# SEISMIC UPGRADE OF VANCOUVER TECHNICAL SCHOOL, CANADA

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**ABSTRACT :**

Vancouver Technical School is the largest school in British Columbia, Canada. The 4-storey heritage building was built in two sections in 1928 and 1955. The 1928 structure has a reinforced concrete frame, with hollow clay tile, unreinforced concrete block and glass block partition walls, and ribbed concrete slabs. It contains an auditorium constructed of concrete walls and timber roof. The 1955structure is constructed of concrete beams and slabs. The concrete columns have nominal reinforcing.

A cost effective and environmentally friendly seismic upgrade was achieved by minimizing demolition, retrofitting key elements and adding new components, while preserving all heritage characteristics of the buildings. Dynamic analysis and soil structure interaction were carried out. Seismic demand was evaluated using the 2005 National Building Code of Canada. Principles of performance-based design were applied taking advantage of all existing components.

Pounding and torsion were eliminated by connecting the 1928 and 1955 sections. This enabled a 20% reduction in seismic demand per Code provisions, which led to optimization of the new structural elements. A steel diaphragm was installed, hidden above the auditorium ceiling. Pilasters and walls were upgraded using bonded fabrics of Fibre-Reinforced Polymers (FRP) and steel plates. New shear walls were strategically located to balance torsional effects. No interior columns were retrofitted. Instead, the drift was reduced to a level where the columns can safely carry gravity loads.

The perimeter hollow clay walls were restrained using FRP anchors; partition walls with steel studs; glass block walls by bonding glass fibre rods into existing mortar joints.

**KEYWORDS:** Seismic upgrade, heritage, school, FRP, performance-based

## 1. INTRODUCTION

The Vancouver Technical School is the largest school in British Columbia (BC), Canada and one of 109 schools owned and operated by the Vancouver School Board (VSB). It comprises 5 different buildings: Main Building, Gymnasium/Cafeteria, Shops, Music Room and Junior Classroom Block. In 2002 and 2003 Sandwell Engineering Inc. (Sandwell) carried out the detailed design of the seismic upgrade for the Music Room, Gymnasium/Cafeteria and Shops buildings to suit the BC Seismic Mitigation Branch guidelines. In 2005 Sandwell carried out the Detailed Design of the Main Building and Junior Classroom Block, which is described in this paper.

The two buildings were analyzed using 75% of the Code design seismic base shear as recommended at that time by the Seismic Mitigation Branch guidelines. The base shear was amplified by an importance factor of 1.3 that brings the life-safety of the school structures essentially to the same level of a new residential or commercial building designed for 100% of the Code forces, thus bringing the buildings to a safety level that is intended to save lives but allows damage to the building after the design earthquake. In addition performance-based concepts were implemented as outlined in new Bridging Guidelines for the Performance-Based Seismic Retrofit of School Buildings (Bridging Guidelines) issued by the Ministry of Education of British Columbia at the time of the design.

The 4-storey building represents the most extensive application of Fibre Reinforced Polymers (FRP) in Canada for a building seismic upgrade project. Sandwells upgrade scheme combined concrete infill of unused wall cavities, FRP, and internal and external steel plates. All components were weather protected and were designed to meet the City of Vancouver's heritage considerations.

## 2. SEISMIC CRITERIA

### *2.1 Regional Seismicity*

The West Coast of British Columbia, an area that includes the major portion of British Columbia's population has been determined by geoscientists to be susceptible to large earthquakes – in fact, the largest expected in Canada. There is evidence that several large earthquakes in the range of magnitude 7 have struck this area in the last 200 years.

Geoscientists suggest that the area is due for an earthquake of magnitude 8 or greater originating from the subduction zone off the West Coast of Vancouver Island, or a shallow crustal earthquake of magnitude 7.
Past earthquakes have caused little damage because of the low population density in the affected areas, and the preponderance of wood-frame houses. Today, however, with the area's large population, the great variety of building types, and mounting evidence of damage from recent earthquakes in California and nearby Washington State, it is clear that extensive damage can be expected to both structural and non-structural systems.

