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DELIVERING ENHANCED SEISMIC PERFORMANCE AND RESILIENCY THROUGH THE USE OF KRAWINKLER FUSES: THREE CASE STUDIES

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Abstract

After the 1989 Loma Prieta earthquake, structural engineers working on complex facilities in the Western United States have been required to meet the challenges of elevated performance standards chosen by their clients, as well as the evolving regulatory guidelines for solving structural problems. One reality that has emerged during the last thirty years is that the controlling performance target for most of these complex facilities has expanded from preventing loss of life to reduction of property damage so that downtime is reduced and resilience is increased. Increasingly, complex facilities are expected to survive major earthquakes; it is less acceptable for the entire building to be “sacrificial.” It is no longer sufficient for facility occupants and visitors to escape with their lives when the complex facility weathers a foreseeable MCE, DBE or service level earthquake.

Three recent examples will illustrate new design approaches that can assist structural engineers in meeting the increasingly common owner requirement that the lateral force (seismic) resisting system for its new facility outperform minimum code standards, and prevent significant disruption of core operations following a major earthquake by using designated fuses to protect the primary structure rather than sacrificing the primary structure itself..

The essential dynamic characteristic of the designated fuse concept is that the fuse is inserted into the lateral load paths at key locations to limit the amount of force to a predetermined level. In each of the three projects, the topologies of the load paths are unique to the particular project and the position of the fuses (denominated “Krawinkler Fuses”) are different, but this function remains constant. The fuses have a predictable robust hysteresis curve, absorb energy and reduce the overall structural response. The damage resulting from inelastic deformations is concentrated in the designated ductile fuses through the use of custom connection components and detailing in the rest of the structure to assure that the it can accommodate the MCE target displacements without damage.

Design for the Cinematic Arts Complex, a four story building with a basement at the University of Southern California, was initiated during 2006 and included an owner’s mandate that it remain functional for at least 100 years (hence, owner’s performance target was a “100 year building”).

The structural design for the Isle of Capri, which is a one story long span structure located in the New Madrid fault zone of influence on the Mississippi river in the central US, incorporates Krawinkler Fuses into a system of Prestressed Self-Centering Prefabricated Rocking Vertical Braced Frames founded on an analytically rigid basin wall and a system of fused Horizontal Moment Reaction Trusses.

Krawinkler Fuses are an essential element in a BRB Guyed Pinned Strongback system that forms the seismic resisting design for several modules at Gigafactory 1 in Sparks, Nevada.

In all of these case studies, the structure is designed using conventional code seismic strength design approaches considering the force limitation produced by the fuses. The fuses and special detailing produce superior post-elastic performance minimizing accelerations, displacements, and non-structural damage and virtually eliminating damage to the primary structure.

Keywords: performance-based design, resilience, Krawinkler Fuses

1. Introduction

Back in 1995, Professor Helmut Krawinkler, Professor Emeritus of Structural Engineering at Stanford University, and a world-renowned expert in earthquake engineering, said "It is no longer reasonable to design structures based solely on the strength approaches contained in current codes. We must focus on performance-based design that considers all limit states relevant to the owner and society and pays tribute to life cycle cost considerations."

The International Building Code references ASCE 7-16 for seismic design requirements. ASCE 7-16 lists 85 types of structural seismic resisting systems. Every one of the systems listed can be considered a sacrificial structure, i.e., the earthquake energy is dissipated through the inelastic deformation of the primary gravity structure. We have designed buildings this way since 1933, going on 87 years now.

Not only has the mindset that earthquake resistance is a matter of strength and ductility persisted in our codes, the codes are written in a way that reinforces and promotes that mindset. The word fuse is not mentioned in ASCE 7 or in the IBC. Yet, it is possible to design structures that both adhere to the code, perform in an exemplary manner after the onset inelastic action in the structure, and that are designed to perform in a manner that preserves the structural integrity of the primary gravity structure.

In 2006, Professor Krawinkler was consulted regarding an appropriate objective for the design of the structure for the new School of Cinematic Arts at the University of Southern California that would be worthy of such a stellar organization. Without hesitation, he responded "Make it repairable after an MCE." This paper provides a brief overview what has transpired since then in response to his dictum. Professor Krawinkler provided guidance in the conceptual design of the first two projects. His legacy is reflected in the third project which was designed after his passing in 2012.

