Revitalizing a Community Space Using Performance-Based Seismic Design

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Synopsis: This paper summarizes the benefits and challenges of implementing performance-based seismic design (PBSD) for two concrete buildings of the Lower Sproul Plaza Redevelopment Project in one of the busiest areas of UC Berkeley campus. The project included new construction of Eshleman Hall and the additions to Martin Luther King (MLK) Hall, and the seismic retrofit of the existing MLK Hall as a result of the expansion. The peer-reviewed PBSD implemented three-dimensional nonlinear response history analyses at two levels of seismic hazard. The analytical simulations using pairs of near-fault ground motions, scaled to match the site specific spectrum, were intended to establish the expected seismic behavior of the buildings under rare and frequent earthquakes. The choice of PBSD over code-prescriptive procedures was prompted by multiple layers of complexity of the project. Several challenges including those related to the horizontal and vertical irregularities, or connecting new and existing concrete buildings with different lateral force-resisting systems would have made a code-prescriptive design a cumbersome analytical endeavor without providing reliable insight about expected seismic behavior of the buildings. The PBSD methodology, however, proved a powerful tool to: 1) Design for a reliably predictable seismic behavior with sufficient ductility by providing solutions for how to optimize the layout of the concrete walls of the additions to the MLK Hall to minimize the impact on the existing structural elements, and how to create designated ductile hinge zones with sufficient confinement and shear capacity, 2) Avoid unnecessary conservatism to deal with the complexities, when designing diaphragms, drift- and acceleration-sensitive elements including the cladding system, and the bridge that connects the Eshleman and MLK Halls, and 3) Include all potential modes of failure of concrete elements retrofitted by FRP material including the debonding failure between FRP material an substrate.

Keywords: Acceptance criteria, Fiber-reinforced polymer (FRP), Hazard level, Nonlinear Response History Analyses (NLRHA), Reinforced concrete, Seismic design, Seismic evaluation

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INTRODUCTION

The Lower Sproul Redevelopment Project was a student-based initiative to reinvigorate an outdated existing central campus space in UC Berkeley. Encompassing a site area of approximately 184,000 SF, the project included strengthening of three existing buildings, namely Martin Luther King Jr. Student Union (MLK Hall), Plaza/parking Garage, and Anthony Hall, and replacement of Eshleman Hall. This paper summarizes the motivations, methodologies, and benefits of implementing performance-based seismic design throughout this project focusing only on MLK Hall (strengthening/retrofit) and Eshleman Hall (new design).

PROJECT SCOPE AND COMPLEXITIES

The Lower Sproul Plaza project redeveloped one of the busiest areas in UC Berkeley Campus, and included work in multiple adjacent structures that were separated by seismic joints from each other. New construction, retrofit of existing structures, use of different construction materials, desire for a higher seismic structural performance for the buildings, reduction of impact on the existing structures, and the close proximity among these different buildings in a limited construction site, including the adjacent Zellerbach Hall and Cesar Chavez Student Center, which remained open during construction, all added layers of complexity to the project. Given the large number of different goals for the use of concrete in various elements and buildings, including elaborate board-formed patterns in the exposed concrete walls, the project had nearly 20 different mix designs.

Existing MLK Hall

The existing 117,000 SF, five-story MLK Hall, constructed in 1958, was renovated in this project by two major additions (on the west and south sides), adding 29,333 SF of student mixed-use space to the existing building. Both south and west additions consist of a concrete basement and two-story steel framed structure above the Plaza level with reinforced concrete shear walls, connected to the existing structure at all levels.

Figure 1- Two new two-story buildings structurally connected to west side (top image) and south side (bottom image) of existing MLK Hall (images by project architect, Moore Ruble Yudell)

New Eshleman Hall

The existing Eshleman Hall (constructed in 1965) was demolished and replaced with a new building that consists of a 64,800 SF, five-story (plus basement) concrete structure with special reinforced concrete shear walls as lateral force-resisting system. The gravity load-carrying system uses concrete slabs supported on concrete beams and columns. Some of the gravity beams were post-tensioned to control long-term creep deflections at the cantilever floors, which overhang the plaza at the upper levels on the north façade of the building.

