Seismic Behavior of Precast Parking Structure Diaphragms

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This paper presents a study of the role of diaphragm deformations in the seismic performance of precast parking structures. A prototype parking structure typical of structures in the Los Angeles area prior to the 1994 Northridge earthquake was studied. Nonlinear static analyses of the diaphragms and shear walls were conducted. The response of the diaphragms and shear walls in these analyses provided stiffness and strength properties for a dynamic analysis model of the structure. Significant issues related to the role of diaphragm deformations in the seismic performance of precast parking structures were identified. These include large drift demands on the gravity load system due to excessive diaphragm deformations; shear wall locations that cause the diaphragms to twist in plan, amplifying these deformations; and cross sections in critical locations in the diaphragms that have insufficient strength.

During the 1994 Northridge earthquake, nine parking structures in the Los Angeles area suffered severe damage, including six structures that experienced partial or complete collapse. Most of these parking structures were built in the late 1980s. Several of the damaged parking structures employed a precast concrete gravity load system with cast-in-place shear walls for lateral load resistance. Reinforced topping slabs acting compositely with the precast floor system were designed to serve as stiff diaphragms between the shear walls.

The shear walls in these parking structures appeared to be relatively undamaged. In two of the three precast parking
structures located within 2 miles (3.2 km) of the earthquake epicenter, only regions of the structures adjacent to the shear walls remained standing. The third structure, smaller in plan area, sustained only minor cracking in the slab and precast members.

Observations indicate that the damage to the parking structures may have been caused by failure of the gravity load system due to large lateral displacements (drifts) of the floors at regions away from the shear walls. Also, design procedures may have produced diaphragms with inadequate strength, and the diaphragms may have failed under the in-plane forces that developed during the earthquake.

This paper describes research on the seismic behavior of precast parking structure diaphragms. The research involves analysis of a prototype precast parking structure representing typical design and construction practice in Los Angeles prior to 1994 (see Fig. 1). The research examined: (1) the static in-plane behavior of the diaphragms to lateral forces; and (2) the influence of this behavior on the dynamic response of the prototype parking structure.

Elastic static analyses were performed to obtain information on diaphragm flexibility, including modes of deformation. Nonlinear static analyses were conducted to identify critical regions within the diaphragm. A final set of static analyses including detailed nonlinear models of critical regions between precast members was used to quantify the diaphragm behavior under seismic loading.

Nonlinear models of the diaphragms, based on the results of the static analyses, along with nonlinear models for the cast-in-place shear walls, were used to create a multi-degree-of-freedom dynamic analysis model of the entire parking structure. The dynamic analysis model was subjected to an array of ground motions. The resulting force and drift levels were compared to the levels anticipated in design. An evaluation of these forces and drifts was made and the seismic behavior of the structure was assessed.

The research considered only precast parking structures typical of design and construction practice in the Los Angeles area prior to the 1994 Northridge earthquake. The research focused on the influence of diaphragm flexural deformation on the dynamic response of these structures. Diaphragm strength issues not related to flexural behavior were not examined. Investigations of the strength of the lateral load resisting system (shear walls and diaphragms), including shear failure of the diaphragms or loss of lateral load transfer to the shear walls, are found elsewhere. Also, alternate explanations regarding failure of the gravity system due to large vertical accelerations of the structure have been proposed.

PRECAST PARKING STRUCTURES

Parking garages require a structure that is, in general, large in plan, durable, and exposed to the weather and environment. These requirements can be satisfied economically by precast, prestressed concrete structural systems. To minimize the internal stresses due to temperature change and shrinkage, and to facilitate ease of and speed of erection, simple connections are provided between the members.
Recent seismic design practice for precast parking structures uses a lateral force-based approach. In this approach, the components of the lateral load resisting system, including the floor diaphragms, the shear walls, and the shear wall foundations, are designed to resist code-specified lateral forces. The design approach assumes implicitly that the lateral load resisting system will be loaded beyond its elastic limit under the design level earthquake, and that inelastic behavior will occur.

Recent design practice assumes that this inelastic behavior will occur in the shear walls and they are designed and detailed accordingly. The other components of the lateral load resisting system are assumed to have sufficient strength to remain elastic under lateral forces corresponding to the ultimate strength of the shear walls. In particular, the floor diaphragms, which are not designed and detailed for inelastic behavior, are assumed to remain elastic.

Often the lateral load resisting system configurations in parking structures include only a few shear walls. To provide lateral load resistance to the structure at locations away from the walls, the rest of the structure (intended to resist only gravity loads) is tied to the walls by the floor diaphragms. For parking structures in regions of high seismicity, a reinforced topping slab, acting compositely with the precast floor elements, serves as the diaphragm. A rigid diaphragm assumption is employed in current design practice. Hence, the entire precast gravity load system, even at locations far from shear walls, is assumed to undergo the same drift as the shear walls.

