SEISMIC RETROFIT DESIGN OF HISTORIC CENTURY-OLD SCHOOL BUILDINGS IN ISTANBUL, TURKEY

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ABSTRACT :

This paper presents a seismic retrofit design and implementation plan for two of the historic century-old school buildings located on the campus of Robert College of Istanbul in Turkey. The four and five-story buildings have unreinforced concrete bearing walls along their perimeters, which are connected by reinforced concrete floor slabs to steel framing encased in concrete in the buildings' interior. The thick perimeter walls of these Beaux-Arts style buildings resist most of the lateral loads. Seismic hazard at the school site was evaluated. Structural analysis showed that the buildings' perimeter wall piers have insufficient in-plane shear and bending capacities to resist seismic demands, especially in the buildings' transverse direction. The proposed retrofit design employs the use of horizontally oriented carbon fiber reinforced polymer (CFRP) composite fabrics on the interior and exterior faces of the buildings' perimeter wall piers and vertically oriented CFRP fabrics on the jamb faces of the wall piers. CFRP anchors in conjunction with the vertically oriented fabrics are recommended to develop the tensile strength of the vertical fibers into the pier-to-spandrel joints. A site investigation was performed to identify the buildings' architectural features, structural details, and mechanical systems to devise a retrofit scheme that minimizes the impact on these systems and preserves the buildings' historic fabric. Architectural impact is minimized by strategically placing the CFRP fabrics on the wall piers that have exterior surface offsets, and selecting a finish material that allows the CFRP fabrics to blend in with the other bare concrete wall surfaces.

KEYWORDS: Seismic retrofit, historic building, unreinforced concrete, CFRP fabric

1. DESCRIPTION OF BUILDINGS

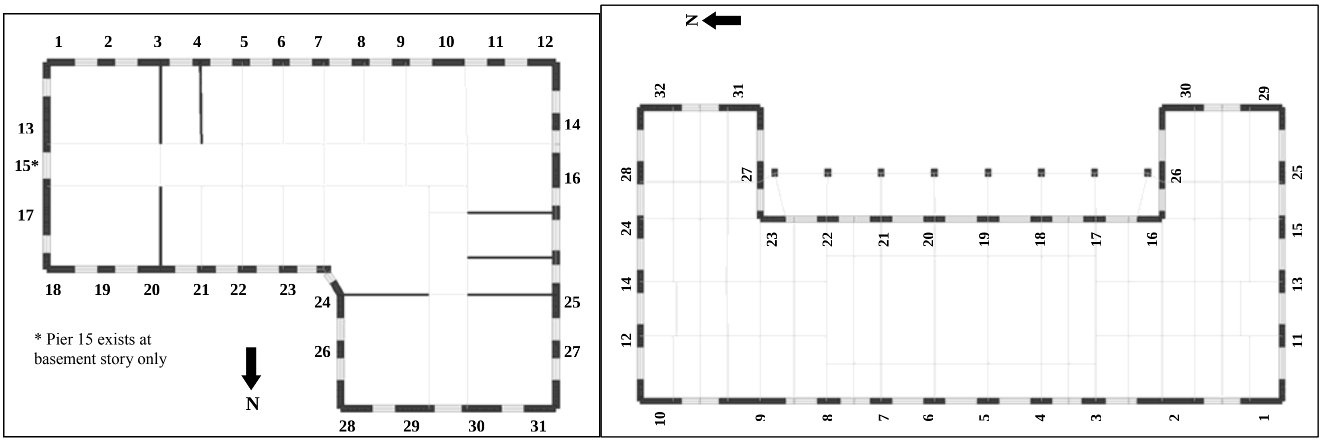
The site of the Robert College of Istanbul is located off Kuruçeşme Avenue in Arnavutköy on the western side of the Bosphorus Strait in Istanbul, Turkey. The site was built on a graded terrain which moderately slopes downward towards the Bosphorus on the East.

The building plan for Sage Hall (Figures 1a and 2a) is L-shaped with lengths of 35.4 m and 24.4 m (116' and 80') for the outer edges and a width of 15.2 m (50'). The four-story structure has story heights of 4.11 m (13'-6") except for the basement story which is 3.50 m (11'-6") tall. Sage Hall is the home to the girls' dormitory, the infirmary, the art studio and a student lounge.

