BRIDGING GUIDELINES FOR THE PERFORMANCE-BASED SEISMIC RETROFIT OF BRITISH COLUMBIA LOW-RISE SCHOOL BUILDINGS

DEMONSTRATION PROJECTS

MARCH, 2006

BRITISH COLUMBIA
Ministry of Education

A P
E G

Bridging Guidelines
Second Edition

Demonstration Project No. 1

for

SEISMIC RETROFIT OF WOOD/MASONRY SHEARWALLS FOR SENATOR REID ELEMENTARY SCHOOL

(School District No. 36 – Surrey)

Prepared by:
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March 2007

1.0 Executive Summary

The single storey, ten classroom block of Senator Reid Elementary School located in Surrey, BC is used as a sample for this Demonstration Project. This sample will illustrate the proposed seismic retrofit of existing interior wood stud walls and exterior hollow brick (SCR) masonry walls using the Bridging Guidelines (BG), 2nd edition issued in November 2006. Also, all effort is made to incorporate changes agreed upon at the BG meeting held on March 12, 2007.

This report will present the proposed seismic retrofit of two major LDRS components mentioned in the paragraph above and review the existing roof diaphragm consisting ofplank decking with plywood overlay.

We estimated that the use of Bridging Guidelines, 2nd edition will probably achieve a cost savings of 10% to 20% compared to an upgrade using the 2006 BC Building Code with an importance factor of 1.

2.0 Project Description

2.1 Introduction

Senator Reid Elementary School is located in Surrey BC. The south classroom block, which is the focus of this demonstration project, was constructed in April 1961, with an addition in September 1961. The structure is a single storey, mixed wood frame and unreinforced hollow clay brick (SCR) wall construction with wood deck roof and wood floor framed over a crawl space. The roof structure was upgraded in 1999 when 12.5 mm plywood sheathing and a metal strip drag strut/chord was added on top of the existing T&G spruce plank deck spanning over glulam beams and interior bearing partition walls. The roof glulam beams are supported by a mix of hollow clay brick (SCR) walls and wood posts on the building perimeter, and on wood posts concealed in the wood frame construction along the corridor. The brick walls rest on reinforced concrete foundation walls and strip footings. The interior wood posts and wood stud walls are supported by continuous concrete strip and spread footings. The ground floor is a wood floor framed over a crawl space. The block is attached to the rest of the school building along its north walls. Partial floor plan with walls layout is presented in the Appendix I "Design Calculations and Details" of this report.

2.2 Description of Existing Elements

- Existing interior bearing walls are 38x64 at 200mm o.c. staggered wood stud walls. Walls are covered with lath and plaster both sides and various school necessities. Walls continue through to the crawl space. Due to site conditions we were not able to confirm exact wall construction at floor level. Walls in crawl space are 38x140 studs at 400mm o.c., not sheathed and they are nominally fastened to a 150mm wide by 450mm deep concrete foundation.

- Existing hollow clay brick (SCR) walls are not reinforced and they have continuous windows along full wall length. At each end, walls have returns that support glulam beams located on each side of interior partition wall. In 1991, most of the hollow clay brick walls were reinforced for out-of-plane seismic loading using exterior vertical HSS sections. This strong back HSS reinforcing does not comply with current Bridging Guidelines requirements for spacing and connection design.
- Existing 64x133 spruce plank decking was reinforced in 1991 by adding 12.5mm D. Fir ply sheathing and steel strap drug struts/chords.

3.0 Seismic Assessments

In 2004, Senator Reid Elementary School was assessed following prescribed forms provided by the Ministry of Education. The Classroom block that is part of this demonstration project was identified to have medium to high risk and in need of seismic retrofit.

This assessment was confirmed in the Feasibility Study stage 1 and 2 report. The estimated cost for seismic upgrade was less than 70% of the replacement cost and as such was recommended to proceed to detailed design.

Senator Reid Elementary School is located in Seismic Zone 4, per the ‘Seismic Zone Map of British Columbia First Edition APEGBC/UBC 2005’ prepared by AXYS Environmental Consulting Limited, dated March 23, 2005.

The site class has been confirmed, by a site specific geotechnical investigation, to be an equivalent of Site Class C per Table 4.1.8.4.A of the National Building Code of Canada, 2005. The investigation found the subsoil profile within the site to consist of clay/silt soil overlying very dense till-like soil. There are no apparent geotechnical risks.

4.0 Proposed solution

The structural seismic upgrade scheme for Senator Reid Elementary School is developed in accordance with the procedures recommended in the Bridging Guidelines 2nd Edition.

In the north/south direction, the load resisting structural system consists of the perimeter exterior SCR hollow clay brick walls having returns each end (see Figure 1, Appendix I). Walls are not confined at the top due to the presence of continuous windows between top of the SCR brick wall and roof diaphragm. The walls are also neither reinforced nor anchored to the foundation wall. Therefore, the SCR brick walls do not meet all of the out-of-plane requirements prescribed in the Bridging Guidelines.

The SCR hollow brick material is not addressed as such or defined in the Bridging Guidelines, but could be compared to hollow concrete masonry based on similar material properties.

Proposed retrofit is as follows:

- Remove existing windows and add cast in place concrete along the full length of SCR brick walls
- Reinforce existing SCR brick walls and dowel them into concrete foundation walls
- Provide new connections between roof diaphragm and newly formed walls
- Add HSS columns at each SCR brick wall return to ensure gravity stability for roof structure when SCR brick walls are subjected to out-of-plane load and drifts > 4%

In the east/west direction, the load resisting structural system of the south classroom block consists of the wood stud bearing walls (see Figure 1, Appendix I). The stud walls are covered with gypsum lath and plaster, which has a lateral capacity of 0.88 kN/m, per side, as provided by UBC team. After assessment of existing shear resistance it was concluded that lath and plaster does not have sufficient shear resistance and that designated walls will have to be reinforced.

Proposed retrofit is as follows:

- Remove existing lath and plaster on one side of interior bearing walls to expose existing studs.
- Provide new blocked plywood sheathing
- Connect walls to roof diaphragm
- Provide load path between wall above and below floor framing
- Provide new anchors between walls and concrete foundation
- Provide hold-down anchors and new footings

Existing roof diaphragm and metal strap drag strut/chord were found to have sufficient capacities.

For calculations and details refer to Appendix I.

5.0 Cost Analysis

Seismic upgrade cost contains a significant percentage that is not a structural cost. Based on our experience, structural cost will be approximately 50% of total project cost. The additional component is associated with architectural, mechanical and electrical cost, fees, contingencies and contractor’s overhead. Also, a further cost of approximately 10% of the upgrade cost associated may be required to provide temporary classrooms during construction.

The table below presents a comparison of shear force demands based on either NBCC 2005, BG 1st Edition or the BG 2nd Edition.

Table 1
Shear force demand as a % of weight

Prototype	NBCC 2005 (I=1.0)	Bridging Guidelines 1st Edition	Bridging Guidelines 2nd Edition
W-1	13%	12%	8%
M-2	30%	21%	21%
D-1	13% / 30%	------	8%

W-1 : Blocked plywood shearwall
M-2 : In-plane reinforced masonry shearwall
D-1 : Blocked plywood diaphragm

Table 1 shows that, the Bridging Guidelines 2nd Edition has 62% lower shear force demand for blocked plywood shearwall and 43% lower shear force demand for reinforced masonry wall comparing to the NBCC 2005. Considering that structural cost is only 50% of total project cost, total structural savings based on shear force demand comparison is 31% for prototype W-1 and 21% for prototype M-2. These savings are also largely dependant on other components such as building layout, constructability, minimum requirements and various others constraints

To seismically upgrade this school, the estimated cost, based on BG 2nd Edition, in September, 2006 dollars was 1.96M or 36% of replacement cost that was estimated to be approximately

5.50M for the school of the same size. (Note: the ten classroom block only represents a portion of the overall area of the school being upgraded).

6.0 Conclusion and Recommendations

The Bridging Guidelines 2nd Edition design forces for this project are either same or significantly lower than design forces based on either Bridging Guidelines 1st Edition or NBCC 2005 respectively.

In our opinion the biggest benefit of BG 2nd Edition is the "toolbox" approach. This approach allows designers to use existing systems and utilize existing strength of certain materials, such as lath and plaster in this demonstration project. Another, significant advantage is that designers are allowed to use different LDRS systems with different drift limits for flexible diaphragms in the same direction of seismic force, as long as overall performance of the diaphragm is not compromised and does not exceed maximum diaphragm inelastic strain. Different drift limits are shown in the Figure 2, Appendix I.

As the total project cost is significantly influenced by architectural, mechanical and electrical costs related to the seismic upgrade, plus other indirect costs, total savings discussed in Section 5 may not be as high as may be expected. During design, all effort shall be made to minimize disturbance of architectural finishes, cabinetry, major mechanical and electrical components and extensive foundation work.

Appendix I – Design Calculation and Details

South Wing Classroom Block – one storey over crawl space area

The design calculations for three major elements are presented:

Diaphragm

1. Prototype D-1, Blocked plywood diaphragm

Lateral Deformation Resisting System (LDRS):

2. E-W direction: Prototype W-1, blocked plywood shearwall
3. N-S direction: Prototype M-2, reinforced masonry shearwall

N

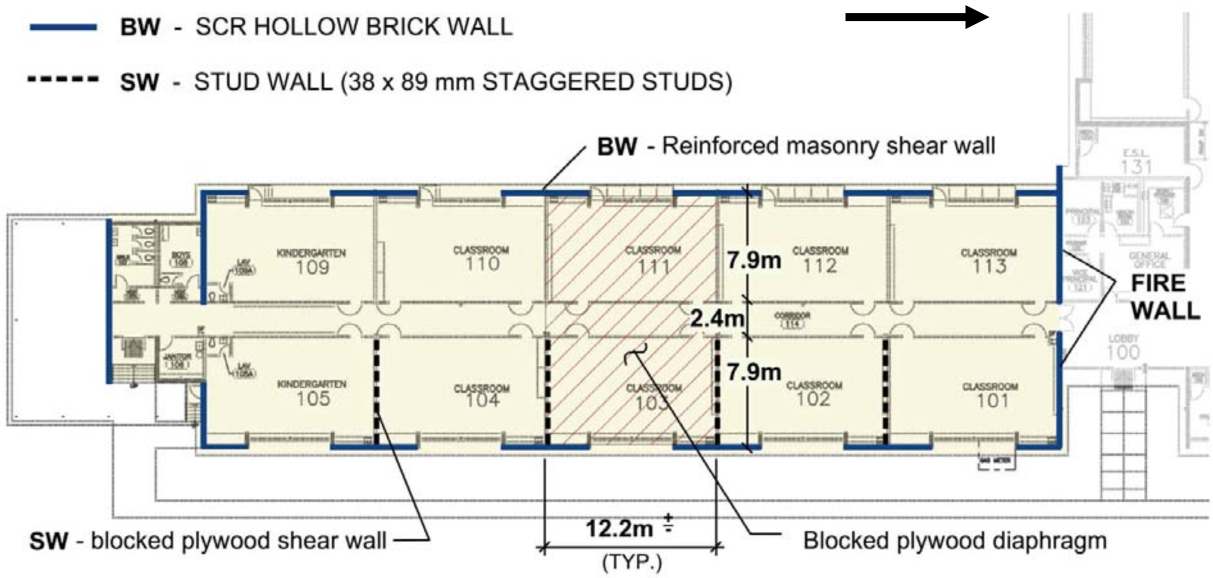


Figure 1

Diaphragm Check

Prototype D-1 ISDL = 4% R_o =1.7

Diaphragm span, L_d = 12.2 m

Minimum required factored resistance for diaphragm at each end of span, $R_{md} = 8\%W_d$

Weight of diaphragm plus weight of walls normal to shaking direction W_d = 402 kN

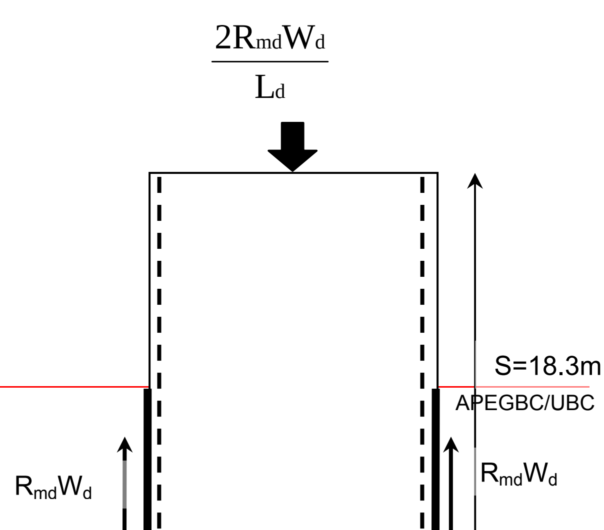
$R_{md} = 8\%W_d = .08 \times 402 = 32.2$

Shear force in diaphragm = $R_{md}W_d$ / S

= 32.2 / 18.3 = 1.8 kN/m

Existing roof diaphragm:

½" plywood overlay nailed with
64mm nail @ 100 along all plywood
edges and 300 each way between



on 3” T & G decking

V_r = 6.31 kN/m > 1.8 OK

Ref: Wood Design Manual 2005
p. 466 – Diaphragm selection Table
Factored shear resistance for
blocked SPF diaphragm.

Following standard practice 3 x 6 planks should be fastened with min 1 – 153mm nail at support.

Plank width = 133 mm

n = 1000 / 133 = 7.5 plank per meter

$V_{r\ nails}$ = 7.5 x 2.48 (1.0 x 1.0 x1.0 x 1.3) = 24.2 kN/m

Ref: Wood Design Manual 2005
p. 242 – Table 7.3, Basic factored lateral
resistance for nails $\Phi n_u J_y$

Because nailing pattern could not be confirmed, safety factor is used to assess nailing adequacy.

FS = $V_{r\ nails}$ / V_r = 24.2 / (2* x 6.31) = 1.91 Acceptable

* Connection supports roof diaphragm at each side of the wall

It is concluded that existing plank nailing has sufficient strength to provide effective diaphragm support.

Chord force, $F_c = \left(\frac{R_{md}W_d}{100}\right) \bullet \frac{L_d}{2S}$ = 32.2 x 12.2 / (2 x 18.3) = 10.7 kN

Existing chord: Galvanized metal strip 1.22mm x 150 continuous nailed to
roof with 64mm nail at 100 staggered.

$T_r = \phi A_g F_y$ = 0.9 x 1.22 x 150 x 240 = 39.5 kN > 10.7 **OK**

Design of blocked plywood wood shearwall:

Prototype: W-1 Site class: C Seismic zone: 4 ISDL= 4% R_o= 1.7

Minimum required factored resistance for LDRS, R_m = 8%W

W: weight of building above the mid height of the first story of the building plus 25% of snow load, for tributary area 12.2m by 18.3m.

Roof level W_R = 412 kN

Floor level W_F= 294 kN

Total Weight, W_T= 294 + 412 = 706 kN

At floor level, R_m = 8%W = 0.08 x 412 = 33 kN

R_m = 33 / 6.5m = 5.1 kN/m Length of wall = 6.5m

Existing wall: 2 x 6 stud wall with lath and plaster both sides.

Vr = 2 x 0.88kN/m = 1.76 kN/m < 5.1 kN/m **NG**

Wall to be reinforced for: 5.1 – 0.88 = 4.22 kN/m (one side lath & plaster 0.88kN/m)

Lath and Plaster on one side of the wall will be replaced with plywood sheathing.

Use 12.5mm D. Fir blocked plywood sheathing at one side of wall nailed with 64mm long, 3.25mm diameter nails, spaced at 150mm O.C. at panel edges.

