A FIVE STORY PRECAST CONCRETE TEST BUILDING FOR SEISMIC CONDITIONS - DESIGN DETAILS

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SUMMARY

In the third and final phase of the PRESSS (PREcast Seismic Structural Systems) program, a large five story precast concrete building was tested under simulated seismic loading. A companion paper provides an overview of the main features of the building and the test procedures, and this paper describes details of the design of the building. The lateral load resisting system consisted of a wall in one direction and two perimeter frames in the other. Those frames contained four different connection systems that had previously been evaluated in component tests. Two different flooring systems (hollow-core and double tees) were used.

INTRODUCTION

The PRESSS (Precast Seismic Structural Systems) program is a US-Japan joint research program whose goals are to develop innovative seismic structural systems and to formulate design recommendations for precast concrete buildings [Priestley, 1991; Nakaki et al., 1999]. On the US side, attention has been focussed on jointed systems, in which the deformations are largely concentrated in the connections, rather than on emulative systems which seek to mimic the behaviour of cast-in-place concrete. The use of jointed systems minimises the quantity of site cast concrete required. Concepts were developed in Phase I of the project, and they were evaluated through testing of individual connections in Phase II. Phase III consists of the design, construction, testing and analysis of a complete building, and the development of design recommendations from the results. The concepts used in Phase III were selected from those tested in Phase II. At the time of writing, testing of the building had just been completed.

STRUCTURAL FEATURES OF THE TEST BUILDING

The test building was based on a prototype that was five stories high with plan dimensions of 100 ft x 200 ft. The test building is a 60% scale model of a two bay by two bay segment out of that prototype, with plan dimensions of 30 ft x 30 ft and a storey height of 7.5 ft. This was the largest model that could be accommodated in the structural testing laboratory at the University of California at San Diego. The scale was large enough to permit the use of conventional structural components, such as reinforcing bars, but special measures were still required to account for the effects of scale on the gravity system.

N-S perimeter frames and a central E-W wall provide the seismic resistance. Gravity frames are located on the north ansd south faces. The floor system on the lower three floors consists of double tees that span between the seismic frames, while the upper two are made from hollow core that spans from the gravity frames to a steel header attached to the wall. Details of the layout are shown in Sritharan et al. [1999]. The floors must not only

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resist gravity loads but also act as diaphragms to transfer the inertia forces from their points of origin in the floors to the wall or seismic frames. A single design concept is used for the wall, but four different connection systems are used in the frames. On one side of the building they are based on the use of unbonded prestressing, whereas on the other they use reinforcing bars. On each side, one system is used in the lower three floors and another in the upper two. The configuration and construction of the systems are given here. Details of the design and intended performance of the connections are given in a later section of the paper.

The connection system in the lower floors of the prestressed frame is the “Hybrid Frame” connection [Stanton et al. 1997]. Precast beams are first erected on temporary corbels on the columns. The beam-column interfaces are then grouted. Unbonded post-tensioning (PT) strand is then threaded through matching ducts in the beams and columns, stressed and anchored at each end. In the test building the anchors were set in beam stubs in order to minimise steel congestion. The PT clamps the beams and columns together, and creates an elastic restoring force when the column undergoes lateral drift. Reinforcing bars are also grouted in ducts at the top and bottom of the beam and through the column. Their function is to yield cyclically in tension and compression as the building vibrates, and thus to dissipate energy.

In the upper two floors of the prestressed frame, a pretensioned (Pre-T) beam system is used. In concept, it resembles the Hybrid Frame but the construction details differ. The beam is made in one piece, so the columns segments are discontinuous through the beam and are connected by bars that are grouted in vertical ducts in the beam-column joint. The prestressing in the beam is bonded directly to the beam ends in order to anchor the strands, but it is sleeved over the central region where it passes through the beam-column joints in order to minimise local strain changes and to ensure an elastic resistance to column drift. Reinforcing bars could be added to dissipate energy by yielding, but in the test building they were not. In the field, the seismic framing may extend over more than two bays. If a single beam were to extend over more than two columns, aligning the column bars in the beam ducts could be difficult. The beams could then be made in two-span pieces and be joined in alternate bays at midspan, where the seismic moments are smallest.

