

Warren Hall Seismic Retrofit vs. Demolition – A Feasibility Analysis - Case Study

Arjun R Pandey¹, Farzad Shahbodaghlou², and Saeid Motavalli³

¹ Illinois Institute of Technology, Chicago, USA

² California State University East Bay, Hayward, USA

³ California State University East Bay, Hayward, USA

ABSTRACT: This is a case study about the demolition of a 13-story concrete and steel building named Warren Hall located on California State University, East Bay campus Hayward, California. The case study covers the study of vulnerability of the building, which posed life threats to the occupants, structural deficiency when subjected to lateral forces, the proposed seismic retrofit systems to strengthen the building and the other selection of alternatives, and the final demolition decision and the demolition of the building.

1 INTRODUCTION

The Warren Hall was a 13-story concrete and steel frame building located on California State University East Bay (CSUEB) campus in Hayward, California. The total area of the building was approximately 145,000 square feet. Each floors area was approximately 12,000 square feet. There was a two-story connector bridge that spanned over West Loop Road and connected with the Library building. The connector bridge was structurally attached to the Warren Hall and abutted the library with a seismic joint.

CSUEB undertook several studies of the Warren Hall and the attached Library building. In 1999, Rutherford and Chekene performed a seismic study of Warren Hall in order to assess the performance of the structure when subjected to strong seismic ground motions. The major finding of this study was that there would be a brittle shear failure of the primary lateral force-resisting system. The firm also designed two retrofit schemes (Schemes 1 and 2) focusing on reducing the longitudinal reinforcement of the perimeter beams by cutting some of it without compromising the vertical loading carrying capacity. These retrofit schemes were not accepted by CSUEB seismic review committee, because of the length of time require to perform the model testing for the beam-column joint recommended. These alternatives could have caused unacceptable delays. In the more detailed 2001 seismic analysis study by Rutherford & Chekene Consulting Engineers, structural deficiencies were identified and it was determined that a major seismic event at the near by Hayward Fault could result severe damage or the loss of life. Rutherford and Checkene also developed several additional retrofit schemes. Schemes 3 and 4 were proposed in order to develop the beam and column flexural capacities in the moment-resisting frame. Then in mid 2002, the firm prepared a complete and detailed budget for the proposed solutions.

A preliminary study in May 2005 explored options for mitigating the structural deficiencies in Warren Hall by the partial deconstruction and structural retrofit of the building. A December 2005 study reviewed an alternative to completely demolish the building. It was this alternative that the University ultimately selected as the best approach to resolve the seismic deficiencies of the Warren Hall.

In January 2013, the CSU board of trustees authorized \$50 million to replace Warren Hall. The building was rated the least earthquake safe building at any CSU campus, in an earthquake, by the CSUEB Seismic Review Board. The project budget included \$12.8 million to cover the cost of removing hazardous materials such as asbestos in ceiling and floor tiles, recycling aluminum window frames and imploding the Warren Hall building.

The demolition of a seismically unsafe landmark building located on a San Francisco Bay Area major fault line, gave researchers a unique opportunity to study the nearby Hayward Fault. When the 13-story Warren Hall was imploded by demolition experts, the geologists Hayward campus of California State University East Bay and the U.S. Geological Survey scientists monitored the pulse of energy on nearly 600 seismometers temporarily placed in a two-mile radius around the building. Observing the paths taken by this pulse of seismic energy through the ground underlying the Hayward area would enable USGS to create 3D images and map the Hayward Fault in unprecedented detail.

2 VULNERABILITY OF THE BUILDING

2.1 *Objective of the study*

The section of the Hayward Fault that runs near the University is presented in Figure 1. The Warren Hall was built approximately 600 meters from the Hayward fault on a hill. This building was one of the most seismically unsafe buildings in the Bay Area. The USGS estimates that there is a 63 % chance of a major earthquake in the Bay Area within the next three decades.

