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EVALUATION OF PARKING GARAGE RESPONSE DURING THE 1994 NORTHRIDGE EARTHQUAKE

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ABSTRACT

Structural analyses of two precast parking garages located less than two miles from the epicenter of the 1994 Northridge earthquake are summarized. The buildings have common structural features: structural walls comprise the lateral–load resisting systems and precast concrete members form the gravity–load resisting systems. Portions of both garages collapsed during the earthquake. Two possible failure scenarios which highlight the importance of diaphragms as part of the lateral–load resisting system are presented.

KEYWORDS

Precast concrete parking garages; diaphragms; structural walls; lateral–load resisting system; earthquake damage; structural collapse.

INTRODUCTION

Precast parking garages were susceptible to structural damage during the Northridge earthquake (Iverson and Hawkins, 1994). This paper describes the results of an investigation to evaluate the seismic response of parking garages in the epicentral region. Two parking garages that sustained significant damage during the earthquake are presented to illustrate potential modes of failure. Analyses of the garages have focused on the influence of the diaphragm on the response of the structural systems.

DESCRIPTION OF THE PARKING GARAGES

The two parking garages discussed in this paper were less than five years old at the time of the earthquake and were located within 2 miles of the epicenter. Cast–in–place reinforced concrete walls formed the lateral–load resisting system in both buildings. Precast concrete columns, beams, and double tees comprised the gravity–load framing system. The double tees were overlain with a topping slab to form the floor diaphragms. Both garages were supported on pile foundations.