2. USC School of Cinema

Phase 1 of the USC School of Cinematic Arts is a state-of-the-art facility with over 130,000 sq-ft of floor area devoted to offices, classrooms, sound labs, and screening rooms. The building comprises two four story wings flanking a courtyard over a full basement designed as a Mediterranean Revival architectural style building that required a joint-free stucco façade. The AEC team was challenged to develop a structural system that would respond to the unique architecture, the desire for "100-year building", and the high seismicity of the site in a manner consistent with a world class, forward-looking institution of higher learning. [1]

2.1 Project Constraints

In developing a structural solution there were a number of competing interests, cost and schedule being uppermost on the mind of the contractor who was engaged during the design phase:

- The desired construction schedule could most easily be met with steel framed structure with composite floors
- The owners desire for a Venetian Plaster Interpretive Italian Renaissance façade with no joints which dictated something comparable with traditional masonry construction.
- For optimal performance and reliability, the structure would require ductility which would normally be a moment frame in a building with no place for braces.
- To be repairable, the structural damage should be limited to easily replaceable components, which is incompatible with a traditional concrete or steel moment frame.
- To minimize non-structural damage both drift and floor accelerations should be limited. [2] [3]

- There was no time in the design schedule to go through the time-consuming process of pursuing the testing and peer review processes normally associated with an innovative structure.

2.2 Conceptual Design.

The project team, which included an active and assertive owner, launched an intensive and wide-ranging conceptual design phase that was focused on the structure [1]. The concept that was chosen was basically a steel framed beam and post building with infilled exterior concrete shear walls. The steel floors would go up quickly, enabling the roof to go in and interior services installed early in the construction schedule. The concrete walls would be pneumatically placed concrete (shotcrete) installed from exterior scaffolding which would then double as the work platforms for the Venetian Plaster installation. The shotcrete was high strength, shot against interior forms which left a smooth interior finish, and left with a rough wire screed finish on the outside which was an ideal substrate for the finish. This relatively straightforward and conventional structure satisfies the stiffness requirement and the architectural constraints but given the punched windows that are larger at the bottom of the structure, it would fall short of the ductility and repairability criteria.

To solve these challenges the exterior columns were designed to be 10 inches deep and spaced at 10 ft on center, turned with webs perpendicular to the wall, pushed out into the plane of the wall so that the inside face was flush with the inside face of the wall and 2” of concrete would cover the outside face, thereby creating short wall segments separated by predetermined crack locations. The segments of the wall were separated from the foundation wall by 2” of expanded polystyrene (EPS) so they would be free to rotate, connected to the foundation at the center by a steel lug, and to the adjacent columns by yielding plates. The steel columns act as boundary elements for the individual walls, already an improvement over concrete boundary elements that will crush in compression, crack in tension, and deteriorate with cyclic loading.

The amount of shear that is carried by the walls is limited by the shear capacity of the connections to the columns. At the time of the conceptual design there was a collaborative Stanford – University of Illinois research project studying fused rocking walls. Professor Krawinkler suggested testing a butterfly plate for use in the School of Cinema project. The structural engineer provided the plates that were tested as part of the program [4]. Figure 1 is a photo of one of the plates tested and the corresponding plot of the cyclic testing