Gould Hall is the center piece of Robert College high school. Its building plan (Figures 1b and 2b) is essentially a C-shape with dimensions of 54.0 m by 25.0 m (177' by 82'). The five-story structure has story heights of 4.11 m (13'-6") except for the basement story which is 3.50 m (11'-6") tall. Gould Hall is home to classrooms, the school library, a museum, administrative offices, the alumni office, faculty rooms, all service facilities including the kitchen, the dining hall, a café, and storage spaces.

(a) (b)

Figure 1 (a) Sage Hall building and (b) Gould Hall building



(a) (b)

Figure 2 Perimeter wall pier layouts for (a) Sage Hall and (b) Gould Hall

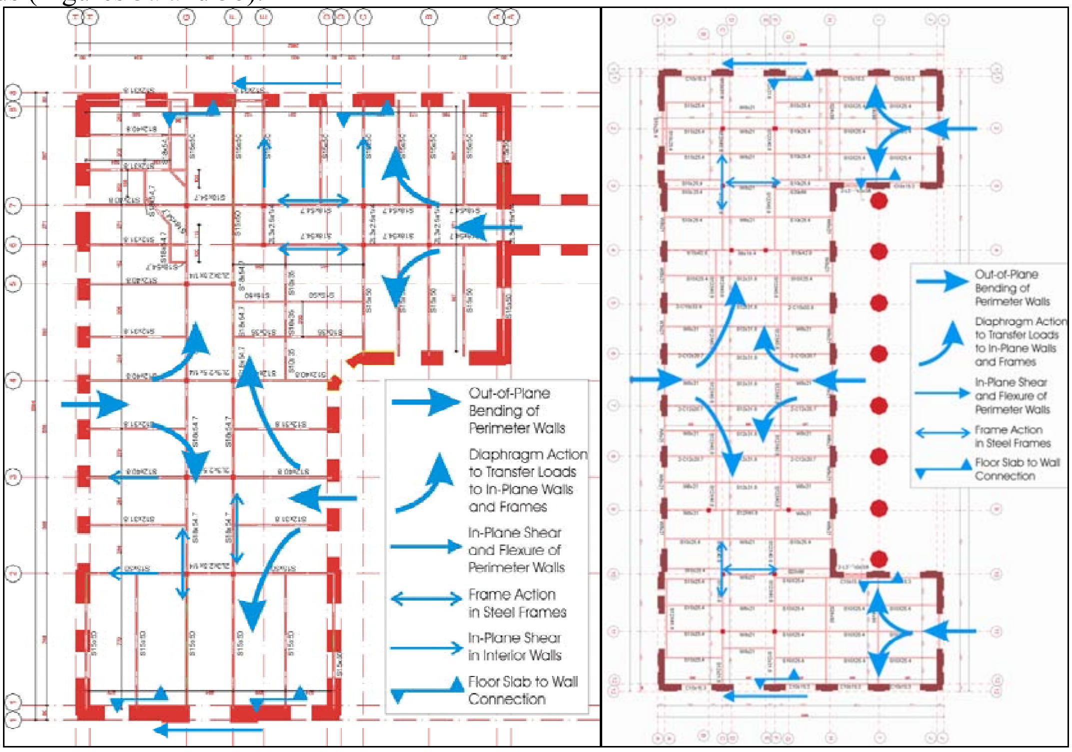
The structural system of both buildings consists of plain concrete bearing/shear walls along the buildings' perimeter, and a composite frame system in the buildings' interior. The perimeter walls have varying thickness along the height and along the perimeter of the buildings: Sage Hall: varying from 610 mm to 710 mm (24" to 28") at the basement and first stories, and 510 mm to 610 mm (20" to 24") at the second and third stories; Gould Hall: 810 mm (32") at the basement story, 610 mm (24") at the first story, and 510 mm (20") at the top three stories. The perimeter wall piers at the top three stories have 102 mm (4") thick offsets on the exterior façade, bringing the total thickness of the walls to 610 mm (24") at these locations

The interior frame system is composed of steel built-up columns and standard shape floor beams with riveted connections encased in plain concrete. The connection details on the original construction drawings show that the floor steel beams frame into the orthogonal perimeter beams at the floor levels. These riveted connections with shear tabs are embedded deep inside the concrete perimeter walls. The steel frame system extends into the attic and forms cross-diagonal braces connected to steel rafters supported on perimeter walls. Unlike Sage Hall, Gould Hall has a steel roof truss system also supported on the perimeter walls. The interior partition walls and staircase walls are composed of 102 mm (4") thick unreinforced concrete masonry blocks that are fully grouted.