V_r = 5.72 kN/m > 5.1

Ref. Wood Design Manual 2005 page. 488 Shearwall selection table

At crawl space level, R_m = 8%W = 0.08 x 706 = 56.5 kN

R_m = 56.5 / 7.5m = 7.5 kN/m Length of wall = 7.5m

Use 12.5mm D. Fir blocked plywood sheathing at one side of wall nailed with 64mm long, 3.25mm diameter nails, spaced at 100mm O.C. at panel edges.

V_r = 8.29 kN/m > 7.5 **OK**

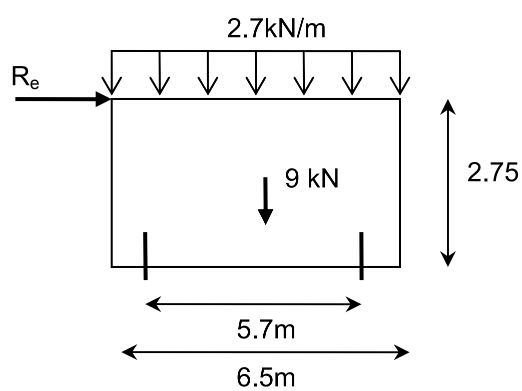
Base moment at floor level:

Roof DL on Wall = 2.7 kN/m

DL of Wall = 9 kN

Factored resistance of LDRS:

R_e = 5.72 x 6.5 = 37.2 kN



Overturning moment:

37.2 x 2.75 = 102.3 kNm

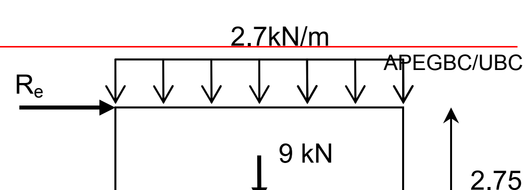
Resisting moment:

(9 + 2.7 x 6.5) x (6.5/2) = 86.3 kNm < 102.3 Hold down is required

Hold down force = (102.3 – 86.3) / 5.7 = 2.8 kN

Use Simpson Hold down PHD2 T_r = 24.0 kN

Ref: Simpson Strong Tie catalogue effective 1/1/2006 page 33



Base moment at crawl space level:

Floor DL on Wall = 0.21 kN/m

Factored resistant of LDRS:

R_e = 8.29 x 7.5 = 62.2 kN

Overturning moment:

62.2 x (2.75 + 1)= 233.3 kNm

Resisting moment:

(0.21 x 7.5) x (7.5/2)
+ (9 + 2.7 x 6.5) x (6.5/2) = 92.2 kNm < 217.5
Hold down is required

Hold down force = (217.5 – 92.2) / 5.7 = 22 kN

Use Simpson Hold down PHD5 Tr = 33.6 kN

Ref: Simpson Strong Tie catalogue effective 1/1/2006 page 33

Design of connection between roof diaphragm and shearwall

Design for minimum of:

1. Factored resistance of diaphragm = 2* x 6.31 = 12.62 kN/m
2. Factored resistance of shearwall = 5.72 + 0.88 = 6.6 kN/m

* Connection supports roof diaphragm at each side of the wall

Factored shear resistance of blocked plywood shearwall = 6.6 kN/m

Factored lateral resistance of nail (4" long, 4.88 dia. , 39mm penetration), N_r =1.87 kN

6.6 / 1.87 = 3.5 Use 4 nail per meter, spacing 250mm

It is assessed that existing nails hve sufficient strength.

Expected nail resistance = 24.2 kN/m

Connection demand = 6.6 kN/m

FS= 24.2 / 6.6 = 3.6 Acceptable

Drag strut to be designed for:

$Rc = 6.6kN/m \times 6.5m = 42.9$ kN

$A_{req} = 42.9 \times 10^3 / (0.9 \times 240MPa) = 199 \, mm^2$

199 /75 = 2.65mm thick

Use 16 ga (1.58mm) x 75mm Plate each side of the wall c/w

2 – 12 SDS ¼ x 21/2" wood screws

$Vr = 2 \times 12 \times 1.87 = 44.9$ kN

Ref: Simpson Strong Tie catalogue, effective 1/1/2006 page 15

Design of masonry brick wall:

The existing masonry walls are not confined at the top due to the presence of continuous windows between top of the SCR brick wall and roof diaphragm. In order to provide the load path, the existing windows are removed and a new cast in place concrete beam is provided along the full length of SCR brick walls.

The existing masonry walls are not reinforced and they sustain 4% drift in out– of – plane. Since these walls do not meet the requirement of Bridging Guidelines– section 6.5, the minimum vertical reinforcement based on CSA- S304.1-04 should be provided.

Use 15M at 1200 and 2- 20M at each end

In plane shear capacity of masonry wall

Prototype: M-2	In plane reinforced masonry wall	Site class: C	Seismic zone: 4
		ISDL= 1.5%	R_o= 1.5

Minimum required factored resistance for LDRS, R_m = 21%W

W: weight of building above the mid height of the first story of the building plus 25% of snow load for tributary area 9.1m by 12.2m.

Roof level W_R = 206 kN

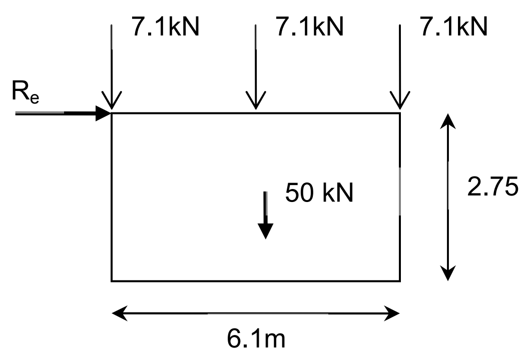
Shear force at floor level, R_m = 21%W = 0.21 x 206 = 43.3 kN

The 6.1 m long vertically reinforced masonry wall with an opening (1m x .75m) at mid, has a factored shear capacity of 345 kN.

Vr = 345 kN/m > 43.3 **OK**

At crawl space level, there is a reinforced concrete wall.

Base moment at floor level:



Roof DL on Wall = 3 - 7.1 kN

DL of Wall = 50 kN

Factored resistant of LDRS:

R_e = 345 kN

Overturning moment:

345 x 2.75 = 949 kNm

Resisting moment:

(7.1 x 6.1) + ((7.1 + 50) x 6.1/2) = 217 kNm < 949

The vertical rebar should be anchored to the existing concrete wall foundation at crawl space area

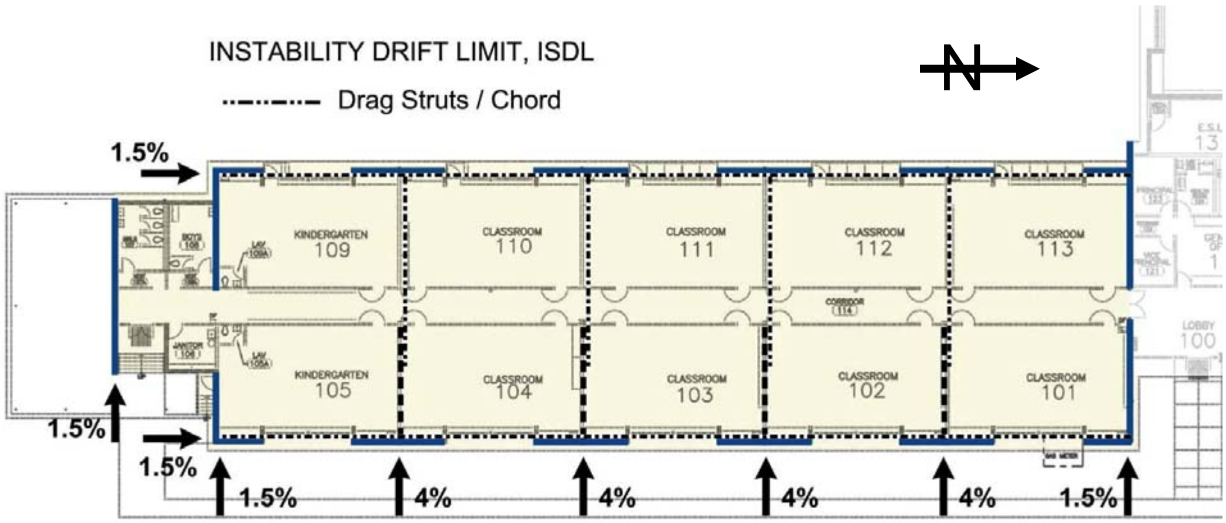


Figure 2

Exterior of Classroom block- Masonry brick wall

Typical interior load bearing partition wall

Typical floor structure, diagonal shiplap over joists

Crawl space area, typical pony wall under load bearing partition wall

Bridging Guidelines
Second Edition

Demonstration Project No. 2

for

SEISMIC RETROFIT OF STEEL/MASONRY GYMNASIUM FOR FRANK HURT SECONDARY SCHOOL

(School District No. 36 - Surrey)

Prepared by:
Bush, Bohlman & Partners

March, 2007

Executive Summary

This document has been prepared to provide engineers with an example of how steel braced frames, masonry walls and steel deck diaphragms are assessed and retrofitted using the Bridging Guidelines.

The retrofit of the Frank Hurt Secondary School Gymnasium is documented in this report. It includes detailed drawings as well as hand calculations using the 2nd Edition Bridging Guidelines. These include:

1) Retrofit of stack bond masonry wall with FRP reinforcement.
2) Replacement of existing steel braces with new steel braces in conjunction with a steel stud shearwall.
3) Upgrading of the existing steel deck roof diaphragm.

The use of the 2nd Edition Bridging Guidelines would have yielded a probable cost savings of 15% compared to an upgrade using the 2005 National Building Code of Canada (NBCC) with an importance factor of 1 (i.e. I=1.0).

Project Description

Frank Hurt Secondary School is located in Surrey, BC. The Gymnasium, which is the focus of this demonstration project, was upgraded in 2006 as part of the BC Schools Retrofit Program. The retrofit design was based on the 1st Edition Bridging Guidelines.

The Gymnasium was originally constructed in 1972, and has a floor area of 1780m^2. An addition was made to the north side in 1976. The exterior cladding is a combination of insulated metal panels and a masonry veneer. An exterior view of the block is shown in Figure 1.

The roof structure was a 38mm, 22 gauge steel deck on 1370mm deep open web steel joists (OWSJ) spaced at 1.82 meters. There were 3x6" wood "nailers" in between the deck and OWSJ, to which both were attached. The deck was fastened to the nailers with screws and sidelapped with button punching. The spacing of the connections was unknown. There was a W-section around the perimeter of the roof, which could act as the chord, but no shear lugs were present to transfer the chord forces to it. Tension only horizontal cross-bracing (single L51x51x6.4) was present in the roof, which would have acted as a diaphragm. Figure 2 shows a detail of this connection, including the roof bracing.

Three of the walls (East, West and South Elevations) had a steel rod (25mm diameter) braced frame LDRS located 2.43 meters above the floor level. Figure 3 shows an interior view of the braced bay. The braces were connected to the roof beam (W-section) above, a 203x203 HSS wind girt below, and to W200x42 columns on either side. The connection (see Figure 4) was well below capacity design requirements, and also did not meet any "bail out" conditions. Braces were also present below the HSS wind girt (not shown in Figure 3, because they are behind the finished and insulated wall on one side and a brick veneer on the other). These braces were comprised of HSS64x64x6.4. While the single lower braced bay (HSS) members likely had sufficient capacity to take the loads of the upper two braced bays (1" rods), the connections of the lower bay were unknown.

The columns were connected to the foundation with four ¾" anchor bolts.

The North wall was comprised of 250mm wide hollow concrete blocks arranged in stack bond. The wall was lightly reinforced with two grouted 15M vertical bars at 1200mm o/c. The wall was anchored to the foundation with two 15M bars at 4 ft. o/c, however it was not connected at the top (see Figure 5).

The columns on all walls are W200x42. They are spaced at 5486mm on the East and West walls and at 4876mm on the North and South walls.

The main floor of the gymnasium consists of flooring on slab on grade. There is a mezzanine area (stage) on the south side of the gymnasium that was upgraded, but that is not covered in this demonstration project.

The foundation consisted of a 250mm deep x 300mm wide strip footing around the perimeter of the gymnasium. Spread footings (900 x 900mm, 300mm deep) were located under each column.

Seismic Assessment

The school was assessed in August of 2004. The assessment indicated that the school was located on Site Class C, with a base shear demand on the gymnasium of 33%W. The gymnasium was given an overall risk of Medium/High, with possible adjacency issues. The following deficiencies were noted:

- Weak roof diaphragm
- Weak steel bracing (vertical lateral system)
- Poor connections at roof level
- Poor connections of adjacent blocks
- Insufficient foundation capacity

<u>Phase I Feasibility Study</u>

The Phase I Feasibility Study was conducted in November, 2005. This assessment was done using the 1st Edition Bridging Guidelines. The gymnasium was specified to be located in Seismic Zone 4 on Site Class C. The governing prototypes were LDRS 1 (Concentric Steel Braced Frame (Tension Only)) in the North-South direction and LDRS 13 (Reinforced Concrete Masonry) in the East-West direction. The gymnasium was assigned a medium level of risk in the North-South direction and a high level of risk in the East-West direction. The list of items to be covered in the retrofit was given:

- Strengthen existing metal deck roof diaphragm or add horizontal trussing below roof
- Improve connections on existing brace bays and add additional bracing as required
- Strengthen drag strut and diaphragm chord connections
- Strengthen out-of-plane and in-plane resistance of existing stack bond masonry wall
- Address concrete masonry wall connections at non load-bearing walls at mezzanine
- Strengthen connections at deck to drag strut and to reinforced masonry wall

Phase II Feasibility Study

The Phase II Feasibility Study was undertaken in March, 2006. The Phase II study confirms the assessment and risk levels determined in the Phase I study. It also confirmed there were no major non-structural risks in the gymnasium, the adjacency issues could be addressed by the seismic upgrade, and enrolment projections indicate that the school is required. A review of the site conditions indicated that it was located on Site Class C, and that liquefaction was not an issue. The following is the recommended seismic upgrade program:

- Strengthen the existing deck roof diaphragm
- Connect the roof diaphragm to continuous perimeter steel beams acting as drag struts and chord members
- Provide new steel bracing connecting the steel roof beams to an existing continuous steel HSS girt
- Provide new steel stud shear walls to transfer shear from horizontal HSS girt into the foundations
- Reinforce and strengthen the existing unreinforced masonry wall for in-plane and out-of-plane seismic forces
- Connect the existing wood-framed mezzanine to existing reinforced load-bearing walls

Drawings of the proposed retrofits were included.

Proposed Solution

The seismic upgrade of the Frank Hurt Secondary School Gymnasium was carried-out in the Summer of 2006, which meant it used the 1st Edition Bridging Guidelines. However for the purposes of this demonstration project, the calculations will be given using the 2nd Edition Guidelines. The following is a list of issues that had to be addressed:

- Roof Diaphragm
    - Connections between steel deck and OWSJ uncertain
    - Diaphragm capacity low
    - Chord not connected to diaphragm
- East, West and South Lateral Systems (Braced bays)
    - Connections of braces are inadequate
    - Braces below HSS girt have very poor connections
- North Wall (Lightly Reinforced HCB Stack Bond)
    - Insufficient capacity because not reinforced horizontally
    - Wall not connected to roof diaphragm (major out-of-plane hazard)
- Foundation
    - Small footings under braced bays cannot resist any significant uplift

<u>Roof Diaphragm</u>

The existing roof diaphragm was classified as a Type B steel deck (Prototype D-4) because it had button punching for its sidelap fastener. The diaphragm was upgraded to have screw sidelap fasteners and additional screws were added to the frame, which met the requirements on a Type A steel deck (Prototype D-3). Calculations for the diaphragm capacity were done using the Hilti Steel Deck Program. See Figure 6 for the demand calculations. The diaphragm details are shown in Figure 7 and Figure 8.

