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DESIGN EXAMPLES USING MODE SHAPING SPINES FOR FRAME AND WALL BUILDINGS

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ABSTRACT

This paper presents design examples of frame and wall buildings that use mode shaping spines to control drift by ensuring a global tilting mode, while precluding story mechanisms. The improvements to seismic performance are significant and cost effective with several projects achieving savings in comparison to conventional systems. Performance Based Design frees designers from the constraints of a force-based code. This allows the synthesis of overall project goals – the improvement of seismic resistance, the reduction of first costs, the protection of architectural skin, and the reduction of damage and loss of use.

In addition to mode shaping deformation control, several examples employ rocking and re-alignment strategies. A cantilever wall with a flexural mechanism at its base is an obvious example. Rocking steel frames and mast frame examples are also presented. Restoring force characteristics are provided in the form of post-tensioning reinforcement to re-align buildings back to plumb after an earthquake.

Mode Shaping Spines

A mode shaping spine is a stiff vertical element which serves to preclude soft stories by ensuring a global tilting mode. Current US codes are often silent when explicitly addressing global deformation patterns. For example, “special concrete walls” can still form extremely damaging story shear mechanisms. Spines are very effective in improving performance and they can take on many forms including cantilever walls, rocking frames, stiff columns, etc. An example is the central column of a Japanese pagoda. It stabilizes the stacked levels of structure by precluding a kinking or story shear mechanism. Because the central column is continuous, any soft story shear mechanism would be resisted through bending of the spine. The column has little fixity at its base and easily tilts. While having negligible lateral strength of stiffness itself, it interacts with the surrounding stacked structure to maintain an overall stable tilting.

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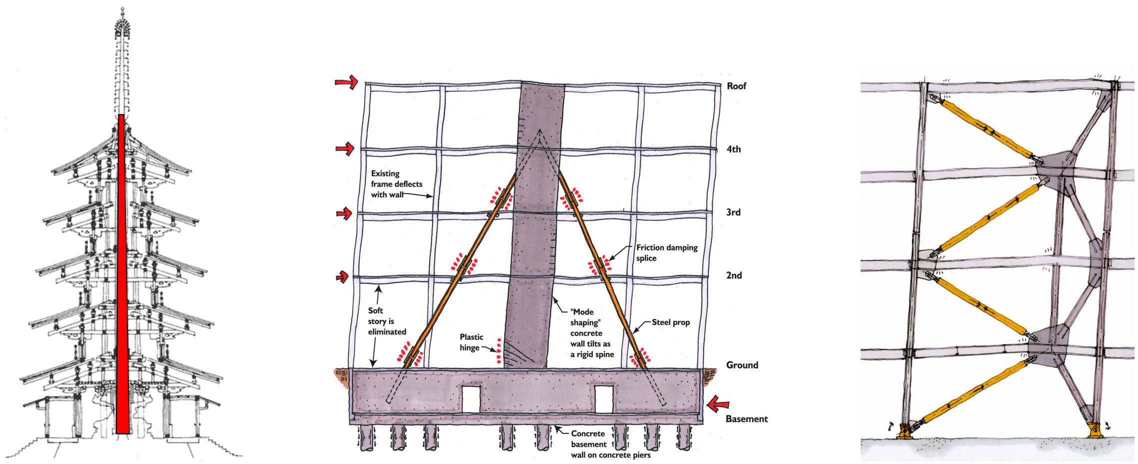

Figure 1. Examples of mode shaping spines: the central column of a pagoda, a propped shear wall, and a trussed mast.

Propped Shear Wall

Another example is the propped shear wall such as that used for the retrofit of the I. Magnin Building in Oakland, California. This structural system consists of a central wall or spine, braced near the top of the wall. The braces are joined with friction dampers. When the wall forms a plastic hinge at its base, the tilting mode forms. Additional energy is absorbed as the prop joints slip to activate the friction dampers. The mode shaping effect works in two ways when the system is used in a retrofit. The first way is the spine acts like a vertical collector of story forces, transferring them up to the braces and down to the base. Since the wall is stiff and strong above the weaker base, it forms a plastic hinge and tilts as the braces deform.