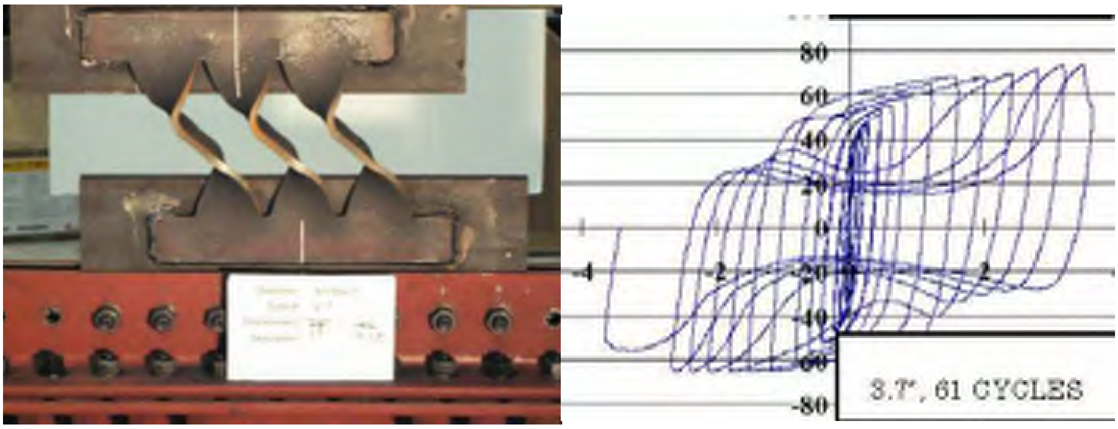


Fig. 1. Slit Shear Plate Test

The walls created by this system are properly called “rotating” or “pivoting” walls because they rotate about a center point. In this system, the boundary elements, the columns, translate with no vertical movement due to lateral loads at the floors, and the center of wall also translates with no vertical movement. This makes it possible to connect the floor beams to the columns and the floor diaphragm centered between the beams to the wall with a pin that transfers shear into the wall. Figure 2 shows an overall view of the concept as presented to the project team during concept design. The movement takes place between the boundary column and the wall. This behavior has the advantage of solving the conundrum of incompatible displacements at the corner of the building when the walls in each direction are isolated for in-plane movements but restrained for out of plane (wind & seismic) movements.

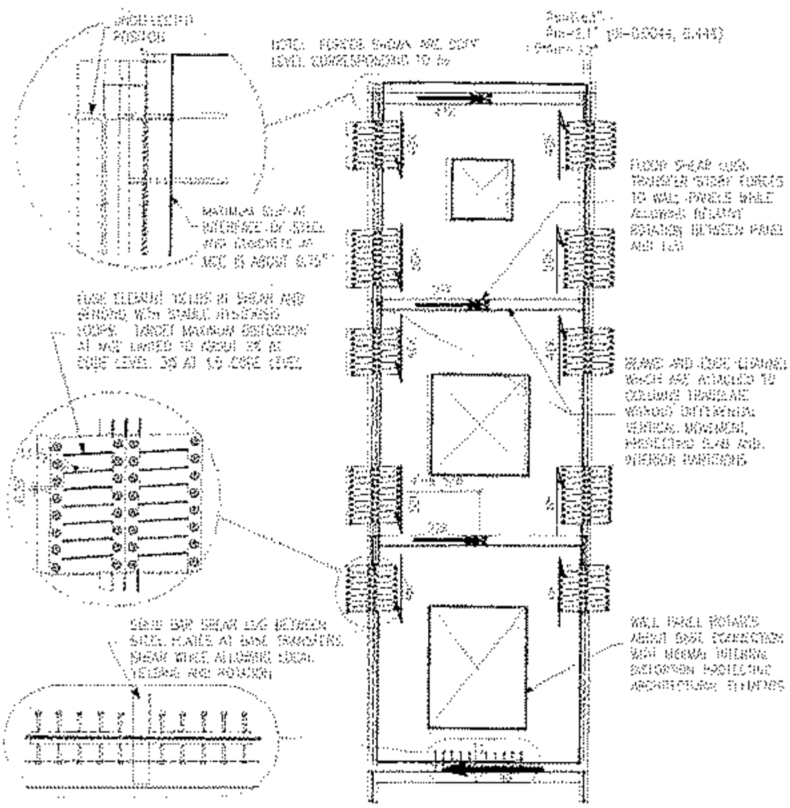


Fig 2. CIP System Concept Forces and Displacements

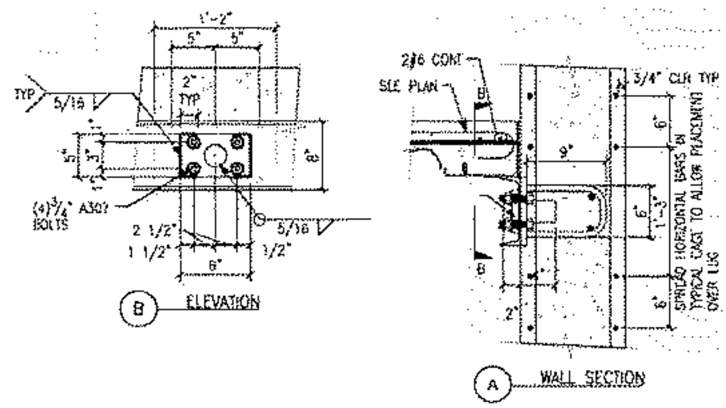


Fig 3. Floor Shear Lug

The key to controlling damage was to follow through on the details required to provide for the anticipated movement. Figure 4 shows a photo and drawings of the typical pin connection that occurs at the center of each wall at each floor. These lugs were designed for the maximum of the code forces or 1g on the floor. The latter value was recommended by Professor Krawinkler based on his experience with non-linear time history analyses of a range of structures.

