COMPREHENSIVE SEISMIC UPGRADE OF A NON-DUCTILE CONCRETE MOMENT-RESISTING-FRAME BUILDING APPLYING FRP

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Abstract

Non-ductile concrete moment-resisting frame (MRF) buildings are known to be susceptible to earthquake damage. Many past retrofit efforts mainly focused on primary lateral-force-resisting elements. However, potential life-threatening hazards caused by the insufficient displacement capacity of non-structural components such as precast cladding panels are often left unaddressed. Comprehensive building seismic upgrades must address both the deficiencies of the primary structural system, as well as insufficient displacement compatibilities of any heavy non-structural components prone to falling. A seismic upgrade design of a thirteen-story non-ductile concrete MRF building was recently completed with the construction currently ongoing. Findings from a non-linear time-history analysis of the building revealed a lack of ductility in MRFs primarily associated with inadequate shear strength within various regions of beams. The seismic strengthening currently being implemented utilizes an innovative strengthening technique that combines over-the-top Fiber Reinforced Polymer (FRP) anchors with externally-bonded U-shape FRP strips to increase beam shear strength within MRF beam end-yielding region while adding shallow concrete crack-inducement grooves in the region to promote flexural yielding, which greatly improve the global ductility of an existing non-ductile MRF system. These MRF beam strengthening details were proven to be effective in a series of 4/5-scale cyclical T-beam component tests. This paper explains the process to develop the design procedure of this new and effective confinement technique and the correlation between the analytical and experimental results. The beam strengthening technique substantially reduced the necessary scope of work in the retrofit construction, and could be apply to future seismic upgrade of existing buildings. In addition, the design team evaluated compatibility between the resulting building deformations and heavy precast cladding panels with multiple analyses including a three-dimensional finite-element sub-structure model of the cladding to simulate interaction between the adjacent cladding and structural framing components. Mechanisms determined by the cladding analyses were verified in the laboratory using ¾-scale cyclical component testing of various cladding panel configurations. Based on the analytical results, innovative strengthening details were developed to improve the deformation capacity of the cladding panels to accommodate inter-story drifts. This paper describes mechanisms of the modified cladding connection details and their applications to address the cladding system's deflection compatibility issues in future projects. The comprehensive approach of the retrofit design that includes adequacy of primary structural system, compatibility of critical non-structural components and necessary component tests to calibrate analytical results ensures all potential seismic deficiencies of the building structure are fully addressed.

Keywords: externally bonded FRP, FRP anchor, precast cladding strengthening, concrete moment-resisting frame, ductility

1. Introduction

The seismic vulnerability of non-ductile reinforced concrete (RC) moment-resisting frames (MRF) has been well documented in past earthquakes. Based on building code changes most notably associated with damage observed in the 1971 San Fernando Earthquake, concrete MRF buildings built after the mid-1980s in general have performed satisfactorily in recent earthquakes. However, deviations from the prescriptive procedures described in the building code or unconservative simplifications used in design can create deficiencies in the seismic performance. Most past efforts to seismically upgrade non-ductile MRF buildings have focused on the strengthening of primary lateral force resisting (LFR) elements, while other potential life-threatening non-structural hazards, such as heavy cladding panels prone to binding, were left unaddressed. To maintain life safety, building seismic upgrades must comprehensively address the deficiencies of the primary structural system, as well as the attachment of heavy non-structural components prone to falling.

A recent seismic evaluation of a 1995-vintage thirteen-story concrete tower revealed that its MRF system is non-ductile and cannot achieve its intended seismic performance due to inadequate shear capacity of the beams. This deficiency of the MRF beams is the result of a reduced cross section due to the presence of large utility openings, missing crossties, and underestimation of probable seismic shear demand due to an oversimplified design assumption. The evaluation also noted that precast concrete (PC) cladding panels have limited lateral drift capacity, with panel binding at the clips, panel edges, building corners, and adjacent framing identified. This deficiency could result in loss of support in multiple locations.

To verify the inadequate shear strength of MRF beams identified through the analytical process, a series of 4/5-scale cyclical T-beam component tests was performed confirming the identified deficiency. Subsequent testing of two beam specimens strengthened in the opening region with externally bonded Fiber Reinforced Polymer (FRP) proved to adequately mitigate the deficiency. For the noted MRF beam “2d” plastic hinging region deficiencies, an innovative strengthening detail was developed and cyclically tested to verify its effectiveness. This 2d region strengthening combined over-the-top FRP anchors with externally-bonded U-shape FRP strips to increase shear strength, along with shallow sawcut crack-inducement grooves in the adjacent concrete to promote distributed flexural yielding. The ductility of the existing MRF system was greatly improved with the FRP strengthening as demonstrated in subsequent component tests summarized below.