Therefore the objective of the study was to analyse the Seismic Performance Levels of the building which were primarily based upon the following two performance criteria:

- a) Deterministic Median Hayward Earthquake – Life Safety: Substantial damage would be tolerated as long as significant hazards of serious injury or death such as partial collapse were eliminated. This earthquake, characterized as a major earthquake, is very similar to the 20% probability for exceedance in 50 years probabilistic earthquake, which has a return period of 224 years.
- b) 150% of the Deterministic Median Hayward Earthquake – No-collapse: In spite of substantial structural damage, global stability of the structure was to be maintained for the subject earthquake. This earthquake, characterized as a great earthquake, which has a return period of 475 years.



Figure 1. Location of the Hayward Fault and the Warren Hall

2.2 Assessment Methodology and Strengthening Criteria

The 2001 study followed FEMA-273 “NEHRP Guidelines for the Seismic Rehabilitation of Buildings” and the guidelines contained in ATC-40 “Seismic Evaluation and Retrofit of Concrete Buildings”. But in the 2002 study the retrofitted building’s response to a major seismic event was evaluated using FEMA 356, which superseded FEMA 273.

The main design difference between the two is the adoption of the coefficient method, as detailed in FEMA 356, to determine the roof lateral deflection, as opposed to the capacity spectrum method that was adopted in the original assessment, consistent with an alternative procedure of FEMA 273. Both methods were performed for completeness and judged that the capacity spectrum results were too small, hence not conservative enough. Therefore, the seismic lateral roof deflection demands and consequently local element rotation demands were calculated based on the coefficient method.

Deficiencies identified by the evaluation were addressed, with strengthening designed in accordance with the FEMA-356 guidelines: a) Deterministic Median Hayward Earthquake-Life Safety by designing the retrofit system to Life Safety (S3) performance level; b) 150% of Deterministic Median Hayward Earthquake-No collapse by designing the retrofit to Collapse Prevention (S5) performance level.

Because of the symmetry of the structure, a two-dimensional computer model of the building was created, and analysed using SAP 2000 non-linear computer program by CSI Inc. The site specific response spectra used in the design of the retrofit systems for the type of earthquakes mentioned above (major and great) were developed by Geomatrix Inc.

3 STRUCTURAL DEFICIENCY WHEN SUBJECTED TO LATERAL FORCES

There were two major deficiencies of the building that could result in a substantial structural damage. First, the perimeter moment-resisting frame was non-ductile due to several brittle failure modes. Second, the double story height of the lobby floor (1st Level) resulted in a weak and soft story. These two deficiencies are discussed in more detail below.

a) Non-Ductile Perimeter Moment-Resisting Frame: A reinforced concrete frame forms flexural hinges at beam ends, adjacent to the column. These hinges in the system yield and sustain the yield force while undergoing inelastic deformation to dissipate earthquake input energy, thus forming a ductile system. To maintain the integrity of the lateral force-resisting system (i.e. to develop and sustain the flexure hinge mechanisms), sufficient strength of the beam, column and beam-column elements of the frame is required, as well as the ability of the hinge to sustain cyclical and inelastic actions. But, the Warren Hall perimeter frame was non-ductile due to the frame elements having insufficient strength to develop the desired beam flexural hinges.

Two insufficient elements/mechanisms in this case were: 1) Column and beam shear: The shear capacity of the beams, consisting of the concrete and reinforcing stirrups, was weaker than flexural capacity of the beam. The beam shear reinforcements consisting of overlapping “C” shaped bars were not developed. The ability of this reinforcement to contribute diminished rapidly as concrete cover spalled, worsening the shear failure deficiency. The shear capacity of the columns was weaker than the flexural capacity of the beam; 2) Beam-Column Joint Shear: Joint failure was expected to occur prior to the development of the strength of the beam longitudinal reinforcement, hence prior to the development of the beam flexural hinge.

Both of these types of shear mechanisms were associated with few but wide diagonal crack formation and significant loss of load carrying capacity. Therefore, the conclusion was that partial to total collapse was likely during or following the designed seismic event.

b) Weak and Soft Story at 1st Level: Although the existing structural form endeavored to offset the double story height effects by introducing “C” shaped interior frames, the story strength and lateral stiffness at the 1st Level were less than those of the stories above and below. A weak story resulted in the majority of the inelastic action occurring at one level, which typically the structural elements could not accommodate.