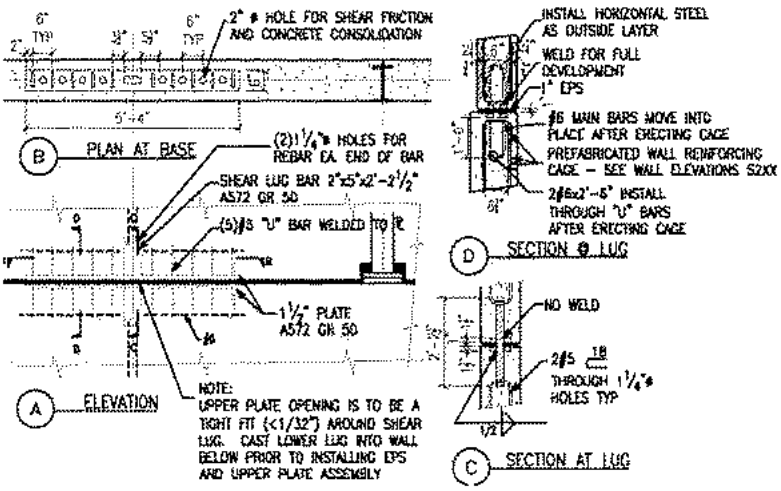


Fig 4. Wall Base Shear Lug

Figure 5 shows a photo and drawings of the typical base shear lug that occurs at the center of each wall at the top of the basement wall.

2.3 Permit Process

At the initial meeting with LADBS, it was made clear that if the team were to pursue a full performance-based design approach, the system would have to be subjected to a rigorous testing program and the peer review process. However, it was noted that the system utilized ordinary elements and materials and could be designed using conventional practice if a suitable R factor could be determined based on provisions of the then-applicable 2001 CBC/2002 LABC.

The team opted for the latter approach and submitted justification for using an R factor of 5.5, the same as for shear walls, since the primary shear load path is through the concrete infill to the base lug. The LADBS approved this approach.

While this development greatly simplified the permit process, it resulted in additional complexities in the analysis and design. The original objective was twofold: develop a robust structure that would exceed code minimum requirements and protect the architectural and non-fuse structural elements. It is relatively simple to control the maximum strength of a system at an assumed limit state, which was the organizing philosophy prior to the meeting with LADBS.

It is somewhat more difficult - if not impossible - to have the same strength limit when the system is designed based on an elastic analysis as the elastic force distribution does not necessarily match the plastic distribution. In order to try to match the elastic and plastic force distributions to avoid unintended negative consequences to the "protected" elements from higher forces due to overstrength, the team developed a range of both "butterfly" and straight blades of varying strength to stiffness ratios.

2.4 USC Summary

The team moved forward with an innovative structural system, approved by the LADBS based on code conventional construction requirements that has the following attributes in its favor:

- Concentration of inelastic deformations at well-defined locations by using tuned fuses and protecting the remainder of the structural system from overloads that might lead to undesirable damage or failure modes at other locations. This protects the structural system from unexpected damage and deterioration.
- The fuses are installed in a manner that greatly facilitates replacement in case of an earthquake causing significant inelastic deformations. This will reduce downtime and structural repair costs in case of a severe earthquake.
- The structural system has a large elastic stiffness, which will reduce drift dependent nonstructural damage.
- The strength of the fuses is tuned to code strength requirements without considerable over strength, which causes localized ductile yielding and limits floor accelerations to tolerable values that will control acceleration dependent nonstructural damage.
- The fuses possess very large inelastic deformations capacity without loss of strength, which provides a large margin of safety against collapse at an MCE level event.

The construction of the system took advantage of the opportunity to prefabricate the rebar cages for the 10-foot-wide 12" thick walls as well as the expedited early delivery of the composite steel floors and the building was deliver ahead of schedule and under budget. Ultimately there were two more phases at the School of Cinematic Arts that used the same lateral system.

3. Isle of Capri, Cape Girardeau, Missouri

As the design of the third phase of the USC School of Cinematic Arts was getting under way, the design started for the Isla of Capri Casino in Cape Girardeau, Missouri. The owner, the contractor, and the entire design team except for the structural engineer was from St Louis where seismic design is a mild irritant in the background of most designs. However, the site in Cape Girardeau had an Ss = 1.5 and an S1 = 0.64, 80% of the values in Los Angeles at the site of the School of Cinematic Arts. The challenge was to provide a state-of-the-art performance-based design for a seismic resisting system to a group of people who were not familiar with the sorts of design and construction requirements that people in the cities of California are used to.