The seismic deflection capacity of the PC cladding panel system was evaluated to assess the risk of becoming dislodged and falling. Identifying the failure mechanisms of the complex art deco cladding system was challenging due to the myriad of configurations and connections present. Several factors contribute to the identified seismic deficiencies of the cladding system, including prying and binding at adjacent panels and the perimeter MRF, inadequate displacement capacity at slotted connections, and misalignments during assembling and installation processes. Analytical procedures including both local and global models were supplemented by ¾-scale cyclical cladding panel testing to determine critical displacement limit states. A summary of mitigation measures is described herein. Occupant safety would be significantly improved by the comprehensive approach that addressed both the structural and non-structural deficiencies.

2. Seismic Evaluation of Existing Structure

The subject structure is a 13-story office building located in downtown Los Angeles with a two-level subterranean parking podium and large mechanical roof penthouses located at levels 6 and 12 (Fig. 1). In plan the building is L-shaped below the 6th floor, with approximate maximum dimensions of 300 feet by 352 feet. At levels 7 through 13, the projecting tower has maximum dimensions of 210 feet by 117 feet. Story height is 20 feet at the ground level, and 13.5 feet in the upper levels. The superstructure’s structural system consists of cast-in-place (CIP) RC MRFs for lateral force resistance (LFR) and PC gravity framing with CIP concrete slabs. The MRFs generally consist of 48-inch-deep by 30-inch-wide beams and 48-inch deep by 36-inch-wide columns, except at the top floor and penthouse levels. At grade, the MRFs discontinue with only columns extended to the foundation below. The MRF frame is laterally supported by the transfer diaphragm at grade, which transfers lateral loads to interior and perimeter subterranean concrete shear walls. Foundations consist of deep isolated spread footings and continuous wall footings.

The building was built in late 1990s and the discovery of stress cracks on several precast beams triggered an overall structural evaluation in early 2010s. One focus of the evaluation was on large utility openings adjacent to beam plastic hinge zones and their effect to the ductility of the MRFs (Fig. 2). The concern was the reduction in shear capacity due to the openings may cause premature shear yielding prior to the development of plastic hinges and result in non-ductile behavior of the system. Based on the results of the evaluation, it was concluded that the primary LFR system did not possess the necessary strength or ductility to achieve the desired performance objective. Concerns of pre-mature shear yielding at openings was later confirmed by component testing.

Fig. 1 Thirteen-story building

Fig. 2 Utility opening in MRF beam

The project team applied provisions in California Building Code (CBC) [1] regarding earthquake evaluation and retrofit design for existing buildings to develop the criteria. Although the building structure did not meet the desired performance objective, it was not in dangerous conditions and the upgrade project was entirely on a voluntary basis as part of the goal of the building owner's organization to improve seismic resilience. CBC refers to ASCE 41 [2] for specific evaluation and design procedures, and minor modifications were made in the project-specific criteria to achieve the selected performance objective.

The building's primary LFR system is required to achieve both Life Safety (LS) performance under the design basis earthquake (DBE-level) and Collapse Prevention (CP) performance under the maximum considered earthquake (MCE-level). The owner opted to not apply the reduction in earthquake intensity allowed by ASCE 41 for existing buildings and using the basic safety earthquakes (BSE) for new buildings (BSE-1N and BSE-2N as the design-level and MCE-level earthquakes, respectively) to ensure the upgraded building would perform similarly to a new one. As a public agency, the owner wanted to achieve the overall seismic performance objective for state-owned buildings, which required LS performance of heavy non-structural components under the BSE-1E earthquake. Consequently, each PC cladding panel and its connections were evaluated and upgraded to accommodate inter-story drifts of the building without becoming a falling hazard. Because of the building's irregular shape and the selected LS performance which allows structural members to deform beyond elastic range, an analysis using the ASCE 41 non-liner dynamic procedure (NDP) was performed and peer reviewed by a third-party engineer.

3. MRF Beam Opening – Testing and Strengthening

The subject structure's concrete moment-resisting frame beams in many cases are inherently non-ductile because their strength and deformation capacity are limited by shear-critical mechanisms. Specifically, the probable moment capacity (M_{pr}) used in the original beam design was based on a rectangular cross-section with only the tension reinforcing steel being considered, and the contribution of the flange as well as beam compression and jamb bars neglected. The design simplification resulted in the beam probable seismic shear demand to be underestimated.