Shear lugs were added, as the only transfer of load to the LDRSs was through the OWSJ. The connections between the shear lug and the diaphragm were calculated using Equation (11-1). Figure 9 shows the calculations for the connections. Note that this is not consistent with the details shown in Figure 10. This is because the diaphragm was designed using the 2005 NBCC (i.e. 1st Edition Bridging Guidelines did not have provisions for diaphragms or connections), and a bail-out force was used to determine the connection capacity. Under the 2005 NBCC, only 11 screws per shear lug were required. Similar connections are provided on walls in the East-West direction.

The existing horizontal steel bracing had some capacity in the rods, but the connections were inadequate. It was costly to remove the roof braces, so they were left in, but their contribution is ignored. Had the connections been suitable, it is possible that they diaphragm would only have needed to be upgraded with the shear lugs.

Braced Bays

The existing braces had sufficient capacity in the rods, but had connections that did not meet capacity design. The old braces were removed and replaced. There was a concern that high uplift forces would be a problem for the small footings. The retrofit on each wall was to include three braced bays side-by-side. Calculations for the braces are given in Figure 11. One bay of the existing braces was left, but not included in the resistance calculations.

The retrofit design of the braces is shown in Figure 12. Note that the demand for these braces is 11%W using 2nd Edition Bridging Guidelines, while the original design under the 1st Edition Bridging Guidelines had a demand of 23%W. This is why it appears that the rods are over designed.

Connection design calculations are given in Figure 13. Note that according to Section 11.4, these are to be designed in accordance to CSA-S16-01, which will allow "bail-out" forces on the connections, for our R_d=2.0 (limited ductility concentrically braced frame). However, to use the bail-out forces one must use the 2005 NBCC base shear demands, which in most cases will be higher than the capacity design requirements using the 2nd Edition Bridging Guidelines. The retrofit connection is shown in Figure 14.

The lower braced bay was left in place, but its capacity was not accounted for, as it did not meet capacity design requirements and would suffer a brittle failure at the connection. The wall finish and insulation were removed and new steel studs with "Sureboard" sheathing (one side) were added. "Sureboard" is a steel plate/drywall composite material. This system was designed to have a higher capacity than the overstrength capacity of the braced frames. Hold downs were included under the extreme ends of the braced bays. This system was connected to the foundation with Hilti anchors. An elevation of the "Surboard" system is shown in Figure 15, and a section is shown in Figure 16. The use of this system prevented high uplift forces which would have required a costly upgrade of the foundation.

Masonry Walls

The existing masonry walls had adequate connections to the foundation, and some vertical reinforcement. However, because it was stack bond, it did not have adequate shear capacity, and it was not connected at all to the roof diaphragm.

Horizontal strips of FRP were added to develop shear resistance, in lieu of cutting in horizontal reinforcement. The in-plane design is shown in Figure 17, and calculations are shown in Figure 18. Note that the masonry wall has an ISDL of 1.5%, while the steel braced frame in the same direction has an ISDL of 4%. This is permitted because the LDRS are more than 5m apart, and the diaphragm is flexible (Section 10.6(4)).

Connecting the masonry wall the roof diaphragm ensured that the lateral load path was complete, and also to enhance the out-of-plane performance of the wall (it was reinforced but acted as a cantilever). This connection was made up of HSS strong backs bolted to the top of the wall, and to four separate cells below the top of the wall. See Figure 19 for calculations on the connection to the diaphragm, and the out-of-plane restraint. See Figure 20 for details of the connections.

Cost-Benefit Analyses

The total cost of this retrofit was $295,000, for all the work done below the roof membrane. The re-roofing and exterior diaphragm work will be done at a later date.

This project was done using the 1st Edition Bridging Guidelines which had significantly higher demands for the steel braced frames and no guidance on diaphragms or stack bond walls. As such, this project did not see significant cost savings compared to the 2005 NBCC with an I=1.0.

Had this project taken advantage of the 2nd Edition Bridging Guidelines, potential saves could have been made in the diaphragm and steel braced frames. It is likely that only the perimeter elements of the diaphragm would have needed extra fastners to enhance their existing strength. The number of braced bays would have been reduced to 2 from the current 3. The probable cost savings would have been in the order of 15%.

Conclusions and Recommendations

The major cost savings of this retrofit was in avoiding any changes to the existing foundation, and leaving as much of the interior finish untouched as possible. The large existing wind girt provided a convenient location to connect the two different systems (i.e. steel braced frame to steel stud "Sureboard" shearwall).

Unfortunately in this building very little of the existing material were useful in the retrofit. The one LDRS that was kept (i.e. stack bond wall) has no real capacity until it has been reinforced vertically and horizontally.

This retrofit used the 1st Edition Bridging Guidelines, which had more conservative values for the steel braced frames and had no guidance for diaphragms. Had this retrofit been done with the 2nd Edition Guidelines, additional savings might have been found in the braced frames and diaphragm.

The diaphragm will be upgraded in the near future. The connections to the diaphragm will be reassessed using the 2nd Edition Bridging Guidelines, and more potential savings (compared to 2005 NBCC) are possible.

Appendix

Figures

Figure 1 Exterior of Gymnasium

Figure 2 Roof Connection Details

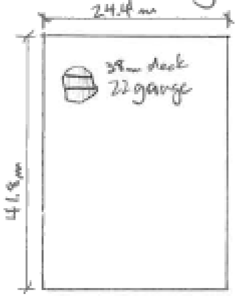
Figure 3 Interior View of Braced Bay

Figure 4 Existing Braced Bay Connection

Figure 5 Top of Existing Masonry Wall

Calculations for BG2 Demonstration Project

Roof Diaphragm - Steel Deck Type B (Poor)



To Frame Screws or Nails into wood 3x6"
nailer 36/7

Side lap - Button Punching (8 per span)

Resistance = 3.3 kN/m (Hilti Program)
Wd = 1121 kN

Site Class C, Seismic Zone # Prototype D-4

42 m → 59.5% Wd = 667 kN → 27.3 kN/m NG
24 m → 50% Wd = 560 kN → 13.4 kN/m NG

Upgrade to Type A Deck Prototype D-3

42 m → 16.5% Wd = 184 kN → 7.6 kN/m
24 m → 20% Wd = 224 kN → 5.4 kN/m

Upgraded Deck

To Frame - Screws 36/9
Side lap - Screws 13 per Span

wf = 12.9 kN/m OK

Chord Forces (Equation 10-2)

$$\text{Span - } 42\,m = \frac{R_{md}\, L_d\, W_d}{400 s} = \frac{16.5 \cdot 42 \cdot 1121}{400 \cdot 24} = 80.9\ kN$$

$$\text{Span - } 24\,m = \quad = \frac{20 \cdot 24 \cdot 1121}{400 \cdot 42} = 32\ kN$$

Chord is W310×39 → Tr is OK

Figure 6 Diaphragm Calculations

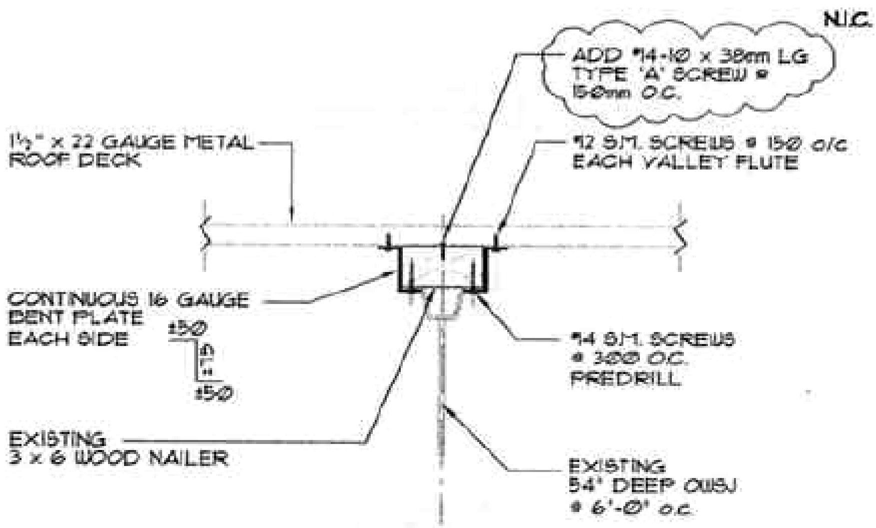


Figure 7 Deck to Frame Connection

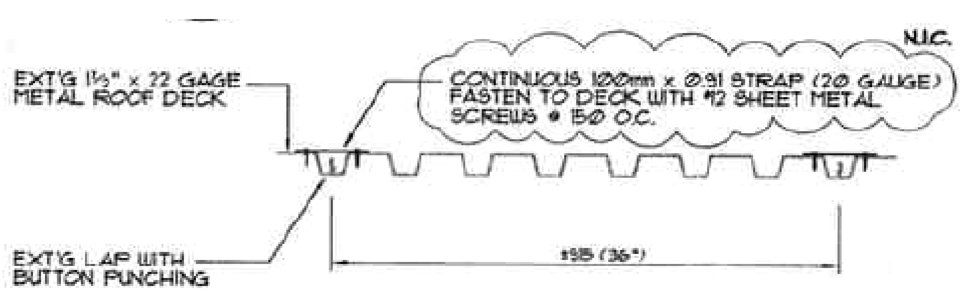


Figure 8 Deck to Deck Sidelap Connection

Connection to Frame

Equation (11-1) $R_{mc} = \frac{R_{ed}}{n_c}$

$n_c R_{mc} = 12.9$ kN/m

N-s dir #14 Screws ~ Shear lug every 1.83m

$R_{mc} = 2.2$ kN/screw

$n_c = \frac{12.9 \times 1.83}{2.2} = 11$ screws

Figure 9 Diaphragm Connection to Shear Lug Calculations

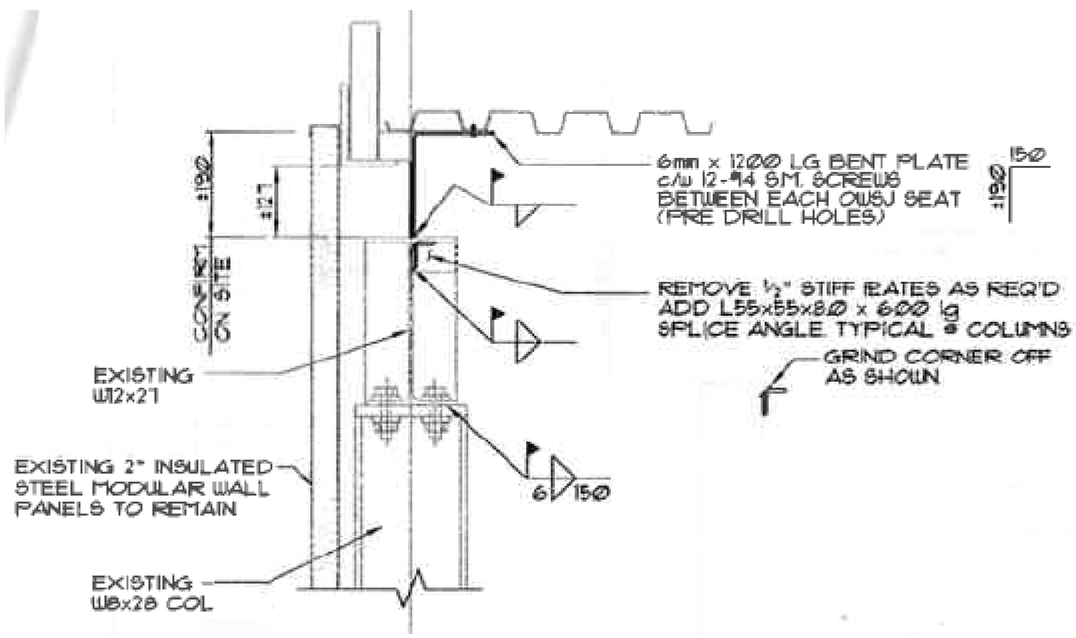
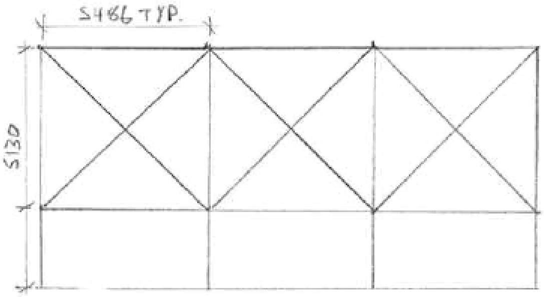


Figure 10 Diaphragm Chord, Shear Lug and Diaphragm Connection

Braced Bays - Concentrically Braced Steel Frame
(Tension Only - Prototype S-1)

RetroFit



Seismic Weight = 870 kN

Site Class C
Seismic Zone 4

Prototype S-1

Figure 4-1(c) Governing Drift Limit = 4%

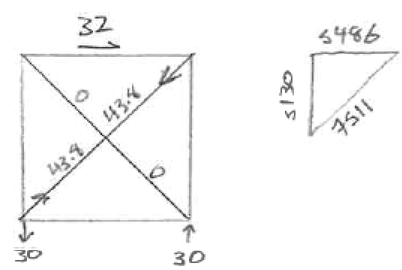
$R_m = 13\%\ W$

Height > 3.0 m Equation 4-1 $K_{mr} = 1.45 - \frac{4}{6.67} = 0.85$ Max

$R_{mr} = 0.85 \cdot 0.13 = 11\%\ W$

$R_e \geq 95.7\ kN$

Look @ 1 bay $\frac{95.7}{3} = 32\ kN$



Use 22 mm ϕ Rods Area = 380 mm² $A_n = 298\ mm^2$

Full
$T_r = 0.9\ A_g F_y = 0.9 \cdot 380 \cdot 300 = 102.6\ kN$ OK

Threaded
$T_r = 0.9 A_n F_u = 0.9 \cdot 298 \cdot 450 = 120.7\ kN$ OK

Figure 11 Brace Calculations

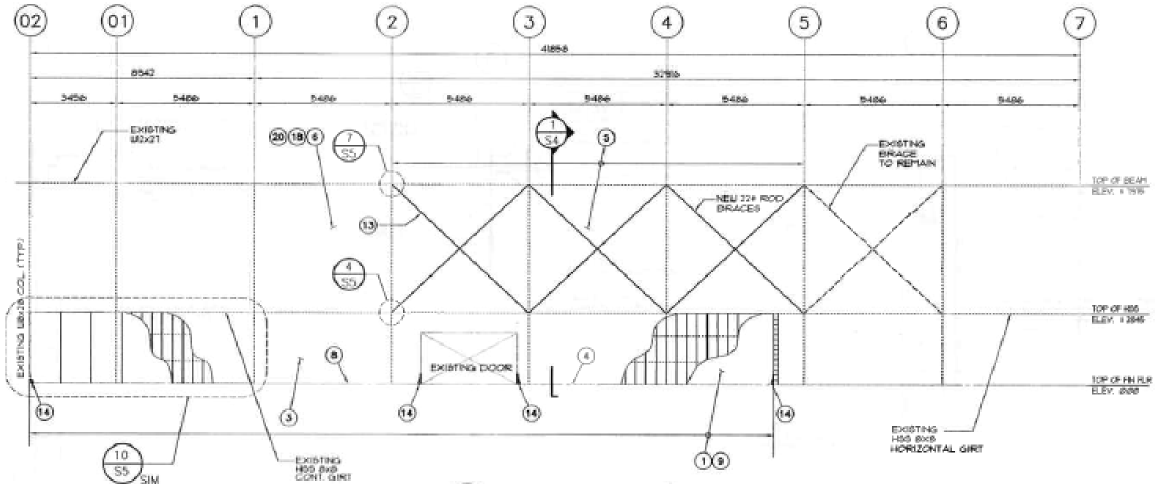


Figure 12 Typical Steel Braced Frames

Braced Frame Connection Design. Forces

Capacity Design

$R_y F_y A_g$

= 385 · 380

= 146.3 kN

Bail out

→ Requires Code Forces

~25% W × Rd ← too high

= 199 kN

Design all Connections to ≥ 146.3 kN.