On the lower three floors of the non-prestressed frame the connections are made using a TCY-gap connection, so called because the important reinforcement consists of conventional bars that yield alternately in tension and compression, and because a gap exists over part of the beam-column interface. Only the bottom of the beam-column interface is grouted, and a post-tensioning tendon running the length of the frame clamps that grout pad between the beam and column. At the top of the interface there is a gap between the beam end and the column face, large enough to accommodate the design drift without concrete-to-concrete contact. The reinforcing bars cross the gap and are grouted in ducts in the beam and column. The beam is intended to rock about the grout pad while the top bars yield. One of the benefits of the concept is that the beam undergoes no elongation, so the floor slab does not need to crack in order to maintain compatibility with the beam displacements. Maintaining the integrity of the floor diaphragm has obvious advantages. Beam elongation occurs in almost all other systems, including cast-in-place moment frames, and, if precast slabs are used, could cause their unseating. The TCY-gap concept was tested in subassemblage form by Palmieri et al. [1996] prior to its incorporation in the test building.

On the upper two floors, reinforcing bars are grouted in ducts through the columns to form a conventional TCY connection. The beams are erected on temporary corbels and the beam-column interface is grouted. Bending is resisted by tension and compression in the bars, which are designed to yield. Shear is presumed to be carried across the interface by shear friction and dowel action. Connections of this type are widely used in industry, and this one was selected so that the test building would include one example of current technology.

DESIGN FORCES

The design strength of the primary framing elements was determined using Direct Displacement Based Design (DDBD). This method of design [Priestley 1996] is an alternative to the force-based approach presently advocated in codes such as the UBC [Uniform 1997]. Its advantage is that it leads to estimates of drift in response to a given ground motion that are more reliable than those available from force-based methods. This is important, because damage is closely related to maximum drift. The accuracy of the drift estimate depends on the ability of a linear elastic model with viscous damping to simulate the dynamic behaviour of a hysteretic system.

The DDBD process involves two systems: the MDOF hysteretic prototype and an "equivalent" SDOF viscously damped elastic system. The design method is iterative and is explained here using the wall as an example. It requires an elastic displacement response spectrum (DRS), constructed with several different damping levels, for the design ground motion.

First the target displacement under design conditions, target, and is selected. For the wall, the design criterion selected was first yield of the PT bars at 2% roof drift. If the target drift had been smaller, the wall would have had to be stronger or to dissipate more energy. Next the deflected shape of the building must be chosen. It is often referred to as a mode shape, but does not possess the mathematical properties, such as orthogonality, associated with a true elastic mode. Despite this, the deflected shape is used to compute "modal" properties, such as modal mass, using conventional equations for modal analysis. Priestley [1996] gives recommendations for deflected shapes for frames. A linear shape was used for the wall, because the rigid body displacement of the rocking wall was expected to be much larger than the in-plane deformations of the individual panels. The generalised co-ordinate (or "system displacement"), eq, associated with the shape of the mode and the magnitude of target is computed. The viscous damping of the equivalent elastic system, eq, is then estimated from prior experience. (The initial value is not important, except that a better value speeds convergence).

The DRS is now used to find the equivalent period, Teq, that corresponds to eq and eq. (This is the reverse of the way in which spectra are used in force based design, when the period is assumed to be known and the acceleration or displacement is sought). The equivalent modal mass, Meq, which is obtained from the deflected shape and the real distribution of mass, and Teq then lead to the equivalent stiffness, Keq. The equivalent base shear is given by

$$Veq = Keq * \Delta eq$$

The true base shear and the floor inertia loads can then be obtained from the modal properties. The member forces needed to resist these loads can be computed and the member properties can be selected to give adequate design strengths. It should be noted that the inertia forces are not increased by load factors, but rather the structure is expected to reach the target displacement when the members are inelastic and loaded up to their design strengths. This condition forms the basis of the relationship between the real and equivalent systems. A pushover analysis can now be conducted to establish the energy dissipated per cycle in the prototype system, and leads to a corrected value of the equivalent damping. If it differs significantly from the assumed value, the calculations are repeated with the new damping value. If it is close enough, the remaining components of the structure are designed using capacity design principles.

