panel (viewed from the west) and a plan view of the north-east corner of the diaphragm (viewed from the top) are included in the figure to assist with interpreting the location of the instruments.

As expected, the amplitudes of the transverse displacements of wall panel WP4 were smaller near connections to the adjoining structural members. Wall panel WP4 was connected to the footing along the bottom edge and to the adjacent transverse wall panel in the upper north corner. Displacements measured along both the north and south edges of the wall panel tended to increase with increasing height above the base. The amplitudes of the displacements measured along the south edge were approximately four times larger than those measured along the north edge throughout the entire loading history.

During the first three stages of loading, the amplitudes of the transverse displacements increased nearly linearly with height above the base, and the amplitude of the response was nearly equal in the positive and negative directions of loading. This implies that wall panel WP4 behaved essentially as a rigid body during the initial stages of loading and movement in the east–west direction may be attributed to rotation about the base of the panel. A large increase in the displacements at the top of the wall panel was observed during stage IV, however. A diagonal crack formed in wall panel WP4 during the first positive cycle to this displacement level. The approximate path of the crack is shown in Fig. 8. The two instruments located above the crack, ODLW3 and ODLW6, experienced appreciable increases in displacements with respect to the instruments located below the crack. Thus, the discontinuities in the transverse displacement profiles during stages IV and V may be attributed to the cracking of wall panel WP4 during stage IV.

CONCLUSIONS

A model of a tilt–up structural system was constructed and subjected to a series of lateral load reversals. The test specimen included reinforced concrete wall panels and a plywood roof diaphragm. The structure had an aspect ratio of two and was loaded in the transverse direction. The response of the system was controlled by the behavior of the flexible diaphragm.

Transverse displacements measured at the center of the roof were approximately six times larger than the displacements measured at the top of the end wall panels. The stiffness and strength of the diaphragm degraded as the amplitude of the imposed displacements and number of loading cycles increased. The effective stiffness of the diaphragm at the end of the test was approximately one–third of the initial effective stiffness. This reduction in stiffness was observed along the entire length of the diaphragm and did not depend on the location considered.

Measurements of transverse displacements taken over the height of a longitudinal wall panel indicated that the panels experienced nearly rigid–body rotation about its base during the early stages of loading. Cracking of the longitudinal wall panels was observed when the maximum transverse diaphragm displacement exceeded 1% of the height of the structure. The displacement profile became nonlinear after the crack formed.

The strength of the system was limited by buckling of the $^3/_8$–in. thick plywood panels in the roof diaphragm.

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