This shear deficiency manifested itself in two major ways. First was the inadequate strength in the region of large HVAC openings present at numerous beams throughout the structure. Second was a condition of inadequate shear strength within the yielding region of MRF beams with single-tie shear reinforcing and with short span/depth ratios. The extent of MRF beam structural deficiencies was verified in a detailed non-linear time history (NLTH) structural analysis that utilized compound beam elements (Fig. 3) to assess structural performance in the design- and MCE-level seismic events. The analysis confirmed that significant limited-ductile and non-ductile behavior will occur in existing MRF beams (Fig. 4).

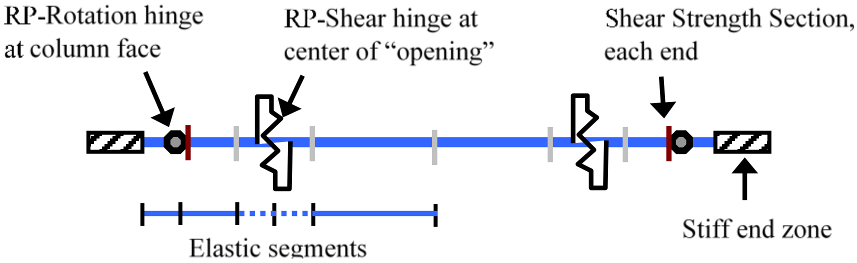


Fig. 3 – Compound beam elements with nonlinear deformation hinges

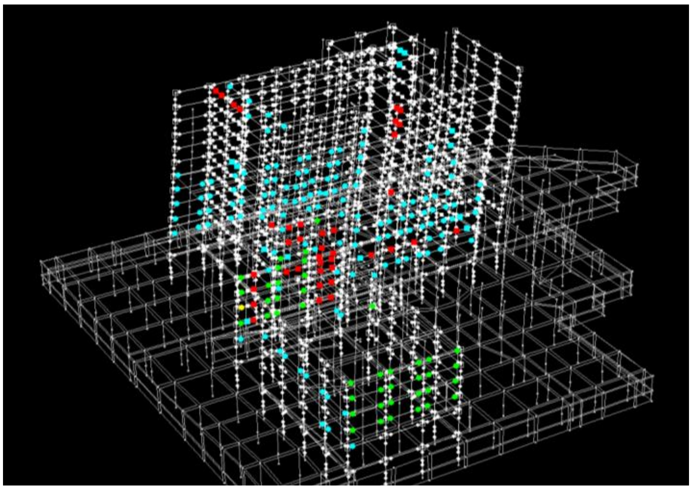


Fig. 4 – PERFORM-3D [3] BSE-1N level deformed shape without strengthening.
Colors represent the level of beam plastic deformation or force-level for a given limit state and NLTH run step. Usage ratio (LS2): red>2, yellow 1.5-2, green 1.5-1, blue 0.5-1.

3.1 **MRF Beam Openings**

Many of the 48-inch-deep by 30-inch-wide MRF beams have large 48-inch-wide by 16-inch-tall duct/utility openings on the 16-foot, 26-foot, or 28-foot clear span beams. Where present on the longer MRF beam spans, openings are commonly present at both ends of the span, located just inside the beam 2d end plastic hinge region (Fig. 5). Most of the openings are 1/3 of the beam height, with a geometric width/height ratio of 3:1.

Large openings are rarely installed in Special MRF beams, and assessing their suitability requires an understanding and careful appraisal of forces as they occur in the beam opening top and bottom segments (opening segments). An obvious weak point, the behavior in the region of the beam openings must be wholly elastic. Any yielding of the beam should occur outside the opening region with a reasonable margin on the design. Load transfer from the segments into the main beam body adjacent to each opening must also be considered.

At the opening, vertical and horizontal shear as well as shear lag must be considered. From the detailing standpoint, a suitable spacing to depth ratio (s/d) of the beam shear ties must be provided across the opening segments and into the opening reentrant corner regions of the beam body. In simple terms, the reinforcing tie s/d ratios of the opening segments and reentrant corner regions must be more tightly spaced than the beam in general. Horizontal shear (VQ/Ib) transfer at the top segment beam/flange cold joint should be checked (Fig. 6), since the horizontal shear stress is very near the maxima for the cross-section.

The shear strength of the top and bottom segments will be affected by the flexure-induced instantaneous axial load (N_U) coming from the force couple, which reduces or increases the concrete shear contribution (V_c) depending on whether the segment is in compression or tension. Concrete shear strength as a function of axial load can be calculated in accordance with ACI 318-14 [4] §22.5.6 and §22.5.7 (Equations 22.5.6.1 and 22.5.7.1).