In the wall, the PT and energy dissipating shear connectors between the walls were selected using DDBD, but the bending and shear strength of the wall panels themselves, the strength of the floor-to-wall connections and the diaphragm must all be conducted using capacity design in order to ensure that the inelastic action occurs in the vertical structure and not in the floors. In the case of the frames, capacity design principles should be used for quantities such as the column shear strength, the floor-to-beam connection strength, etc. They should be used in conjunction with overstrength factors that represent the effects of higher vibration modes.

The design forces for the walls and frames in the test building were obtained by calculating the values for the prototype and scaling them down. The prototype building contained four walls, compared with one in the test building, and the scale factor on linear dimensions was 60%, so the test building lateral loads were 0.25* (0.62) = 9% of those in the prototype. In the protoype, the story height was 12.5 ft, the building weight was taken as 195 psf, the design drift was 2%, the deflected shape was assumed to be linear and the displacement spectrum was idealized as bilinear, with the two lines intersecting at 4 seconds and 21.9 in. The converged value of equivalent viscous damping was 12.4%, from which the prototype building base shear was found to be 2167 kips. The test building wall was therefore designed for 9% of this value, multiplied by 1.1 to allow for eccentricty of mass, as required by the UBC. The design base shear was thus 215 kips. The maximum floor inertia force, at the roof level, was 71.5 kips.

The overstrength factors used with the capacity design principles were selected to represent the effects of both material overstrength (assumed to be approximately 1.25) and higher modes (taken as 1.5). The final value was rounded up to 2.0. For the PRESSS test building, the same details were used at each floor in order to simplify construction so that the real overstrength factors were higher at the lower floors. The maximum inertia force in the wall direction at the top floor computed from the DDBD approach was 71.5 kips, so the load used for design of the floor components and connections and the actuator connections was 143 kips/ floor, rounded up to 150 kips/floor.

CONNECTION DESIGN AND DETAILS

Wall and panel connections

The design of the wall is discussed in detail by Galusha [1999]. The primary design elements are the post-tensioning (area and initial stress) and the strength of the energy dissipating connectors. The behaviour of the precast wall is illustrated in Fig. 1. As the wall displaces, each panel rocks about its bottom corner, thereby stretching the PT tendon and causing shear displacements in the energy-dissipating connectors that link the panels. One of the benefits of the rocking precast wall is that the residual displacement is theoretically zero if the PT remains elastic up to the design drift. Under the maximum credible drift, taken here as 3% at the roof, the tendon must not fracture in order to ensure life safety. Both of these criteria can be satisfied by using geometry to calculate the stress increase caused by the drift and then selecting the appropriate initial stress. A tendon with a high elastic strain capacity, such as strand, is advantageous under these circumstances, but Dywidag bars were selected for the test building because of the construction difficulties of anchoring strand in the foundation.

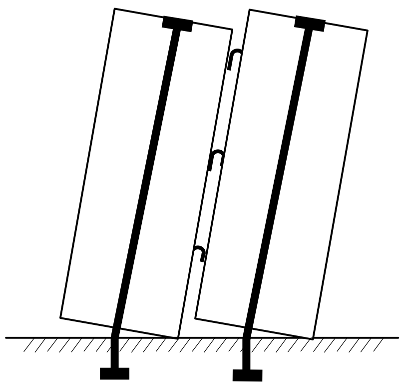

Fig. 1. Rocking of Precast Wall.

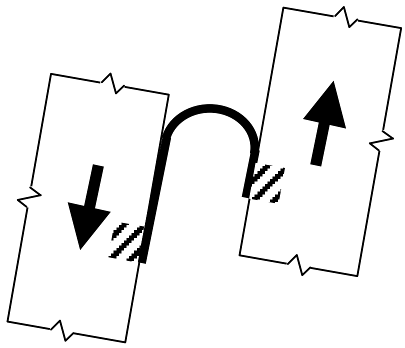

Fig. 2. Behaviour of UFP.

The energy dissipating connectors used in the building were U-shaped Flexural Plates (UFPs), initially developed by Skinner et al. [1975] and tested in the PRESSS program by Schultz [1996]. They are shown in Fig. 2. A flat plate is bent into a U-shape and is bolted or welded to plates embedded in the edges of adjacent wall panels. As the wall rocks, the edges of adjacent panels undergo relative shear displacements and force the UFP to roll like a tank track. The curved part straightens and the straight part becomes curved. The plate dimensions and material properties can be selected to ensure that yielding occurs and that energy is dissipated by the rolling action.