Figure 13 Connection Design Forces

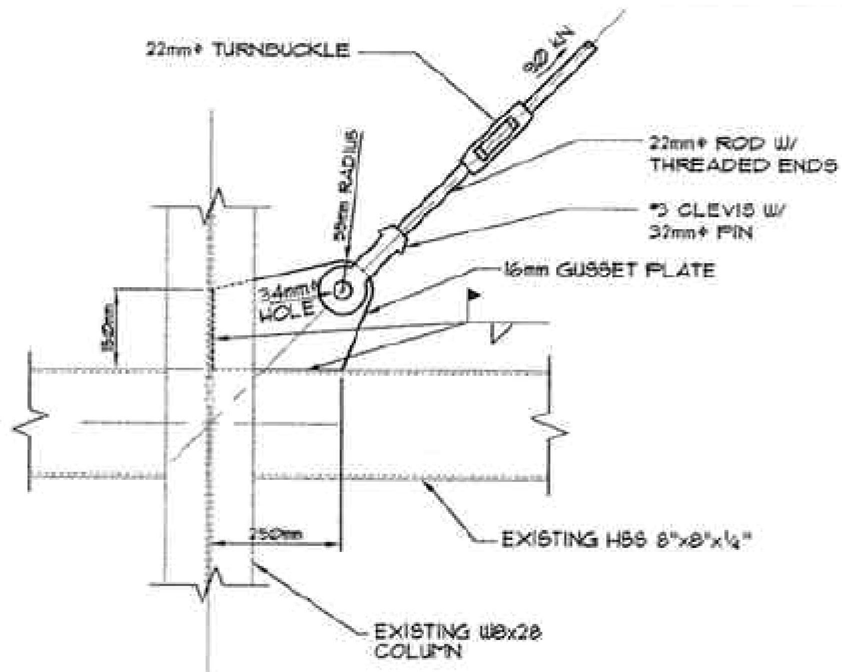


Figure 14 Brace Connection Details

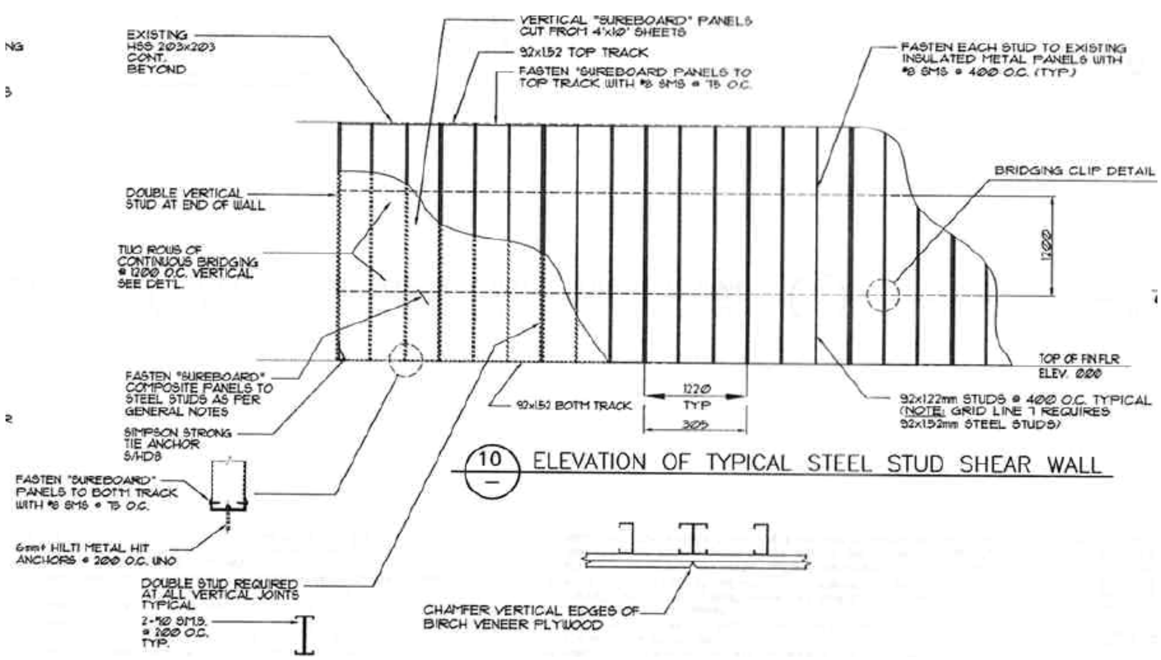


Figure 15 Elevation of "Sureboard" Wall

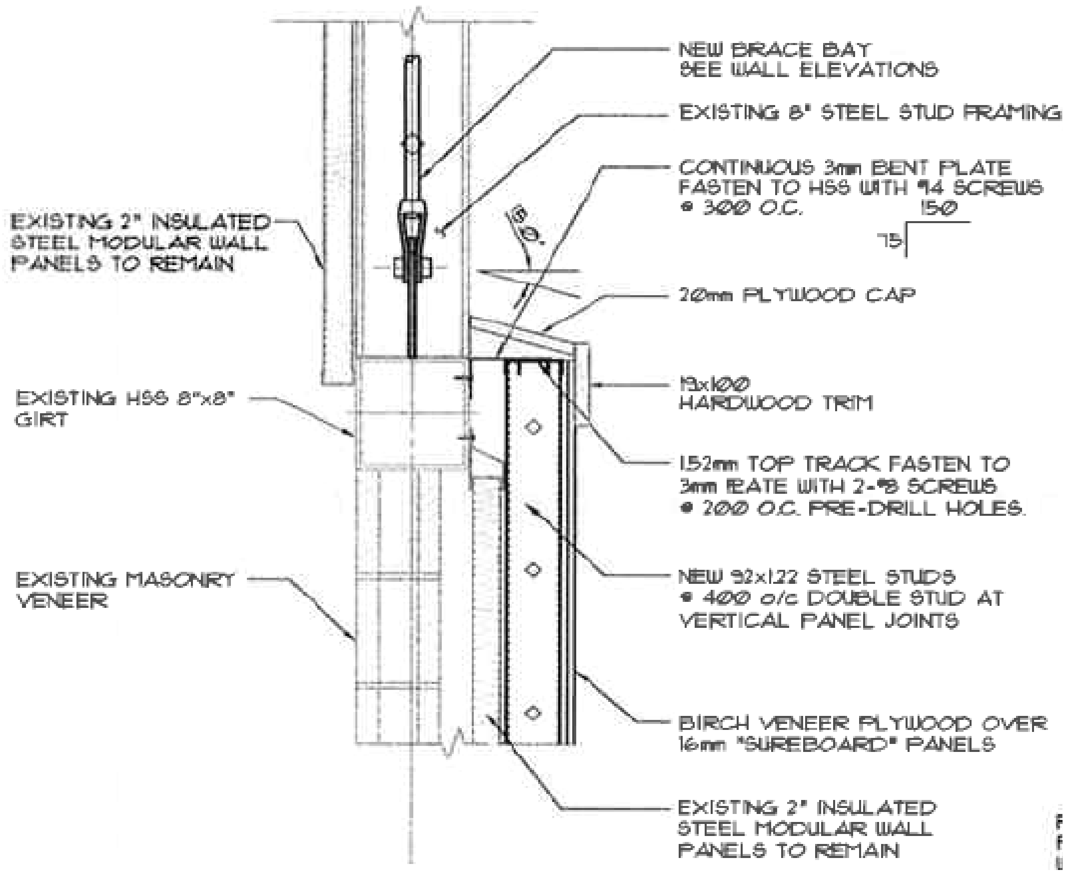


Figure 16 Section of "Sureboard" Wall

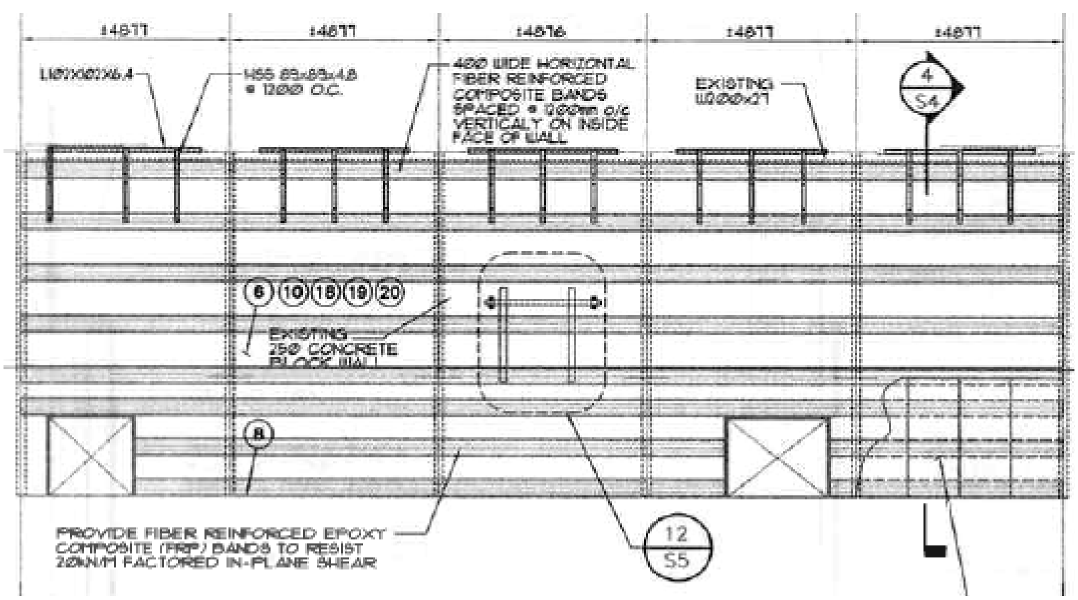


Figure 17 Masonry Wall Retrofit

Masonry Wall - Stack Bond, Light Vertical Reinforcement
- Will add Horizontal FRP Strips in lieu of Horizontal reinforcement

Prototype M-2   Site Class C   Zone 4   ISDL = 1.5%

$R_m = 21\% W \quad W = 1400\,kN$

$h_{sc} > 4m \quad K_{mr} = 1.45 - \frac{4}{6.67} = 0.85$

$R_{mr} = 17.9\% W \quad R_{mr} = 250\,kN$

Wall length minus openings 19.5 m

Assuming Stack Bond provides no shear resistance

Shear required $v_r = \frac{250}{19.5} = 12.8\,kN/m$

Using FRP ~ Elastic material use a factor of 1.5

$v_r = 12.8\,kN/m \times 1.5 \simeq 20\,kN/m$

Figure 18 Masonry Wall In-plane Calculations

Connection to Diaphragm

$R_{mc} = R_{ed} \div n_c$ $\quad$ $R_{ed} = 12.9\ kN/m \times 24.3$

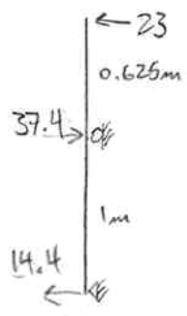
$n_c = 45$

$= \dfrac{313.5}{45}$

$= 7\ kN/bolt$

Hilti HAS Rod 16mm φ $\quad V_r = 23.3\ kN/bolt$ OK

HSS Strong backs (Out-of-plane Restraint)



$\frac{1}{2}Wb / 15 = 345\ kN / 16 = 23\ kN$

Max Tension on connections is 36 kN (2) 3/4" HAS 200mm embedment

$T_r = 54\ kN$ OK

Max Moment on HSS = 15 kN-m

HSS 89x89 -4.8 ~ $M_r = 15.2\ kN\text{-}m$ OK

Figure 19 Masonry Wall Diaphragm and Out-of-plane Connections

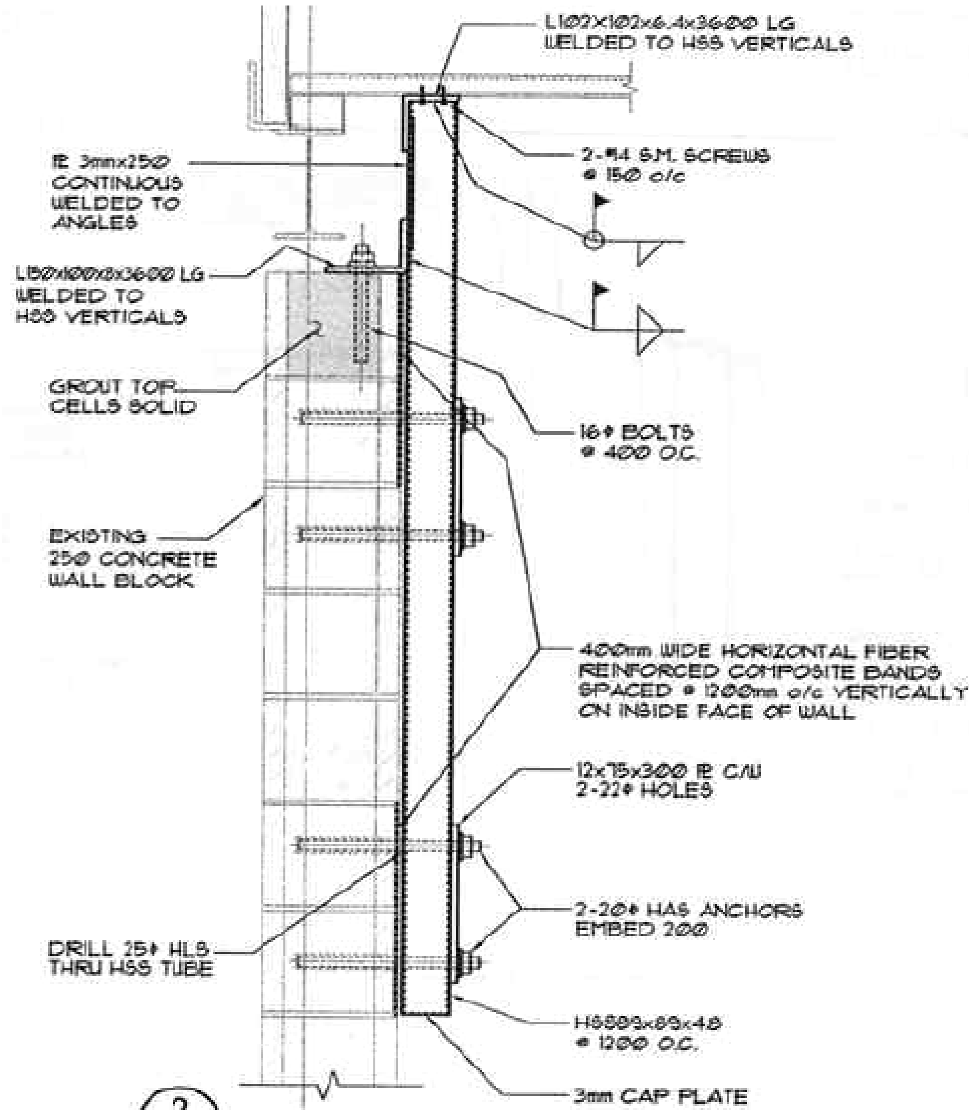


Figure 20 Masonry Wall Out-of-plane Retrofit

Bridging Guidelines
Second Edition

Demonstration Project No. 3

for

SEISMIC RETROFIT OF 3-STOREY CONCRETE CLASSROOM FOR TRAFALGAR ELEMENTARY SCHOOL

(School District No. 39 – Vancouver School Board)

Prepared by:
Read Jones Christoffersen Ltd.