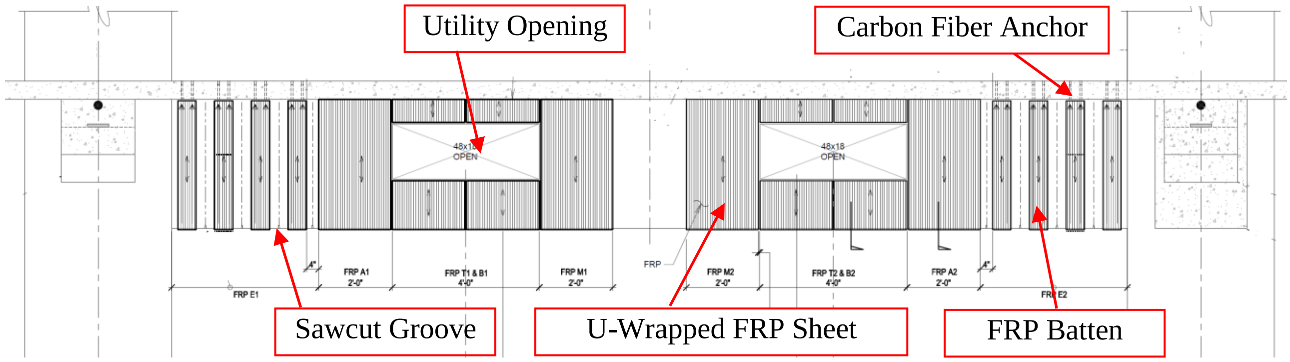
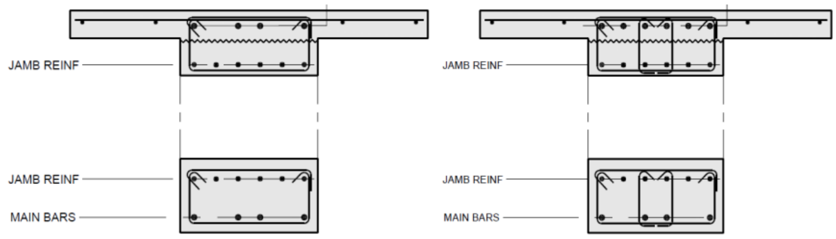


Fig. 5 MRF beam elevation with FRP strengthening



Single-tie sets Double-tie sets

Fig. 6 – Existing MRF beam cross sections at the openings

The ACI 318-14 §22.5.6 and §22.5.7 shear equations correlated well with the observed strength loss state at the Beam Specimen 1 (representing existing condition) of the cyclical component tests [5]. However, determining distribution of the load between the top and bottom chords of the opening segments is difficult because of the progressive cracking and axial load reversals that affect each segment's relative stiffness. A parametric bounding study of relative stiffness and load distribution would need to determine which chord in the opening segment would control the stress check in the analytical process.

Alternatively, the beam opening segments can be treated as a beam-column frame element under shear and axial forces using the ASCE 41-13 Standard. The moment shear ratio (M/Vd) used in the equation per ASCE 41 methodology is based on the larger M/Vd ratio of the overall beam and the beam segment, and in this case is in the range of 3.2 to 3.5 controlled by the overall beam. Concrete shear strength (V_c) in ASCE 41 is conservatively neglected for the opening tension segment.

Flexural-axial design checks of the opening segments should reliably ensure elastic behavior in the horizontal reinforcing steel within the opening region. All cases of factored loading and probable seismic moment (M_{pr}) demands should be considered including the effect of simultaneous axial loads due to the horizontal force couple (P-M interaction). The potential for longitudinal bar buckling in the beam segments is usually slim if nominally spaced ties and crossties are present.

The tendency of the beam web to crack diagonally at opening corners is intuitively obvious. Based on the cyclical testing results, the cracking pattern is associated with beam segment “diagonal strut” action. These reentrant corner cracks occur over a range of 45 to 75 degrees from the vertical and can be mitigated by improving shear capacity of the regions adjacent to the openings. Note that the design concerns at the beam opening segments can be mitigated by limiting the span to depth ratio of opening segments to 2, and a minimum segment depth of about 10 inches, not to exceed 25% of the beam overall depth.

3.2 **Adequacy at the Existing Beam Openings**

Each of the MRF beam with large openings was split into the following segments based on their geometry and behavior in the analytical and design processes: (1) opening top segment, (2) opening bottom segment, (3) end-adjacent region, and (4) middle-adjacent region. The opening top segment is a shallow T-beam, prone to issues of shear transfer through the flange/web cold joint. The opening bottom segment is a rectangular beam prone to shear failure. The retrofit design of both the top and bottom segments was focused on increasing their shear capacity. Regions on each side of the opening act as runouts, distributing loads from the top and bottom segments into the body of the beam. Runouts are prone to cracking aligned with the beam segment compression strut angle at reentrant corners. Top segment runouts have an additional concern with horizontal shear transfer through the cold joint.