Unlike Sage Hall, the 457 mm (18") thick perimeter walls in the attic are very tall extending much beyond the fourth story ceiling. This enhanced structural roof system supports the fourth floor in the central portion of the building via built-up steel hangers extending down from the roof trusses. This variation in the structural system of Gould Hall is due to the discontinuation of columns in the central portion of the building at the second and third stories to accommodate an open space for the library. The library is located on the second floor and has an atrium overlooking it on the third floor.

Except for the attic floor and the roof diaphragm, the floors of the two buildings consist of reinforced concrete slabs. The reinforced concrete floor slabs are 127 mm (5”) thick with a 76 mm (3”) terrazzo topping that are supported on the concrete encased steel floor beams. The concrete of the floor slabs is reinforced with #3 and #4 deformed steel bars at 6” spacing in the two orthogonal directions. Reinforcement is heavier for the library floor slab (#5 at 6”). The attic floor and the hipped roof have single straight-sheathed wood diaphragms with two nails at each end of each sheathing board on 2”x6” wood joists at 20” on center secured to steel floor beams or roof rafters.

Vertical loads are transferred from the roof and the floor slabs to the interior composite beams and columns and the perimeter concrete bearing walls, and then down to the foundation. The roof and the floor slabs, the perimeter and interior walls, the cross-diagonal braces and the trusses in the attic, and the interior composite columns resist lateral loads (Figures 3a and 3b).



(a) (b)

Figure 3 Seismic load path for (a) Sage Hall and (b) Gould Hall

It has been determined from drawings, pit holes and structural element exposures by other consultants that the perimeter walls bear on strip footings, while the interior columns have individual footings varying in size from 500 mm by 500 mm (20” by 20”) to 750 mm by 750 mm (29” by 29”). All concrete footings reportedly rest on bedrock composed of limestone and/or shale.

Gould Hall has eight, 1160 mm (46”) diameter plain concrete portico columns in series along its front (East) elevation supporting the roof terrace. A steel-framed walkway bridge added in 1993 at the rear (West) provides access to the building at the third and fourth floors.

Based on observations on site and the reviewed construction photographs, the steel sections used in the structural framing of the buildings and the concrete masonry blocks used to build the interior walls were manufactured in the U.S. and shipped to Turkey. The construction of the buildings was completed in 1914.

2. SEISMIC HAZARD

The soil conditions at the site have been extensively investigated by several engineering firms prior to the involvement of Weidlinger Associates, Inc. Therefore, we relied upon the information obtained by other engineering firms.

A comprehensive seismic hazard analysis was performed for the site, considering the seismic sources in the vicinity of the site and the various attenuation models available in literature. Results were compared with estimates from the Turkish Seismic Design Code (TSDC 2007) and 1997 UBC (ICBO, 1997). For brevity, this study is not described in this paper. However, it was determined from this study that the TSDC 2007 recommended spectra for the site is quite conservative and adequate to represent the design ground motions, compared with the most recent 10% in 50 years hazard estimates. Figure 4 shows the response spectrum based on life safety requirements of TSDC 2007 and used in the structural analysis of the school buildings.