The Uniform Building Code (UBC) specifies a pattern of equivalent lateral loads \( F_i \) over the height of the structure similar to the first vibration mode of a shear wall (see Fig. 2a). This loading is used to design the shear walls. The diaphragm design loads at each level \( F_{px} \) are specified using a related pattern of loads (see Fig. 2b).

When the diaphragm reinforcing steel in the topping slab is designed for the UBC specified pattern of loads, the amount of reinforcing steel increases over the height of the structure. A portion of the reinforcing steel (chord steel) is designed for in-plane bending due to the lateral loads transverse to the long dimension of the structure (see Fig. 2c).

Reinforcing steel (collector steel) is also provided in the topping slab to carry the lateral loads into the shear walls. The reinforcement that serves as chord steel for lateral loads in the transverse direction also serves as collector steel for lateral loads in the longitudinal direction (see Fig. 2d).

Flange-to-flange connectors between the double tees are often used to transfer diaphragm in-plane shear forces across the joints between the double tees. However, in typical practice in the Los Angeles area, flange-to-flange connectors are not used and shear reinforcement is included in the topping slab. The lateral loads are transferred from the floor system to the shear walls through dowels in the topping slab.

Prototype Precast Concrete Parking Structure

A prototype parking structure was designed for this study by a Los Angeles area design firm. The structure was designed to be typical of design and construction practice in the area before the 1994 Northridge earthquake. The structure is a typical three-bay side-by-side parking garage (single interior ramp) comprising five levels and measuring 320 x 189 ft (97.5 x 57.6 m) in plan (see Fig. 1).

Dual shear walls are located at each end of the structure for lateral load resistance in the transverse (short) direction. Individual shear walls, offset...
Table 1. Prototype diaphragm reinforcing schedule.

<table>
<thead>
<tr>
<th>Level</th>
<th>Inner chord</th>
<th>Outer chord</th>
<th>Additional at midspan</th>
<th>Transverse collector</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>7 - #6</td>
<td>11 - #6</td>
<td>3 - #5</td>
<td>4 - #6</td>
</tr>
<tr>
<td>3</td>
<td>9 - #6</td>
<td>13 - #6</td>
<td>4 - #6</td>
<td>5 - #6</td>
</tr>
<tr>
<td>4</td>
<td>10 - #6</td>
<td>15 - #6</td>
<td>5 - #6</td>
<td>5 - #6</td>
</tr>
<tr>
<td>R</td>
<td>14 - #5</td>
<td>21 - #5</td>
<td>7 - #5</td>
<td>7 - #5</td>
</tr>
</tbody>
</table>

![Fig. 3. Diaphragm reinforcing steel details: (a) typical detail at exterior spandrel; (b) typical detail at interior column.](image)

from the center due to the precast framing, are provided in the longitudinal (long) direction. The ramp at the ground level is a cast-in-place slab on fill with a small region at the upper end similar to the precast gravity load system. Each column is precast in a plant, transported and erected in one piece.

The precast gravity load system employs 24 in. (610 mm) deep double tee floor units spanning 63 ft (19.2 m) in the transverse direction of the structure. The double tees are supported along the column lines by precast spandrels. The bays in the transverse direction provide two flat regions flanking a ramp region. In this paper, these regions are referred to as diaphragm segments. At interfaces between the ramp landing and the flat segments, the double tees are supported by precast inverted tee beams.

Chord and collector mild reinforcing steel (Grade 60) is placed within the slab for each diaphragm segment as shown in Fig. 1. The amount of reinforcing steel per level is listed in Table 1. Flange-to-flange connectors are not included in accordance with typical practice in the Los Angeles area. The diaphragm reinforcing steel details at the spandrel girder are shown in Fig. 3a. The corresponding detail at the interior column line along the ramp is shown in Fig. 3b.

The shear walls are cast-in-place and designed according to UBC specifications in use prior to the Northridge earthquake. The details of the shear walls in the transverse direction of the structure are shown in Fig. 4a, while details of the shear walls in the longitudinal direction are shown in Fig. 4b. Changes in the UBC specifications put in place near the time of the earthquake require fewer longitudinal reinforcing bars in the shear wall boundary regions, and fewer ties for confinement. Therefore, a second set of shear wall details was generated (also shown in Fig. 4) using the 1994 UBC specifications.

**DIAPHRAGM BEHAVIOR UNDER LATERAL LOAD**

Two-dimensional nonlinear static analyses of the diaphragms were conducted to study the in-plane behavior of the diaphragms including the stiffness, strength, and failure modes. Planar analyses of diaphragms at each level of the prototype structure examined the response to transverse and longitudinal lateral loads. Loading directions, diaphragm segments, and the diaphragm segment interfaces, discussed later in this section, are shown in Fig. 5.

For this study, the in-plane resistance of the diaphragm was assumed to include only the resistance from the reinforced topping slab and the double tee flanges. Any supplemental resistance from the tee stems, spandrels and inverted tee beams, and the columns was neglected. The resistance of the shear walls to loads transverse to their length was neglected as well. The planar analyses focused on in-plane flexural behavior of the diaphragm, and the shear capacity across and between double tee units was assumed to be sufficient to avoid shear failure. The connection of the diaphragm to the shear walls was also assumed to be sufficient to carry the forces developed in the planar analyses.