The acceptability of the MRF beam opening region is reliant on its combined seismic plus gravity demand as resisted by the provided reinforcing steel detailing. Beam opening segments are critical under shear which is a force-controlled action, and the adequacy check was based on the shear demand from a suite of 14 ground-motion records in the NLTH analysis. Where reinforcing ties with added crossties (double-tie) had been provided at tight spacing across the opening segments (Fig. 6) and runouts, strengthening in the opening region was found unnecessary as confirmed by cyclical testing and by calculation. For the case of beams with single ties (Fig. 6) as in the case of Beam Specimen 1, those opening segments and adjacent regions would require strengthening including additional capacity to transfer horizontal shear through the flange-web cold joint.

3.3 **FRP Contribution to Beam Opening Segment Shear Strength**

Strengthening in the opening region (Fig. 5) consisted of vertically oriented uniaxial FRP fabric U-sheets, externally bonded to the beam web, and anchored overhead into the beam flange with splayed carbon fiber anchors (CFA). A vertical fiber inclination is effective at precluding diagonal cracking and in developing additional shear friction at the web-flange joint. Where possible, CFAs partially embedded into beam flanges were applied to limit construction work. Where additional shear strength at the code joint was necessary, over-the-top pass-through CFAs [6] which require simultaneous work above and below the beam were used. Beam shear strength at the top and bottom segments and adjacent transition regions are contributed by concrete, reinforcing steel and FRP fabric. The contribution of FRP fabric ($\psi_f V_f$) was determined using the shear strength equation in AC125 §7.3.2.6 [7], as simplified below:

$$\psi_f V_f = \psi_f A_f E_f \varepsilon_f d_f / s_f \tag{1}$$

where

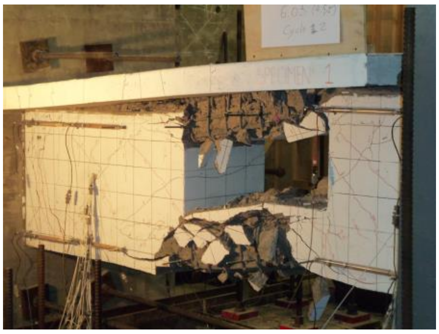
V_f = FRP fiber shear strength

ε_f = 0.004 or $\varepsilon_u \kappa_n$ (maximum permissible strain for FRP)

d_f = Fabric partial height below the flange for embedded CFA, full segment height for over-the-top CFA

ψ_f = FRP shear effectiveness coefficient, taken as 0.95 for fully developed FRP fabric, and 0.85 for partial-height fabric U-wraps that are partially developed with shallowly embedded CFA

Figure 7 shows the comparison of Specimen 1 (w/o opening strengthening) and Specimen 1A (w/ opening strengthening) at the ultimate limit state as the former failed prematurely with excessive damage at beam opening and the latter developed a plastic hinge at the end. The component tests definitively confirmed the effectiveness of FRP strengthening at the opening.



Specimen 1 (existing)

Specimen 1A (FRP strengthening)

Fig. 7 Photos of MRF beam specimens at ultimate limit state

4. MRF Beam End Region – Testing and Shear Strengthening

The design team considered several factors when selecting the final MRF strengthening scheme. First, the shear strengthening of the non-ductile beams should not significantly increase the flexural capacity of beams to be stronger than the attached columns. Second, it should not appreciably increase the beam depth due to overhead space limitations. Third, the strengthening must result in a progressive development of flexural plastic hinge at the beam ends. Finally, the strengthening should minimize disruption to building's operation which rules out options involving extensive concrete placement.

Considering the limitations of modifying the existing concrete MRF system and the fact that the tested FRP strengthening at beam openings was effective, the FRP wrapping was selected to strengthen the end regions. The effect of the beam flange was recognized by the design team and therefore, two initial 4/5 scale T-beams (Specimens 1A and 1B) utilizing solid sheets of unidirectional externally bonded FRP in the end region in addition to the opening strengthening were tested in laboratory. Specimen 1B included embedded CFA over the end region. However, in both specimens, the FRP sheet material in the end region was prematurely damaged, and in the latter, CFA pulled out during flexural cracking.

4.1 **FRP Batten Detail and Testing**

The second iteration of the end region shear testing [5] was performed to resolve the FRP detailing issues observed in the initial two specimens. Here, the strengthening concept for Beam Specimen 3 utilized over-the-top carbon CFA passing through the flange [6] on each side of the T-beam web to lap with the FRP sheets to resist CFA anchor pull-out. CFA passing over the beam flange (floor level) were left un-recessed to prevent the inducement of flexural cracking there. Then, shallow crack-control grooves were saw cut in the T-beam web and the flange at about 12-inches on center over the end regions to induce initial flexural cracks and to ensure these cracks would be away from the anchored externally bonded FRP. Finally, two-layer carbon fiber U-battens 6-inches wide were installed between the sawcut grooves and lapped to over-the-top CFA, which created a solid FRP hoop functioning similar to a reinforcing closed-tie. See Figure 5 for an elevation of the strengthened beam.