Figure 3- Old Eshleman Hall (left) was demolished and replaced

Figure 4- New Eshleman Hall (image by project architect, Moore Ruble Yudell)

New and existing structures connections

The connection between existing and new structures of MLK Hall required very complicated detailing that dealt with various existing conditions, which in several instances differed significantly from the as-built information available. The structures also have to accommodate their relative displacement through their perimeter seismic joints. Moreover, the bridge connecting MLK Hall with Eshleman Hall was designed with an articulated end to accommodate the movement between the two buildings under a seismic event.

WHY PERFORMANCE-BASED SEISMIC DESIGN (PBSD)

The reasons and motivations for choosing PBSD over code-prescriptive design for this project can be summarized in three categories:

1) *Reliable prediction of building response despite the project complexities:* Several challenges such as the geometry of the buildings, connecting new and existing concrete buildings with different lateral force-resisting systems, and connecting two buildings with a pedestrian bridge would reduce reliability of the code-prescriptive design for this project. PBSD, on the other hand, enabled the design team to find insight into complex aspects of the response at the global and element level (e.g. the extent of hinge zone of concrete walls, or distribution of story shear between the interconnected new and existing buildings).

2) *Limitations of code-prescriptive design:* Some of the current limitations of the code-prescriptive design such as lack of guidelines and provisions for using fiber elements to model flexural behavior of concrete walls (the current ASCE 41 code is based on lumped plasticity modeling), or modeling of existing concrete elements retrofitted by FRP materials would impact the understanding of the building seismic behavior and would limit the retrofit options.

3) *Design optimization (avoiding unnecessary conservatism):* The code-prescriptive design of structures with complexities such as the buildings in this project with plan and vertical irregularities is often a cumbersome analytical endeavor with accumulative layers of conservatism. Implementing PBSD provided the design team with justifications to avoid those unnecessary conservative assumptions that would impact the design of diaphragms and drift-sensitive components such as the cladding system or the bridge connecting the two buildings.

PBSD METHODOLOGY

The PBSD methodology for this project was developed by the design team at Rutherford+Chekene as an "acceptance criteria document". The document was peer-reviewed by Tipping Mar Associates and was approved by UC Berkeley Seismic Review Committee (SRC) before implementation. The "acceptance criteria document" for each building provided answers to three main questions:

1) What levels of earthquake hazard concern the seismic evaluation and/or design of the building?

2) How to establish the building responses at each hazard level (modeling assumptions) reliably?

3) What are the accepted global building responses and component-level responses at each hazard level?

The following sections provides a summary of the acceptance criteria document for the two buildings that are the focus of this paper.

Earthquake hazard levels

For design of new Eshleman Hall, two different levels of seismic hazard were considered: the hazard level corresponding to a return period of 475 years (Design Basis Earthquake or DBE), and the hazard level corresponding to a return period of 975 years (Maximum Considered earthquake or MCE). For the renovation of MLK, on the other hand, the two hazard levels correspond to a return period of 225 years (BSE-R) and 975 years (BSE-C) in accordance with the Chapter 34 requirements of CBC 2013. Note that in 2013 CBC Section 3417.5, if the floor area of an addition is greater than 50% of the floor area of the existing building, the BSE-R and BSE-C levels are replaced with the somewhat larger BSE-1 and BSE-2 levels, respectively. Since the size of the two additions to MLK Hall did not exceed the 50% threshold, the hazard levels were BSE-R and BSE-C. For the purpose of brevity and clarity, from this point on, this paper refers to DBE (BSE-R in case of MLK Hall) as "LS" (short for life safety), and to MCE (BSE-C in case of MLK) as "CP" (short for collapse prevention).

Seismic response simulations

Seismic responses of each building were established by nonlinear response history analyses (NLRHA) on three-dimensional (3D) analytical models developed on the Perform 3D Program (Version 5) platform. In the following section, selected key assumptions and procedures used to develop the analytical models for NLRHA are described.