### *2.2 Seismic Design Criteria*

The intent of seismic design requirements of the National Building Code of Canada 2005 (NBCC 2005) is to provide an acceptable level of public safety. Structures designed in conformance with these provisions should be able to resist moderate earthquakes without significant damage and major earthquakes without collapse.
The NBCC 2005 specifies the intensity of design earthquakes based on a probability of annual exceedance of 2% in 50 years or 1/2475 (or sometimes referred to as a 1 in 2500 year earthquake). Ground shaking intensity is typically expressed in terms of Peak Ground Acceleration (PGA). According to the code the recommended firm ground PGA in the horizontal direction for the site is 0.46g where g is the acceleration of gravity. The design earthquake is approximately a Magnitude 7.0 event with a distance to the epicentre about 50 to 60 km.

The commercially available structural analysis program, ETABS non-linear version 9.0.9 was used to prepare the 3D computer model of the building to perform seismic dynamic analysis. The model included beams and columns simulated as FRAME elements. The concrete slabs and walls were modeled as SHELL elements. The cracked section properties of the concrete elements were based on factors suggested by FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA). The soil foundations were simulated as springs. A three-dimensional view of one of the ETABS models of the building is shown in Figure 1.

The building was analyzed using the dynamic analysis procedure prescribed in the NBCC 2005, using response spectrum analysis. The seismic lateral loads were applied in the two orthogonal directions. The inherent torsion in the system is included in the dynamic analyses and the accidental torsion prescribed by code was added to the seismic load cases for the response spectrum analysis.

Natural vibration periods, mode shapes and modal participation factors were calculated to ensure sufficient mass participation as required by code; greater than 90% mass participation was achieved in all simulations. Modal responses are combined using the CQC (Complete Quadratic Combination) method to estimate the maximum values of force and displacement.

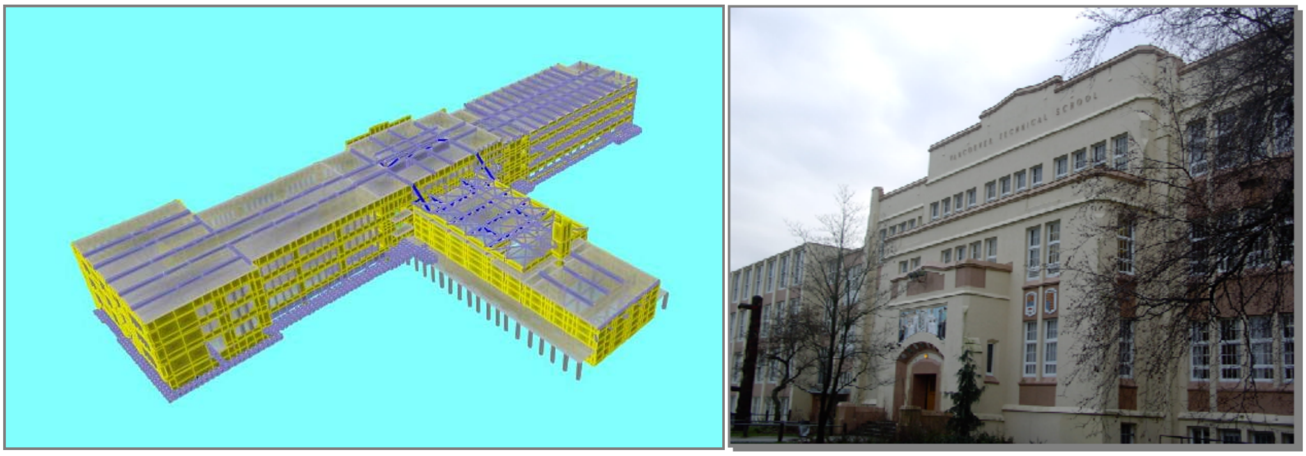


**Figure 1: 3D View of a Computer Model and Photograph of Main Entrance**

### *2.3 Base Shear Calculation*

The parameters and base shear equation per NBCC 2005 are:

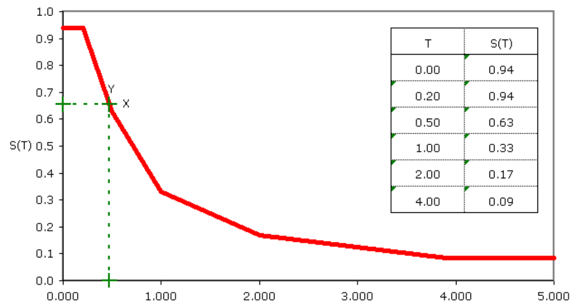
$$V_{Code} = S(T_a) \times I_E \times M_v \times \frac{1}{R_o \times R_d} \times W$$

- $T_a$ = Fundamental lateral period = $0.05\ (h_n)^{3/4}$ for shear wall structures; $h_n$ is the total height in meters above base; $T_a$ = 0.50 secs for this building
- $S(T_a)$ = Design spectral acceleration at fundamental period of structure
- $I_E$ = Importance factor = 1.3 for school building
- $M_v$ = Factor to account for higher mode effect on base shear = 1.0 for this building
- $R_d$ = Ductility force modification reduction factor that reflects the capability of a structure to dissipate earthquake energy through inelastic behaviour, commonly defined as ductility. This factor also recognizes the existence of alternate load paths or redundancy in the critical structural elements, thus increasing a number of locations where energy can be dissipated, and reducing the risk of structural collapse because of failure in the individual elements. In NBCC 2005, values of R factor vary from 1.0 for non-ductile (i.e. brittle) structures to 5.0 for well-detailed ductile steel or concrete structures. The original structure did not have structural elements with suitable reinforcing and details that can develop plastic hinges that absorb energy and allow the structure to survive displacements much larger than the structure was designed for on an elastic basis. Retrofit of the existing concrete walls and buttresses was based on the use of FRP that requires $R_d$ not exceed =1.5. Therefore, a value of $R_d$ =1.5 was considered appropriate considering that there is a very limited redundancy and ductility capacity associated with structures of this type; this value corresponds to the indicated value in the code for “conventional construction” of shear walls
- $R_o$ = Overstrength related force modification reduction factor that accounts for the dependable portion of reserve strength in a structure; it varies from 1.0 to 1.7. The selected value of $R_o$ = 1.3 corresponds to conventional construction of shear walls and was considered appropriate for this structure
- $W$ = Dead load plus 25% of the design snow load,

$V_{Code}$ = 0.31 $W$ and $V_{Design}$ = $0.25W$ for this building

*2.4 Modal/Dynamic Analyses*

NBCC 2005 requires that the spectral response acceleration values are modified by acceleration-based and velocity-based site coefficients, $F_a$ and $F_v$; the hazard site values previously discussed were not affected by these factors to obtain the response spectra shown in Figure 2 for the site class C soil for 1/2500 return period event for 5% damping because $F_a$ = 1.0 And $F_v$ = 1.0.




| T | S(T) |
|---|---|
| 0.00 | 0.94 |
| 0.20 | 0.94 |
| 0.50 | 0.63 |
| 1.00 | 0.33 |
| 2.00 | 0.17 |
| 4.00 | 0.09 |

**Figure 2: Response Spectra per NBCC 2005 for Vancouver**

Due to the irregularity of the buildings, response spectral analyses of each of the two existing buildings using the spectra above needed to be scaled to seismic base shears of $V_{Code}$ in the east-west and north-south directions. NBCC 2005 requires that if the dynamic base shear of an irregular building is less than 100% of the code base shear, $V_{Code}$, then the dynamic base shear shall be taken as $V_{Code}$.