The batten/sawcut shear strengthening concept was proven affected in the component test, sustaining the beam end region shear through a total rotation of 0.038 radians. Cyclical testing results represented by the force-deflection curve for Specimen 3 are shown in Fig. 8. Not only did the observed rotation approach the code SMRF rotation requirement of .04 radians, but it also well exceeded the computed ASCE 41 prescriptive total rotation parameter of 0.027 radians.

Beam plastic behavior in the end region as observed in the testing was very stable, with beam flexural cracking developing at the sawcut groove near the column face, and sequentially progressing into the other grooves within the "1d" region. Throughout the test, FRP battens and CFA remained undamaged until the local failure at the bottom edge of the second batten from the end which occurred simultaneous with the strength loss of the beam specimen. The FRP batten detail's compatibility with the concrete and overall effectiveness in supplementing shear strength was demonstrated in the component test.

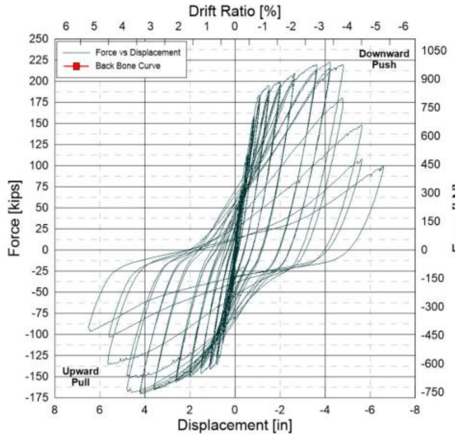


Fig. 8 –Setup of Specimen 3 (batten/groove strengthening at end) testing & force-deflection curve

4.2 **FRP Batten Contribution to End Region Shear Strength**

The shear strength contribution of FRP U-Battens ($\psi_f V_f$) was determined using AC125 §7.3.2.6 as simplified in Eq. 2. Here, an effectiveness coefficient (k_f) has been added to account for the FRP Batten loss of effectiveness at increasing beam plastic rotations well beyond the elastic range as demonstrated from the project-specific Beam Specimen 3 cyclical testing.

$$\psi_f V_f = k_f \psi_f A_f E_f \varepsilon_f d_f/s_f \quad (2)$$

where

ε_f = 0.004 (AC125 permissible strain for fully developed FRP)

d_f = full beam height for over-the-top CFA

k_f = FRP plastic yielding reduction factor deduced from the limited cyclical testing, as follows:

For ductility $\mu < 2$: $k_f = 1.0$ but $\psi_f V_f$ for design at this ductility was not taken larger than V_c.

For ductility $\mu = 12$: $k_f = 0.25$. Linearly interpolated at intermediate points.

k = 0.7 for yielding at the beam end region.

The contribution of the added FRP battens to the MRF beam end-region shear strength is relatively small at large rotations but can contribute significantly to the rotational capacity before fracture. This is shown by the two red stars on the force-deflection backbone in Fig. 9, representing the existing (0.022 rad) and the FRP Batten strengthened (0.038 rad) configurations based on the results of the component tests. Also apparent from the figure is the relatively conservative estimation of the ASCE 41 prescriptive beam force-deflection (moment-rotation) curve versus that observed in the component testing.