For the planar analyses, the ramps were brought into the horizontal plane but were connected to the outer diaphragm segments only at the interface between the ramp landings and the outer diaphragm segments (see Fig. 5). Also, for the analyses of the diaphragms under loading in the longitudinal direction, the south longitudinal shear wall was moved to a location directly across from the north longitudinal direction to make the structure symmetric about its longitudinal axis.

**Flexible Diaphragm Behavior**

Elastic analyses were conducted to understand the deformation modes of the diaphragms. Two distinct diaphragm deformation modes were identified: in-plane bending due to lateral loads in the transverse direction; and in-plane twisting due to lateral loads in the longitudinal direction.
Transverse Direction — The elastic analyses of the diaphragms of the prototype structure indicated that significant flexural deformation of the diaphragms occurs under lateral loads in the transverse direction (see Fig. 6). This flexibility is due in part to the configuration of a typical precast parking structure, which is constrained by functional requirements. The parking and traffic requirements, combined with limits on the ramp slope of 5 to 7 percent, result in aspect ratios in plan of approximately 2:1 in the longitudinal direction.

This overall aspect ratio is not extreme, and can be modified by introducing additional bays of double tees to extend the width of the structure. However, the ramp partitions the floor into independent diaphragm segments. Each diaphragm segment can have aspect ratios in plan between 3:1 and 5:1. Shear walls in the Los Angeles area are often located along the perimeter of the parking structure for public safety reasons. This results in spans between shear walls equal to the length of the diaphragm segment. Diaphragm flexibility is a concern for aspect ratios of this magnitude.11

Longitudinal Direction — Partitioning of the floor diaphragm into segments by the ramps of the prototype structure also produces significant diaphragm deformations under lateral loads in the longitudinal direction. The individual behavior of each diaphragm segment renders the shear wall resistance eccentric to the center of mass of the diaphragm segment (see Fig. 7). Elastic analyses indicate a tendency for the diaphragm segments to twist in plane due to this eccentric load path. The twisting affects the structure unfavorably in three ways:

1. Tension due to flexure is added to the regions acting as tensile collectors for the shear walls, and this flexural behavior will cause yielding of the reinforcing steel at lateral loads less than the design loads.
2. The twisting deformation is sufficiently large that the transverse displacement due to twisting is significant.

3. The outer (north and south) segments separate from the ramp because the ramp does not twist, and columns intended to support both the ramp and an outer segment (see Fig. 3b) will not be able to do so if significant separation occurs.

**Identification of Critical Regions**

The elastic forces that develop in the diaphragm due to lateral loads in the transverse direction are shown in Fig. 8. Each diaphragm segment develops in-plane flexure. In-plane forces develop at the interfaces between diaphragm segments as a result of the end restraint provided to the outer segments by the ramp landing. In addition to the individual segment behavior, the continuity across the interfaces causes in-plane deformations associated with overall floor bending. These deformations create a resultant tension force in one outer segment and, due to symmetry, an equal compressive force in the other outer segment. The transfer of these forces between the outer diaphragm segments causes shear forces to develop at the interfaces between the ramp landings and the outer diaphragm segments.

A set of inelastic static analyses of preliminary coarse-mesh smeared-crack finite element models were used to obtain the extent of diaphragm damage (concrete cracking leading to steel reinforcement yielding). As shown in Fig. 8, damage occurs: (1) in the diaphragm segments where the tension from in-plane flexure of an individual diaphragm segment coincides with the tension from flexure of the overall floor diaphragm (i.e., the north segment in Fig. 8); and, (2) near the interfaces between the outer diaphragm segments and the ramp landings. Regions of the diaphragm under compression from flexure of the overall floor diaphragm have much smaller regions of damage (i.e., the south segment in Fig. 8).

Under lateral loads in the longitudinal direction, the major region of damage occurs where tension due to in-plane twisting of the outer diaphragm...
High tension due to twisting and collector tension causes region of significant damage.

Fig. 9. Diaphragm damage under longitudinal lateral load.

The shaded area shown

Damage due to high shear and flexural restraint at interface.

Interface shear forces due to ramp thrust

Tension or compression depending on intensity drift

Ramp thrust

Line of Symmetry

Segment coincides with the tension collector forces (see Fig. 9). The diaphragm interfaces are subjected to tension (due to diaphragm twisting) and shear forces (due to ramp thrusts).

For loading in either direction: (1) the rotational restraint provided at the interfaces between the ramp landings and outer segments tends to reduce the in-plane deflections and the in-plane moment in the outer segments; (2) the interfaces develop high forces due to this restraint and subsequently crack; and, (3) the beneficial effects (to the outer segments) of the restraint are reduced by this cracking.