3.1 Project Constraints

The primary feature of the building is a 7 foot deep 176 ft x 233 ft water-filled basin in which floats a 5 ft deep precast concrete and CIP barge that is the casino floor (in Missouri all casinos must be floating). The roof of the casino is higher than the surrounding areas such as restaurants and non-gambling entertainment and, of course, is a deep long-span structure. The long-span diaphragms would be a challenge no matter what lateral system was chosen. Even more challenging would be the support of all the mechanical systems in the ceiling area of the casino.

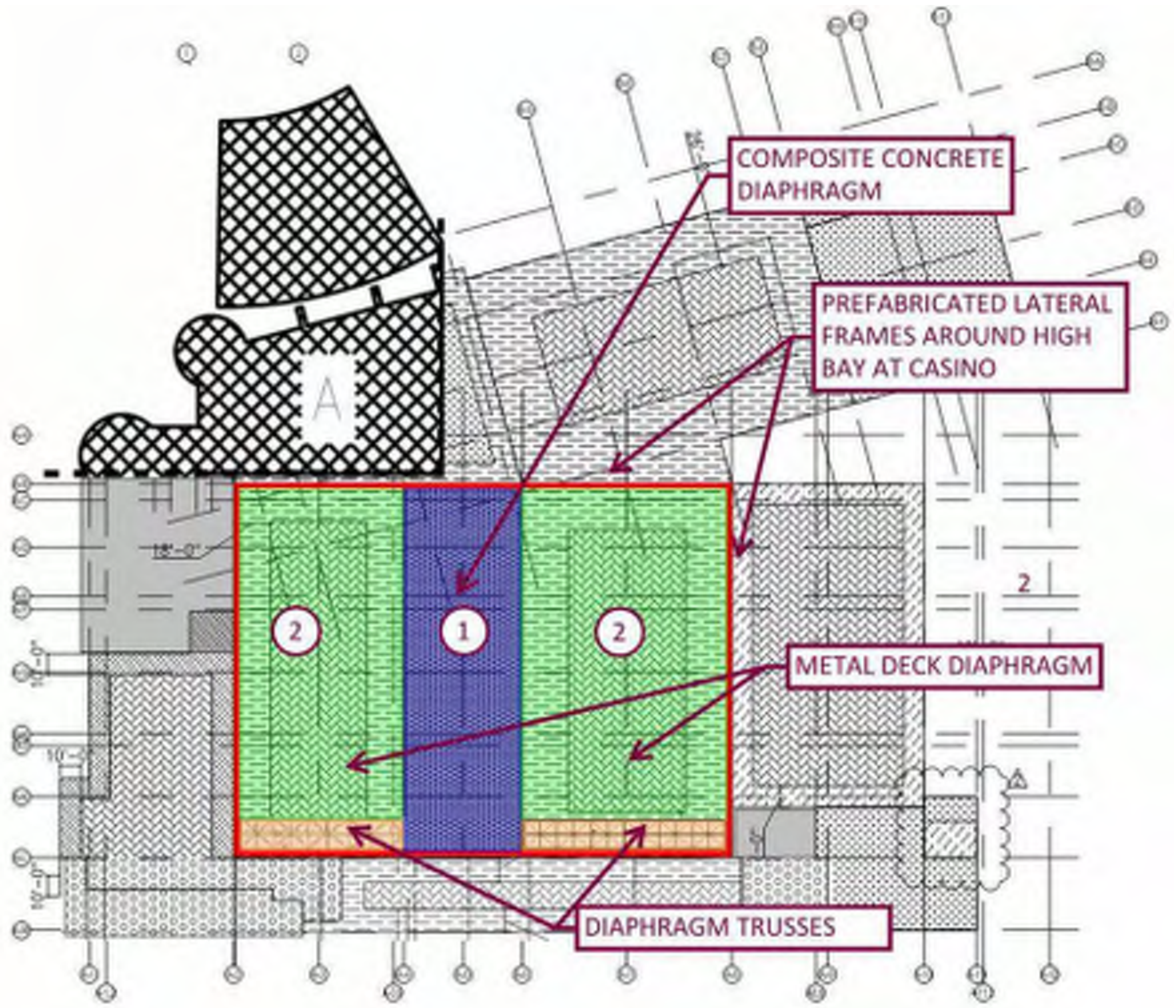
3.2 Conceptual Design

As with the School of Cinema, the approach that the engineers took was to provide a code compliant structure with some modifications to alter its post elastic performance.