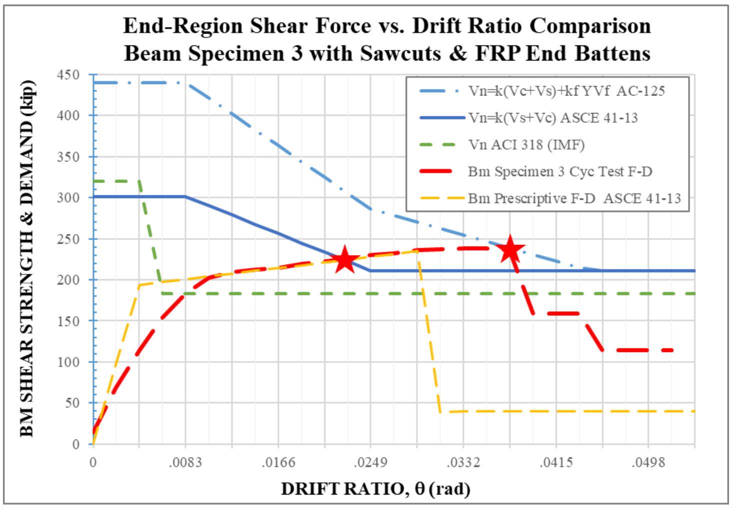


Fig. 9 – End-region shear vs. rotation comparison

5. Precast Concrete (PC) Cladding Seismic Strengthening

5.1 Background

The structure's PC cladding system consists of normal weight concrete panels approximately 10 feet long, 4 to 6 inches thick, with a ¾-inch expansion joint between adjacent panels, most weighing between 7,000 and 12,000 pounds. In general, there are three panels per frame bay (one middle panel and two end panels), so that typical cladding dimension for the building is 13'-6" nominal height by approximately 10'-0" wide, with a perimeter SMRF beam directly adjacent at the top, and SMRF columns with a wraparounds at one end of each end panel.

The deflection capacity required for PC panels is determined by the lateral stiffness of the building's LFR system, the panel geometry, and the ground shaking intensity used as the design basis. Concrete SMRF systems are more flexible laterally compared to other structural systems such as braced frames and shear walls, and their design has generally been drift-controlled. Maximum building inter-story drifts were determined from the NLTH analysis, taken as the average of the maximum from the suite of 14 ground-motion records. The largest inter-story drifts occur in the north-south direction at the building's mid-height floors. BSE-1E shaking level per ASCE 41 was used for the cladding checks and retrofits, and the maximum inter-story drift was computed to be 0.011 radians, or about 1.8 inches nominally.

Furthermore, the curvature of the perimeter MRF beams directly cast against the PC cladding panels causes additional rotation in the panels (Fig. 10). In this case, one-inch of nominal lateral story drift could result in 0.83 inches of drift at the bottom clips of some panels, while other panels could have as much as 1.35 inches of drift at the clips. Relative displacement between cladding panels in opposite directions is not entirely intuitive and tends to open and close (in some cases with binding) the ¾-inch expansion joints at the typical panel-to-panel edges. The described drift demands well exceed the existing cladding system's deflection capacity.

Cladding panel out-of-plane restraint is typically provided at the four panel corners while permitting articulation when subjected to seismically or wind-induced inter-story drifts. Except for waterproofing at low roofs, PC cladding is typically supported vertically at the overhead connection with cast-in rebar and rebar welded to embedded angle at the slab-edge. Panel in-plane movement is accommodated by deformable push-pull rods or by slotted clips at the anchor bolts, resisting out-of-plane forces while permitting in-plane relative drifts between the floors under seismic and wind forces.

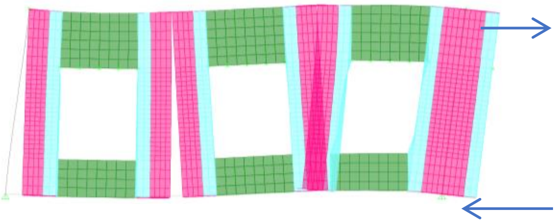


Fig. 10 – Effect on cladding panel drift from rotation of perimeter MRF beam

Several factors limit the cladding system's deflection capacity. A typical existing bottom clip has 3-inch-wide slots which provide about 1.25 inches (equivalent to .008 radians) of in-plane drift, but could be reduced by as much as 50% due to anchor bolt misalignments. In-plane panel drift at a typical end cladding panel with MRF column wraparounds is limited by a 2-inch nominal gap. Furthermore, the PC panels were used as forms for perimeter cast-in-place MRF beams, causing additional binding that restrains out-of-plane movement of the panels. Because cladding panels slide in-plane but are fixed out-of-plane, adjacent orthogonal panels at corners of the building have high likelihood of binding earlier than the rest of the panels.

5.2 **PC Cladding Testing**

Due to the complex mechanism of cladding prying and binding, a program of destructive panel testing developed in accordance with ASCE 41was implemented to verify the analytical results. Of primary concern was defining the panel deflection limit with the effects of internal panel binding considered. The LS performance permits cracking in the panels and damage to waterproofing and window glazing panels, but does not permit loss of vertical panel support at the overhead connections, nor does it permit large concrete spalling.

For the testing, four ¾-scale PC concrete panel specimens of various configurations matching the original PC panel shop drawings were constructed off-site, and then set in the testing laboratory. A typical panel testing setup is shown in Fig. 11. Panels were cast into the testing block top connection (Fig. 11 right) and mounted to the bottom loading rail (Fig. 11 middle). Two different testing-rig setups were developed. Out-of-plane cyclical displacement testing (Fig. 11 left) up to the BSE-2N drift level was performed first on each panel. Then, the loading ram was set in-line to the loading rail and bottom clips for the subsequent in-plane cyclical testing (Fig. 11 middle) with progressively increased displacement until connection failure. Slotted bottom clip angles permitted slip during the testing, though only the forces and displacements after binding were of importance in design as the available slip in each clip was accounted for separately.