Reinforced concrete walls- The in-plane bending and axial behavior of the reinforced concrete walls were modeled using fiber elements with the corresponding expected material properties. The shear stress-strain relationship of the wall, on the other hand, assumed nominal material properties for the LS-level analyses, and expected material properties for the CP-level analyses. The stress-strain relationship of the steel fibers of the reinforced concrete walls used a simplified tri-linear model. During the peer review process, it was verified that for the range of acceptable seismic response of the building, consideration of strength loss or cyclic degradation of steel fibers was unnecessary. The stress-strain relationship of concrete fibers of the reinforced concrete walls used a simplified tri-linear model as well, however, the model considered strength loss under large compressive strains, but did not include cyclic degradation. The shear behavior of the reinforced concrete walls was modeled using nonlinear shear stress-strain behavior. The ultimate strength of the shear stress-strain curve of shear materials was defined using the ACI 318-08 formula for shear capacity of reinforced concrete walls. The elastic shear modulus was assumed to be 20% of that of uncracked reinforced concrete (based on recommendations of ATC 72-1, Chapter 5), and the yield shear strength was assumed equal to 60% of the ultimate shear strength which was established without a reduction factor. The idealized stress-strain relationship considered the strength loss (beyond 1% shear strain) but would ignore the effects of cyclic degradation. Although the model included strain limit states associated with the nonlinear shear materials of the walls, shear design of the walls was not based on such limit states. Rather shear design of the walls was based on stress limit states that would confirm shear stress demand would not exceed the shear stress capacity calculated per ACI 318-08.

Reinforced concrete beams and columns- To model the concrete beams and columns, lumped plasticity models were used. To establish simplified backbone curves of each concrete beam or column cross section, expected material properties were used as input to a fiber-based section analysis program. Both LS- and CP-level analyses assumed expected material properties to model flexural behavior of beams and columns. For the range of acceptable seismic response of the building, consideration of strength loss or cyclic degradation for frame elements was verified unnecessary.

Diaphragms- Except for Level 2 of Eshleman Hall, all analytical model nodes at each level were constrained to each other by a rigid diaphragm assumption. At Level 2 of Eshleman Hall, the nodes on east and west side of the bridge were assigned to separate rigid diaphragms. The total translational mass in horizontal direction and mass moment of inertia around the vertical axes were lumped at the location of center of mass of each diaphragm.

Global P-Delta effect- The gravity loads contributing to the global P-Delta effect were assigned to the end points of columns in the model.

Soil-structure interaction- The soil compression forces under the foundation and behind retaining walls were simulated in the analytical models of the buildings using compression-only elastic springs. The stiffness values of the springs representing the compression forces under the foundations were calculated based on the tributary area of the spring. During the peer review process, it was verified that assuming elastic behavior under compression forces was justified for the soil springs as they were not subjected to the forces corresponding to the soil yielding.

Bounding analyses- Based on the results of a bounding analyses conducted for stiffness values of the springs representing the soil-structure interaction, it was concluded that most building response quantities are not affected by variation of spring stiffness values between lower and upper bounds. However, for some response quantities including the total drift and soil pressure, the analyses using lower-bound values of soil spring stiffness resulted in larger responses. Thus, it was decided to perform the entire NLRHA using only lower-bound values of soil spring stiffness.

Ground motions- For each seismic hazard level, site-specific response spectra were developed which considered all aspects of the site including the base-slab averaging and embedment effects per FEMA 440. Then seven pairs of horizontal ground motions with fault-normal and fault-parallel components were selected and scaled to match the corresponding spectrum. The angle to apply the fault-normal component of the ground motions where defined based on the location of the buildings relative to the Hayward fault.

Acceptance Criteria

The structural design of the buildings in this project aimed to achieve a seismic behavior that is governed by ductile nonlinear actions at targeted elements, and essentially elastic response of remaining structural elements. The targeted nonlinear actions that will help dissipate earthquake energy include limited flexural yielding of the concrete walls at designated hinge zones, and limited flexural yielding of frame elements. On the other hand, the design ensured that brittle nonlinear actions such as shear failure of concrete wall piers or frame elements are avoided.

A set of acceptance criteria was developed for the building global response and individual structural element response under each of the LS and CP levels. The demands at LS level were calculated using a numerical model that assumed expected material for flexural behavior, and nominal material properties for shear behavior of structural walls. The demands at CP level, on the other hand, were calculated using a numerical model that used expected material properties for both shear and flexural stiffness.