Section Strength and Stiffness Evaluation

Strength analyses of the critical regions indicate that the cross section details of the precast floor diaphragm system tend to concentrate the cracking in the joints between precast members. Therefore, joints between precast members in each critical region play a crucial role in the diaphragm behavior. The critical cross sections are: (1) joints between the ends of the double tees along the inverted tee beams at the interfaces between the ramp landings and the outer diaphragm segments, referred to in this paper as seams; and, (2) joints between double tees in the regions of high tension within the diaphragm segments.

Diaphragm Seam Section — The details of the joint at the interfaces between the ramp landings and outer diaphragm segments are shown in Fig. 10. Stirrups (not shown) protruding from the inverted tee enable it to act compositely with the topping slab. However, a reduced cross section (seam) consisting only of a topping slab exists on either side of the inverted tee beam. Staggered reinforcing bars (#3 reinforcing bars 5 ft (1.52 m) long spaced at 18 in. (460 mm) for the prototype structure) are provided in the cast-in-place topping slab for force transfer across this joint. The seam enables the ramp landings to restrain in-plane bending of the outer diaphragm segments. This restraint decreases when the seam cracks, and decreases even further when the ultimate strength of the reinforcing steel crossing the seam is reached.

Double Tee Joint Section — The typical double tee joint detail is shown in Fig. 11a. The joint between the double tee panels is weaker than the region within the double tee because the joint consists of only the reinforced topping slab. Typically, the strength of this critical cross section is further reduced by a troweled construction joint used for shrinkage crack control. The topping slab will crack (or is already pre-cracked due to shrinkage and temperature effects or concentrated moving gravity loads) at these sections in response to in-plane diaphragm forces that develop during lateral loading. After cracking, the diaphragm behavior is dominated by deformation of the reinforcing steel crossing the cracked joint. Little cracking, if any, takes place within the double tee itself.

Two distinctly different types of steel reinforcement cross the joint between double tees: steel reinforcing bars and welded wire mesh. The welded wire mesh is not intended to provide in-plane flexural strength to the diaphragm segments. However, in
comparison to the relatively small amount of diaphragm reinforcing steel, the amount of wire mesh is quite large; thus, its contribution to the flexural behavior of the diaphragm is considered. However, the mesh is inherently brittle due to the manufacturing process required to draw the wire to its small diameter. Therefore, moderate inelastic deformations will fracture the mesh.

Accurate modeling of the diaphragm in-plane behavior involves the assessment of the load-deformation response of the steel reinforcement. The distance over which each steel bar deforms independently between the double tee panels is a critical parameter of the response. This distance is controlled by the bond relationships between the steel and topping slab. An equilibrium-based bond stress-slip model was used to determine nonlinear force-deflection relationships for the steel reinforcing bars and welded wire mesh between the double tees.

A static model for precast diaphragms was developed that captured the stiffness and strength of the joints between double tee members. The model, termed a panel model, consists of discrete double tee panels joined together by joint strips (see Fig. 11b). Each panel represents a double tee flange with an uncracked topping slab. Each joint strip represents a precracked joint between double tees. The joint strip contains ductile elements to model the steel reinforcing bars in tension and brittle elements to model the welded wire mesh in tension. These elements use the nonlinear force-deflection relationships mentioned above.

**Static Modeling of Precast Concrete Floor Diaphragms**

The panel model was employed within two separate analytical frameworks: (1) a fiber element approach to examine diaphragm behavior under lateral loads in the transverse direction; and, (2) a finite element approach for diaphragm behavior under lateral loads in the longitudinal direction. The interfaces were handled differently for each direction. For response in the transverse direction, individual diaphragm segments were analyzed with end forces to model the seam forces at the diaphragm segment interfaces. For response in the longitudinal direction, finite element models of the entire floor diaphragm, including nonlinear models of the seam cross section, were analyzed.

**Static Modeling of Transverse Behavior** — A fiber element approach was employed to develop models for the transverse load behavior of diaphragm segments using the structural analysis software DRAIN-2DX. The concrete fiber model incorporates both the compression and tension resistance of concrete (see Fig. 12) to model the uncracked concrete within the double tee panel region. A concrete fiber model with no tension resistance is used to model the joints between precast members (assumed to be precracked). The joint region has a finite width in the fiber model, which was chosen to be the elastic development length of the chord steel. The model neglects shear deformations within the double tee panels and neglects shear sliding between adjacent double tee flanges in the joint region.

Each diaphragm segment is analyzed independently under distributed lateral loads. Seem forces developed at the diaphragm segment interfaces are included as forces acting on the ends of the diaphragm segment to produce accurate in-plane displacements. These end forces are estimated by integrating stress distributions from results of an elastic finite element analysis of the entire floor system subjected to a uniform transverse inertial loading. The normal stresses across the seams are resolved into a
normal force and moment; the shear stresses across the seams are resolved into a shear force.

The end forces and lateral loads are treated as proportional until the shear capacity of the seam cross section is reached. At this point, the end forces are held constant while the lateral loads increase. The shear capacity of the seam cross section is determined using a shear-friction approach based on the strength of the reinforcing bars and wire mesh that cross the seam. A coefficient of friction, \( \mu \), of 1.0 was assumed.