The UFP strength must be limited so that the PT is able to return the structure to zero residual drift. The variables controlling the design are therefore the combined strength of the PT and the UFP (which governs the wall strength), the ratio between the initial PT force and the UFP strength (which controls the residual drift) and the energy dissipation capacity of the UFP (which controls the equivalent damping).

The shear strength of the UFP can be shown [Galusha, 1999] to be

$$V = Mp/R = (0.25bt2Fs)/R$$

Where

Mp = the plastic moment capacity of the plate

R = the mean radius of the bent plate

b = plate width

t = plate thickness
F_s = steel strength (= F_u at large strains)

The strongest connector is obtained by using thick plate and a small radius. However, this configuration also causes large strain, because the bending and straightening strains are both given approximately by

strain = t/R

Therefore the cyclic strain capacity of the plate limits t/R and, by inference, the shear strength. In the PRESSS test building, stainless steel was used, because it has a strain capacity larger than that of carbon steel. However it exhibited considerable isotropic strain hardening, which meant that the shear resistance of the UFP increased with cycling, which in turn made ensuring zero residual drift more difficult. In order to provide the necessary strength, two pairs of UFPs were provided at each floor level.

Hybrid Frame connections

The Hybrid Frame concept is shown in Fig. 3. As the frame drifts, the top or bottom of the beam lifts off the column and a wedge-shaped gap opens between the two. The PT tendon is anchored at each end of the frame but is unbonded in between, so that the additional strain due to the gap opening is kept to a minimum. The tendon is placed at mid-height of each beam to minimise the additional strain caused by the frame deformation. The rebars are grouted in ducts at the top and bottom of the beam for ease of construction. They are unbonded over a short length in the end of the beam to prevent their fracturing through excessive cyclic strain.

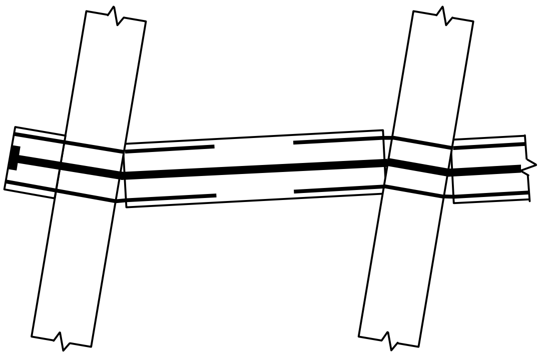

Fig. 3. Hybrid Frame

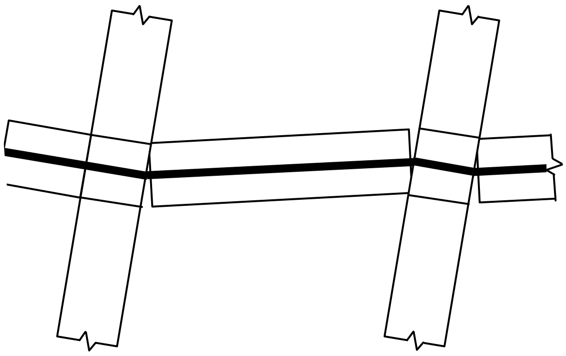

Fig. 4. Pre-T Frame

As with the wall, the PT and the energy dissipating rebars in the Hybrid Frame must be designed together to ensure sufficient overall strength, zero residual drift, and the desired level of damping. The design criteria are similar to those for the wall. The residual drift can be eliminated if the initial PT force is strong enough to close the gap despite the resistance of the rebars in compression. If the centre of the concrete stress block coincides with the compression bar location, which is usually a good approximation, the optimum strategy is to generate half of the flexural strength from the rebars and half from the PT, because this approach maximises the damping. However, it is important to use the true bar strength rather than the specified minimum: if the bars are too strong, then some residual drift may occur. (Small oscillations near the end of the earthquake will, in practice, probably minimise the residual drift even if the bars are theoretically too strong for the PT to close the gap, but the size of the residual drift then depends on the ground motion characteristics, which are unknown a priori). The area of the PT tendon depends on the initial stress in it, because the gap closing criterion is expressed in terms of an initial PT force. However, for the PT tendon not to yield, the initial stress, fpe, must be less than [Stanton et al., 1997]

$$fpe < fpy - 1.8\,(h/L)Ep * drift$$

Where

fpy = yield stress of prestressing strand

Ep = Young's modulus of prestressing strand

h = beam depth

L = beam clear span

drift = design drift angle

The interface grout must be fibre-reinforced, so that it does not drop out when the beam lifts off from the grout pad. Experiments [Stone et al., 1997] have shown that the fibre-reinforced grout pad remains in a single piece and fulfils its function well.