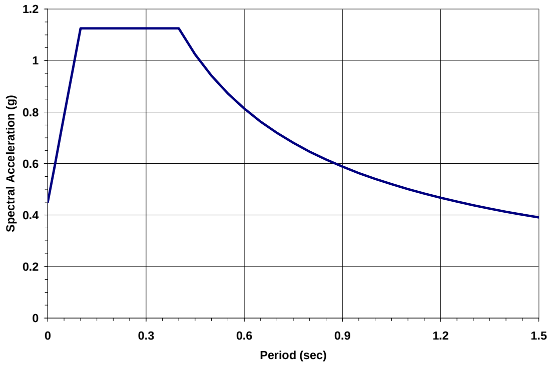
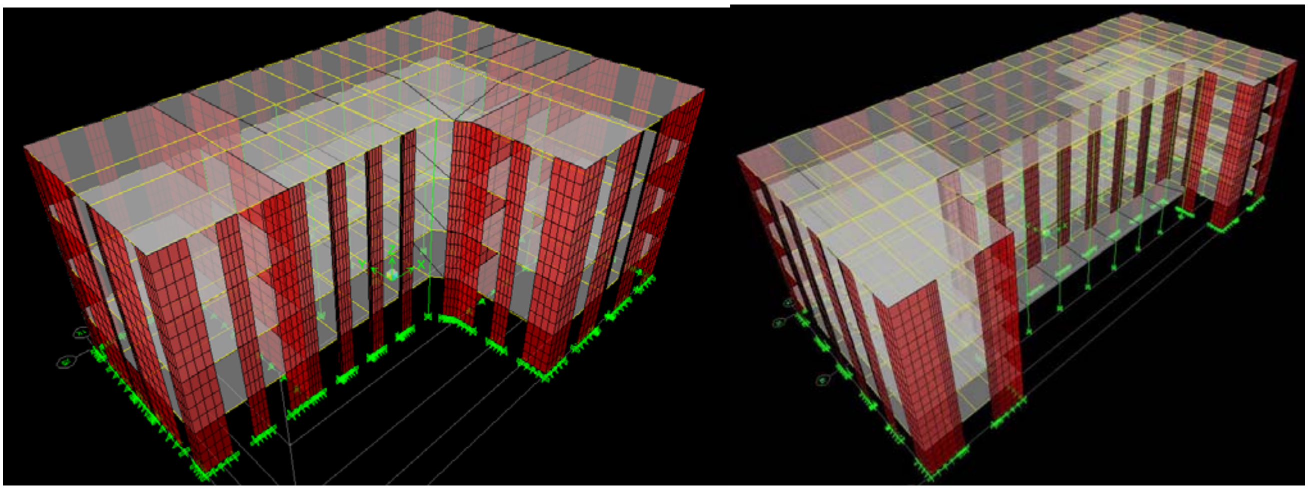


Figure 4 Response spectrum for school buildings for life safety per TSDC 2007

3. STRUCTURAL ENGINEERING ANALYSIS

ETABS computer models of Sage and Gould Halls, initially created by prior consultants, was substantially modified for a more accurate representation of the buildings. The finite element models use shell elements with cracked section properties for the wall piers to consider their in-plane and out-of-plane bending response. Figure 5 shows the computer models for Sage Hall and Gould Hall. The results of the computer model analysis were used to determine the earthquake load demands on various components of the lateral force resisting system of the buildings. The earthquake demands were then compared to the calculated capacities of the building components to identify those components that require structural strengthening.

The in-plane shear strength of the perimeter wall piers was calculated based on the guidelines for the seismic retrofit of existing buildings (ICBO 2001) and the concrete building code (ACI 318-05) equations for plain (unreinforced) concrete in shear. The shear strength of the piers includes contributions of the concrete ($1.33\sqrt{f'c}$) and the axial load on the concrete. The in-plane flexural strength of the perimeter wall piers was calculated based on the ACI 318-05 equations for plain concrete in bending. The bending strength of the piers includes contributions of the tensile strength of plain concrete ($5\sqrt{f'c}$) and the axial load on the concrete. The compressive strength of concrete (f'c), which is used in the shear and bending strength calculations, is based on the actual in-place strength as derived from the concrete core test results. The calculated in-plane shear and flexural capacities for each perimeter wall pier at each story in the buildings are compared to the earthquake demands on that pier, and the corresponding D/C ratios are calculated. The D/C ratios are larger than 1.0 for all piers with the exception of some of the piers at the top story. Furthermore, the largest D/C ratios occur for the piers located along the buildings' transverse direction. Therefore, most of the perimeter wall piers need structural strengthening for in-plane shear, especially those piers aligned in the transverse direction.



(a) (b)

Figure 5 ETABS computer model for (a) Sage Hall and (b) Gould Hall
(model modified from an earlier version by other consultants)

The vertical axial stress demand on the wall piers is small compared to the axial compressive strength of the wall piers. The large axial strength is achieved by the large cross-sectional area provided by the thick perimeter walls. Hence, the perimeter wall piers do not need strengthening for vertical compressive earthquake load demand. Some of the wall piers show a net tensile load demand that is greater than their tensile strength; however, it is expected that the strengthening of the piers for flexure will provide the necessary capacity in tension also.