The diaphragm segments (north, south, ramp) of each floor level of the prototype structure were analyzed using the panel model. The static response of the north and south diaphragm segments for the third floor are shown in Fig. 13. The figure shows the diaphragm segment lateral load plotted against the deflection of the diaphragm segment at a location halfway between the transverse shear walls, i.e., the diaphragm segment midspan. The deflection of the diaphragm segment midspan is given relative to the ends of the segment (near the transverse shear walls). The repeated unloading branches in the force-displacement response are caused by the progressive fracture of welded wire mesh. Note that the north diaphragm segment, which is under tension from flexure of the overall floor diaphragm, has less lateral stiffness and strength than the south diaphragm segment, which is under compression.

**Static Modeling of Longitudinal Behavior** — In the longitudinal direction, the entire diaphragm at a floor level was modeled. The panel model is created using two-dimensional finite elements to model the double tee panel regions, and axial spring elements to model the reinforcing bars and wire mesh together with contact elements to model compression at the joints between double tees. Shear transfer is accomplished through friction capabilities of the contact elements and through shear springs that model the dowel action of the reinforcing bars.

The model of the seam in the interface region is similar to the model of
the joint region with the following differences: (1) the seam cross section is initially uncracked; (2) the force-deflection relationship for the reinforcement crossing the seam includes bond stress-slip behavior of the 5 ft (1.52 m) reinforcing bars; and, (3) nonlinear shear resistance of the seam is included. The seam shear strength was determined from a shear-friction calculation based on the strength of the reinforcing bars and wire mesh.

To study the effects of the restraint provided by the interface between diaphragm segments on the behavior of the outer diaphragm segments, analyses of diaphragms with intact seams and failed seams were performed. Fig. 14a shows the deflected shape of the diaphragm at incipient wire mesh fracture for an intact seam analysis. The outer diaphragm segment deflected shape has reverse curvature due to interface restraint. Inelastic deformation is concentrated between the double tee panels in the region of high tension resulting from twisting and collector forces. Inelastic deformation also occurs between the panels in the regions of the outer diaphragm segments near the interfaces with the ramp landings (i.e., near the seams). In contrast, a diaphragm with a failed seam has less pronounced reverse curvature in the outer diaphragm segments, with greater diaphragm flexibility (see Fig. 14b).

Fig. 15 compares the response of the preliminary and panel diaphragm models under longitudinal lateral loads. The figure shows the lateral load (expressed in terms of an equivalent acceleration) plotted against the deflection of the diaphragm. The deflection of the diaphragm is a relative displacement between the end of the diaphragm near the ramp landing (shown in Fig. 14b) and the longitudinal shear walls.

The preliminary (smeared crack) model provides an upper bound to the diaphragm resistance. The response of the panel model with the nonlinear seam model is bounded by the response of the panel model with the intact seam (i.e., for initial behavior), and the response of the panel model with the failed seam (i.e., for ultimate behavior). The increased resistance of the model with the intact seam indicates the potential for improving diaphragm behavior by strengthening the seam cross section.

**Ramp Interaction Between Levels**

The traffic ramps form a secondary load path through the structure for longitudinal lateral loads. For certain
ramp configurations (single and double helix), this lateral load subsystem behaves similarly to a truss structure and is recognized as a means to carry lateral loads due to wind. Relative displacement between the floors (story drift) in the longitudinal direction, therefore, will introduce a concentrated force, or ramp thrust, to the diaphragm in addition to the lateral loads from inertial effects. In the prototype structure, the interaction of the ramps between adjacent floors will tend to restrain or amplify the displacement of a level, depending on the overall drift profile of the structure and the instantaneous direction of the earthquake motion longitudinal to the structure.

Assuming each floor level has a greater drift than the one below it (this assumption may be reasonable for most structures, but may not be reasonable for structures with flexible diaphragms due to the importance of diaphragm-driven modes of vibration), two cases termed up-ramp (see Fig. 16a) and down-ramp motion (see Fig. 16b) can occur. Both cases involve restraint of floor level drift from the ramp below and amplification of floor level drift from the ramp above. However, in the up-ramp case, the ramps are in tension while in the down-ramp case the ramps are in compression.

The force delivered to the diaphragms is opposite in sense; ramp tension tends to compress the diaphragms (Fig. 16c); and vice versa (Fig. 16d). The force delivered to the diaphragm by the ramp is limited by the strength of the double tee joint at the ramp landing in tension and the shear strength of the seam for ramp compression.

Properties for Dynamic Model

The results of the panel model analyses performed on each floor of the prototype structure were used to create spring properties for a nonlinear multi-degree-of-freedom dynamic analysis model. The flexibility of each spring represents the relative displacement between a diaphragm segment and a shear wall degree-of-freedom as a result of forces in the diaphragm. In the longitudinal direction, the static analyses of the diaphragms under lateral loads were augmented by static analyses of the diaphragms under thrust loadings from the ramps to obtain quantitative measures of the stiffness and strength properties of the diaphragm segments.