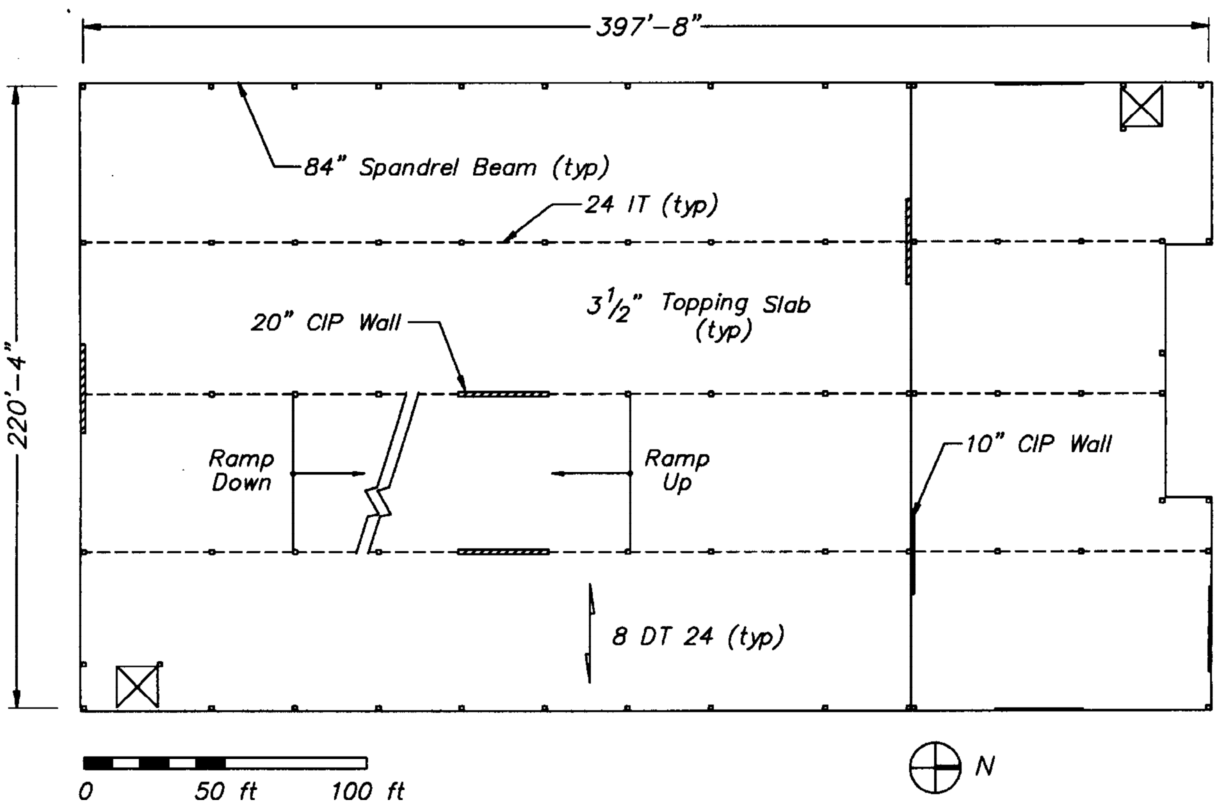

Fig. 1 Second–Floor Plan of Parking Garage A

Parking garage A was approximately 400 ft. by 220 ft. in plan (Fig. 1). It consisted of two adjacent and independent structures, physically separated by an expansion joint. Parking was provided on three levels: one on grade and two supported levels. The ramps that connected the parking levels were located near the center of the south section of the building. Both sections of the garage contained two structural walls in each direction. The walls were 20–in. thick in the south section, and 10–in. thick in the north portion of the building. All walls were approximately 30 ft. long, corresponding to an aspect ratio of 0.75. Cast–in–place beams extended beyond the walls at the elevations of the floors and served as drag struts. The drag struts were positioned on one side of the wall at some locations and on both sides of the wall at others.

All columns were precast in a single two–story section and connected to the foundation with four, 1–in. diameter bolts. The columns supported interior inverted tee beams and exterior spandrel beams on corbels. Ledges on these beams carried the double tees that formed the floor system. Elastomeric pads were placed on the corbels and beneath the stems of the double tees. There was no direct connection between the flanges of adjacent double tees.

The 3½–in. thick topping slab was reinforced with welded wire fabric. Its area was approximately 80% of the minimum temperature and shrinkage requirements in the ACI Building Code (ACI 318–89) based on the topping thickness alone. Additional reinforcing bars were located in the topping slab adjacent to the walls and along the drag struts to facilitate force transfer from the diaphragm. Unstressed strands were positioned along the edges of the diaphragms to act as chord reinforcement. The topping slab thickness was increased to approximately 5½ in. in these areas to accommodate the added reinforcement.

Parking garage A sustained extensive damage during the earthquake. Large sections of the garage collapsed. Only those portions of the structure immediately adjacent to the structural walls remained standing.

Garage B was a single structure that was approximately 300 ft. by 270 ft. in plan (Fig. 2), and also contained three levels of parking. The structural system used in garage B was essentially the same as that in garage A. Four cast–in–place walls formed the lateral–load resisting system and precast members formed the gravity–load resisting system. Wall thicknesses ranged from 12 to 26 in., wall lengths ranged from 30 to 88 ft., and the wall aspect ratios ranged from 0.25 to 0.75. The ramp was located along the east side of the structure. Eight bays in the east half of the parking garage collapsed during the earthquake.

Both parking garages were demolished shortly after the Northridge event.

EVALUATION OF BUILDING RESPONSE

Based on the observed damage, several hypotheses have been developed to explain the collapse of portions of the two precast parking garages. Two likely modes of failure are described in this paper.

The first possible failure mode is identified by tracing the transfer of seismic forces through the structure. The structural systems were designed such that the inertial forces induced during the earthquake would be carried by the diaphragm into the structural walls and to the foundation. Photographs taken of the buildings immediately after the earthquake show that most of the walls did not crack, indicating that the full inertial forces were not transmitted to the walls (Iverson and Hawkins, 1994).

Flexural failure of the diaphragm constitutes another possible failure mode. In–plane inertia forces could cause flexural cracks in the diaphragm. If the chord reinforcement is perpendicular to the joints between adjacent double tees, the cracks will form along the joints. Photographic evidence from other damaged parking garages confirms these likely crack locations (Iverson and Hawkins, 1994). The topping slab is the only structural element that crosses these potential planes of cracking. The influence of concentrated cracks on the flexural strength of diaphragms was evaluated.