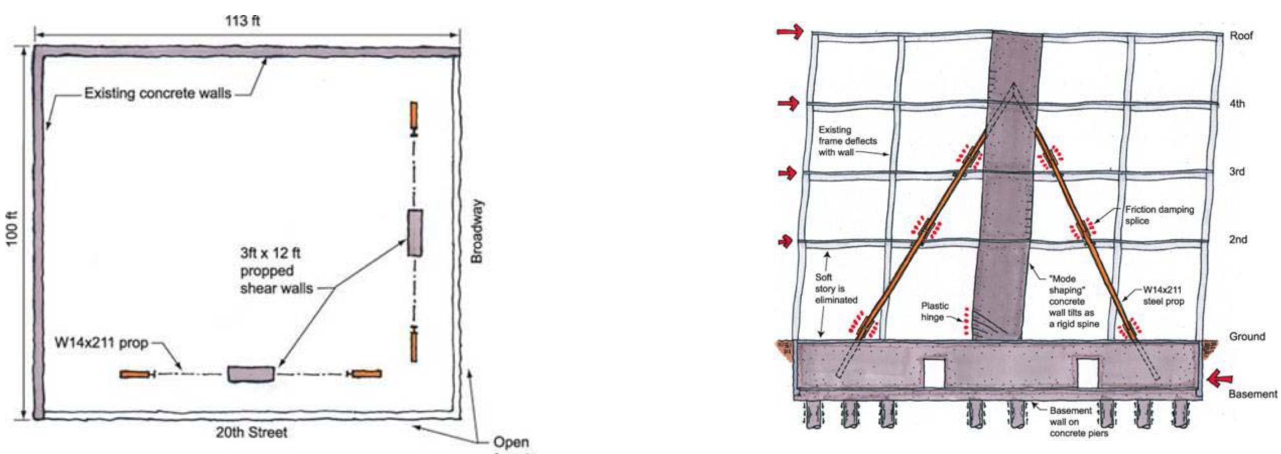

Figure 2. Plan and frame elevation of the propped shear wall retrofit.

Spine and Frame Interaction

The second means of mode shaping comes through the spine's interaction with the existing structure. The I. Magnin Building retrofit illustrates this. The existing structure is a bolted steel moment frame with a soft and weak story at the ground floor. The building would form a soft story mechanism in its original state. Through interaction with the spine, the frame is forced to undergo a tilting mode because, like the pagoda, the soft story mechanism is prevented by the additional flexural strength of the spine. If the effect is considered in terms of strong column and weak beam design requirements for moment frames, the spine functions as if extra columns have been added. The balance of column strength to beam strength is effectively changed in the frame, with the diaphragm forcing deflection compatibility of the frame and spine. The overall result is to heal the frame's weakness. Column flexural demands are reduced in a tilting mode as compared to a shear mode where deformations are concentrated in a single story. Similarly, beam rotations are distributed throughout the frame rather than at the soft story. This increases the frame's ability to sustain drift because local rotation demands are reduced at the soft story. Additionally, the frame's energy absorbing potential and strength are increased.

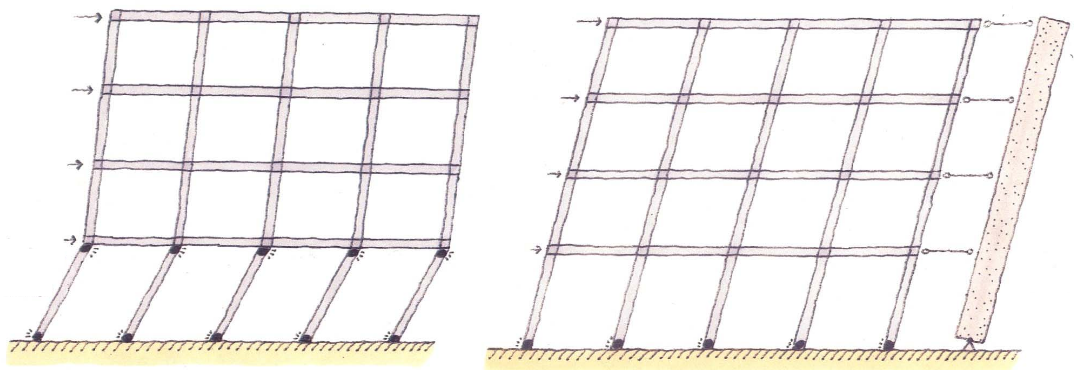

Figure 3. Mode shaping effect to correct a soft story mechanism.