The out-of-plane bending strength of the perimeter wall piers is calculated based on the ACI 318-05 equations for plain concrete in bending. The bending strength of the piers includes contributions of the tensile strength of concrete and the axial load on the concrete. The compressive strength of concrete, which is used in the determination of the tensile and flexural strength of concrete, is based on the actual in-place strength as derived from the concrete core test results. The out-of-plane bending demands on all wall piers are smaller than their calculated capacities. Furthermore, the height-to-thickness (slenderness) ratios of the perimeter walls do not exceed the ratios permissible by the ICBO 2001 guidelines for these unreinforced wall piers. The above analyses are based on the fact that the perimeter walls are properly connected to the roof and the floor diaphragms at every story so that the maximum unbraced height of the wall is the story height. The original construction drawings indicate that the perimeter steel members of the floor framing are fully embedded in the concrete perimeter walls and the floor framing is anchored into the perimeter walls via steel plates and/or anchor rods. This establishes the connectivity required to resist earthquake demands, and the validity of the above analyses. Hence, the perimeter wall piers do not require structural strengthening for out-of-plane bending.

The perimeter walls extend beyond the roof line as parapets. The height-to-thickness ratio of the parapet walls is less than 2.5, the maximum allowable ratio for parapets according to the ICBO 2001 guidelines for the seismic retrofit of existing buildings. Thus, the parapets do not need structural strengthening for out-of-plane bending.

The perimeter wall spandrels act like deep beams. Their shear and flexural capacities, calculated based on concrete compressive strength derived from in-situ test data, are greater than the shear and bending demands on the spandrels due to earthquake loads. Furthermore, the steel beams embedded in the concrete spandrels and the reinforcing bars below the steel beams provide increased shear and bending capacity to the spandrels. The D/C ratios for the perimeter wall pier-to-spandrel joint shear were also calculated. The shear strength of a joint was conservatively taken as $6\sqrt{f'c}$ to reflect the strength of an unconfined concrete section and is 50% of the specified strength by ACI 318-05. The D/C ratios for the pier-to-spandrel joints do not exceed 1.0 anywhere in the buildings. Structural strengthening is not required for the perimeter wall spandrels and the pier-to-spandrel joints.

The in-plane shear and bending strengths of the interior concrete masonry walls were calculated by taking into account the axial load on the walls in addition to the contribution of the concrete strength. The calculated in-plane shear and bending capacities for the interior walls at each story in the buildings are compared to the earthquake demands on those walls. The D/C ratios are smaller than 1.0 for the interior walls at every story; therefore, the interior walls do not need strengthening for in-plane shear and bending.

The in-plane shear strength of the reinforced concrete floor diaphragm slabs was calculated based on ACI 318-05 by taking into account the contribution of concrete ($2\sqrt{f'c}$) and the contribution of the steel reinforcing bars (ρtfy, where ρt is the ratio of the steel area to concrete area in the slab, and fy is the yield strength of steel). The in-plane bending strength of the floor slabs was also calculated by considering the contribution of both the concrete and the steel reinforcing bars, and ignoring the contribution of the steel floor beams. The demand on any of the floor slabs does not exceed its calculated shear or bending capacities. Associated with the in-plane diaphragm action of the floor slabs is the formation of tensile and compressive forces along the diaphragm chords (in this case, along the perimeter walls on opposite sides of the building). The demands on the perimeter wall spandrels are smaller than the chord tension capacities. The axial tension capacity of the steel beams embedded in the perimeter walls at floor levels immensely contributes to the tension capacity of the spandrels. The expected tensile strength of the existing structural steel was obtained from FEMA 356 (2000).

The in-plane shear strength of the wood diaphragms of the attic floor and the hipped roof was calculated based on FEMA 356 (2000). The diaphragms alone do not have sufficient capacity to resist the shear demand at the roof level. However, the steel cross-diagonal braces made up of double-angle sections together with the wood diaphragms have enough capacity to transmit the shear forces across the building from one perimeter wall to the opposite wall. Based on our observations on site, the perimeter walls are interconnected at the roof level to form diaphragm action. This connectivity is provided by a line of steel profile located around the periphery of the attic floor and embedded in the perimeter walls, which is connected to the cross-diagonal braces via the attic floor steel framing and the steel roof rafters.