At the first design meeting it was decided to limit the height of the casino roof to 35 ft and the period of the structure to less than 0.35 seconds so that an ordinary concentric braced frame structure could be used. This comes with an R=3.25 resulting in very high level of design forces. The advantage is that it minimizes the special procedures that would be required of the local sub-contractors.

The second thing that was done was to make all the braced frames 10 ft wide, because 10 ft x 35 ft frames could be shop fabricated and shipped, and locate them around the perimeter of the basin so the basin wall could be used to distribute the concentrated forces from the braced frames. Ultimately, this allowed a fabricator in West Point, Mississippi, who had a lot of steel left over from a Gulf Coast project that was stopped because of Hurricane Katrina, to supply the steel. Since all the complicated steel was fabricated in the shop and field bolted, it allowed the erection to be done by a small erector who came across the bridge from his farm in Illinois to do the erection.

The third thing that was done was to put all the mechanical units at the center of the high roof on a composite concrete slab supported on steel beams and girders that would penetrate through the barge to the basin slab. The composite slab provided an analytically rigid rectangular geometric shape at the primary diaphragm and served to span the 176 ft diaphragm span. The corners of the rigid diaphragm were supported on one side of the casino in the back-of-house area by two large braced frames dividing the main diaphragm into three sections, one of which was the rigid diaphragm itself, cutting the span of the metal deck



diaphragm to 101 ft.

Fig. 5. Isle of Capri Lateral Frame and Diaphragm Plan

The final thing that was done to complete the initial code compliant design was to design the diaphragms of the various areas outside the casino area to the casino lateral system so the braced frames would support the majority of the roof areas.

The code compliant design using non-ductile ordinary braced frames was then transformed into a state-of-the-art high-performance lateral system by:

1. detailing the base of the frame columns to allow them to lift up while being restrained horizontally;
2. providing (4) post tensioned Dywidag rods on the frame columns anchored into pilasters that are built integral with the basin walls and prestressed to exactly balance the code uplift forces with sufficient reserve capacity to reach target displacements without exceeding 75% of the ultimate capacity of the rods;
3. providing a rocker connection at the top chord of the horizontal trusses that tie the vertical frames together that allows rotation but resists horizontal forces;
4. and providing Krawinkler Butterfly fuses tying the bottom chord of the horizontal truss to the vertical braced frames.

Up until the forces reach code level, the structure performs as an ordinary braced frame. Once rocking starts the base moment of the frames is constant. However, a mechanism is not reached because the moment can be resisted by the horizontal trusses at the top. The force deflection-curve changes because of the difference in the stiffness of the moment load path once the Dywidag rods and the fused horizontal trusses take over. This change reduces the response of the structure. The structure still doesn't reach a mechanism when the fuses reach their capacity because the Dywidags haven't reached their capacity and can supply additional overturning resistance, but the damping is increased.

The members are designed for 125% of the forces at target displacement to assure that that the frames will actually rock and relieve the forces before any of the primary frame members reach a limit state. This is an artifact of the code development over time from Allowable Stress Design to Load and Resistance Factor Design. It seems illogical that the steel in the rocking frame design has to be designed for higher forces than a code compliant design.