A similar healing effect can occur with braced frames. Many braced frame configurations form global shear mechanisms when braces buckle or yield. This reduction of story stiffness can concentrate and potentially exhaust the rotation capacity of the gravity columns and increase the story shear demands from P-delta effects. Global shear mechanisms can be problematic even when the brace is ductile, as with buckling restrained braces – although much less so as compared to braces.

Dynamic Shear Loads

The design of a mode shaping spine requires accounting for the dynamic shear effect where lower shear resultants can produce significantly higher peak shears compared to those suggested by first mode (triangular) load distributions. This shifting of shear resultants is not accounted for with conventional code-based prescriptive design methods. The spine itself needs sufficient flexural strength and stiffness to tilt about the base without yielding above the base. This may require a deliberate weakening of the flexural capacity of a wall at its base relative to upper levels. This discourages optimum tapering of the moment capacity relative to the demands. In the case of the pagoda spine column, it is a pin joint. For braced frames, these need to be capable of a stable rocking mode. This precludes most wide multi-bay frames.

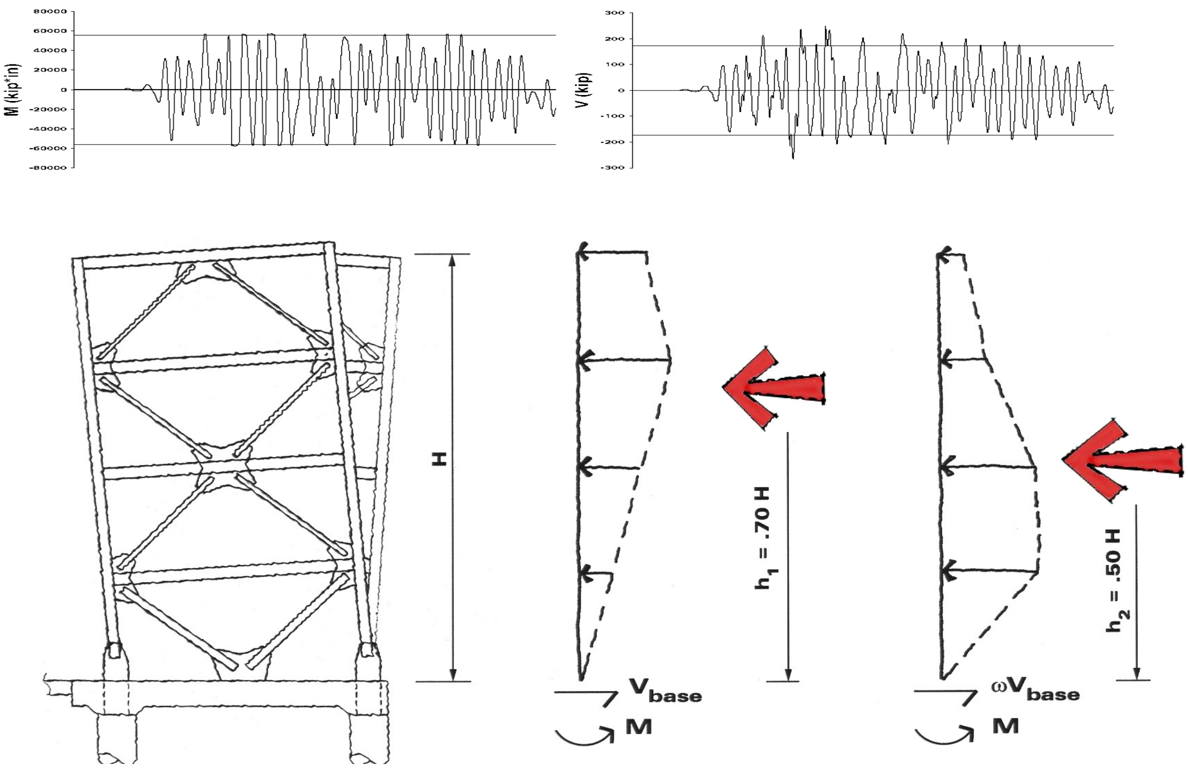

Figure 4. Dynamic shear effect with lower shear resultant and higher peak shear in a rocking steel frame. Above: moment vs. time trace and shear vs. time trace for the frame.