Pounding of the adjacent buildings and torsional sensitivity were eliminated by providing a suitable configuration. The buildings were connected together and new structural elements were sized and located in a manner to eliminate all irregularities as defined in the Code. This resulted in a reduction of the seismic demand by 20% while diminishing the torsional effects. For 'regular' buildings the dynamic analysis is allowed to be reduced to 80% of the lateral earthquake design force, $V_{Code.}$

*2.5 Non-structural components*

Partition walls and their connections, parapets and capstones were restrained to withstand the lateral force, $V_p$, prescribed by NBCC 2005 applied to through the centre of mass of the element, that is equal to:

$$V_p = 0.3F_aS_a(0.2)I_ES_pW_p \qquad \text{where} \qquad S_p = C_pA_rA_x/R_p$$

$F_a$ $S_a(0.2)$, and $I_E$ as previously described. $W_p$ is the weight of the element.
$C_p$ = Element factor, 1.0
$A_r$ = Element force amplification factor, 2.50
$A_x$ = Height factor $(1 + 2h_x / h_n)$, being $h_n$ the height to level that is uppermost in the main portion of the structure and $h_x$ being the height above the base to level x, where the earth motions are imparted
$R_p$ = Element response modification factor, 2.50
$V_p$ varies from 0.29 $W_p$ on the ground floor to 0.76 $W_p$ on the fourth floor.

**3. PROJECT DESCRIPTION**

The existing building structures are of cast-in-place reinforced concrete construction. The main classroom block is composed of two separate buildings with an expansion joint in between. The Main Building, which includes the west wing and main entrance area were constructed integrally with the auditorium in 1928 in an 'L'-shaped configuration. The Junior Classroom Block or east wing was added in 1955. Structural analysis identified two global deficiencies as well as a number of local vulnerabilities particular to each building. Figure 3 illustrates this description and numbers the relevant deficiencies and upgrades. Combined global and local deficiencies together had the buildings at very high collapse risk for the design level earthquake.

In general, architectural finishes and fixtures affected by the new structural work were selectively removed and re-instated or replaced with new. Existing exterior windows and roofing were retained, with localized work occurring only where prompted by structural interventions. Some exterior concrete shear walls required architectural treatment to be in keeping with the heritage character of the building, or to incorporate window or door openings in them. On the south wall of Junior Classroom Block, some windows were filled in to provide additional shear capacity. Other windows or glass block walls were modified and retrofitted to balance natural lighting levels and suit structural restraint requirements.

New shear walls were strategically located and incorporated with basic accessibility upgrades required in the project scope. This upgrade included providing an elevator servicing all four floors (new 4 sided concrete shaft) as well as an intermediate grade level entrance. Upgrading of areas of refuge and horizontal exiting were also required in accordance with the City of Vancouver Building Bylaw. Some stairwells were rebuilt and reconfigured to incorporate shear walls and to improve vertical circulation, including removal of the encroachment in the corridor space and to provide an exit directly outside.

### *3.1 Global Deficiencies*

The global deficiencies were the torsional effects generated by the asymmetry of the original ‘L’-shaped 1928 building and the pounding effects created between the 1955 addition and the original building. Pounding was a serious concern because the expansion joint gap was insufficient and the floors are at different levels at the interface of the two buildings. By replacing the expansion joint with a structural connection joint (2), the problematic effects of asymmetry and pounding were simultaneously eliminated; as a result, the seismic demand could be reduced by 20% as previously explained. The existing north-south concrete walls of the auditorium were upgraded (3) to buttress the main classroom area, thus resisting seismic forces in the central region of the building.

Additional concrete shear walls were required to stiffen the wings of the connected building, particularly in the west wing, see (1), (4) and (6) in Figure 3. Some of these walls were placed on the exterior face of the building to reduce interior disruption (4). Steel drag struts in the central zone transfer seismic forces into the auditorium walls. Many existing walls and buttresses were reinforced with Fibre Reinforced Polymers (FRP) (5).