Possible Load Paths

Based on the assumption that the inertial forces were transferred by the diaphragm through drag struts and into the walls, the lateral strength of the structure may be limited by the shear strength of one of three components: the walls, the connections between the diaphragm and the walls, and the diaphragms (Quinn, 1995). The strength of each component was evaluated using the applicable provisions of the ACI Building Code (1989).

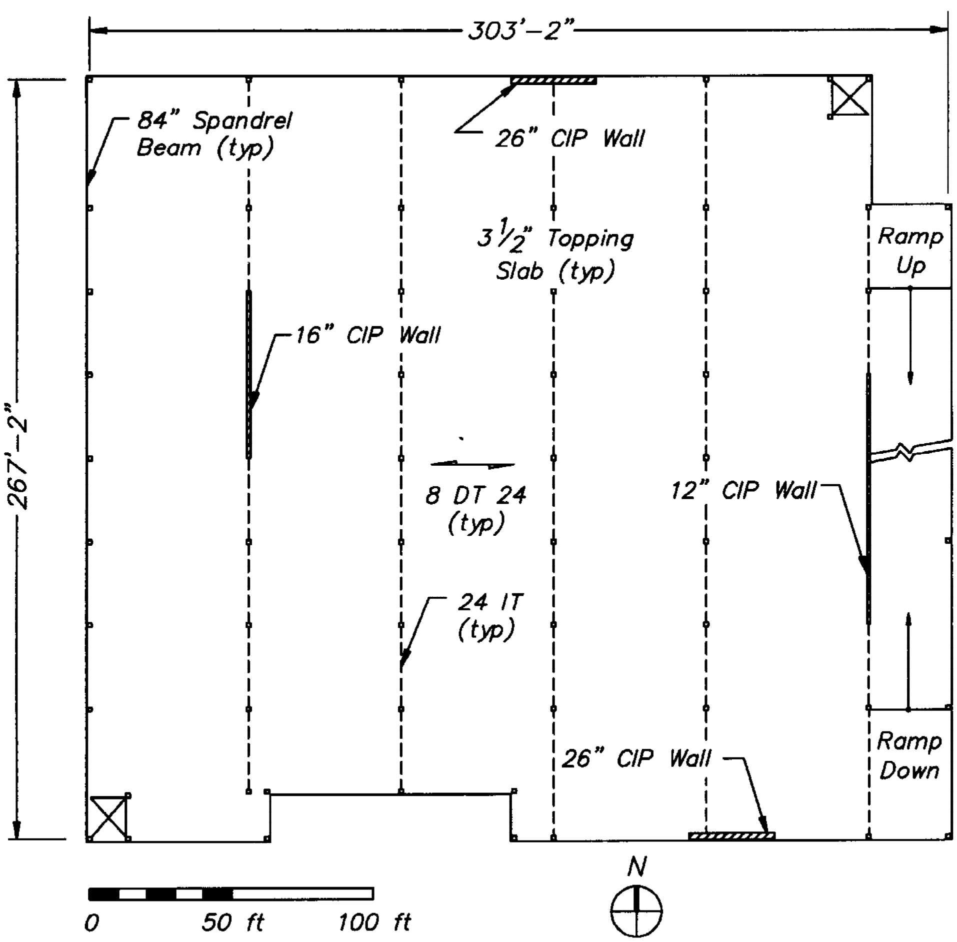

Fig. 2 Second–Floor Plan of Parking Garage B

The shear strength of the structural walls, V_n, was evaluated as:

$$V_n = \left(\alpha_c \sqrt{f_c'} + \varrho_n f_y\right) A_{cv} \quad (1)$$

where α_c was taken equal to 3.0 for structural walls with an aspect ratio less than 1.5, f_c' is the compressive strength of the concrete in units of psi, ϱ_n is the web reinforcement ratio in the wall, f_y is the yield stress of the web reinforcement, and A_{cv} is the net area of the wall cross section (ACI, 1989).