The shear demands on a spine can be significantly higher than those predicted from non-linear static pushover analysis based on set loading patterns such as with a first mode load distribution. There are design recommendations to predict this effect[1,2]; however these are currently not reflected in U.S. building codes. A non-linear time-history analysis can directly capture the increased dynamic shear. The design for the four story dormitories at Stanford University's Escondido Village illustrates this. The lateral system is rocking braced frames with friction dampers at the base of the columns. The stiff and strong braced frame forms the spine. When the overturning moments reach a set limit, the frames uplift at the base of the tension column and rock about the compression column. The global mechanism is similar to a flexurally controlled cantilever concrete wall rocking about the compression block. The moment vs. time trace shows the fusing of overturning moment as the frame slips under friction loads and rocks (Note the plateau in the moment trace). Based on the first mode pushover, the predicted shear is shown in the horizontal bounds of the shear vs. time trace. The trace shows peak shear 40% higher than the first mode pushover limit. The snap-shot of the forces at peak shear show force distribution with a resultant lower in height than that of first mode, accounting for the higher shear even under a fusing moment.

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[1] T. Pauley and M.J.N. Priestley, *Seismic Design of Reinforced Concrete and Masonry Buildings*, (New York: Wiley Interscience, 1992), Chapter 5 section 5.4.4 p. 413.
[2] Farzad Naeim, *The Seismic Design Handbook – Second Edition*, (Kluwer Academic, 2001), Chapter 10 p.490.

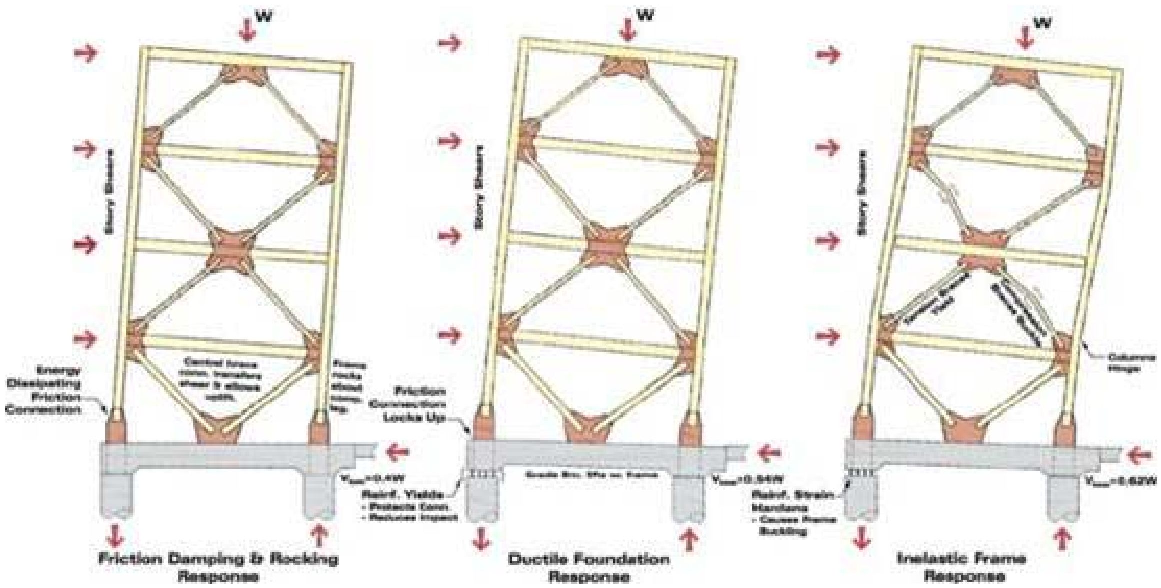

Figure 5. Modes of a rocking wall: Primary – rocking with friction damping at the uplifting column, Secondary – yielding of vertical reinforcement in caissons and stable buckling of braces.

Redundant Mechanisms

The frame design for this projects incorporates several back-up and redundant mechanisms other than first mode rocking (See Figure 5). If the rocking mode causes the uplift of the tension column to reach the physical limit of the friction joint, the secondary modes will be activated. The next mechanism is yielding of the vertical steel in the frame caissons. This steel has been de-bonded to increase the deformation capacity. Eventually the steel would strain-harden and trigger brace buckling. The braces were designed to be compact, and the connections can develop the tension capacity of the braces.

Trussed Mast Frame with BRBs

Another lateral system that uses a mode shaping spine is a trussed mast frame. We credit Professor Robert Tremblay of the Ecole Polytechnique Montreal for introducing our firm (via a presentation) to this configuration. Figure 6 shows the trussed mast frame in comparison to a conventional chevron frame. The frames were developed for the Garr Building in Berkeley, California. Both frames were evaluated for structural performance, architectural impact and cost. The trussed mast frame proved consistently superior and it is now being built (the building is under construction at the time of this writing).