At both hazard levels for each building, Per Section 16.2.4 of ASCE/SEI 7-05, the mean value of peak response under seven pairs of ground motion was used to show the corresponding design criterion is met. Tables 1 through 5 summarize the key parts of the project acceptance criteria.

Table 1- Acceptance criteria for global response

Design Check Name (Type)	**Acceptance Criteria**
Peak transient interstory drift (deformation controlled)	For all story levels, mean value of interstory drift under seven pairs of ground motion shall not exceed 2.5%[a] at LS and 3.0%[b] at CP level.
Residual interstory drift (deformation controlled)	Mean value of three worst residual drifts under seven pairs of ground motion shall not exceed 1.0%, and peak residual interstory drift shall not exceed 1.5%[c] at both LS and CP levels.
Sliding at foundation-soil interface (force controlled)	Maximum base shear under seven pairs of ground motion normalized by seismic weight shall not exceed ultimate coefficient of friction at the foundation-soil interface multiplied by a reduction factor of 0.67[d] at LS level and without a reduction factor at CP level.

[a] Per Section 16.2.4.3 of ASCE/SEI 7-05, the allowable drift per Table 12.12-1 can be increased by a factor of 1.25 (total interstory drift includes contribution of rotational displacements (due to building rocking over the foundation), as well as deformation of structural elements).

[b] Per professional judgment and a similar set of design criteria in Section 8.7.1.1 of PEER/TBI Guidelines for Performance-based Seismic Design of Tall Buildings (2010)

[c] Per professional judgment and similar set of design criteria in Section 8.7.12 of PEER/TBI Guidelines for Performance-based Seismic Design of Tall Buildings (2010)

[d] $\phi = 0.67$ corresponds to the safety factor of 1.5.

Table 2- Acceptance criteria for reinforced concrete walls

Design Check Name (Type)	Acceptance Criteria
Elastic shear behavior of walls (force controlled)	For all wall piers and spandrels, mean value of shear force demand under seven pairs of ground motion shall not exceed their shear capacity[a].
Tensile strain of flexural steel reinforcement (deformation controlled)	For all wall piers and spandrels, mean value of tensile strain of flexural steel reinforcement under seven pairs of ground motion shall not exceed 6%[b] at both LS and CP levels.
Concrete crushing (deformation controlled)	For all wall piers and spandrels, mean value of concrete compressive strain under seven pairs of ground motion shall not exceed 0.15%[c] at both LS and CP levels.

[a] Shear capacity of the walls (both existing and new) is calculated per Section 21.9.4, ACI 318-08 using nominal material properties and $\phi = 0.75$ at LS level and $\phi = 1.0$ at CP level.

[b] Flexural steel rebars are directly modeled using the expected strength value and a nonlinear model considering strain hardening. The 6% strain considered for the acceptance criterion is the fracture strain recommended by Priestly et al. (Displacement-Based Seismic Design of Structures, 2007), which considers concentration of strain due to concrete cracking.

[c] The compressive strain of concrete is directly monitored in the nonlinear analytical model using fiber elements with expected properties of confined concrete. The 0.15% compressive strain considered for the acceptance criterion is half of the specified concrete crushing strain defined by ACI 318-08 based on a study by Orakcal and Wallace (2006) that showed nonlinear models such as the one used for this building underestimate the actual demand of concrete compressive strain by a factor of 2.

Table 3- Acceptance criteria for reinforced concrete beams and columns

Design Check Name (Type)	Acceptance Criteria
Elastic shear behavior of frame elements (force controlled)	For all concrete beams and columns, mean value of shear force demand under seven pairs of ground motion shall not exceed their shear capacity[a].
Concrete beam plastic rotation (deformation controlled)	For all concrete beams, mean value of plastic rotation shall not exceed 0.02 and 0.04 radian[b] at LS level and CP levels, respectively.
Concrete column rotation (deformation controlled)	For all concrete columns, mean value of plastic rotation shall not exceed 0.018 and 0.025 radian[c] at LS level and CP levels, respectively.