Two modes of failure were considered when evaluating the strength of the connection between the wall and the diaphragm. A shear friction model was used to determine the capacity of the dowel bars that extended from the wall and drag struts into the diaphragm:

$$V_n = A_{vf} f_y \mu \quad (2)$$

where A_{vf} is the effective area of reinforcement crossing the potential failure plane and μ was taken equal to 1.0 (ACI, 1989). The axial of the connection between the drag strut and the wall was evaluated as:

$$P_n = A_s f_y \quad (3)$$

where A_s is the area of longitudinal steel in the drag strut that was anchored in the wall.

Potential failure planes through the diaphragm were assumed to form along column lines and did not cross through any precast or cast–in–place elements. A shear friction model (Eq. 2) was used to evaluate the strength of sections where the plane of weakness ran parallel to the direction of the applied load. In sections where the plane of weakness was perpendicular to the direction of the applied load, the strength of the diaphragm was determined by the axial strength of the reinforcement crossing the plane of weakness (Eq. 3).

The capacity of each wall, connection, and potential plane of weakness through the diaphragm was calculated using the procedures outlined above. Demand to capacity ratios were used to evaluate the relative strengths of the structural members and to identify those that would be most highly stressed during an earthquake. The demand placed on each structural element was evaluated using a simple representation of the garages in which the inertial force in a given direction was distributed equally to the walls parallel to that direction. Torsional response was ignored.

The demands placed on the structural elements were calculated using the equivalent lateral force procedure defined in the Uniform Building Code (1994) assuming elastic response ($R_w = 1$). Elastic forces were selected to provide a basis for comparison, and were not intended to represent the actual lateral forces resisted by the buildings during the Northridge earthquake. Ratios of the elastic shear demand to the nominal capacity of the structural elements for the two garages are plotted in Fig. 3. The two sections of garage A are considered separately.

As expected, the elastic shear demand exceeded the nominal shear capacity of the structural members and connections at all locations. At the base of the walls, the elastic shear demand was approximately twice the nominal capacity. The elastic demand placed on the connection between the walls and the diaphragm was two to three times the nominal capacity. The diaphragm appeared to be the critical element in the south section of garage A and in garage B, where the demand to capacity ratios reached five to six. Demand to capacity ratios for the diaphragm were not calculated for the north section of garage A or for the east and west walls in the south section of garage A because all possible load paths intersected cast–in–place members.

The data shown in Fig. 3 indicate that the shear capacity of the diaphragms was the weakest of the three failure modes considered. Seismic forces were not transferred into the structural walls because the diaphragms did not have sufficient strength to carry the inertial loads.

Flexural Response of Diaphragm

The influence of discrete cracks on the flexural response of the diaphragms was evaluated by calculating moment–curvature relationships (Maund, 1995). The diaphragms were assumed to have a rectangular cross section with a width equal to the depth of the topping slab, a depth equal to the transverse dimension of the parking garage, and a span equal to the longitudinal dimension of the garage. Inertial loads parallel to the axes