March, 2007

1.0 EXECUTIVE SUMMARY

The three storey concrete classroom block of Trafalgar School is used as a template for a Demonstration project illustrating the use of new and existing concrete walls in an assessment and upgrade using Version 2 of the bridging guidelines.

It is seen that upgrade force levels and requirements for Version 2 of the bridging guidelines are lower than for Version I and NBCC 2005 when a 2% drift limit is chosen. This is an advantage and will result in cost reductions.

Concrete walls exist in the E-W direction of the school and using the toolbox approach. They are used to resist about 40% of the load in this direction.

A new "Moderately Ductile" tied-down concrete wall is introduced in the east-west direction and combined with the existing Conventional Construction concrete walls, which are governed by rocking. This is allowed in the guidelines but is not allowed in NBC 2005. This is a clear advantage and results in cost savings.

A 2% drift level is chosen for assessment, which results in low design force requirements. However, the columns are at risk at this drift level and steel prop columns will probably need to be added.

Since this is a Demonstration project, detail solutions are not presented. However, the following are pointed out as issues to be considered in a final design.

- Forces for the diaphragm are developed and diaphragm load path weaknesses identified.
- Foundation sliding is identified as a concern and solutions are identified.
- Alternate solutions and systems which would usually be investigated as part of a complete assessment are discussed.

2.0 PROJECT DESCRIPTION – TRAFALGAR SCHOOL ADDITION

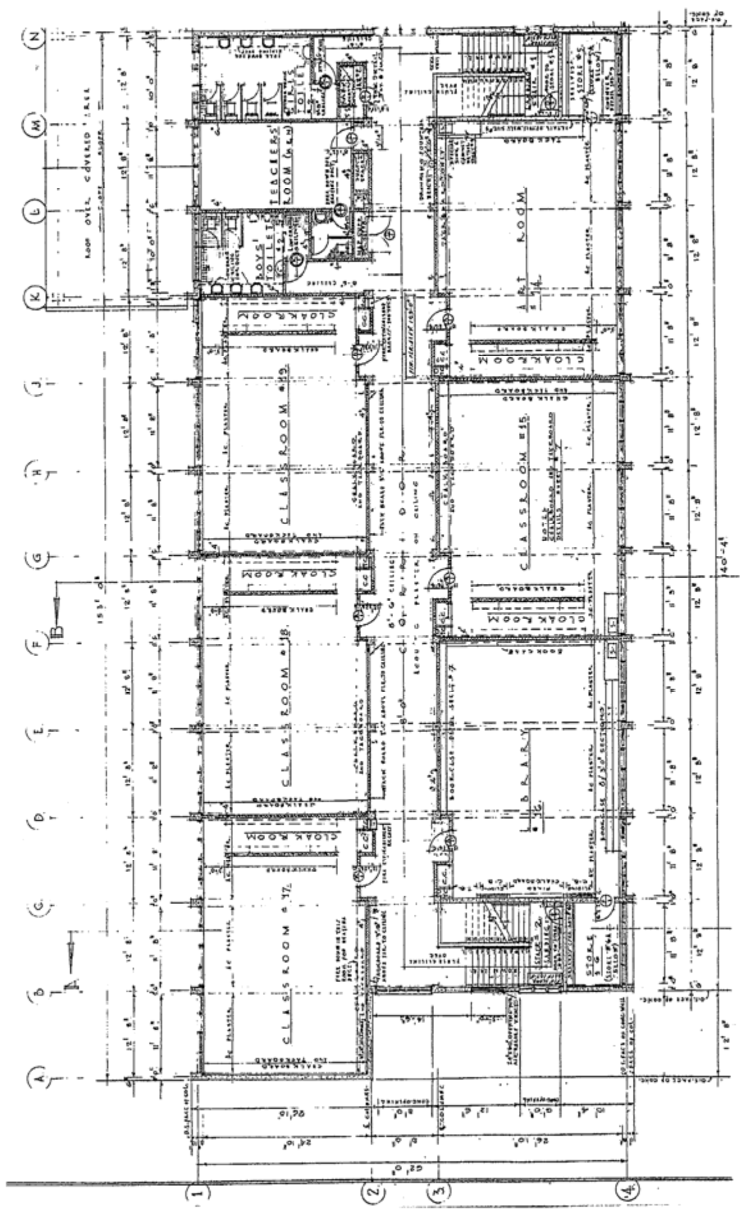
The project is a 3 storey concrete building built in 1950. It consists of:

- Roof slab (typically 4") spanning about 12 ft to 12" x 20" beams which span from the exterior to columns at each side of the corridor.
- Two floor slabs (4-1/2" typically) with 1" of topping spanning to 12" x 23" beams.
- 8" walls across the two narrow ends with 8" x 26 ft long parallel walls one bay over (these walls run East-West).
- One very small 8" wall at the North end running about 12 ft North-South.
- The walls are full height.
- In the lowest floor there are some 1 storey 8" walls around a mechanical room.
- The interior corridor columns are 12" x 12" and the edge columns are 12" x 18", all at 12' o.c.
- The edge columns are "short" columns and are restrained by a 3 ft deep spandrel.
- Floor heights are 10 ft in the lower level and 12 ft after that. The footings are about 2 ft below the slab-on-grade, making the lower structural height about 12'.
- All interior partitions are 4" clay tile.
- Footings are typically 4 ft x 4 ft at columns and 1 ft wide strip footings at the walls.
- Material Properties:
  - Soil 8 ksf allowable.
  - Concrete – f_c= 2,500 psi
  - Reinforcing Bars – f_y = 40,000 psi
- The floor/roof area is about 8,840 sq ft per suspended slab.
- The roof weight (including snow) is about 1,126 kips.
- The two floor weights are about 1,460 kips each.
- The total weight at the lower level is about 4,046 kips.
- Eccentricity – the building does not satisfy the requirements of the guidelines if a wall line in the middle is proposed for the north-south direction as an upgrade as eccentricity is defined as a % of the width, and width is defined as the distance between the outermost lateral resisting elements. For a single wall line this has no meaning, and therefore the building would be outside the guidelines.

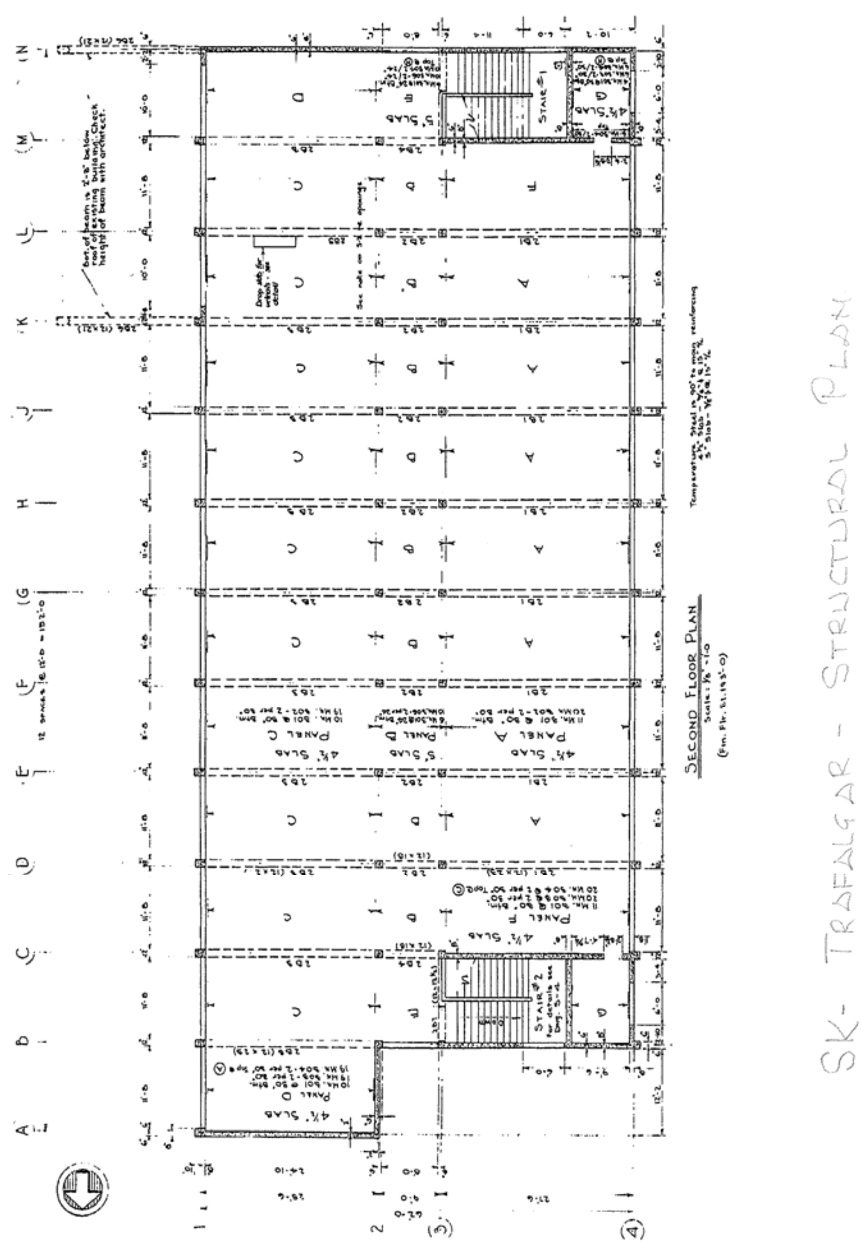
  However-judgement – This is a reasonably symmetric building with well distributed walls in the other direction so assume the guidelines apply (i.e., They are deemed to apply.).

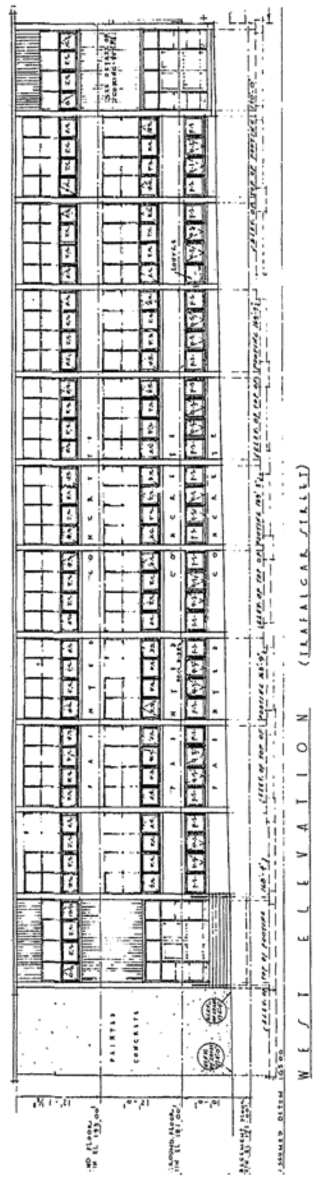
  If this is a problem, place similar systems on the East and West exterior walls to resist the North-South loads, and the building will satisfy the guidelines.

- See attached sketches, architectural plan, structural plan, and architectural elevations.



SK- TRAFALGAR - ARCHITECTURAL PLAN.


SECOND FLOOR PLAN
SK- TRAFALGAR - STRUCTURAL PLAN



SK - TRAFALGAR - ARCHITECTURAL ELEVATION

3.0 SEISMIC ASSESSMENTS

Previous work on the school has rated the buildings as moderate-high to high risk.

This was the conclusion of the 2004 work and the Phase I and Phase II assessment.

The estimated costs for upgrades were less than 70% of the replacement costs, so the schools are candidates for upgrading.

Estimated costs for upgrading the three storey concrete classroom block are:

2004 - $2,613,400.00
2006 – Phase II $2,699,730.00

The “structural only” portion of the estimate is about 50% of the total costs.

4.0 PROPOSED SOLUTION

4.1 This is a Demonstration Project and is meant to:

- Illustrate use of the second edition of the guidelines (Version 2-November 2006). Any changes since November are not part of the Demonstration Project.
- Discuss systems and why they were chosen.
- Discuss alternates.
- The concrete portion of Trafalgar is used as a template for illustration only. This is not intended as a design for this school.

4.2 Demonstration Project Solution

The Solution Proposed is developed in some detail, sufficient to illustrate the guidelines and to develop the concepts. However, several approximations have been made with some simplifying assumptions. This should help keep the explanations clear and leave the inevitable clutter (but important clutter!) of details to the final design. For instance, the mechanical room walls in the lower level are ignored, which will probably the case for the final design as well.

It is assumed that users of the guidelines are familiar with dead load calculations and detailed design of steel and concrete systems.

4.3 Initial Observations based on Visual Examination of the Drawings.

It seems fairly clear that:

- The building is a heavy “Conventional Construction” concrete building.
- Most of the dead load is coming down some place other than the walls, and the walls have very small footings.

The walls will probably rock for E-W loading.

- There is virtually no resistance from the short wall for the N-S direction.
- The short perimeter columns will probably be an issue because of drift.
- The clay tile partition walls will need to be removed or encapsulated, with removal being the preferred method as stated in the guideline. This will be a cost decision, but removal will free up the space for new walls (and reduce the building weight by about 10%).

4.4 Proposed Systems

All references such as S1, S2, C1, M2, etc., are the appropriate figures and systems in Sections 3 to 8 of the guidelines – Version 2.

The building is in Vancouver on good ground, so it is a Zone 4, Class C site.

Potential upgrade systems are concrete, steel, or masonry. Wood is not going to be appropriate for this building simply because (even though the percent of weight "strength requirements" are small) the shear resistance value per foot are too small to be practical for this building. However, this will be looked at later as a point of interest with some preliminary details given in Sketch 2 – Appendix.

There are several factors that are used to adjust base shears for floor to floor heights, base moments, and force distributions to calculate base moments. These will be developed first as they are necessary for the analysis.

They are given in Table 1.

Note: The shear and moment relationship is developed from the force distribution given in the guidelines (Equation 1-1) and not the base moment equation given in (1-2). Equation (1-2) is very conservative. While this may be appropriate for "shear" mechanism systems such as wood shear walls or braced frames, it is too conservative for flexural systems such as concrete walls in flexure or in rocking mode. Basically, Equation (1-2) is intended to give a minimum base moment equal to the base shear times the total height. When actual upper story strengths are used, as defined in (1-2), the base moment can be greater than the base shear times the height. This is quite conservative.

TABLE 1

1. Floor to floor height correction for Figures in Sections 3 to 7 (Note: Not Section 8)

Steel:	1.45 – h/6.67 = 1.45 – 3.6/6.67 = 0.91
Concrete:	1.45 – h/6.67 = 0.91
Masonry:	1.45 – h/6.67 = 0.91

2. Base Moment Reduction Factors

Steel	1.0 for 1 through 3 stories.
Concrete	0.9 for 2 or 3 stories.
Masonry	1.0 for 1 through 3 stories.

3. Base Moment from Force Distribution

$$F_x = \frac{R\,(W)\,(W_X)\,.\,h^2_X}{100 \sum W_i\,.\,h_i} \quad \text{(Equation 1-1)}$$

$M_{Base} = \sum F_i\,.\,h_i$

$(RW/100) = V_{Base} = V_B$

For this structure:

F (roof) = 20.9 V_B (70% of V_B)

F (2nd) = 7.9 V_B

V(1st) = 1.3 V_B

M_B = 29.8 V_B without moment factors, and using 12 ft = 10 ft + 2 ft for ground to first floor to get to bottom of footing.

4. Rocking Wall/Footing Factors

1.33 – H/9

Where H is height to centre of mass, and is 6 m maxium

For our 3 storey building, H is about 2 x 3.6 m = 7.2 m

Therefore, 1.33 – 6/9 = 0.67

There is also a height to length ratio factor which determines what curve to use.