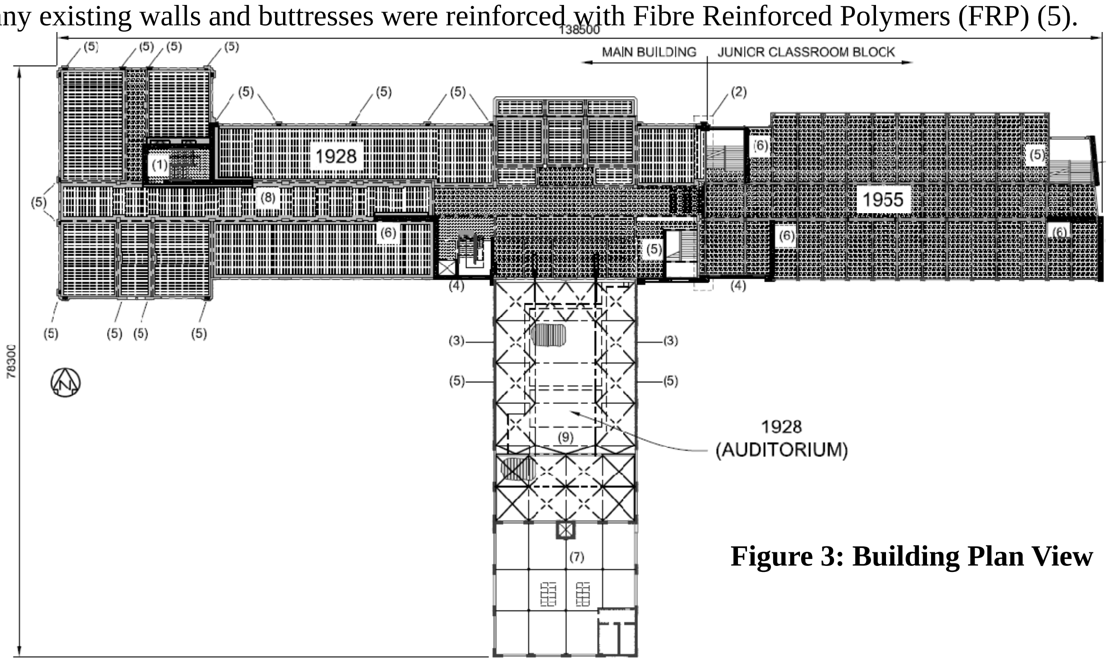



**Figure 3: Building Plan View**

### *3.2 Local Vulnerability*

The most relevant deficiencies to present local vulnerability and their remedial works are also numbered in Figure 3 for better illustration.

*3.2.1 Excessive drift and insufficient reinforcement of load-bearing elements.*

The average building drifts of the as-is buildings were calculated to be in excess of 6%. This excessive deflection to the load-bearing framing elements of the buildings was reduced to less than 1% by adding new shear walls and retrofitting existing concrete walls, piers and buttresses as described in the previous section. Over 360 non-ductile concrete columns were beyond the instability drift limit (ISDL), because of their short dimensions and insufficient lateral and longitudinal reinforcement. Following the performance-based concepts of the Bridging Guidelines the maximum permissible drift limit needed to be reduced to 1% to avoid retrofitting the columns. The boiler room columns (See (7) on Figure 3) were restrained at mid height with I-shaped steel elements to control the excessive slenderness.

The Bridging Guidelines require that load-bearing framing elements be capable of maintaining their support of vertical load for inelastic building deformations up to the ISDL. This issue is especially important for non-ductile concrete columns. The equation given below provides an estimate of the maximum clear height drift that a non-ductile concrete column can accommodate with a low probability of axial load failure.

$$ISDL(\%) = 4x \frac{1 + tan^2(65°)}{tan(65°) + P\left[\frac{s}{A_{st} f_{yt} d_c tan(65°)}\right]} \quad -1$$

but need not be taken as less than 1%

*ISDL* = instability drift limit; maximum permissible drift limit
*P* = axial load
*s* = spacing of transverse reinforcement
$A_{st}$ = area of transverse reinforcement
$F_{yt}$ = yield strength of transverse reinforcement
and *dc* = depth of core (centerline to centerline of ties)

*3.2.2 Flexible diaphragms.*

The roof slab of building west wing (See (8) in Figure 3) and the auditorium roof (See (9) in Figure 3) had insufficient lateral load capacity to act as diaphragms; the rest of the slabs had sufficient diaphragm capacity, therefore no upgrade was required. The west roof slab is a concrete ribbed structure with hollow clay tile infill and a 50 mm concrete slab at top; glass fibre fabrics of FRP were designed to take 100% of the shear demand, and were bonded to the soffit of the slab. The auditorium roof diaphragm was upgraded by eliminating the mass of the original heavy mortar ceiling, and installing a horizontal truss system made of steel angles anchored to the auditorium walls; the new steel angles are supported from the existing timber joists that previously supported the mortar ceiling, thus reducing buckling length and maximizing capacity, which resulted in a very light and efficient diaphragm. The ceiling was replaced by light acoustic panels.