[a] Shear capacity of beams and columns is calculated per Section 11 of ACI 318-08 using nominal material properties and corresponding reduction factor at LS level and without a reduction factor at CP level

[b] Per Table 6-7, ASCE/SEI 41-06 (minimum acceptance criteria for secondary concrete beams with conforming transverse reinforcement at Life-Safety (LS) and Collapse-Prevention (CP) performance level).

[c] Per Table 6-8, ASCE/SEI 41-06 (minimum acceptance criteria for secondary concrete columns with conforming transverse reinforcement at Life-Safety (LS) and Collapse-Prevention (CP) performance level).

Table 4- Acceptance criteria for diaphragms

Design Check Name (Type)	Acceptance Criteria
Elastic behavior of diaphragms (force controlled)	For all diaphragms, the mean value of demand under seven pairs of ground motion shall not exceed the corresponding capacity calculated using nominal material properties and reduction factors at LS level and expected material properties and without reduction factors at CP level.

Table 5- Acceptance criteria for foundations

Design Check Name (Type)	Acceptance Criteria
Elastic shear behavior of foundation (force controlled)	Mean value of shear demand under seven pairs of ground motion shall not exceed shear capacity of the foundation calculated using nominal material properties and reduction factors at LS level and expected material properties and without reduction factors at CP level.
Soil pressure (force controlled)	Mean soil pressure under seven pairs of ground motion shall not exceed allowable soil pressure at both LS and CP levels.

KEY BENEFITS OF IMPLEMENTING PBSD

Optimized design of ductile lateral force-resisting systems

PBSD enabled the design team to detail the lateral force-resisting systems of the two buildings to achieve a reliably predictable seismic behavior with sufficient ductility. The detailed insight about the global and local behavior of the building gained by implementing PBSD facilitated the optimization of the concrete walls layout of the additions to the MLK Hall (to minimize the impact on the existing structural elements with different material and behavior). Moreover, modeling the flexural behavior of the walls could by fiber elements provided the key to determine the hinge zones and the horizontal and vertical extent of boundary zones. Figure 5 showcases how the PBSD relied on the results of analyses to detail the boundary elements of Eshleman Hall concrete core walls.

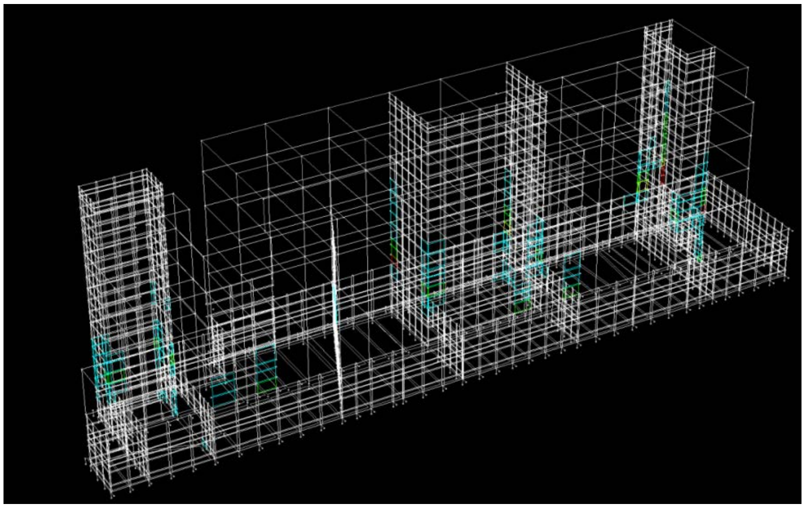

Figure 5- Using colored elements to check concrete compressive strain does not exceed its corresponding acceptance criteria and to establish the horizontal and vertical length of boundary zone; red, yellow, green and blue elements respectively correspond to compressive strain of 0.15, 0.11, 0.08, and 0.04%

Optimized design of drift- or acceleration-sensitive components

The PBSD provided the design team with drift values (see Figures 6 and 7) that were used to design the cladding system, and the bridge connecting MLK with Eshleman Hall, with an articulated end to accommodate the movement between the two buildings under an earthquake (see Figure 8). The corresponding design values per code provisions would be conservatively larger than those based on PBSD. Similarly, the response history analyses based on PBSD provided less conservative forces to design new diaphragms (or evaluate the existing ones), and to design the connections between the existing MLK diaphragms and those of the west and south additions such as the one shown in Figure 9.