From a performance perspective, the trussed mast frame is obviously better for precluding story mechanisms. It was also much less expensive than the conventional frame. The biggest factor came by eliminating the redundancy penalty required in the IBC for the conventional configuration. Using a rational Performance Based Design, our office successfully demonstrated - via a peer review process - that the mast frame activates all BRBs as it deforms (independent of loading pattern). The tilting mode allows vertical redistribution of forces if any brace at any level were to fail. We demonstrated this redundancy by comparing the limits of strength loss from a failed brace through our analysis. Consequently, a building with conventional frames would need to be 30% stronger than a trussed mast frame. The reduction in base shear allowed the total number of frames to be reduced from seven (7) to four (4). This had a very positive impact on the architectural layout. Another large driver in cost comes from reducing the number of expensive BRBs per frame from eight (8) to four (4). The total cost saving was $400,000 out of a structural cost of $6.2 Million and a project cost of $25.4 Million.

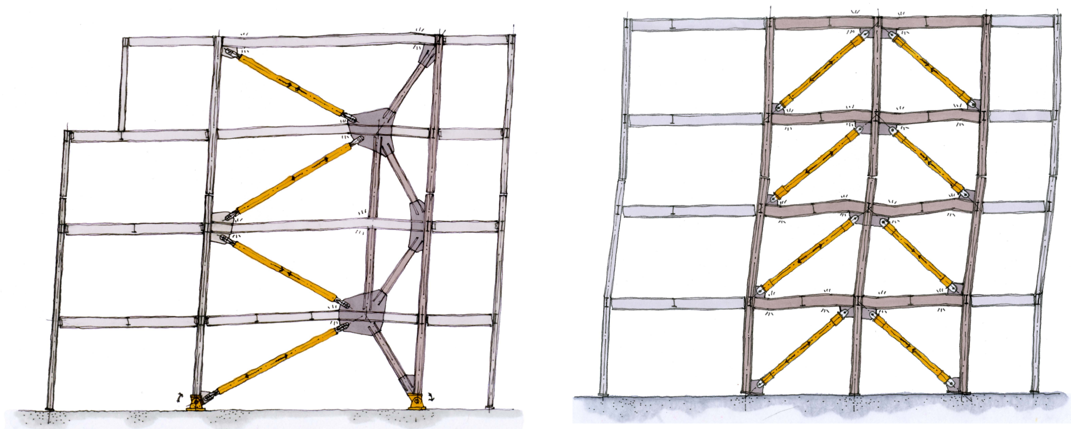

Figure 6: Trussed mast frame (left) with half the BRBs of a conventional chevron frame (right).

To ensure that the mast was made adequately strong, the frame was designed based on non-linear time-history analysis. The trussed mast is designed to stay elastic under MCE loading (from the peak loads of seven time history sets).

Rocking and Restoring Features

Frames and walls configured to rock can utilize gravity loads and post tension loads to create a non-linear elastic restoring force. Numerous configurations are possible and a steel and concrete example will each be presented in this paper. Our office is very enthusiastic about the benefits of rocking and restoring systems. We have completed three of these types of projects, with another four currently in design.

Post Tensioned Steel Frames

The lateral system for the new Orinda City Offices, in Orinda, California, is single bay rocking steel braced frames with vertical post tensioning. The post tensioned cables are located near the webs of the steel columns. Connecting the bases of the columns to the foundation are paired steel angles bolted to the frame and the foundation with anchor bolts. The angles are a set of simple and inexpensive L8x8x3/4 pieces, designed to yield when the frame rocks. The mechanism is that of simple bending parallel to the axis. The inspiration for the fuse came from tests of a bolted moment joint (connected by L8x8 angles) performed by Professor Hassan Astaneh at U.C. Berkeley (See Figure 7 for the test specimen). They function in a similar way to the mild reinforcement in the concrete walls above. The design intent is that these will be replaced after an earthquake, while the frame and the post tensioned cables remain elastic.

This design illustrates the deflection limits of the post tensioned cables. The material has a limited stress and strain range. Part of the available strain is used in the initial prestress of the frame. The remainder is needed to accommodate the stretch induced by the rigid body rocking. The design used 0.8 GUTS (guaranteed ultimate tension strength) as the maximum stress under the ultimate deflection. To ensure adequate drift capacity, the cables were looped under the frame to make the available cable length two times the frame height. The loop was formed by a pair HSS sections, welded to the anchor bolt template. The tendons remained individually sheathed and the tubes were filled with grease.