The buildings' interior composite frame system (steel framing embedded in concrete) functions mainly to provide support for the vertical loads on the buildings, while the perimeter concrete walls resist most of the lateral loads from earthquake and wind. Although strengthening the interior frame system would increase its lateral load resistance, the lateral load path in the buildings would also be impacted and the contribution of the already strengthened perimeter walls to lateral load resistance would be reduced. Thus, structural strengthening of the interior frame system is not proposed.

The plain concrete portico columns on the front elevation of Gould Hall have their calculated capacities larger than the earthquake demands regarding shear forces, biaxial bending moments, and their interaction with the axial loads. The cracking strength of the concrete is not exceeded by the earthquake loads on the columns because although the unbraced length of the portico columns is rather large, they have massive cross-sections and do not provide support at floor levels.

4. SEISMIC RETROFIT

Analyses showed that the perimeter wall piers of Sage Hall, in their current condition, would reach their limit state (failure mode) in flexure rather than shear if they were subjected to a code-level earthquake. Hence, the wall piers need to be strengthened primarily to increase their flexural capacity. The perimeter wall piers in Gould Hall, however, would fail in shear rather than flexure. Hence, the piers of Gould Hall need to be strengthened primarily to prevent shear failure that can present itself in the form of sliding, rocking or diagonal cracking of the piers depending on the location of the pier in the building. Secondarily, the piers need to be strengthened to increase their flexural capacity to resist seismic demands.

The seismic retrofit work envisioned for the buildings involves strengthening the perimeter walls to increase their in-plane shear and bending capacities using sheets of carbon fiber reinforced polymer (CFRP) fabric attached to the interior and exterior faces of the wall piers with epoxy. The carbon fibers in the CFRP fabric are unidirectional, and therefore the proper orientation of the fibers is essential. The CFRP fabric is to be oriented vertically on the pier jambs to provide extra in-plane bending capacity to the wall piers, and horizontally on the pier surface to provide extra in-plane shear capacity to the wall piers. In addition, the top and bottom ends of the vertical fabric are to be anchored into the pier-to-spandrel joints using CFRP anchors in order to develop the tensile strength of the vertical fibers into the joints. The CFRP anchors are to be epoxied into predrilled holes in the pier jambs where the vertical CFRP fabric will be located.

Based on the structural engineering analysis explained in Section 3, the extra bending and shear capacity needed for each wall pier was determined. The number of layers of CFRP fabric required to provide this additional capacity was determined based on the equations of the FRP design guide ACI 440.2R-02 (2005). The perimeter wall piers in the buildings' transverse direction require the most number of layers or the largest amounts of CFRP fabric per square meter (foot) of wall surface. Some of the piers in the building transverse direction must be fully wrapped with 3 layer thick CFRP fabric, which requires removal and replacement of the window framings on either side of these piers.

Another option for the placement of the horizontal CFRP fabric is the two-sided application with CFRP anchors. This option has the benefit of avoiding removal and replacement of window frames because the horizontal CFRP fabric is epoxied only to the exterior and interior faces of a pier, but the horizontal fabric on each face has to be tied together by the CFRP anchors. This requires predrilling of holes through the thickness of the pier to epoxy-place the anchors in the holes. The CFRP anchors are to be located at each end of a 12" or 24" wide band of CFRP fabric.