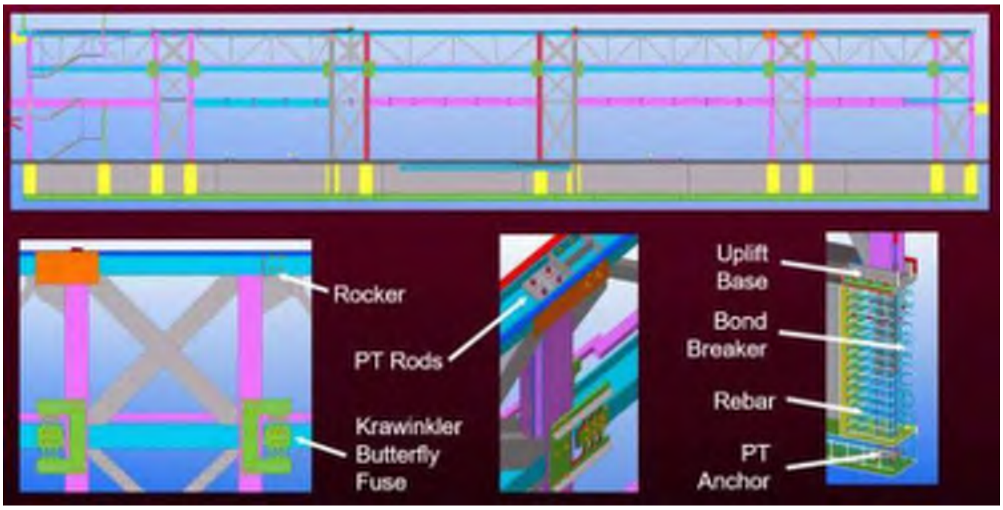


Fig. 6 Components of the Pre-stressed Fused Rocking Frame System.

Figure 6 show portions of the 3D design and detailing model for the one of the Pre-stressed Fused Rocking Frames. Figure 7 shows construction photos of the frames.

Fig. 7 Construction Photos of the Pre-stressed Fused Rocking Frame System.

Other aspects of project involved performance-based design.

The barge is not subject to seismic motion because water does not transmit shear, but the basin side walls will move in an earthquake. The basin side walls are designed for an impact force corresponding to the momentum of the barge moving at a velocity corresponding to a sign wave with an amplitude of 36 inches impacting the elastic side wall. The design forces were huge. That would seem to be a place where viscous dampers could be put to good use.

All of the mechanical systems are required to be braced. As part of the structural package, horizontal L4x3 angles were provided on a 12 ft x 12 ft grid suspended from the joists at 8 ft on center (8x12 grid). The angles were designed to support the vertical weight of ceilings and mechanical systems and carried the seismic forces in tension back to the structure at the sides of the high bay. The grid is reinforced locally where the architect wants to suspend a special ceiling feature.

3.3 Permit Process

The local authorities reviewed the structure based on the elastic analysis of the ordinary braced frame.

3.4 Summary of Cape Girardeau

It is possible to fulfill all the seismic code requirements and go beyond that to provide a high performing seismic resisting system and convenient and effective bracing for non-structural elements of the building in a region where the local design professions, contractors, and tradesmen are unfamiliar with seismic construction details and traditions that are common in high seismic areas like California.

4. Gigafactory 1, Sparks, Nevada

At the beginning of April, 2016, the owner was in a big hurry for the factory in Sparks, Nevada to be completed. The engineer was told that the building had to be designed and submitted for permit in 6 weeks. The owner wanted to start erecting steel in July, enclose the building, and turn it over to process 11 months after the engineer received the NTP.

Fig. 8 Gigafactory 1 Layout

4.1 Project Constraints

The site has a Ss = 1.2 and S1 = 0.4, so not as high as the Cape Girardeau site. The site class is C and per the owner's instructions, an I of 1.25 is to be used presumably to make the structure safer. Beyond specifying and I of 1.25, there were no discussions about what the owner's expectations might be in regard to down time or repair costs in the event of a seismic event. It would have been interesting to determine what the present value of an extreme event with a small probability that causes significant interruption and repair cost.

4.2 Conceptual Design

The geometry evolved to appear as giant Chevrons stretching over several bays tying the top of the doubled-up frames to the ground as shown on the left side of Figure 10. The giant chevrons are BRB's so the global behavior of the system is BRB. The upper right of Figure 10 is a screen short of the design and detailing model showing the bolted Krawinkler Butterfly Fuses which are bolted symmetrically front and back of the frame. The lower right side of Figure 10 shows another fundamental property of the system. No matter where forces are applied to the frame (or mass is distributed to the floors), it acts as a strongback to distribute the forces to the top and bottom nodes where they are picked up by the global BRB system.

Having solved the elastic problem, the team turned its attention to constructability and the post-elastic behavior. The two joints which are located close to the neutral axis of the combined section are the ideal location for Krawinkler Fuses. Providing a pinned support at the far side of the strongback maximizes the fuse deformation for a given drift angle. The overturning in the double frame is carried by the reactions in the pins. All of the fuses on this project were the same size. The guyed geometry and stiff strongback enforced reasonable consistent behavior of all of the fuses. .

Figure 9 Typical BRB Guyed Fused Strong Back

It should be noted that where the BRB's cross a horizontal without a vertical, the forces have to balance above and below so the BRB will be the same size at both levels. With his system the variation in shear from floor to floor is handled by the strongbacks while the global shear is carried by the BRB guys.