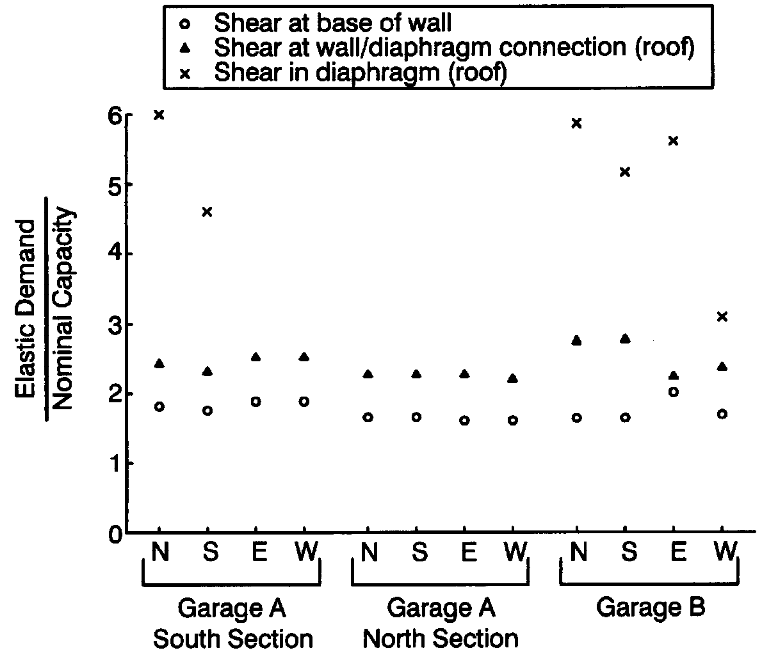

Fig. 3 Elastic Demand to Capacity Ratios for Potential Load Paths in the Parking Garages

of the double tees were considered. The precast elements were not considered to contribute to the flexural capacity of the diaphragm. Cracks were assumed to form only along the interface between adjacent double tees.

Average strains were assumed to vary linearly over the depth of the cross section. However, after cracking all tensile deformations were assumed to be concentrated at the cracks. The crack width, Δ, at a given depth was calculated as:

$$\Delta = \phi \cdot y \cdot b_t \tag{4}$$

where ϕ is the average curvature, y is the distance from the neutral axis, and b_t is the distance between cracks (width of the double tee flange). Because the steel crossing a crack is assumed to be anchored in the concrete on either side of the crack, the steel strain may be estimated by dividing the crack width by the development length of the steel. An iterative procedure was used to establish equilibrium in the cross section for a given curvature.

The calculated moment–curvature relationship for the diaphragm in the south section of garage A is shown in Fig. 4. The lower curve approximates the as–built conditions. Material properties representative of the unstressed strand were used for the diaphragm chords and properties representative of welded wire fabric were used for the distributed web reinforcement. The largest calculated moment resisted by this cross section corresponds to the cracking moment.

After cracking, the flexural capacity decreases with increasing curvature. The strength of the diaphragm at larger curvatures does not exceed the cracking moment because the welded wire reinforcement in the web begins to fracture. As the crack opens, the calculated strains in the wire are concentrated over a development length of 6 in., which corresponds to the mesh spacing. The fracture strain of the wire was assumed to be 0.02. The post–cracking strength of the cross section is reached when the chord reinforcement yields and the web reinforcement in tension has fractured. The calculated curvature capacity corresponds to fracture of the strand that forms the tension chord.

A second analysis was performed using the material properties of deformed #3 reinforcing bars as the web reinforcement (upper curve in Fig. 4). The web reinforcement ratio is the same in the two analyses; however, the fracture strain in the bars was assumed to be 0.08 and the development length was increased to 14 in. The calculated flexural capacity of this diaphragm exceeds the cracking moment. The influence of fracturing of