Setup for out-of-plane test Setup for in-plane test with loading rail Panel top connection

Fig. 11 – Cyclical testing of ¾-scale cladding panel specimens and top connection detail

As observed in the testing, warping and out-of-plane cracking at the vertical corners of the cladding panels resulted in a notable softening of the effective stiffnesses that increased the out-of-plane and in-plane deformation capacity. All the tested panels reached the required out-of-plane drifts with only minor cracking, even in the binding (loading inward against the cast beam) direction. For the subsequent in-plane tests, the LS limit was observed to occur in two distinct variations. First, warping at the cladding corners under large cyclical displacements caused corner or edge spalls of concrete. In the other case, the panel vertical force couple associated with lateral binding caused one of the top connections to lose strength under large cyclical displacements. The component tests identified the potential limit states of the panel system and allowed the design team to target the real deficiencies of the system for mitigation.

5.3 **Cladding Retrofit Concept**

As the component tests indicated that the existing panels would be better equipped to accommodate out-of-plane drifts, cladding panel retrofit consisted mainly of increasing the in-plane deformation capacity of the panels by resetting new slotted clips at the anchor bolts and widening mitered building corner joints using track-mounted saws. In some cases, the overhead support connections were improved using various support frames or by adding sleeved anchor bolts through the face at the window head. Bottom connection modifications entailed replacing existing clips with ones that have longer horizontal slots to allow proper fit-up at the existing connection bolts, and adding PFTE pads to permit sliding at both the vertical and horizontal clip angle legs, or by adding push-pull rods where architecturally possible (Fig. 12) .

Fig. 12 Modified cladding panel bottom connection

Fig. 13 corner joint widened by sawcut

6. Seismic Upgrade Construction

The upgrade construction began in late 2018 and the contractor has so far completed installation of shear wall in the basement, MRF beam strengthening with FRP wrapping and modifications of cladding panel connections in low-rise portion of the building. Construction is divided into multiple phases and building occupants are temporarily relocated out of construction areas in each phase to other floors in the building to maintain business operation. The application of FRP wrapping instead of conventional concrete or steel jacketing to strengthen MEF beams substantially reduced the construction time. Pre-construction inspections to identify inconsistencies between details shown in original shop drawings and as-is conditions of cladding panel connections helped reduce RFIs during fabrication and installation processes. Widening corner joints to accommodate relative movements of cladding panels (Fig. 13) in two orthogonal directions was challenging but eventually successful in part due to extensive joint sealant tests during the design phase to select the appropriate products for the application. The on-site construction is scheduled to complete in winter 2021.

7. Conclusions

Many of the MRF beams in the subject building were found to lack the shear capacity necessary to develop ductile plastic yielding due to large utility openings in the beams, excessive longitudinal reinforcing adjacent to plastic hinge zones, and overly simplified assumptions in the estimation of probable shear demand. It was confirmed through 4/5-scale flanged beam component testing that externally bonded FRP wraps developed with carbon fiber anchors are an effective technique to increase MRF beam shear capacity without adding stiffness. Innovative details were developed to strengthen the beam cross-section: 1) at large openings using conventional FRP wraps, and 2) at end yielding regions with a combination of FRP battens, over-the-top fiber anchors, and saw-cut crack-inducement grooves between battens to distribute flexural cracks evenly.

A comprehensive seismic upgrade of buildings has to consider both the primary structural system and heavy non-structural components that may potentially fall during an earthquake. The precast cladding panels of the subject building were evaluated with local and global computer models to verify compatibility with the primary structural system and adequacy of the connections. Complex mechanisms were identified through a series of ¾-scale panel component tests. Primary mitigation measures included modifications of panel connections and widening of panel joints at building corners. Different sealant products were tested for bond, elongation, and overall material compatibility to ensure the building envelope would still be intact with the widened joints. The improvements in both structural and non-structural performances resulted in the achievement of overall seismic performance objective for a state-owned building.

Seismic renovation projects require innovative details capable of integrating new and existing components to achieve the desired performance in an earthquake. Prescriptive procedures specified in codes and standards for new design of components are not readily adaptable to these cases. This knowledge gap creates uncertainties in the design process. Component testing is an important tool to bridge the gap that allows designers to develop appropriate solutions. The specimen testing performed on MRF beams, precast panels and joint sealants provided engineers with the necessary information to reliably mitigate each unique deficiency and was instrumental to the success of the strengthening project.

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