4 SEISMIC RETROFIT SYSTEM PROPOSED

A key finding of these studies was that there was a possible brittle shear failure of the primary lateral force-resisting system. There were several possible schemes studied but four of them presented in detail. The first two schemes (Schemes 1 and 2) were based on reducing the flexural strength of the beams. They also focused on reducing this tendency for brittle shear failure (reducing the flexural strength of beams) by cutting some of the longitudinal reinforcement of the perimeter beams without compromising the vertical load carrying capacity. As mentioned in the introduction section, because of the length of time required to perform the model testing for the beam-column joint CSUEB’s seismic review committee suggested finding a more conventional solution.

This led to the second phase of the study, which was conducted in 2001, where Rutherford and Chekene developed two other schemes (Schemes 3 and 4). Schemes 3 and 4 were based on improving the existing structure by transforming the non-ductile perimeter frame into a ductile frame. The main difference between the two retrofit schemes was that Scheme 3 consisted of complete frame enhancement, while the Scheme 4 consisted of construction of a shear wall between columns for the full building height. CSUEB review committee chose the retrofit Scheme 3 and this paper has presented only Scheme 3.

4.1 Seismic Retrofit Scheme 3-Retrofit Perimeter Moment-Resisting Frame

The scope of retrofit for this scheme was to improve the existing structure by converting the non-ductile perimeter frame into a ductile frame. This was done by supporting a weak-beam/strong-column strength hierarchy, beam-Column joint strengthening, and providing a series of sub-schemes, which follow. These sub-schemes 1) an elliptical jacket around the beams for shear enhancement, 2) anti-buckling of longitudinal reinforcement and 3) providing an elliptical jacket around the beams for shear enhancement and concrete confinement to all four beam faces. The vertical structural irregularity, which arose from the height of the first floor, was removed by making the columns more rigid and stronger.

The retrofit measures and mitigation mechanisms are discussed in detail below.

4.1.1 Column enlargement by reinforced concrete encasement

Column encasement through the joint region with reinforced concrete increases the beam-column joint effective area and allows for joint horizontal shear reinforcement to be added. The full height column enlargement increases column flexural and shear capacities. This supports conventional strong-column/weak-beam strength hierarchy, mitigates column flexural yielding, and prevents premature column failure. A typical section of the column enlargement by reinforced concrete encasement is shown in Figure 2 below.

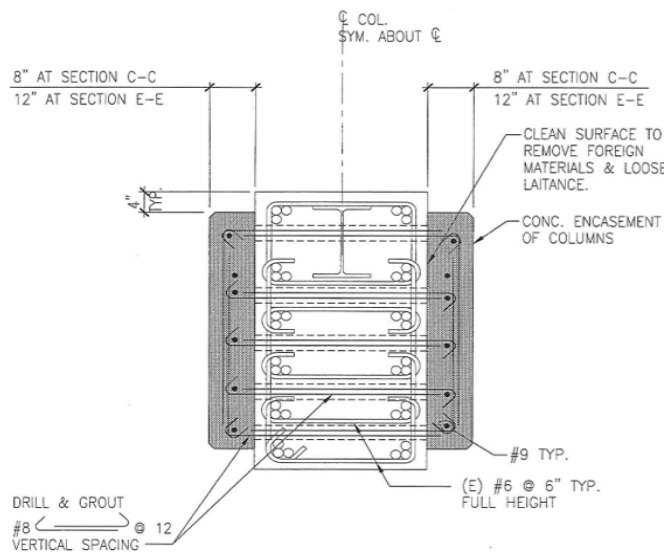


Figure 2. A typical Section of Column Enlargement by Reinforced Concrete Encasement

4.1.2 Elliptical Steel Jacket Encasement of all beams forming the perimeter moment frame

To compensate the excessive beam longitudinal reinforcement contents, an elliptical steel jacket encasement was provided to the level of confinement and shear strength enhancement of these elements. The elliptical steel jacket encasement enabled a radial pressure to be developed and, therefore, confinement to all four faces and to the whole beam section. This allowed concrete strains greater than 0.005 (unconfined concrete limits), which is to be an order of 4 to 16 times

larger than the jacket thickness and effective jacket diameter. This is essential within the potential inelastic flexural region where the maximum plastic rotation demand is 0.04 radians. Providing a $\frac{3}{4}$ " thick jacket for the lower level beams provides an ultimate rotation capacity of at least 0.05 radians. Note that the higher the beam flexural reinforcement content, the greater is the concrete strain demand and hence, for the same geometric conditions, a thicker jacket is required.