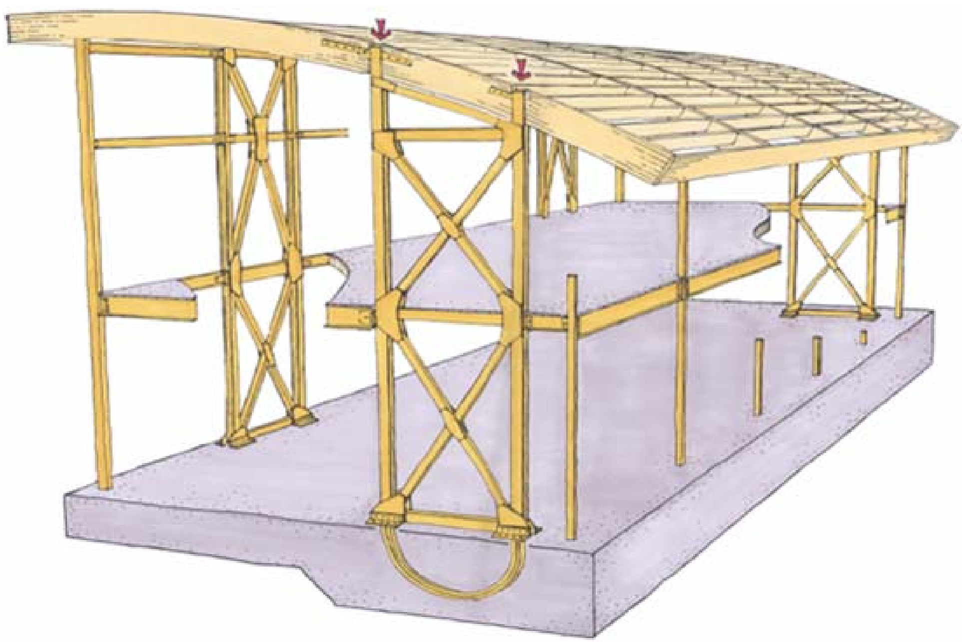

Figure 7. Vertically post tensioned rocking steel braced frame. The loop at the base of the frame houses the PT cable. The photo on the right is the L8x8x3/4 test specimen.

The moment trace vs. drift is a flag shaped hysteretic loop similar to that shown in Figure 13. The form shows the energy absorbing elastic-plastic contribution of the steel angles superimposed with the non-linear elastic contribution of the cables. The post tension force was designed to be greater than the angle yield strength, causing the restoring force to be greater than the angle strength. As such, the PT forces the frame to return to plumb.

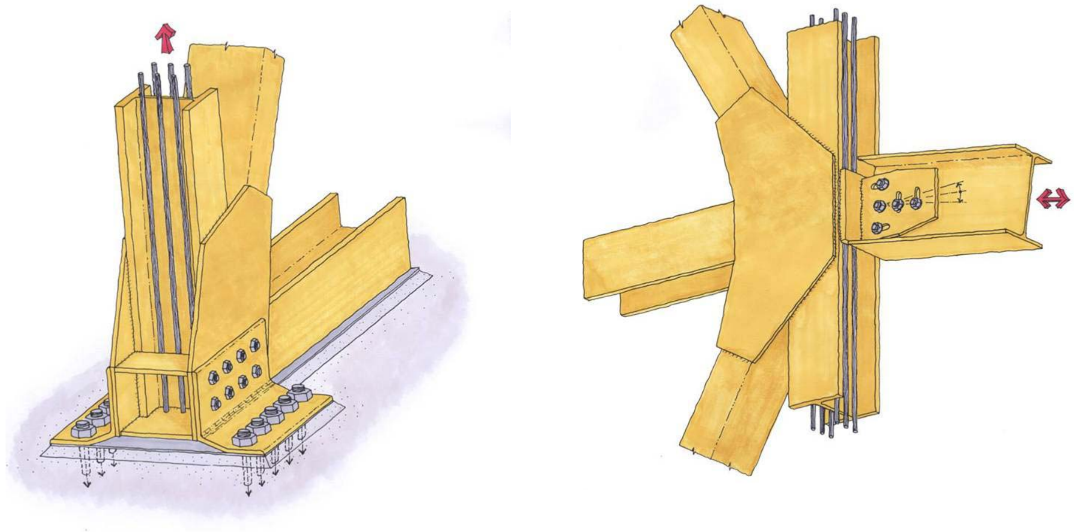

Figure 8. Post tensioned braced frame details. The base of the frame on the right is illustrated in the lifted position with the angles yielding in double curvature. The collector to frame connection on the right shows a pin configuration made up of slotted holes to accommodate the rocking.