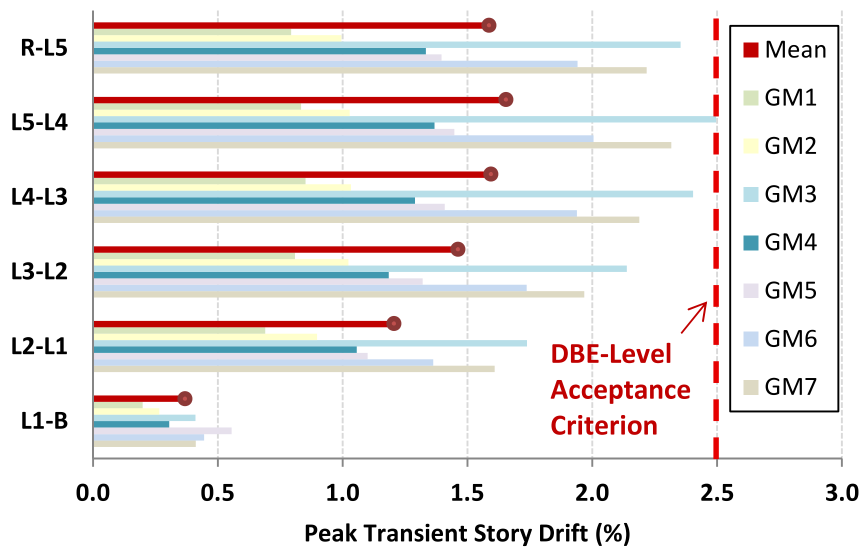

Figure 6- Peak transient interstory drift values of Eshleman Hall under seven DBE-Level ground motions

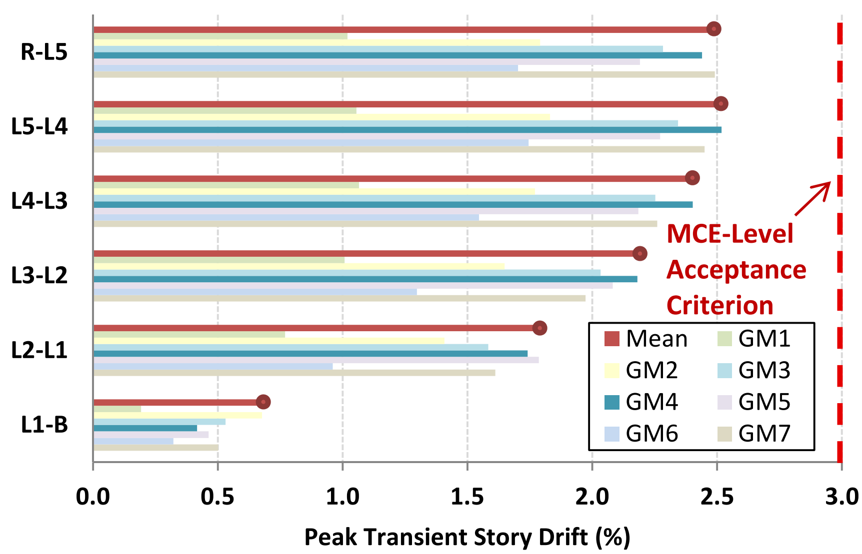

Figure 7- Peak transient interstory drift values of Eshleman Hall under seven MCE-Level ground motions

Figure 8- Bridge connecting MLK with Eshleman Hall, with an articulated end to accommodate the movement between the two buildings under an earthquake

Figure 9- Connections between existing MLK diaphragms and new diaphragms of west addition designed based on forces calculated by PBSD methodology

Modeling and design of existing elements retrofitted with FRP material

The preliminary pushover analyses of MLK Hall indicated that connecting the two new buildings on the west and south sides of the MLK Hall would stiffen the first two levels of MLK Hall, and would drive the seismic forces to the upper stories of the building. The analyses also showed that while in the EW direction, the behavior will be governed by the flexural yielding of the walls at Level 3 of the existing building (see Figure 10), in the NS direction the walls would suffer shear failures that would lead to a story mechanism (see Figure 11).