DYNAMIC RESPONSE OF PARKING STRUCTURES TO EARTHQUAKE LOADS

Two multi-degree-of-freedom dynamic analysis models of the prototype structure were developed, one for seismic loading in the transverse direction of the structure and one for loading in the longitudinal direction. Each degree of freedom in the model represents the displacement of: (1) a shear wall at a floor level; (2) the middle of a ramp (at mid-story); or (3) the midspan of an outer diaphragm segment (transverse model) or the end of a floor diaphragm in the region near a ramp landing (longitudinal model).

Dynamic results are presented for the prototype structure subjected to a commonly used earthquake ground motion (El Centro scaled by a factor of 1.5) and a Northridge ground motion (Sylmar). The dynamic analysis results are compared to the behavior of the structure considered in design, and the influence of diaphragm deformations is shown.
Transverse Seismic Response

Fig. 17a shows the displaced shape of a transverse section through the structure at the diaphragm midspan at the time of maximum drift under the Sylmar ground motion. Differential displacement exists between the leading diaphragm segment on the second floor and the first story ramp. The drift profile of the north diaphragm segment is reproduced in Fig. 17b along with the shear wall drift and the UBC design drift. The UBC anticipated displacements are based on an elastic cracked section analysis of the shear wall subjected to the code-specified distribution of design lateral loads. The resulting elastic displacements at each level are factored by the UBC multiplier \( \frac{3}{8} R_w \) to arrive at the design displacements.

Three important results are shown: (1) the critical story drift is in the first story; (2) the diaphragm displacements are significantly larger than the shear wall displacements; and, (3) the Sylmar ground motion produces significantly larger drifts than the design earthquake.

Fig. 17c shows the same results for the scaled El Centro ground motion. Note that even when the shear wall displacements are similar to those anticipated in design, locations at the middle of the diaphragms undergo much larger displacements. The number of diaphragm and shear wall yield events for the transverse response to the Sylmar ground motion are given in Table 2. The results indicate that the diaphragms yield as often or more frequently than the shear walls.

Table 2. Yield events: transverse Sylmar ground motion.

<table>
<thead>
<tr>
<th>Level</th>
<th>Shear wall</th>
<th>North diaphragm</th>
<th>South diaphragm</th>
<th>Ramp diaphragm</th>
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<tr>
<td>2</td>
<td>3/4</td>
<td>5/0</td>
<td>1/4</td>
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</table>

Longitudinal Seismic Response

Fig. 18 shows displacement-time histories for the diaphragms at the east end ramp landings of the roof (fifth floor) and second floor of the proto-

Fig. 17. Displaced shape at maximum transverse drift: (a) transverse section (Sylmar ground motion); (b) comparison with shear wall drift (Sylmar ground motion) and UBC; (c) comparison with shear wall drift (El Centro ground motion) and UBC.
type structure under the scaled El Centro ground motion. For comparison, the roof displacement at the longitudinal shear walls is also included. The displacement histories have been annotated to show the frequent yield excursions of the diaphragm. In contrast, the shear walls (1991 UBC design) do not yield. During the time of elastic response (prior to 2 seconds), the diaphragm displacements are approximately five times that of the shear walls. After the diaphragm yields, the diaphragm displacements are an order of magnitude larger than the shear wall displacements.

Fig. 19 shows the displaced shape of a longitudinal section through the structure at the time of maximum drift under the scaled El Centro ground motion. The figure shows displacements of the longitudinal shear walls and the displacements at either end of the structure. The diaphragm displacements at the ends of the structure are much larger than the shear wall displacements. The critical story drift occurs in the first story between the free superstructure and the fixed substructure. Displacements at either end of the ramp (for example, west roof and east fourth floor) are similar in magnitude. The greater magnitude of the ramp/diaphragm displacements compared to the shear wall displacements implies that the tie action provided by the ramps does not extend to the shear walls.

This behavior is explained by the significant flexibility of the diaphragm segments due to twisting leading to inelastic softening. Only the inertial forces developed near the shear walls are carried by the shear walls. The remainder of the inertial forces are attracted to the continuous ramp, which acts like a truss to carry the lateral forces to the ground. After the shear capacity of the seams in the interfaces between the outer diaphragm segments and ramp landings is reached, the ramp diaphragm segments (ramps and ramp landings) are isolated from the outer diaphragm segments, and hence, the shear walls.

The response to the Sylmar ground motion is similar. However, as Table 3 shows, the maximum story drifts for the ground motions, the Sylmar response is much larger in magnitude.

Fig. 20 shows a plan of maximum displaced shape under longitudinal loading for a typical floor of the prototype structure during Sylmar motion. The figure indicates that the columns supporting both the ramps and the outer diaphragm segments are subjected to differential transverse displacements due to twisting of the outer diaphragm segments. The columns will be unable to accommodate the displacements of both the outer segments and the ramps, and the spandrel girders of one of the diaphragm segments will pull off the column corbels.