R – 1 Curves – Maximum aspect ratio = 1.0

R – 2 Curves – Maximum aspect ratio = 2.5

Our walls are about 36 ft high/26 ft long = 1.4

Average of R – 1 and R – 2 curves is (1.0 + 2.5)/2 = 1.75

Examination of R – 1 and R – 2 in Zone 4 indicates the R – 2 curve is the highest demand of R – 1 and R – 2 curve.

Therefore, for our case use average, which should be slightly conservative but easier and quicker to calculate.

4.5 Comparing Shear Demands

The values in Table 2 are developed from comparing shear demands for various systems from Figures in 4, 5, 6 and 8, for Zone 4, Class C and using 0.91 factors from above and 0.67 for rocking walls. The rocking walls are the average of R-1 and R-2 rocking walls. All values are from the appropriate figures and tables for 1% and 2% drift.

TABLE 2			
	1%	2%	NBC 2005, I = 1.0
S1 (Tension Only)	.46 (.91) = .42	.25 (.91) = .23	0.25
S2 (T/C Brace)	.38 (.91) = .35	.21 (.91) = .19	0.25
S3 (ECC BRC)	.38 (.91) = .35	.21 (.91) = .19	0.11
S4 (FRM)	.49 (.91) = .45	.35 (.91) = .17	-
C1 (MDWALL)	.28 (.91) = .25	.19 (.91) = .17	0.23
C2 (CCWALL)	.3 (.91) = .27	NP	0.32
C4 (MDFRM)	No Value Given	.26 (.91) = .24	0.18
M2 (RNFWALL)	.26 (.91) = .24	N.P.	0.28
ROCKING	((.51 + .81)/2).67 =.44	((.22 + .38)/2) .67 = 0.2	0.32

Comments on Table 2

- 2% drift requires significantly lower strength.
- 1% drift – probably a problem for short exterior columns anyway.
- Start with 2% drift
  - "Practical" solutions – (judgement!) with low demand are S1, S2, C1, and rocking.
  - Add "prop" steel columns at all columns (review later at detail design stage)
- A conventional construction wall is not allowed at 2% for shear and moment but it is allowed to rock.

Therefore, start with 2% drift, rocking, and Moderately Ductile Walls for any upgrades.

4.6 Complications with Walls

Walls have several "resistance" values:

V_{RSH} = The demand is given in Figures C1, C2, M2 – Lateral load capacity or resistance based on Vc and shear horizontal reinforcement.

V_{RFL} = The demand is given in Figures C1, C2, M2 – Lateral shear resistance based on flexural capacity of vertical reinforcement only – gravity load = zero.

V_{RRF} = The demand is given in Figures R1, R2, R3 - Lateral resistance based on rocking at underside of footing, with moment resistance due to gravity loads including footing weight.

V_{RRW} = The demand is given in Figures R1, R2, R3 - Lateral load resistance of rocking wall above footing with rocking moment resistance due to gravity loads only, excluding footing weight, and with no flexural steel.

V_{RSL} = No figures – Lateral resistance due to sliding and resisted by gravity load only times friction on the soil. (Typically, friction factor is 0.5.)

Therefore – Lateral resistance of wall is the lesser of:

.1 V_{RSH} (Shear)

.2 V_{RRF} (Rocking at underside of footing)

.3 V_{RFL} + V_{RRW} (Flexural capacity due to vertical reinforcement only plus wall rocking above footing).

These resistances are not additive. However, while the “shear” case must always be examined, that is not the approach for the “footing rocking” case and the “wall flexure/rocking” case. The wall will either rock on its footing or the footing will be strong enough to yield the wall in flexure above the footing. It is one or the other, and the least horizontal shear resistance tells us which one of these two cases it is.

Therefore, when combining the ratios of capacity/demand for a wall (or walls) the toolbox method must be done twice – once for the shear case ratio and once for the governing case of footing rocking or walls flexure above the footing.

Since there is no set of figures for V_{RSL}, it must be made to be greater than the governing case.

Clearly this makes the analysis and upgrading for walls a bit tricky.

In summary, the toolbox method is done twice – once using the ratio in .1 below and once using .2 or .3, based on the governing case determined previously.

.1 V_{RSH}/Appropriate values from Figures C1, C2, M2, etc.

.2 V_{RRF}/Appropriate value from Figures R1, R2, R3.

.3 V_{RFL}/Appropriate value from Figures C1, C2, M2, etc. + V_{RRW}/Appropriate values from R1, R2, R3.

If the upgrade chosen ties the footing down, or adds tension zones to the walls, then the governing case may change and the ratios need to be re-checked. For instance, upgrading a footing with tie-downs may force a flexure failure in the wall above the footing and the wall may not have sufficient capacity to make the toolbox approach work.

All the Figures (C, M, R) are given in terms of horizontal shear – therefore any resistances based on rocking moment resistance or vertical steel moment resistance must have their shear values determined from the moment/shear relationship developed from the force distribution relationship (Formulae 1-1 in the guidelines) and in Table 1 of this document.

4.7 Calculate the Wall Demand

.1 Shear demand from C1 – Zone 4, Site Class C – at 2% (See Table 2)
V = 0.19W x .91 = 0.17 W
= 0.17 (4,046 Kips) = 687 Kips (for moderately ductile wall)

.2 Rocking footing demand – R1, R2 curves, Zone 4, Site Class C, 2% - Averaged from R1, R2. (See Table 2)
V = .2W = .2(4,046) = 809 Kips

.3 There is no need to check the "flexural steel capacity plus rocking wall" combination as there is no way to develop the wall vertical reinforcement. Since the footings are very small, this case simply becomes case (.2) above.

4.8 Wall Capacities

For the N-S direction the capacity is basically zero.

For the E-W direction, it turns out the wall capacities are governed by rocking. The capacities are determined by:

- Calculate the dead loads on the wall.
- Calculate the bearing area and the centroid of resistance.
- Calculate the moment resistance for the walls in each direction of loading along the wall.
- Use the shear/moment relationship to calculate the shear capacity for the walls when rocking.
- Add the resistances up. One direction will typically be less than the other.
- Compare to the demand.
- Check that the walls do not slide.

The calculation for rocking of a typical wall is shown in Sketch 1, along with a shear calculation showing shear does not govern.

For this wall, sliding and shear strength are clearly not a problem, and this turns out to be the case for all the walls.

Summing up the governing direction in the East-West direction gives a capacity of about 296 kips. The weight is about 4,046 kips. Therefore:

E – W Rocking Capacity = 296K/4046 = 0.073
Assessment Demand = 0.2(.80) at 2% = 0.16

The walls are good for 0.073/0.16 = 0.46 of the assessment value.

Therefore, the structure must be upgraded in the East-West direction and the existing walls have about 0.073/0.2 = 0.37 of the required “upgrade” base shear.

4.9 Picking an Upgrade System – E–W

Reviewing Table 2, Rocking or a Moderately Ductile Shear Wall seem to be good candidates, along with Steel Tension Compression bracing.

However, while Rocking has low “required” values, work to date indicates that the existing walls are rocking and good for about 40% of the required capacity.

This would require adding about 2.5 times the number of existing walls to get 100% resistance. This would effectively fill the building with walls. This would have minimal foundation cost, but seems impractical and still quit costly.

We will start with the Moderately Ductile Concrete Wall scheme for the following reasons:

- The added wall weight will help resist overturning (structural steel bracing is light).
- Connections of concrete to concrete are easier to make than steel to concrete.
- Tension zones will be easier to do in the concrete scheme.(?)

However, the steel scheme is still appealing and should be developed for costing.

A summary of calculations for the Moderately Ductile Wall scheme are given in Sketch 2.

A brief discussion of using wood walls and 4% drift is given in an Appendix to Sketch 2.

WALL AT STAIRSHAFT – INSIDE BUILDING

– STRIP FOOTING – 1' WIDE
– PADS – 2' x 2'

$P_1 = 16.2^K$

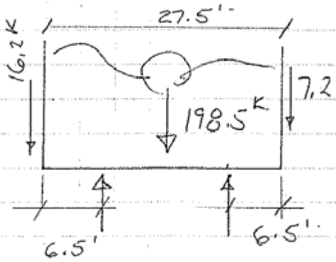
27.5'

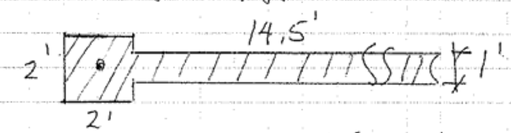
P_2 = WALL + FLOOR
$= 198.5^K$

P_3 = SPANDRELS $= 7.2^K$

$\Sigma P_{DL} = 221.9^K$ $\quad A_{FTG} = \dfrac{221.9^K}{1.5 \times 8\,ksf} = 18.5\,ft^2$

∴ LENGTH OF WALL / FTG
$= 18.5\,ft^2 - 4\,ft^2 = 14.5'$





REACTION AT $\dfrac{14.5'\left(\frac{14.5'}{2}+1\right)}{18.5}$
$= 6.5'$ TO CENTRE OF SOIL BLOCK

RESISTANCE (MOMENT)

⟶ $16.2^K(27.5'-6.5') + 198.5\left(\frac{27.5'}{2}-6.5'\right) - 7.2^K(6.5')$
$= 340^{K\text{-}FT} + 1439^{K\text{-}FT} - 46.8^{K\text{-}FT} = 1732^{K\text{-}FT}$

⟵ $7.2^K(27.5-6.5) + 198.5\left(\frac{27.5}{2}-6.5\right) - 16.2\,(6.5')$
$= 151^{K\text{-}FT} + 1439^{K\text{-}FT} - 105^{K\text{-}FT} = 1485^{K\text{-}FT}$

RESISTANCE (SHEAR)

⟶ $1732^{K\text{-}FT} / (29.8\,(0.91)) = 63^{KIPS}$

⟵ $1485^{K\text{-}FT} / (29.8\,(0.91)) = 54.7^{KIPS}$

v_f IN WALL
$\cong \dfrac{63^K}{.8 \times 27.5' \times 12'' \times 8''}$
$= 30$ psi
CLEARLY OK ✓

SLIDING RESISTANCE $\cong 221.9^K \times 0.5 = 110.95^K$
CLEARLY OK.

SAMPLE CALCULATION OF ROCKING
RESISTANCE FOR WALL AND FOOTING

SKETCH 1.
1 OF 2

SUSPECT WALLS GOVERNED BY ROCKING –
QUICK CHECK TO CONFIRM SHEAR NOT AN ISSUE

$f'_c = 2500$ psi $\quad v_c = .6\,(.2)\sqrt{17} = 0.495\,\text{MPa}$
$= .70$ psi ±

HORIZONTAL REINFORCING IN 8" WALLS – #3@6"

∴ – PER FOOT OF WALL (8")

$\frac{V_f}{ft} = V_c + V_s \cong 8" \times 12" \times .07\,\text{ksi} = 6.7^K/\text{FT}$

$+ \ .85\,(40\,\text{ksi})(.11\,\text{in}^2)\left(\frac{12"}{6"}\right) = 7.5^K/\text{FT}$

$= 14.2^K/\text{FT}$

DEMAND ≅ 687 KIPS (MODERATELY DUCTILE WALL)

RESISTANCE ≅ 6 WALLS @ 26' ±

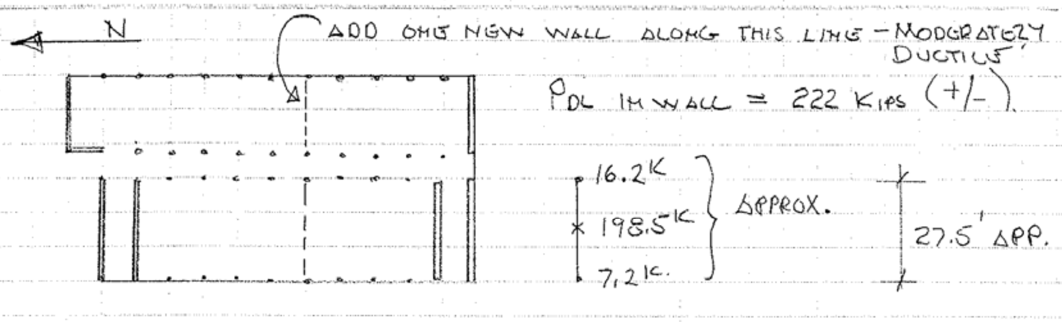
$= 6 \times 26' \times 0.8 \times 14.2^K/\text{FT} = 1772$ KIPS

CLEARLY – RATIO $= \frac{1772}{687} = 2.6 > 1.0$ – OK ✓

FOR 1% – CONVENTIONAL CONSTRUCTION (TABLE 2)
- $V = 0.27\,(W) = 0.27\,(4046^K) = 1092^K < 1772^K$
– STILL OK!

SAMPLE CALCULATION OF ROCKING
RESISTANCE – SHEAR CHECK

SKETCH 1
2 OF 2



APPROACH –

- DETERMINE SHEAR CAPACITY REQUIRED (CI, TABLE 2)
- DETERMINE SHEAR AND MOMENT CAPACITY (CI, ROCKING, TABLE 2)
- COMPARE THE MOMENT/FLEXURE DEMAND IMPLIED BY THE TWO ABOVE APPROACHES
- USE THE RELATIONSHIPS (SHEAR/MOMENT), FACTORS, ETC. IN TABLE 1.
- CHECK DIAPHRAGM, TIE DOWNS, SLIDING ETC. USING OVERSTRENGTHS CALCULATED FROM ACTUAL SECTIONS DESIGNED. SIGNIFICANT YIELDING IMPLIED BY GUIDELINES, EVEN IF WALL IS MODERATELY DUCTILE – ∴ USE $f_y = 1.25 f_y$ SPECIFIED.
- WALL ADDED MID-WAY (+/-) ALONG TO MINIMIZE ECCENTRICITY.

NOW – TWO CALCULATIONS REQUIRED FOR "SHEAR WALL"

– ONE IS FOR "SHEAR" STRENGTH – V_c, HORIZONTAL STEEL

– ONE IS FOR "HORIZONTAL FORCE" CAPACITY BASED ON "FLEXURAL RESISTANCE" CAPACITY – VERTICAL REINFORCEMENT AND DEAD/GRAVITY LOADS.

AND – $M_{BASE} = V_{BASE}(29.8)(.9)$ $W = 4040$ KIPS

SAMPLE CALCULATION FOR NEW EAST-WEST WALL.

SKETCH 2
1 OF 5

EXISTING WALLS – 0.073 W.
UPGRADE DEMAND – 0.2 W (ROCKING)

(1) "SHEAR" RESISTANCE NEEDED FROM NEW WALL.
FROM TABLE 2 - C1 – 0.17 W

FROM TOOLBOX APPROACH

$$\frac{0.073}{0.2} + \frac{x}{0.17} = 1, \quad x = 0.10795 \cong 0.11\,W$$

NEW WALL SHEAR CAPACITY (V_c, HORIZONTAL STEEL)
MUST BE AT LEAST 0.11(4040) = 444 KIPS = V

(2) HORIZONTAL CAPACITY FROM "FLEXURE", AND
WHERE FLEXURE IS THE SUM OF VERTICAL ONLY
REINFORCEMENT (DEALT WITH IN C1) AND
GRAVITY LOADS (DEALT WITH IN ROCKING)

NEED "ROCKING" MOMENT CAPACITY FROM GRAVITY LOADS.
FOOTINGS WILL BE DESIGNED TO TAKE FULL REACTION
AT WALL ENDS.

$$\therefore M_{DL} \cong 7.2\,(27.5') + 198.5\left(\frac{27.5'}{2}\right) = 198^{K\text{-}FT} + 2722^{K\text{-}FT}$$
$$= 2920^{K\text{-}FT}$$

$$V \Rightarrow V(29.8)(.9) = 2920^{K\text{-}FT}, \quad V = 108.9^{KIPS}$$

THIS 108.9 / 4040 = 0.027 W
FROM TOOLBOX

$$\underbrace{\frac{0.073}{0.2}}_{EXISTING} + \underbrace{\frac{0.027}{0.2}}_{ROCKING} + \underbrace{\frac{x}{0.17}}_{FLEXURE} = 1 \qquad x = 0.085$$
$$V = .085\,(4040) = 343^{K}$$

SAMPLE CALCULATION FOR NEW EAST-WEST WALL.