To prevent such brittle behavior the design team decided to use carbon fiber-reinforced polymer (CFRP) strips to increase the shear capacity of the NS Walls of Level 3. The analytical modeling of the walls retrofitted by CFRP material would be a challenge if the project had pursued a code-prescriptive design (the current ASCE 41 code does not have provisions for modeling and acceptance criteria of retrofitted elements). However, thanks to the peer-reviewed PBSD methodology implemented for the project, the design team was able to develop suitable modeling assumptions and corresponding acceptance criteria for the inelastic shear behavior of the walls retrofitted by CFRP material.

The modeling assumed that the elastic shear stiffness and the ratio between yield and ultimate shear strength (F_y/F_u = 0.6) remains unchanged after adding the CFRP to the existing wall. However, the external application of CFRP would increase the ultimate strength of the wall. In the absence of sufficient experimental or numerical studies to capture the shear behavior of concrete walls retrofitted with FRP, it was assumed that under shear strains larger than 0.004 the CFRP no longer contributes to the shear strength due to the bond failure.

Figure 12 is an example to demonstrate how the nonlinear shear stress-strain relationship of the walls retrofitted with CFRP was modeled. The blue line in this figure corresponds to the shear stress-strain relationship of an existing 12 inch thick wall. The orange line in this figure would be a less conservative model that inherently assumes the CFRP overlay and its bond to the existing wall can undergo the same shear strains as the wall without an overlay. The purple line would be a more conservative model which inherently assumes that shortly after 0.004 shear strain, the contribution of the CFRP overlay to the wall shear strength rapidly diminishes and the behavior of the wall would then become identical to that of the existing wall. Commercially available analysis programs such as the one used for this project do not have the option of modeling the post-degradation behavior of concrete material with multiple segments. Therefore, the black line was selected as the simplified version of the purple line such that the area under the black curve is close but smaller than the area under the purple curve.

External application of FRP materials on the walls of MLK Hall to change their behavior from shear governed to flexure governed was an elegant and cost-effective retrofit solution that was possible since PBSD approach allowed the design team to work with the peer reviewer and develop appropriate modeling and acceptance criteria.

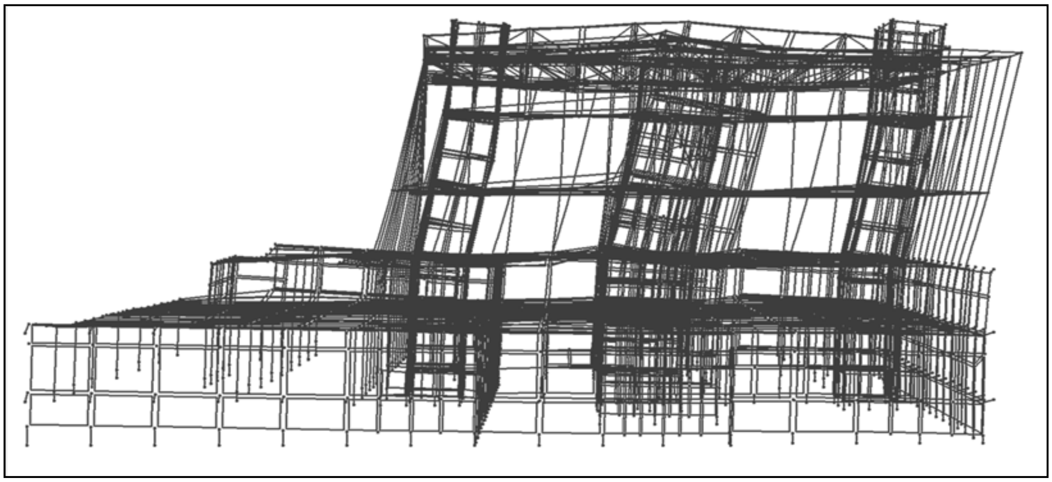

Figure 10- Pushover analyses of MLK Hall (push to East) showed that addition of new buildings on west and south side would result in flexural hinging of EW walls above Level 3