MASTAN and RISA were the primary analytical tools used for preliminary proportioning. A nonlinear SAP beam on elastic foundation analysis with compression only spring was used to produce design forces for the grade beam foundations. A non-linear time history analyses were also performed using ETABS but were primarily used to verify that the structure was behaving as was expected in the non-linear range. Figure 11 shows a comparison of the displacements and acceleration time histories for the fused BRB system and moment frame with equivalent stiffness.

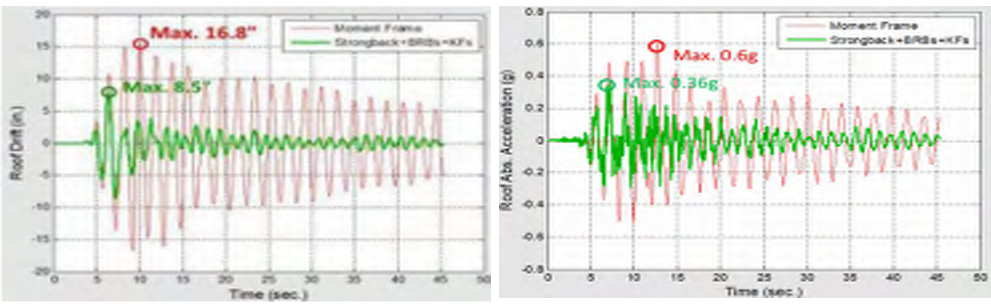


Figure 10 Non-linear Time History Comparison

4.3 Permit Process

The Story County, Nevada, building department was the AHJ for these projects. The design review was performed by a local engineering company who accepted the linear elastic analysis.

4.4 Denoument

When the last two buildings (Buildings A' and F' in Figure 9) the decision was made that they fell under the provisions of section 12.2.1.1 ASCE 7-16 for "ALTERNATIVE STRUCTURAL SYSTEM." As the design schedule was even more ambitious than the 2016 schedule, this process had to be carried out in parallel and simultaneously with the interleaved design and detailing processes. At the end of the day after an excruciating 2 months of non-stop nonlinear response history analyses, the design was judged acceptable. Shortly thereafter the projects were canceled and were never built.

3. Conclusions

There are many questions on which we could use some insight:

- Is it possible that fused systems might reduce the need for precision in the time histories?
- Might the presence of fuses and strongbacks provide the capability of a system to absorb significant impulses without undue distress?
- How can viscous, friction, and inelastic damping be incorporated into our structural systems and should they?
- What is the optimum number of Krawinkler Butterfly Fuses in a system?
- Would it be better to let some fuses yield and start damping out response before code loads are reached?
- Can Krawinkler Fuses be used to add strength and damping to existing buildings?

4. Acknowledgements

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6. References

[1] Luth G, Supriya S, Krawinkler H, McDonald B. (2008) USC School of Cinema: An example of reparable performance based design. Proceedings, 77th Annual Structural Engineers Association of California (SEAOC) Convention, Hawaii

[2] Miranda, E. and Aslani, H. (2003) "Probabilistic Response Assessment for Building-Specific Loss Estimation," *Report No. PEER 2003/03*, Pacific Earthquake Engineering Research Center, University of California, Berkeley.

[3] Mitrani-Reiser, J. (2007) "An Ounce of Prevention: Probabilistic Loss Estimation for Performance-Based Earthquake Engineering,". Ph. D. thesis, California Institute of Technology, Pasadena, California, USA.

[4] MR Eatherton, GG Deierlein, X Ma, H Krawinkler, JF Hajjar (2012) "Toward a Performance-Based Design Framework For Self-Centering Rocking Braced-Frame Spine Systems" 15th World Conference on Earthquake Engineering 2012

[5] Osteraas J, Hunt J, Luth G. 2018 "Innovative Lateral System with Mechanical Fuses and Strongback Frames". Proceedings, 11th U.S. National Conference on Earthquake Engineering, Los Angeles, CA,

[6] Hunt J, Osteraas J, Luth G. 2017 Performance Based Seismic Design of the Gigafactory in Tesla Time. SEAOC 2017 Convention Proceedings, Structural Engineers of California, San Diego, CA,.

[7] Luth G, Osteraas J. (2017) Delivering the Gigafactory in Tesla Time Using HD BIM. SEAOC 2017 Convention Proceedings, Structural Engineers of California, San Diego, CA,