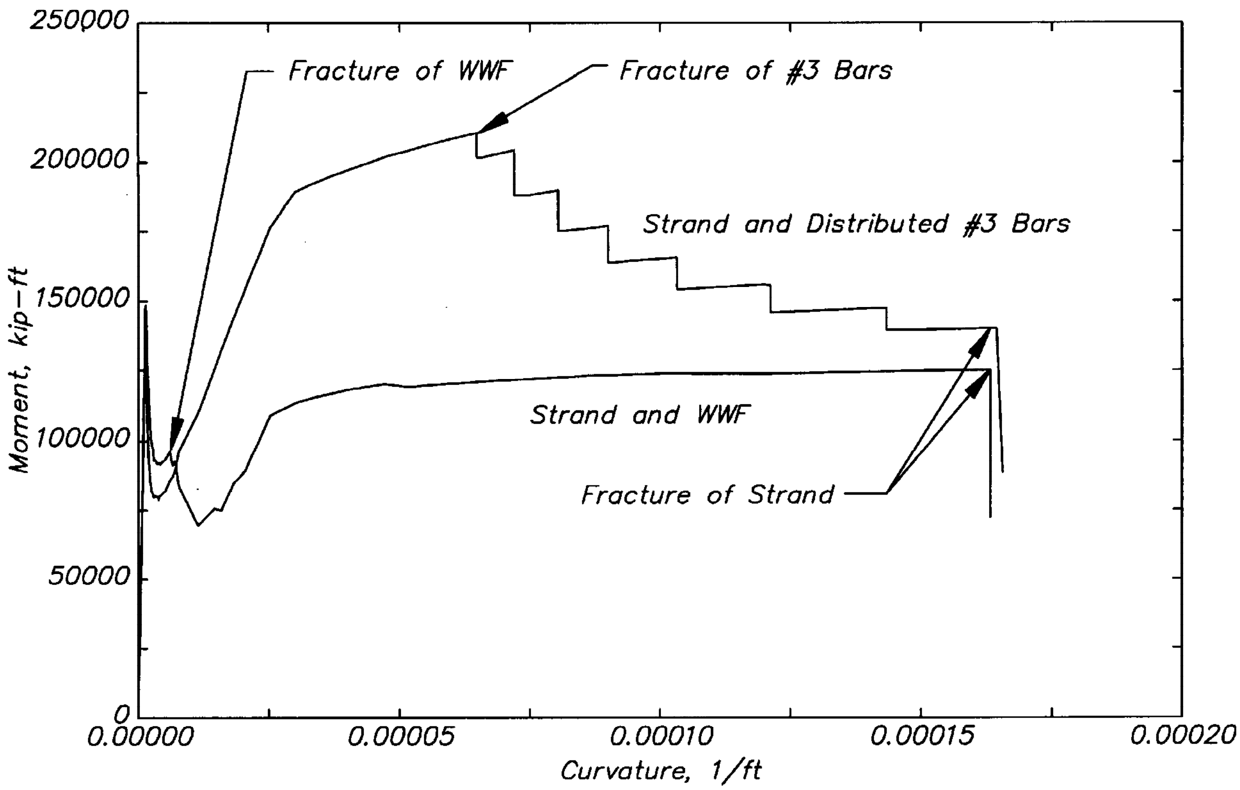

Fig. 4. Calculated Moment–Curvature Relationship for Diaphragm in Garage A

the web steel may be seen in the discrete drops in flexural strength with increasing curvature. The limiting curvature for this diaphragm also corresponds to fracture of the strand that forms the diaphragm chord.

The forgoing arguments suggest that the as–built diaphragms exhibited brittle in–plane flexural behavior. After cracking, the deflections would become large before the strand develops its full strength, and some precast elements could have lost their support, thereby precipitating collapse.

SUMMARY

Two failure scenarios were presented which provide possible explanations for the damage in precast parking garages observed after the 1994 Northridge earthquake. In the first scenario, the weak link in the load path designed to transfer inertial forces into the foundation was identified as the shear strength of the diaphragm. Potential failure paths within the diaphragm were crossed only by the distributed welded wire fabric and by chord reinforcement near the perimeter of the structures. The amount of distributed reinforcement placed in the topping slab was based on shrinkage and temperature requirements (ACI 1989). In the second scenario, the maximum flexural strength of the diaphragm was calculated to occur at cracking. After cracking, the flexural capacity of the diaphragm was limited by fracture of the welded wire mesh crossing the discrete cracks that are likely to develop between the flanges of adjacent double tees. The aspect ratio of the diaphragm would be expected to dictate whether the shear or flexural strength would govern the behavior.

Both failure scenarios highlight the role of the diaphragm on the seismic response of precast parking garages. Cast–in–place structural walls constitute the vertical elements in the lateral–load resisting system. Uniform Building Code (1994) requirements dictate a design approach that is based on the walls being the weak link in the structural system and being the location of the inelastic action. However, the geometry of most low–rise parking garages makes this behavior unlikely. Ductile behavior is not anticipated in walls with aspect ratios less than 0.75. Therefore, the inelastic action must occur in another structural element. The diaphragms served as the weak link in the lateral–load resisting system in the two garages studied. The reinforcement in the diaphragms had not been selected to insure ductile response, and a brittle mode of failure was observed. The design philosophy for such structures should be re–evaluated and code changes that address the critical issues are needed.

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