*3.2.3 Foundation*

The existing foundation consisted of spread footings and was found to be inadequate for seismic loads. To utilize the maximum achievable capacity for the heritage pier and buttresses (See (5) in Figure 3) once upgraded it was required to control rocking at the foundation level. A grid system of foundation beams was built for rocking control, bearing capacity and up lift control, thus significantly minimizing the number of soil anchors and reducing disturbance to the building interior. The Figure 4 below illustrates the foundation layout.

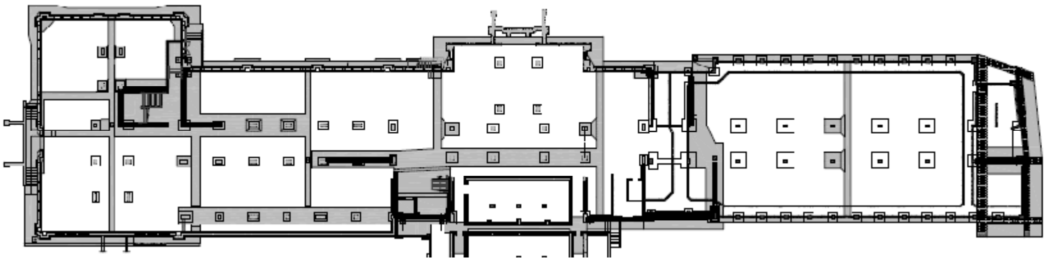


**Figure 4: Building Foundation Plan View**

*3.3.3 Non-structural restraints*

The building has numerous vulnerable non-structural elements such as brittle hollow clay tile partition walls, fragile glass block window-walls, unreinforced hollow concrete block walls and unbonded capstones on the roof parapets. In consideration of providing an "environmentally friendly solution" no demolition of these vulnerable elements was carried out; instead remedial works were implemented. The hollow clay tile and concrete block walls were restrained with light gauge steel studs eliminating the need of removal of the school finishes, boards and millworks by installing the studs behind the lockers on the corridor side of the walls as shown at the left of Figure 5; f the studs were designed to take the entire seismic load disregarding the inherent brickwork capacity. The glass block walls were restrained using part of the capacity of the mortar between the glass blocks by bonding with epoxy glass-fibre rods in the grooves between blocks as illustrated in the center of Figure 5; a plan view of the glass block wall with the rods is depicted at the bottom of Figure 5. Capstones were restrained by anchoring them with steel rods and adhesive epoxy to the solid concrete walls below, as shown at the right of Figure 5.