**COMPARISON WITH UBC ANTICIPATED BEHAVIOR**

In this section, the transverse response of the prototype structure to the Sylmar and scaled El Centro ground motions is compared to the seismic behavior anticipated in recent design practice, and findings regarding the accuracy of the anticipated response are provided. The 1991 UBC is taken to represent design practice in the Los Angeles area prior to the Northridge earthquake.

The maximum displacements of the structure from the dynamic analyses are compared to the design displacements in Table 4. In Table 5, the peak story drifts along each profile for the Sylmar and scaled El Centro ground motion are compared with the UBC story drift limit for the prototype structure. The UBC story drift limit is calculated as $(0.04/R_w) \times (3/8) R_w$.

The results indicate that the response of the prototype structure is much different than anticipated in design practice based on the UBC. The displacements and story drifts are

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**Table 3. Maximum longitudinal story drift (percent).**

<table>
<thead>
<tr>
<th>Story</th>
<th>Sylmar Shear wall</th>
<th>Sylmar Diaphragm</th>
<th>El Centro Shear wall</th>
<th>El Centro Diaphragm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.10</td>
<td>5.39</td>
<td>0.08</td>
<td>1.45</td>
</tr>
<tr>
<td>2</td>
<td>0.12</td>
<td>0.38</td>
<td>0.09</td>
<td>0.52</td>
</tr>
<tr>
<td>3</td>
<td>0.13</td>
<td>0.37</td>
<td>0.10</td>
<td>0.51</td>
</tr>
<tr>
<td>4</td>
<td>0.12</td>
<td>0.33</td>
<td>0.09</td>
<td>0.29</td>
</tr>
</tbody>
</table>
much larger, even for the scaled El Centro ground motion. The drifts in the first story are quite large, a location where the gravity load system may be least able to endure them. Moreover, the differential displacements at the interior columns produced by relative displacement of the ramp diaphragm segments with respect to the outer diaphragm segments creates large deformation demands on the gravity load system. The elevation of the ramp between floor levels creates short column lengths, thereby increasing the local deformation demand.

**UBC Shear Wall Design Modifications**

The 1994 UBC shear wall design has less flexural strength than the 1991 UBC design due to differences in reinforcing requirements (about a 10 percent reduction for the prototype structure). This design change was expected to have beneficial effects on the behavior of the prototype structure by allowing the shear walls to yield at lower levels of seismic load, and thus increasing the possibility that the diaphragms will remain elastic. A dynamic analysis of the prototype structure with the 1994 UBC shear wall design was subjected to Sylmar ground motion in both the transverse and longitudinal directions. The results were compared to the results for the structure with the 1991 UBC shear wall design.

Under transverse load, the 1994 UBC design has larger shear wall displacements — 5.6 in. (142 mm) at the roof — compared to 4.4 in. (112 mm) for the 1991 UBC design. Unlike the 1991 UBC design, which does not yield at the third and fourth level, the 1994 UBC design had a limited number of shear wall yield events (three events) at the third level. However, the effect of the shear wall design change on the inelastic behavior of the diaphragms was inconsequential.

Under longitudinal load, the 1994 UBC design resulted in greater shear wall displacements; however, yielding of the shear walls occurs only at the base. Though the diaphragm deformations are reduced, the total displacements at the ends of the structure are relatively unchanged. Hence, the 1994 UBC shear wall design modifications do not eliminate the influence of diaphragm deformation on the dynamic response of the prototype structure.

**CONCLUSIONS**

The following conclusions are made relative to the effect of diaphragm deformations on the seismic performance of precast parking structures:

1. The individual diaphragm segment flexural deformations are too large. In current practice, the gravity load system is assumed to undergo the same drift as the shear walls. However, diaphragm deformations cause story drifts several times larger than the shear wall story drifts. It is doubtful that the gravity load systems can safely meet the drift demands determined by this study. Diaphragm flexural deformations can be reduced by: (1) reducing the aspect ratio in plan; (2) moving the transverse shear walls (or adding lateral load resisting frames) to the interior of the ramp landings, thereby significantly decreasing the diaphragm span; (3) utilizing a design procedure that recognizes that the diaphragm is composed of diaphragm segments.

**Table 4. Comparison of maximum displacements to UBC design displacements: transverse ground motion (in.).**

<table>
<thead>
<tr>
<th>Level</th>
<th>UBC Shear wall</th>
<th>UBC Diaphragm</th>
<th>Sylmar Shear wall</th>
<th>Sylmar Diaphragm</th>
<th>El Centro Shear wall</th>
<th>El Centro Diaphragm</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.27</td>
<td>1.12</td>
<td>5.83</td>
<td>0.73</td>
<td>2.10</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.71</td>
<td>2.26</td>
<td>5.09</td>
<td>1.52</td>
<td>1.31</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.26</td>
<td>3.50</td>
<td>5.11</td>
<td>2.39</td>
<td>3.12</td>
<td></td>
</tr>
<tr>
<td>R</td>
<td>1.85</td>
<td>4.38</td>
<td>7.33</td>
<td>3.31</td>
<td>5.14</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1 in. = 25.4 mm.