SKETCH 2
2 OF 5

NOTE THAT THE TOTAL SHEAR FOR (2) IS:

$V = 108.9^K + 343^K = 451.9^K$ FOR NEW WALL
"ROCKING OF DL" & FLEXURAL STEEL MOMENT RESISTANCE SHEAR.

COMPARE SHEARS AND MOMENTS FROM (1) & (2) FOR NEW WALL

(1) – SHEAR IS $\underline{\underline{444^K}}$ $Mom = 444(29.8)(.9) = \underline{\underline{11,908}}^{K\text{-}FT}$

(2) SHEAR IS 108.9^K $Mom = 2920^{K\text{-}FT}$ – GRAVITY/ROCK
$+\ \underline{343.^K}$ $+\ 343(29.8)(.9) = 9199^{K\text{-}FT}$
$\underline{\underline{451.9^K}}$ $\Sigma = \underline{\underline{12,119^{K\text{-}FT}}}$

BASICALLY – THE SAME. HOWEVER, THIS MAY NOT ALWAYS BE THE CASE IF ROCKING OF THE GRAVITY LOAD IS A LARGE COMPONENT OF THE RESISTANCE.

THERE IS NOW ENOUGH INFORMATION TO DESIGN THE WALL.

WILL ONE NEW WALL WORK – $l \cong 26'\pm$.

SHEAR – $v_I \cong 452^K / (.8 \times 26' \times 12'' \times 8'') = 226$ psi IN 8" WALL ok.

FLEXURE – ZONES $T \cong \frac{9199^{K\text{-}FT}}{.8 \times 26'} = 442^{KIPS} \Rightarrow A_s = \frac{442^K}{.85 \times 58 \text{ ksi}}$ (58 ksi)
$= 8.96 \cong 9 \text{ in}^2$ FOR ZONE – ok.

ONE "TIED DOWN" WALL WILL WORK.

SAMPLE CALCULATION FOR E-W WALL

SKETCH 2
3 OF 5.

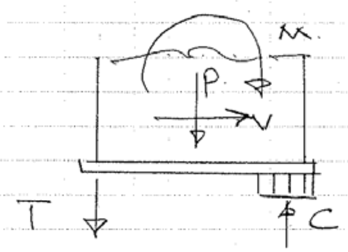
SOIL ANCHORS WILL BE REQUIRED FOR UPLIFT.
DESIGN USING "CAPACITY DESIGN" PRINCIPALS AFTER
FINAL DESIGN OF WALL, USING $1.25 f_y$ IN FLEXURAL
WALL STEEL.

CHECK SLIDING

SLIDING FORCE $\cong 450^K$ X CAPACITY DESIGN
OVERSTRENGTH

RESISTANCE =
C X FRICTION μ.

$\mu = 0.5$



T IS FROM FORCE DIAGRAM – ADDING SOIL ANCHORS
DOES NOT HELP.

$\therefore$ – RESISTANCE $\cong 0.5\,(\underset{\text{DEAD LOAD}}{222^K} + \underset{\text{ZONE FORCE}}{442^K}) = 332^K$

HMM – WILL SLIDE – POSSIBLE SOLUTIONS:

- ADD WEIGHT FROM LARGE FOOTING
- REPLACE S.O.G. WITH STRUCTURAL SLAB AND ENGAGE ADJACENT COLUMN OR WALL DEAD LOADS
- ADD MORE WALLS.

SAMPLE CALCULATION FOR EAST WEST WALL

SKETCH 2
4 OF 5

DIAPHRAGMS

THE LARGEST DIAPHRAGM FORCES ARE AT THE ROOF

FROM TABLE 1, THE ROOF FORCE IS ABOUT $0.7V_B$ TIMES ANY CALCULATED OVERSTRENGTH (O.S).

FOR PURPOSES OF THIS EXAMPLE, ASSUME THE O.S IS 1.4

$\therefore$ LOAD IN THE ROOF IS ABOUT $.7(1.4)V_B \cong V_B$

FROM CALCULATIONS SO FAR:

LOAD INTO EXISTING END WALLS:

$0.073(4040)^K = 295^K$ = TOTAL $\cong 148^K$ EACH END OF ROOF

AT NEW WALL - LOAD AT ROOF INTO WALL $\cong 450^K = 225^K$ EACH SIDE OF WALL.

"GROSS SHEAR" IN DIAPHRAGM (ABOUT 60' WIDE) $- v_f \cong 225^K / (60' \times 12" \times 4"$ SLAB$) = 78$ psi - CLOSE TO v_c, SO PROBABLY NOT AN ISSUE.

HOWEVER - CHECK LOAD PATH AND CONNECTIONS AND "DRAG" STRUTS. IN PARTICULAR, HOW DOES LOAD GET OUT OF 4" SLAB INTO EXISTING WALLS? THIS CONNECTION SHOULD BE UPGRADED.

SAMPLE CALCULATION FOR EAST-WEST WALL

SKETCH 2
5 OF 5

A QUICK LOOK AT WOOD SHEAR WALLS AS A LATERAL SYSTEM

ISSUES –
- DEMAND IS LOW
- HOWEVER, SO IS CAPACITY COMPARED TO OTHER SYSTEMS
- FIRE RATINGS ?? {SINCE THE WALLS ARE LATERAL LOAD RESISTING ONLY, AN ARGUEMENT CAN BE MADE)

TO KEEP DEMAND LOW – LOOK AT 4% DRIFT

AT 4% DRIFT, STEEL BRACES AND CONCRETE AND MASONRY WALLS NOT ALLOWED. !!

ROCKING, HOWEVER, IS!

SO – ROCKING WALL RESISTANCE @ 4% (ZONE 1, C SOIL)

– AVG. OF R1, R2 – TIME 0.67 FOR C.M. CORRECTION

$\therefore$ % G = (12% + 19%)(1/2) × 0.67 = 0.104W

FROM ROCKING RESISTANCE CAPACITY ⇒ 29.6^K = 0.073W

FROM W1 CURVES ⇒ 8% W × (1.75 – 3.6/4) = .068W

$\therefore \dfrac{0.073}{0.104} + \dfrac{x}{.068} = 1, \quad x = 0.02W$ NEEDED FROM WOOD WALL TYPE W1

DEMAND = 0.02 (4046^K) = 82^K

NOW – REVIEW CSA O86 FOR WOOD WALLS

(NOTE – ASSESSMENT = $\frac{.073}{0.104}$ = 0.7 < 0.8 – $\therefore$ UPGRADE

SKETCH 2
APPENDIX (1-3

FOR A WELL DETAILED WALL – 11 ^{KN}/m = 0.75 ^{K}/FT
(POSSIBLE TO GET 14.6 ^{KN}/m)

∴ NEED – 82^{K} / 0.75 ^{K}/FT = 109'± OF PLYWOOD SHEAR WALL

– THIS IS ABOUT 4 x 26'± INTERIOR WALLS

SO – AT 4% DRIFT –

- WOOD OK
- MASONRY, CONCRETE WALLS, STEEL BRACING NOT ALLOWED
- ROCKING WALLS OK
- NEED PROP COLUMNS, BUT DID ANYWAY.
- REMOVE CLAY TILE WALLS – REBUILD AS WOOD SHEAR WALLS.
- CHECK UPLIFT ETC, SLIDING.

POSSIBLE SOLUTION (???)

E-W – 4% DRIFT — ADD WOOD WALLS.

N-S – 2% DRIFT – MODERATELY DUCTILE CONCRETE WALL OR STEEL BRACES.
– DETAIL TO ACCOMMODATE 4% OUT OF PLANE DRIFT

THIS SEEMS LIKE GOOD NEWS!

PERHAPS TOO GOOD???

PROBABLY YES – SEE NEXT PAGE!

SKETCH 2
APPENDIX (2-3)

A PROBLEM - IMPLICIT IN THE CURVES IS THE FACT THAT THE FIGURES AND TABLES HAVE BEEN CUT-OFF AT 60% OF NBCC 2005, $I = 1.0$ AS A MINIMUM DESIGN VALUE.

HOWEVER, THE ROCKING VALUES DO NOT SEEM TO HAVE BEEN CUT-OFF - AND THE 0.67 REDUCTION FACTOR FOR HEIGHT OF CENTRE OF MASS FOR OUR CASE FURTHER REDUCES THE VALUES.

SO - REVIEW - NBCC ROCKING - $R_d R_o = 2$

$$\therefore V_s = \left[0.96(1)(1)/2\right] 0.67 = 0.32$$

└ SHORT PERIOD CUT-OFF

- THIS IS THE VALUE IN TABLE 2
- 0.6 × THIS FOR LOWER BOUND = $0.6(.32) = 0.192$

- THIS IS APPROXIMATELY THE VALUE USED IN TABLE 2 FOR ROCKING AT 2% DRIFT - I.E. - AS PER OUR PREVIOUS ANALYSIS. ∴ REDO @ 2% WITH W1 WALLS.
- W1 @ 2% = 16% × .85 (F-F HT CORRECTION) = 0.136

TOOLBOX @ 2% $= \underset{\text{ROCKING}}{\dfrac{0.073}{0.2}} + \underset{\text{W1}}{\dfrac{x}{0.136}} = 1$, $x = 0.086 W_T$
$= .086(4046^k) =$
$= 348^{\text{KIPS}}$
(NOT 82^{KIPS})

- VERY STRONG W1 $\cong 14.6^{\text{kN}}/\text{m} = 1^{\text{K}}/\text{FT}$
∴ NEED $348^{\text{K}} / 1^{\text{K}}/\text{FT} = 348'$ OF WALLS

HAVE - $5 \times 2_{\text{SIDES}} \times 26' \pm = 260'$ OF WALL = 75% OF REQUIRED

∴ DOES NOT SATISFY REQUIREMENTS BUT IS CLOSE. SOMETHING ELSE NEEDS TO BE DONE.

SKETCH 2
APPENDIX 3-3

4.10 N-S Upgrade

By comparing to the E-W solution, the following can be inferred:

- A single wall running N-S somewhere along the interior column line might work. However, sliding will be an issue.
- Two walls would be a “comfortable” solution, although there may be sliding issues.
- Both of the above would require foundation work and tie-downs.
- Steel bracing could also be considered in lieu of walls.
- Moving the braces (or even the walls) to the exterior walls would:
  - Push the work to the outside of the building, which would simplify it and reduce costs.
  - Result in eccentric footings to design, along with tie-downs. However, the work would be outside the building.
  - The below-grade walls would assist picking up the columns along the wall line to help resist sliding.
  - The building would fall within the eccentricity limits as defined in the guideline.

The design of any walls would follow the outline in Sketch 2.

5.0 COST BENEFIT ANALYSIS

Upgrade costs contain a large component that is not a "hard" structural cost. This additional component is associated with mechanical, electrical, and architectural work, contractor overhead, fees, and contingencies.

In this case, while there are estimates for the 2004 review and the Phase II (2006) proposed work, there is not a cost for this "Demonstration" of the Version 2 of the guidelines.

This work was done with the prime intent of illustrating the application and use of the guidelines on a "real" project by using a "simplified real project" as a template upon which to build a "Demonstration" project.

Seismic upgrades are complex structural challenges that usually have many solutions, and often in developing one solution it becomes apparent that other approaches may avoid difficulties that develop during the design process.

That is the case here. The solution proposed minimizes disruption in the upper floors by adding as few walls as possible. However, this places a large demand on the foundations and requires extensive foundation work.

It may be appropriate at this stage to investigate in some detail other solutions such as:

- Adding more walls to reduce foundation sliding and overturning effects.
- In the N-S direction, add two bays of steel braces to each side of the exterior of the building. This reduces internal disruption in the upper floors and makes the foundation work easier. However, it may be a problem if the building is a heritage building.
- The solutions here all assume 2% drift levels to reduce the demand on the upgrade scheme. However this probably requires "prop" steel columns at all the concrete columns. If this is costly, it may pay to add more walls to reduce the drift so that the steel "prop" columns are not needed.
- Looking at allowing 4% drift and using wood shear walls. (This turns out not to work, but is close.)

With the above in mind, it may be better to infer any cost benefit from a comparison of the demands on the structure with lower values clearly having a cost benefit.

Table 3 illustrates this for selected systems for this project.

TABLE 3 Upgrade force levels as a % of weight for 2% drift levels			
SYSTEM	NBCC 2005 (I=1.0)	GUIDELINE VERSION I	GUIDELINE VERSION 2
S1	25%	41%	23%
C1	23%	21%/30%	17%
C2	32%	23%/33%	NP
ROCKING	32%	28%	20%

Examination of Table 3 shows that, in general, Version 2 of the guideline requires design force levels less than Version 1 or NBC 2005, which is a clear advantage. For instance C1 is about 75% and rocking is about 67% of NBCC 2005 or Version 1 of the guidelines. However, there are two caveats:

.1 C2 is a conventional construction wall and is not permitted at 2% drift in Version 2.

.2 2% drift limits probably requires adding steel prop columns which add some costs over and above the NBCC 2005 solution.

An advantage of the guidelines is being able to use the toolbox approach and utilize existing systems. In this case, in the N-S direction, there is little advantage as there is virtually no existing strength. However, in the east-west direction about 40% of the required capacity comes from the rocking of existing walls. The toolbox approach allows these to be used with a single "tied down" heavily loaded Moderately Ductile Concrete Wall taking about 60% of the load. This is not a recognized approach in the NBCC 2005 for "rigid diaphragm" structures with walls distributed as they are in this project, as we are combining different "R" value walls. Clearly, this is an advantage.

Another advantage of Version 2 of the guidelines over Version 1 is in the treatment of clay tile partitions. In Version 1, they must be removed. In Version 2 removal is the preferred option but "encapsulation" by two stud walls (or the equivalent) is allowed.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Version 2 of the guidelines results in lower design force levels for this project than those in Version I.

Version 2 of the guidelines has new steel systems added and these are candidates to be used in upgrading this school.

The chapter on rocking is much expanded from Version I and is much more useful.

The figures and tables for concrete walls have been simplified from Version I.

The use of the tables and figures for the "simplified" presentation in Version 2 has been illustrated. The distinction between "reinforcing flexural resistance" and "rocking flexural resistance" has been illustrated as has the distinction of rocking "of" the footing and "rocking of the wall" above the footing.

This is a "demonstration" of how to use the document for a concrete school. It is not a detailed design, and additional systems and approaches to those discussed here would be considered in an actual upgrade. Some of these are discussed herein.

Bridging Guidelines

Second Edition - November 2006

Demonstration Project No. 4:

Seismic Retrofit of Wood/Steel/Masonry Shops
Burnsview Secondary School - School District No. 37, Delta

Prepared by:
Pomeroy Consulting Engineers Ltd.

March 2007

Executive Summary

The scope of this document is to illustrate an example use of the *Bridging Guidelines Second Edition* for the assessment and retrofit of a structure consisting of 2” nominal (38mm) T&G wood deck roof, steel load-bearing elements and concrete block walls (mostly infill).