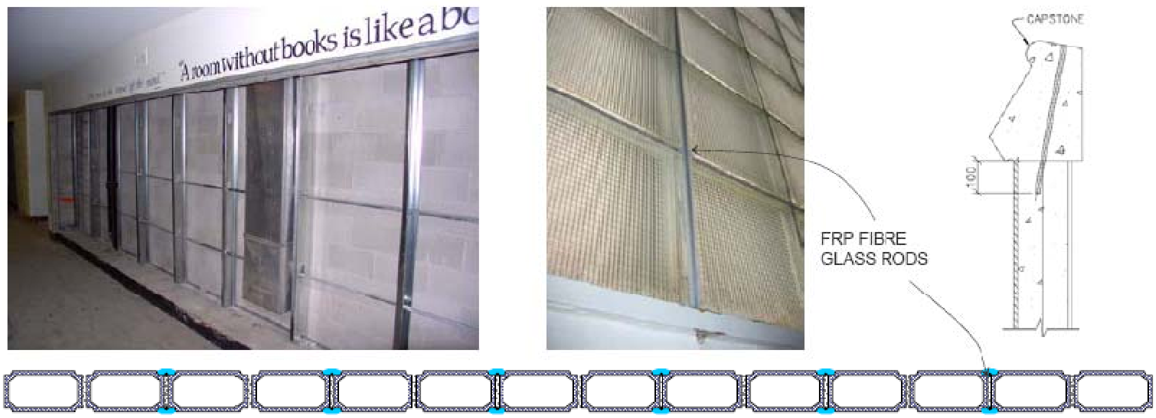



**Figure 5: Non-Structural Restraints**

*3.3.4 Upgrade of concrete walls, piers and buttresses*

Selected concrete walls, piers and buttresses at strategic locations were upgraded with FRP to lower the torsional sensitivity; strain compatibility analysis was performed to allow the concrete to achieve its maximum strain of 0.0035. Some architectural features were replaced by structural steel (matching the original features) to enhance compression capacity of the concrete pilasters allowing the FRP to develop its highest tensile capacity; Figure 6 illustrates a plan view of one the corner buttresses and its photograph. Typically carbon fibre fabrics were used for bending capacity upgrades and glass fibre fabrics for shear capacity upgrades. The solid square steel plate was connected to the concrete with high strength steel bolts fully saturated in epoxy of high hardness to prevent displacement of the steel relative to the concrete, as illustrated in the photograph of a cross section shown at the right of Figure 6; because of the difficulty in welding the square steel sections, the FRP fabrics resist all the tension and the steel plates combined with concrete resist all the compression. When the bending moment acts in one direction all the tension is concentrated in one end, and it is taken by the original reinforcement plus new continuous vertical strips of FRP carbon fibre bonded to the concrete; the ends of the strips were anchored by providing adequate development length beyond the point of maximum moment where the upgrade was required; "fibr-anchors" at the ends of the FRP strips were added to strengthen the part of the concrete that has potential for delamination, as illustrated in Figure 7. In the opposite end of the buttresses, the compression is not taken by the FRP, but the concrete and the steel plates. Voids in the buttress that previously were used for piping were filled with reinforced concrete to provide the required shear capacity; at certain locations FRP glass fibre plies were bonded to provide additional shear capacity required by seismic demand, anchored with fibr-anchors.

Regular rectangular concrete shear walls were upgraded by installing vertical continuous strips of FRP carbon fibre fabrics concentrated at the ends of the walls to provide the tension capacity due to bending, as illustrated in Figure 7; the compression is fully taken by the existing reinforced concrete. The shear capacity was enhanced by bonding fabrics of horizontal glass fibre. Fibr-anchors were also installed.


**Figure 6: Upgrade of Buttress with FRP and Steel Plates**

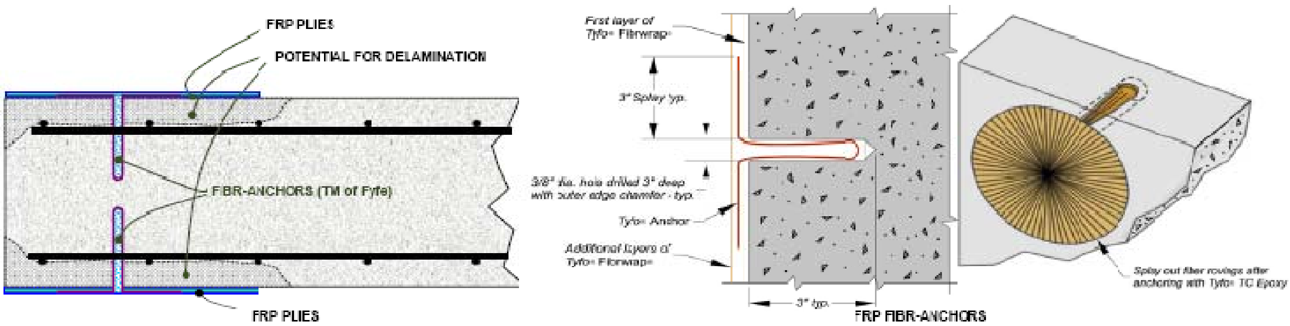



**Figure 7: FRP Anchorage**

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National Building Code of Canada 2005.

Bridging Guidelines for the Performance-Based Seismic Retrofit of British Columbia School Buildings – First Edition.