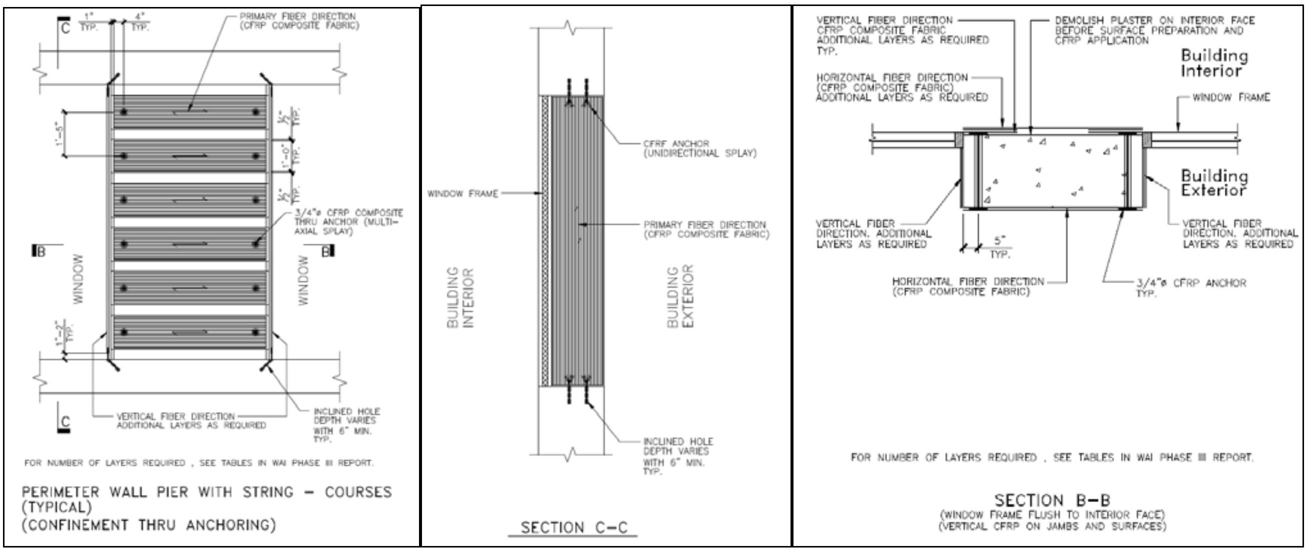
Some of the perimeter wall piers, especially those in the building transverse direction, need too many layers of vertical CFRP fabric for application on pier jambs only. Alternatively, fewer layers of vertical fabric are appropriate provided that the fabric is placed on the jambs and continued on the interior pier face for a horizontal distance of 2 feet from the jambs.

5. RETROFIT IMPLEMENTATION

The exterior faces of the perimeter walls are exposed concrete with several features to complete the façade design. One of these features is the plain string-courses that run horizontally all round the buildings along the first story. The string-courses are monolithic parts of the concrete wall formed by using a profiled formwork during construction. Each of the string-courses has a protruding depth of 25 mm (1"), width of 330 mm (13"), and spacing of 102 mm (4") along the height of the first story. Another feature of the exterior façade is the use of two or three-story high half column pairs. The half-columns of Gould Hall and the frequent use of surface offsets on the perimeter walls create a much more undulated exterior surface than that of Sage Hall elevations.

On smooth surfaced wall piers with no string-courses, 24" wide bands of horizontal CFRP fabric are to be placed adjacent to each other with 0 to ½" butt joints in-between and no overlapping. On the wall piers with string-courses, 12" wide bands of horizontal CFRP are to be placed on the 13" wide string-courses only, and not on the 4" spaces in between the string-courses. Where building corner offsets and other surface undulations are present on the wall piers, a piece of L-shaped CFRP fabric can be placed at the re-entrant corner with the ends of the regular fabric layers epoxied on top of it to achieve continuity of the fibers at the re-entrant corner.

One end of the CFRP composite anchors on the jambs should be epoxied into the drilled holes; the fibers on the other end should be splayed out (in a unidirectional fashion) on to the first layer of CFRP fabric with the additional layers epoxied on top of the splayed fibers. If only one layer of fabric is to be applied, the fibers should be splayed out on to the concrete surface. For the CFRP anchors epoxied into the through holes that are located at the ends of a band of horizontal fabric, the fibers at each end of the anchor should have a multi-axial splay to engage a greater portion of the concrete. Figure 6 illustrates the details of the retrofit design intent.



(a) (b) (c)

Figure 6 Perimeter wall pier (a) exterior elevation, (b) jamb elevation, and (c) cross-section

Spall repairs and epoxy injection of cracks in the perimeter wall piers must be performed during surface preparation and prior to the permanent placement of the CFRP fabrics.

Compared to alternative retrofit methods, the CFRP application is less invasive and less disruptive for the occupants, and more successful in conserving the functionality, architecture and heritage of the school. This retrofit plan minimizes impact on the architecture by strategically placing the CFRP fabrics on the wall piers that have surface offsets and undulations.

6. ACKNOWLEDGMENTS

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