By-products of providing confinement are resistance to buckling of beam longitudinal reinforcement and assurance of stirrup lap splice integrity. The later allows the existing stirrups to be used for shear resistance.

The elliptical casing contributes to the beam shear capacity, thus preventing premature beam shear failure and loss of vertical load carrying capacity. In particular, the casing prevents the tendency for the shear failure at termination points for groups of longitudinal reinforcement. A typical section of the Elliptical Steel jacket encasement of beam is shown in Figure 4 below.

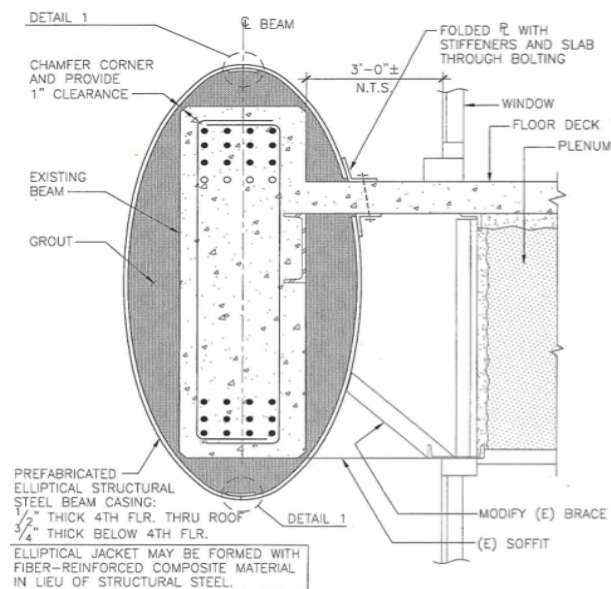


Figure 3. A typical Section of Elliptical Steel Jacket Encasement.

4.1.3 Reinforced concrete encasement of first and second floor beams

Elimination of Weak and Soft Story at 1st Level: In order to mitigate the consequences of a weak and soft story at the 1st Level, the retrofit scope includes reconstructing beams at mid-height of the columns on said floor. Although the new beams were currently limited to the three interior bays per side (and not the outer bays), the new frame system was anticipated to be compatible with the typical floors of the tower.

Note that the construction scope of this retrofit, is based on the full flexural strength of the existing members, that is, excluding cutting of existing reinforcement with the exception of the first floor beams.

4.1.4 Performance

With these retrofit components in place, the pushover curve for the building was analyzed. The result of elliptical beam jackets and concrete encasement in flexural hinges, being the element yielding mechanism, is shown in the Figure 4 below. It is also shown that they provide sufficient ductility for the target displacement to be attained with premature failure.