The building houses numerous city services including a police station. As such, it is designed as an Essential Facility. Adding the restoring and re-aligning capability contributes greatly to the building's resilience. The design was awarded an Innovation Credit, contributing to the projects LEED Gold certification from the USGBC (U.S. Green Building Council).

The frames for the Orinda City Offices are the forerunners of the post tensioned rocking frames tested by Profs. Greg Deierlein of Stanford University and Jerome Hajjar of University of Illinois on the E-Defense table in Japan (Tipping Mar was an advisor to the research team). The frames performed exceptionally well, remaining elastic and returning to plumb through numerous MCE level records and several damper configurations.

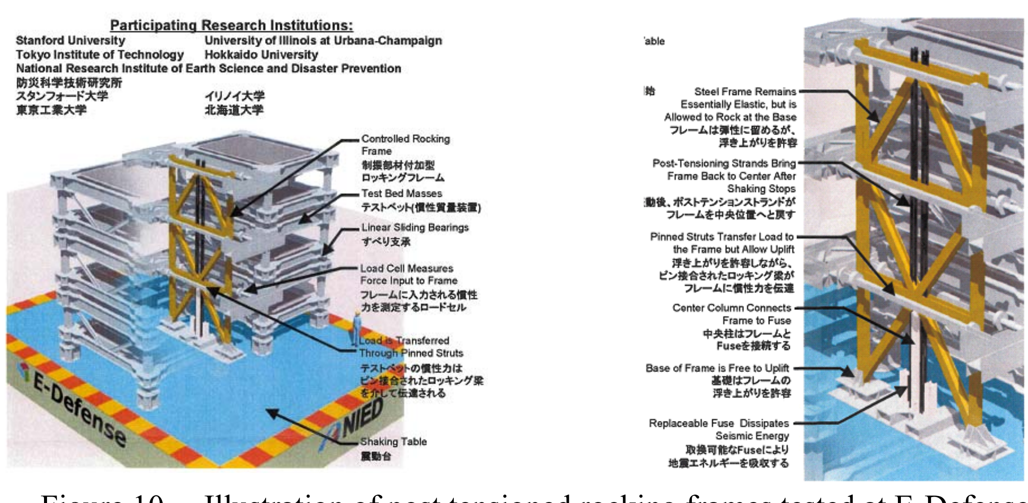

Figure 10. Illustration of post tensioned rocking frames tested at E-Defense in Japan.

Post Tensioned Concrete Walls

The lateral system for Campbell Hall, the new Astrophysics Building at U.C. Berkeley, is vertically post tensioned flexural walls. Like post tensioned frames, these walls are designed to re-align after a major earthquake, resulting in very little structural damage. (The design for the project is nearly complete as of this writing.) Our team capitalized on the technical insights of the University's Seismic Review Committee (SRC) to realize significant performance enhancements and reduce construction costs. Since the post tensioned walls re-align the structure after large displacements, the overall performance can be optimized to minimize overall costs (both first costs and life cycle costs) by minimizing seismic damage, repair, and loss of use.

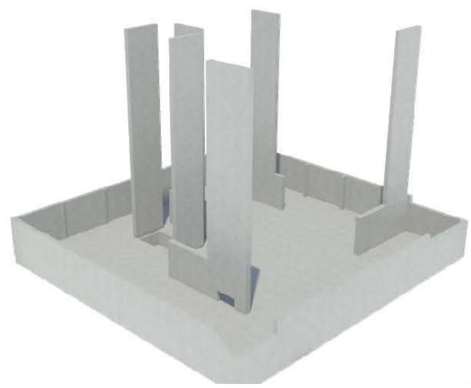

Figure 11. Campbell Hall Astrophysics Building at U.C. Berkeley with PT concrete wall layout.