Pre-T Frame connections

The Pre-T frame is based on principles similar to those that underlie the Hybrid Frame, except that the beam contains no rebar. The concept is illustrated in Fig. 4. The strands are pretensioned, so that the members can be fabricated in a conventional precasting yard, and the need for post-tensioning equipment on site is eliminated. They must be bonded at the beam end and unbonded throughout the rest of the beam. During cyclic loading the stress in the strand changes. Little information is available about the bond of pretensioned strands subjected to cyclic loading, and questions have recently been raised over the bond of monotonically stressed strands [Logan 1997], so a generous development length was allowed in the Pre-T beams to prevent their failing prematurely. The development required a stub beam that projected beyond the corner column. The strands were left projecting through an anchor plate beyond the end of the beam, so that, if they slipped, chucks could be installed and the test could proceed. At points of symmetry along the beam, such as the midspan and at the centre of each column, the strand has no tendency to move relative to the concrete. The strand could therefore be bonded to the concrete at these locations without impairing its basic function, which would provide some measure of security against loss of prestress in all bays if the anchorage failed at one end. However, in the interests of simplicity, such local bonding was not used in the test building, and the lack of slip during testing validated that decision.

TCY-Gap Frame connections

The TCY-gap frame is illustrated in Fig. 5. After grouting the bottom of the gap at the beam-column interface, the beams were post-tensioned with bars located at the centre of the grout pad. In the test building this was done with one-piece bars that extended over both bays, but in the prototype the tendon would be too long to be made from a single bar and could be made from either coupled bars or strand. The prestressing differs from that in the prestressed frame in that the beam is intended never to lift away from the column. The prestressing must be designed so that the compression across the grout pad is always enough to provide the friction needed to prevent slip due to vertical shear force and this criterion assures that lift-off will never occur. The most critical condition occurs when the beam shear is largest, (i.e. at maximum drift) because then the rebars at the top are in compression, and the force on the grout is lowest. The compression strength of the rebars used in design is clearly important here, because excessive strength could lead to vertical shear slip of the beam. The design of the grout pad is a compromise between keeping it small in order to minimise rotation stresses, yet making it large enough to resist the post-tensioning force. The grout should be heavily reinforced with fibres in order to maintain its integrity under these highly stressed conditions. The rebars at the top were sleeved for four inches at the end of the beam in order to prevent fracture through excessive local strain.