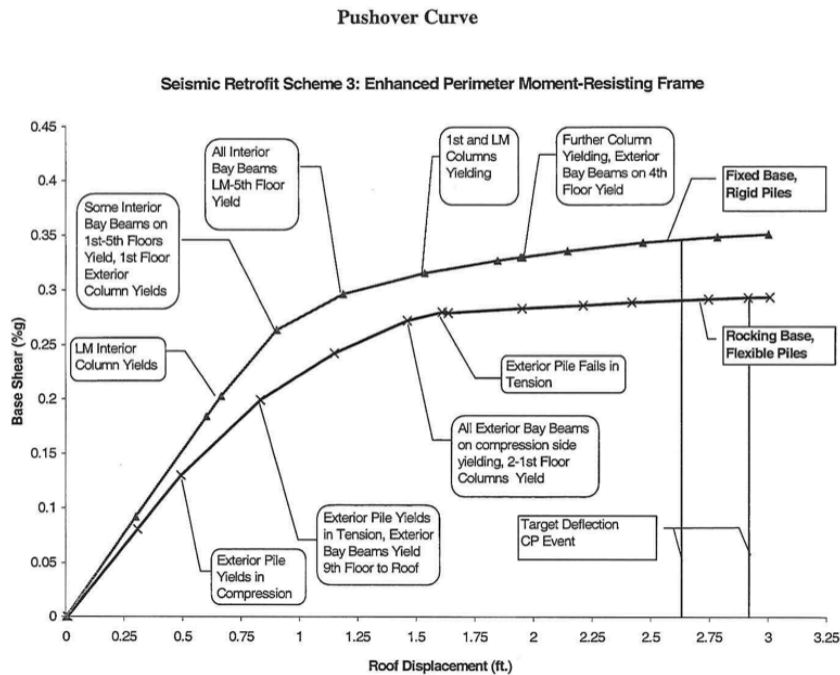


Figure 4.

5 DEMOLITION OF THE BUILDING

Taking down the building posed an engineering challenge because of the tower's proximity to the library and several other campus structures. The demolition contractor, Sundt Construction, placed special timed charges to the vacant building so the building would lean backward and keep the 12,500 tons of concrete rubble and steel away from the other buildings. The demolition experts set over 1,100 detonation charges on the building. That is equivalent to 463 pounds of explosives. The demolition was performed on Saturday August 17, 2013 caused the 13-story landmark building crumbled away from the adjoining library (ABC News, 2013).

All went as planned, except for a 60-foot-high section of the northeast portion of the building that remained more upright than expected. The remaining piece was demolished during the cleaned up period the 60 days following the building demolition.

6 CONCLUSIONS

This paper has presented the first part of the Warren Hall demolition project, which discusses the vulnerabilities of the building and which posed life threats to the occupants, structural deficiency

when subjected to lateral forces, the proposed seismic retrofit systems to strengthen the building and the other selection of alternatives, and the final demolition decision and the demolition of the building.

Upon the release of the USGS data on seismic energy created by the explosion of the building, the data will be analysed and the results of USGS study will be presented in the next SMAR conference as “Warren Hall Demolition Project – Case Study Part Two.”

7 REFERENCES

- ABC News: http://abclocal.go.com/story?section=news/local/east_bay&id=9208862
- Building Seismic Safety Council, 1997, “NEHRP Guidelines for the Seismic Rehabilitation of Buildings”, Washington DC (FEMA-273).
- Building Seismic Safety Council, 2000, “ASCE Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, Washington DC (FEMA-356).
- Computers and Structures Inc. SAP 2000 NL Computer Code.
- Fenves, Gregory L., 1998, “Effects of Footing Rotation on Earthquake Behavior of Pile supported Bridge Piers”, Technical Report for Earth Mechanics, Inc. (NCEER Task Number 106-E-4.1, FHWA Contract Number DTFH61-92-C-00106)
- Huntsman Architectural Group, 2012, “Warren Hall Demolition Project: Design-Build Specification Demolition Narrative”
- International Conference of Building Officials, 1997, “Uniform Building Code Standards (UBC)”, 1997 Edition, Whittier, California.
- Moore, M.A., Ungerer, J. D., 2001, “Warren Hall Seismic Strengthening Study: Addendum Report” Rutherford & Chekene.
- Moore, M.A., Ungerer, J. D. “Warren Hall Seismic Strengthening Study Report” Rutherford & Chekene, March, 2000.
- Priestly, M.J.N., Seible F., 1991, “Seismic Assessment & Retrofit of Bridges”, University of California, San Diego, Report No. SSRP-91/03, pp.117.
- Priestly, M.J.N., Seible, F. Calvi, M., 1995, “Seismic Design and Retrofit of Bridges”, John Wiley & Sons, Inc., New York, NY, Chapter 5, page 348.
- Seismic Safety Commission, State of California, 1996, Seismic Evaluation and Retrofit of Concrete Buildings, Applied Technology Council, Report No. SSC 96-01 (ATC-40).
- The United States Geological Survey (USGS): In the web <http://www.usgs.gov/>
- Rutherford & Chekene, 2003, “Warren Hall Conceptual Design Seismic Retrofit and Code-Complying Modifications”.