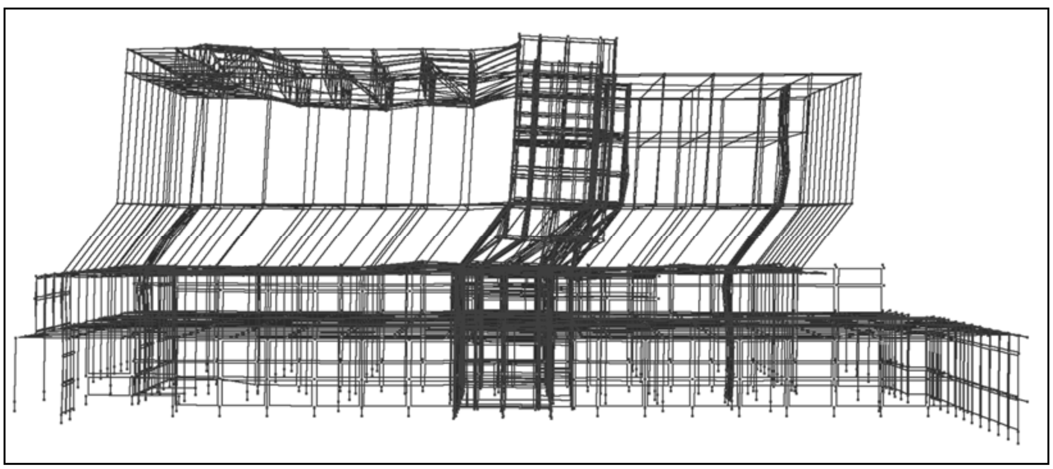

Figure 11- Pushover analyses of MLK Hall (push to South) showed that addition of new buildings on west and south side would result in a shear failure of existing NS walls above Level 3

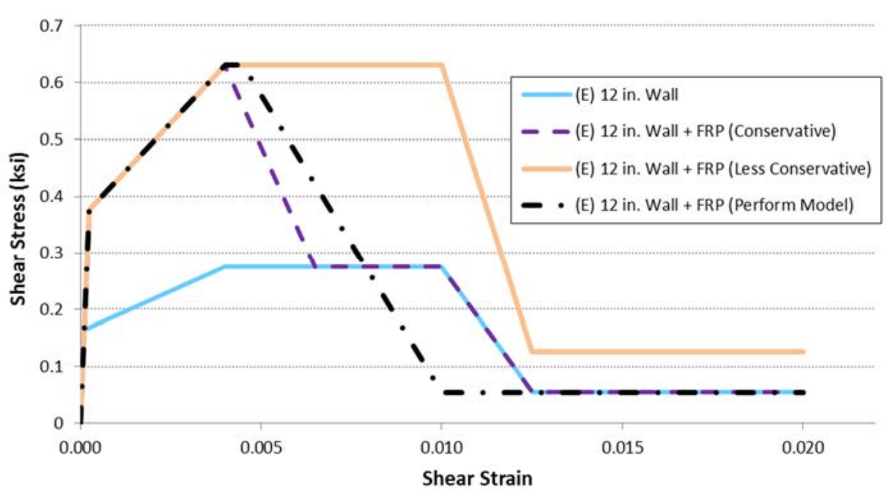

Figure 12- Comparison of inelastic shear behavior modeling of existing and retrofired wall

CONCLUSIONS

Seismic design of for the concrete buildings of the Lower Sproul Plaza Redevelopment Project in one of the busiest areas of UC Berkeley campus is an example of the state-of-practice of performance-based design methodologies for concrete buildings. The peer-reviewed PBSD implemented in this project proved useful and even essential to optimize the seismic design, to avoid unnecessary conservative assumptions that are conventionally used to address the complexity of a project, and enabled the project to take advantage of a retrofit technology (externally applied FRP material) that is currently not addressed by the code-prescriptive methodologies. In many cases, such as design of drift-sensitive elements, diaphragms, or connections between new and existing elements, PBSD provided realistic design values that resulted in cost saving compared to alternative code-prescriptive methods. The readers are advised to note that although the overall framework presented here is applicable to any other project, the specifics of modeling assumptions or acceptance criteria may or may be suitable for another project. The details of the PBSD implemented in this project were developed by the design team and approved by the peer reviewer based on all the characteristics and circumstances specific to the project.

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