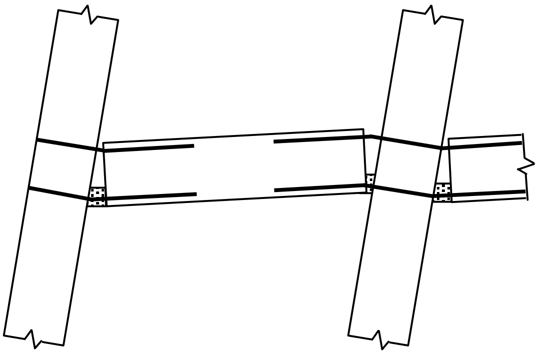

Fig. 5. TCY-Gap Connection

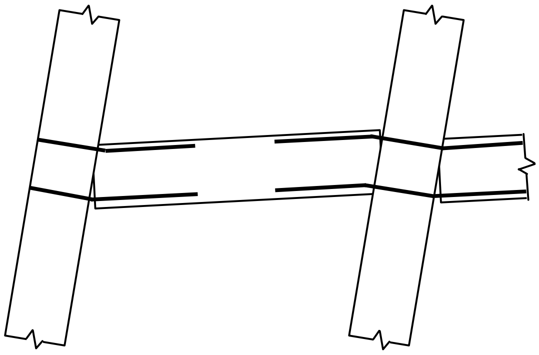

Fig. 6. TCY Connection

<u>TCY Frame connections</u>

The TCY Frame is illustrated in Fig. 6. Each beam spans a single bay and is connected to the columns by rebars grouted in ducts through the column. The system contains no prestressed steel, so some residual drift is likely. The test building was loaded by two actuators per floor, slaved to give the same displacement. They prevented the torsion that might otherwise have occurred because of the different stiffnesses of the two frames. Thus the behaviour of the building was influenced by the properties of both frames together, and the restoring force of the Pre-T frame helped to re-centre the TCY Frame, while the energy dissipated in the TCY Frame helped to overcome the lack of damping exhibited by the Pre-T frame. The integrity of the TCY connection depends on the bond of the bars in their ducts. In the sub-assemblage tests, the TCY connection had failed when the ducts through the column debonded from the surrounding concrete [Palmieri et al. 1996]. In order to prevent this behaviour in the test building, the ratio of column width to bar diameter was kept above the minimum value of 24 recommended by Leon [1989] for cast in place concrete joints. While this does not guarantee that the duct will not slip, it satisfies a lower bound on the ratio. T-heads were also welded on the bars projecting through the corner columns as a further measure to limit bar slip.

Floor Connections

The jointed nature of the precast wall and frames protects the individual members from damage at the expense of opening discrete cracks at the connections between members. The connections between framing members are designed to provide the necessary force and deformation capacities at these discrete cracks, largely through the use of unbonded prestressing tendons, as already described. However, the floors have to undergo deformations that are compatible with those of the frames to which they are attached, so, for example, a crack must open up in the floor in line with the beam-column interface of a Hybrid Frame connection. If it does not, the frame connection cannot function as intended. These floor cracks are an inevitable consequence of the beam elongation in the frames associated with the discrete cracks at the beam-column interfaces. They must also occur in cast-in-place frames, except that in that case the floor cracks are a little more distributed.

The wall presents similar characteristics, but the critical displacements are vertical. When each panel rocks, the centreline of the wall rises. The double tees are supported on the beams of the seismic frames, which undergo much smaller vertical movements than does the wall. Therefore the connection between the double tees and the wall must permit free relative vertical movement, and free relative rotation because rocking causes the wall to rotate, yet it must transfer horizontal diaphragm shear forces to the wall without deformation. In addition, when the building is loaded in the frame direction, the floor must be able to move freely away from the wall in order to accommodate opening of the Hybrid Frame beam-column interface and to allow the wall to tilt out of plane. Thus of the six possible degrees of freedom at the connection, free displacement must occur in four, force must be transferred in a fifth, and only for rotation about the vertical axis is there no requirement.

This specification for forces to be transmitted and deformations to be freely accommodated appears complicated, but it also applies in principle in a cast-in-place building, where the deformations are distributed. The deformations were exacerbated in the test building because the extraction of the test building from the prototype resulted in a reduction of the distance between the end of the wall and the perimeter frame, so any angular changes caused by differential vertical displacements were exaggerated. Despite these circumstances, the connections between adjacent floor elements, and the floor-to-wall and floor-to-frame connections, were designed to satisfy all the design requirements. In a prototype building, the design is expected to be considerably simpler, but in the test building it was deemed important to demonstrate that connections could be designed to satisfy all the requirements, and to show by testing that they would function as intended. Details of the connections are given in (Collins et al. 1999). The most complex one, between wall and double-tee floor, performed without any yielding of steel and without a single crack of any size in the concrete, despite the building's being deformed to 4% and 3% roof drift in the frame and wall directions respectively. This performance demonstrates that suitable connections can be designed and fabricated for difficult circumstances. Designs for connections to satisfy the less stringent conditions expected in prototype buildings are by inference possible, and they are likely also to be simpler.

CONCLUSIONS

The following conclusions were drawn from the study.

1. Precast concrete systems can be designed to resist seismic loading in ways that take advantage of the naturally occurring joints between the precast members.

2. The use of jointed precast systems gives the designer more control over the performance of the building, and in particular the level of damage that will be inflicted, than is generally possible with cast-in-place systems.

3. The use of unbonded post-tensioning for elastic restoring force, combined with a yielding material to dissipate energy, provides system characteristics that offer considerable promise.

4. Floor connections can be designed to satisfy the force and deformation requirements associated with jointed construction.

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