**Table 5. Comparison of maximum drifts to UBC design drifts: transverse ground motion (percent).**

<table>
<thead>
<tr>
<th>Level</th>
<th>UBC Shear wall</th>
<th>UBC Diaphragm</th>
<th>Sylmar Shear wall</th>
<th>Sylmar Diaphragm</th>
<th>El Centro Shear wall</th>
<th>El Centro Diaphragm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.50</td>
<td>0.85</td>
<td>4.35</td>
<td>0.60</td>
<td>1.57</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.50</td>
<td>0.96</td>
<td>1.04</td>
<td>0.71</td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.50</td>
<td>1.03</td>
<td>1.07</td>
<td>0.78</td>
<td>1.62</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.50</td>
<td>1.09</td>
<td>2.64</td>
<td>0.81</td>
<td>2.42</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 20. Schematic of dynamic analysis results in longitudinal direction for Sylmar ground motion — plan of third floor at t = 4 seconds.
2. Inelastic diaphragm behavior occurs. Diaphragm deformations increase dramatically when the diaphragms yield, exacerbating the effects of the elastic diaphragm flexibility. This inelastic behavior occurs because: (1) the strength of the diaphragms is too small relative to the strength of the shear walls; (2) twisting of the diaphragms increases the possibility of yielding; (3) the interaction of the ramp with adjacent floors due to interstory displacements may amplify the force levels in the diaphragms. Inelastic diaphragm behavior results in large story drifts for which the gravity load system was not designed. In a capacity-based design approach, the diaphragms would be protected from inelastic action (yielding), because they are not detailed for ductile behavior. However, the current practice of designing diaphragms and shear walls for the same level of forces does not protect the diaphragms from yielding, and can lead to diaphragm failure. The strength of the diaphragms should be increased by: (1) increasing the reinforcing steel in the topping slab; or, (2) possibly using longitudinal post-tensioning of the floors.

3. Large drift demands are imposed on the first story of the gravity load system. The dynamic response of the prototype structure is significantly influenced by diaphragm deformations. As a result, the first floor has almost the same lateral displacement as the roof. Hence, the first story of the structure has a very large story drift demand. The lower floor diaphragm, with the least reinforcement according to recent design practice, has a significant influence on the drift demands on the first story of the gravity load system. It may be necessary to provide the columns of the first story gravity load system with large drift capacity.

4. Adverse deformation modes exist. In-plane twisting of the diaphragm under longitudinal loads may adversely affect the diaphragm behavior by amplifying the demands on the diaphragm reinforcing steel and imposing differential transverse displacement demands on columns supporting the outer segments and ramps. To eliminate twisting, shear walls (or lateral load resisting frames) should be provided to support each diaphragm segment without eccentricity between the center of mass of the diaphragm segment and the center of rigidity of the shear walls (or frames) supporting the diaphragm segment. Span lengths between transverse shear walls in the longitudinal direction should be limited if twisting can occur.

5. The diaphragm segment interface does not remain intact. The diaphragm behavior is strongly influenced by the strength of the floor in the interface region between the ramp landings and the outer diaphragm segments. The end restraint provided to the diaphragm segments by the interface region is crucial because it significantly reduces the diaphragm flexibility. In recent practice, the interface regions contain modest amounts of reinforcing steel because the forces carried across the seams are not adequately considered. The details of the seam in the diaphragm segment interface region should be changed to include more flexural and shear reinforcement.

**DESIGN IMPLICATIONS**

The research presented in this paper was not intended to develop specific recommendations for the seismic design of diaphragms in precast parking structures. The paper addressed the static in-plane behavior of the precast parking structure floor diaphragms to lateral forces, and the influence of this behavior on the seismic response of precast parking structures. As shown, large flexural deformations of floor diaphragms, inelastic diaphragm behavior, and adverse diaphragm deformation modes can contribute to inadequate seismic performance, and should be avoided in design.

In some cases, improved diaphragm behavior can be gained by selecting a configuration of shear walls (or lateral load resisting frames) that reduces diaphragm spans and eliminates eccentric diaphragm load paths. In most cases, precast parking structures should be designed so that inelastic behavior occurs in the shear walls (or frames) only, and the diaphragms remain elastic. The current practice of designing diaphragms and shear walls (or frames) for the same level of forces does not protect the diaphragms from yielding.

To ensure that the diaphragms remain elastic, the ultimate strength of the shear walls (or frames) must be accurately estimated, and the diaphragms must be designed to remain elastic under the corresponding diaphragm loads. A recent PCI report suggests possible approaches to design precast floor diaphragms to remain elastic under seismic loading. It should be noted, however, that even when the diaphragms remain elastic, large diaphragm deformations can contribute to large story drift demands on the gravity load system and these deformations should be considered in design. Ongoing research at the University of Notre Dame and the University of Wisconsin should provide additional information on the seismic design of diaphragms for precast parking structures.

**ACKNOWLEDGMENT**

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