The design team was able to reduce construction costs by bypassing prescriptive code requirements in two areas. The first step was to ignore the penalties that increase the minimum design strength incurred due to not meeting redundancy expectations (the **r** factor for redundancy). The second step was to not have the strength constrain the code response modification factor for special concrete walls (R = 6). Our fundamental design approach was to consider the role of strength as it effects peak displacement. Displacement was then related to the damageability and post earthquake repair costs of the pre-cast concrete skin system. The design strength was initially established based on using a response modification factor (R) equal to 8. This reduction of strength was reasonable considering the resilience of the walls.

A non-linear time-history analysis was performed considering the DBE and MCE hazards. For all responses (7 sets of matched records, scaled for each hazard), the residual drifts were negligible. The analysis was performed with fiber models using CSI's Perform. The walls were conservatively designed for the peak shears of the MCE responses, because this is a brittle mechanism.

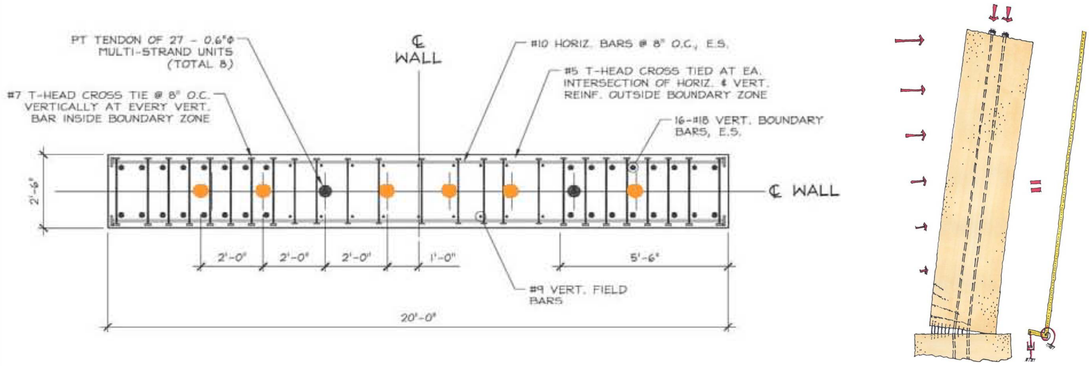

Figure 12. Post tensioned concrete flexural wall section and elevation.

The reduced strength allowed the number of walls to be reduced from eight (8) to six (6) with significant benefits to the architectural layout. The construction savings are estimated to be around $300,000 out of a structural cost of $9.5 Million and a construction cost of $39 Million. In consideration of life cycle costs, the architectural skin supplier was consulted to establish the drifts that trigger damage. Drift less than 1.2% would result in essentially no damage. The drifts under the DBEs were set to be below this limit. Both the structure and skin system are designed to incur very little damage under a DBE event, satisfying Immediate Occupancy requirements at very low costs.

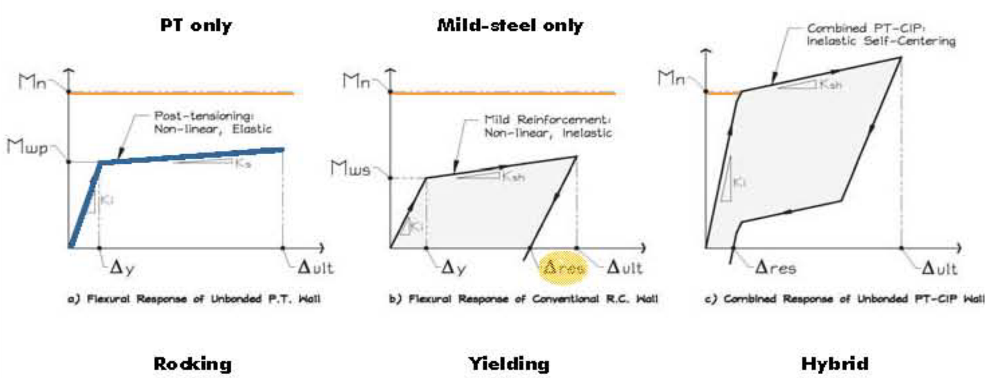

Figure 13. A simple model to describe the hysteretic behavior of PT wall systems as the sum of non-linear elastic rocking plus yielding bi-linear responses.

Conclusion

This paper illustrates several examples of Performance Based Design using mode shaping spines used to control drift. The projects significantly improve performance while being cost effective in comparison to conventional systems. Integrated design using re-aligning frames and walls, allows the synthesis of overall project goals – the improvement of seismic resistance, the reduction of first costs, the protection of architectural skin, and the reduction of damage and loss of use.