The following demonstration will involve the Shops building at Burnsview Secondary School in Delta. We will examine the original conditions of the structure, present the seismic assessment findings and propose a retrofit program which includes improvement to the following:

1. *Roof diaphragm:* Overlay of existing 38mm T&G wood deck with new 12.5mm plywood sheathing and installation of tension chords/ drag struts on the deck.

2. *LDRS* (Lateral Deformation Resisting System): Reinforcing/grouting existing 4.8m high concrete masonry walls for stability and in-plane shear wall requirements.

3. *Load Path:* Upgrade of the connections between the roof diaphragm, the LDRS and the foundations.

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Project Description

Burnsview Secondary School is located in Delta, BC (S.D. #37). At present time, the entire school is in the process of being seismically upgraded. The Shops wing which is the focus of this demonstration, is part of the retrofit project scope. The retrofit program was determined using the of the *Bridging Guidelines First Edition*.

The Shops were built in 1972. The building is single-storey with a floor area of 1100 m^2. The original construction consisted of 38mm thick T&G decking over Open Web Steel Joists supported by a mix of steel beams and columns or unreinforced masonry walls; the ground floor is in part 125mm reinforced concrete slab-on-grade and in part a wood framed suspended floor over a crawl space. The foundation system consists of conventional strip footings. Refer to figures 1 & 2 for foundations/floor and roof plans, respectively.

Seismic Assessments/ Retrofit Solutions

➢ *NBCC 2005-based seismic assessment*

The school seismic load resistance condition was assessed in 2004 as part of the Ministry of Education provincial seismic risk assessment initiative. This evaluation, which was based on the then 2005 draft NBCC, yielded a base shear demand of 87%W on the Shops. The building was given an overall risk level of *High*. The following deficiencies were observed:

a) Weak roof diaphragm;
b) Inadequate connections of roof diaphragm to vertical elements;
c) Deficient vertical system for lateral loads transfer;
d) Brittle unreinforced high (4.8m) concrete block walls; and
e) Lack of connections of the masonry walls for out-of-plane/ in-plane loads; lack of anchorage to the foundations.

The estimated retrofit cost was $518,100 based on the prototypical upgrade unit rates established for the program at that time.

➢ *Bridging Guidelines First Edition-based assessment/Phase 1 Feasibility Study*

Phase I Feasibility Study was undertaken in September 2005 using the *Bridging Guidelines First Edition*. The parameters used in the seismic risk assessment were as follows:

- Seismic Zone: 4
- Site Class: C

- LDRS type 1: Unreinforced concrete masonry shearwalls
  Instability drift limit for masonry LDRS type 1: max. 1%
- Ground snow load: S_s = 2.3 kPa (1/50 years ground snow load)
  (For seismic mass calculations) S_r = 0.4 kPa (1/50 years associated rain load).

Our preliminary analysis validated the 2004 assessment findings for the Shops and it was assigned a high level of risk: the sum of the available resistances from each LDRS was substantially less than that stipulated by the *Bridging Guidelines* therefore, the building required retrofit.

The recommendations for retrofit were:

i) *Roof Diaphragm:* Remove roofing and strengthen existing 38mm T&G wood roof deck by overlaying it with new 12.5mm plywood sheathing. Install tension chords/ drag struts to suit, on top of new deck.
ii) *LDRS:* Reinforce existing concrete block walls and grout reinforced cores; connect newly-designated shearwalls to the diaphragm for in-plane and out-of-plane loads.
iii) *Foundations:* Install dowels in existing foundations to anchor base of masonry walls.

The estimate regarding the cost of this retrofit work was based on in-house experience from previously-undertaken similar projects; it was evaluated that the cost would be considerably less than 70% of the cost of new construction of a building having similar size and facilities.

- *Bridging Guidelines First Edition-based assessment/Phase II Feasibility Study*

Phase II Feasibility Study was undertaken in November 2005, hence, it was also based on the *Bridging Guidelines First Edition*. A more thorough numerical evaluation of the building and further site review of the existing conditions validated the Phase I Feasibility Study findings and the retrofit program remained unchanged.

The cost of the retrofit was estimated by a Quantity Surveyor and amounted to approximately $720,000, less than 70% of the cost of replacement.

For the purpose of this demonstration, the calculations related to the proposed seismic upgrade solution will be presented using the *Bridging Guidelines Second Edition*.

<u>Proposed Seismic Upgrade</u>

- **Roof Diaphragm:**

The decision to upgrade the 38mm T&G wood deck diaphragm was made based on experience and good engineering practice. The Second Edition of the *Bridging Guidelines,* Section 10.0 titled *Performance-Based Earthquake Retrofit Guidelines for Diaphragms in Low-Rise School Buildings* confirmed this decision. Section 10.8 clause 2(b) states that decking must be at least 64mm thick and side spiked in order to avoid upgrading.

The upgrade solution involved overlay of the T&G wood deck with 12.5mm plywood. Section 10.1 prototype D-1 is selected as the upgraded diaphragm condition (the existing 38mm T&G deck will act as the 'blocking' the prototype stipulates). The level of design load is determined using Figure 10-1(c) which is suitable for zone 4 (Vancouver). Selecting Class C soil and a max. diaphragm span of 25m, the graph or the table indicate that the seismic demand on the diaphragm is 8%W. The span and severe weakness of the diaphragm was judged at the time to be unsuitable to permit delay of the upgrade work. Due to the extensive Mechanical and Electrical services in the ceiling space of the shops, upgrade of the diaphragm from the u/s was judged too time consuming and costly. In addition, the existing roof membrane was nearing the end of its useful life.

See attached hand calculations for the diaphragm upgrade solution.

- **LDRS:**

The extent of existing unreinforced masonry walls clearly confirmed their choice as the LDRS. There were no other significant existing elements.

From Section 1.0 titled *Performance-Based Earthquake Retrofit Guidelines for Low-Rise School Buildings-General Requirements, Table 1.1 List of LDRS's* prototype M-2 is selected for the upgraded reinforced masonry walls as the LDRS system of the building. The accompanying ISDL is 1.5% and R_o for connections calculations is 1.5.

From Section 6.0 titled *Performance-Based Earthquake Retrofit Guidelines for Low-Rise Concrete Masonry School Buildings:* Figure 6-2 (c) suitable for zone 4, Class C site and maximum inter-story drift of 1.5% yields a wall seismic demand of 21%W.

See attached hand calculations for LDRS upgrade solution.

Cost-Benefit Analysis

The total estimated cost of this retrofit (work is underway and on budget) is $540,000.00 excluding the new roof membrane. The new roof membrane cost to permit diaphragm upgrade is estimated at $160,000.00.

The upgrade was designed for a base shear of 21% W which is considerably less than that required of 30% W in 2005 NBCC, using I=1.0 was utilized per the original assessment. The cost saving is judged to be relatively small as the masonry reinforcing requirements, due to the height of walls, were governed by out-of-plane forces. Typically, classroom areas of a school have a greater benefit as the masonry walls are lower in height than the shops.

The upgrade cost of this project is good value considering the larger, greater than 50%, increase of construction cost since the 2004 estimate.

Conclusions and Recommendations

The Second Edition of the *Bridging Guidelines*, particularly for wood diaphragms with some reasonable capacity, minimizes required diaphragm upgrades. Where diaphragms upgrades other than building connections can be eliminated, the savings to the project can be as much as $200.00 per sq. m.

Many masonry projects, including large areas of Burnsview with shallow foundations, have sufficient wall lengths available that LDRS upgrades can be completed with little if any requirement for expensive foundation work.

Retrofit utilizing the *Bridging Guidelines* will frequently be less extensive and therefore less costly than upgrades to 2005 NBCC.

BURNSVIEW SECONDARY SCHOOL – SHOPS' SEISMIC UPGRADE

1. Roof diaphragm

Exist. roof deck : 2" nominal T&G wood. (38 mm).

∴ Overlay with 12.5 mm plywood.

From Bridging Guidelines Second Edition:

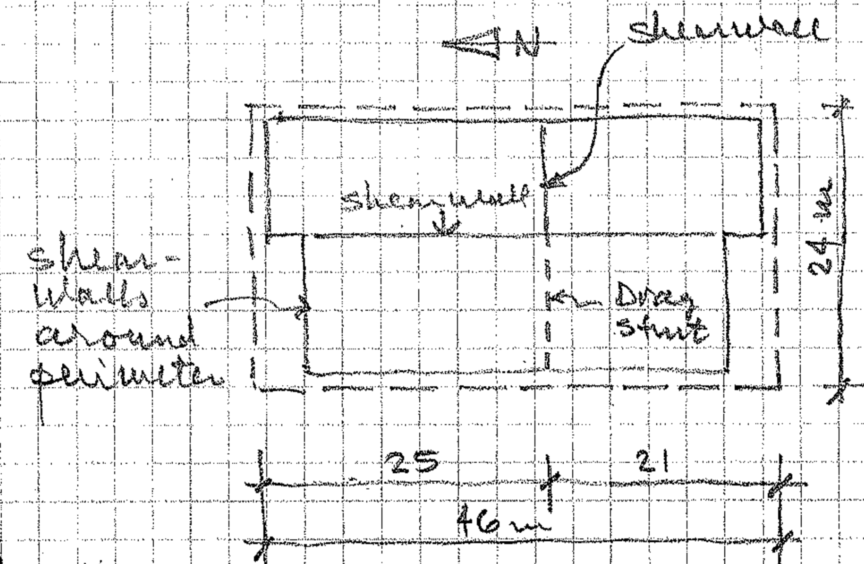
- Prototype D-1 / Zone 4
- Site class C
- Diaphragm max. span = 25 m

} $V = 0.08W$

$W = 2440$ kN

∴ $V = (0.08)(2440) = 195$ kN

Shear stresses in roof diaphragm:



Roof diaphragm & supports plan

- E-W

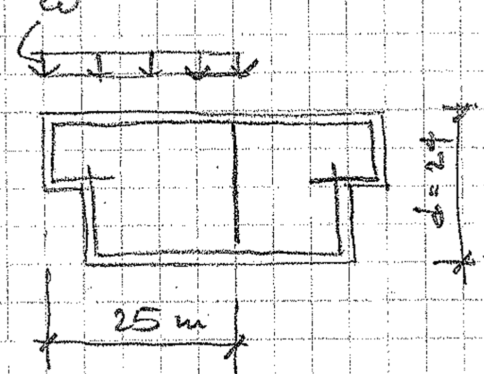
$V_{max} = \left(\frac{25}{46}\right) 195 = 106$ kN

$v_f = \frac{106}{2 \times 24} = 2.2 \frac{kN}{m}$

Plywood sheathing:
Nailed with 64 mm lg.
common nails @ 64 mm c/c
around diaphragm/panel
edges $\rightarrow v_r \cong 9 \frac{kN}{m} > v_f = 2.2 \frac{kN}{m}$ o.k.

- N-S – smaller shear stresses, by inspection.

Diaphragm chords:



$L_{max} = 25$ m

$V = 106$ kN

$w = \frac{106}{25} = 4.3 \frac{kN}{m}$

$M = \left(\frac{wL^2}{8}\right) 2 = \left(4.3 \frac{(25)^2}{8}\right) 2 = 672$ kN·m

In order to reflect the dynamic behaviour of flexible wood diaphragm (Sect. 10.5)

$\therefore\ C = T = \frac{M}{d} = \frac{672}{24} = 28$ kN

$\therefore$ Required: From Simpson Catalogue - Tension strap CMST12 c/w 76 mm lg. common nails around perimeter and at drag strut.

$T_r = 51$ kN $> T_f = 28$ kN. O.K.

S.P.F. plywood.

2. LDRS

Reinforced running bond masonry shear walls:

LDRS prototype M-2, ISDL = 1.5%

∴ M-2 Zone 4 combination presented in Fig. 6-2 (c).

For Site Class C, 1.5% → $V = 0.21\,W$

∴ $V = (0.21)(2440) = 512$ kN

- E-W

Worst case at central wall.

For flexible diaphragm, tributary shear in wall:

$V = (0.5)(512) = 256$ kN

Wall length = 8 m

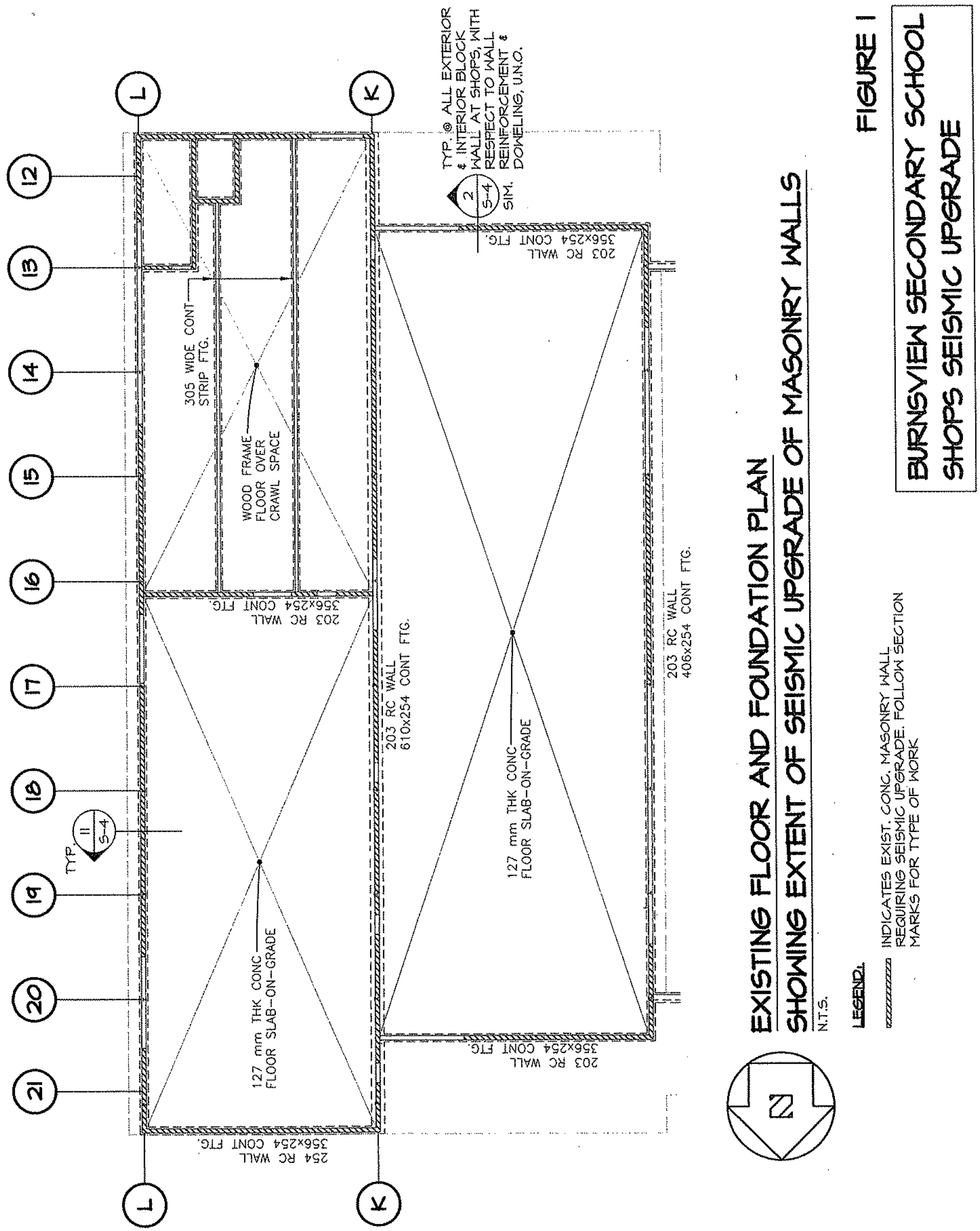
Shear per 1.0 m = $\frac{256}{8} = 32\ \frac{kN}{m}$

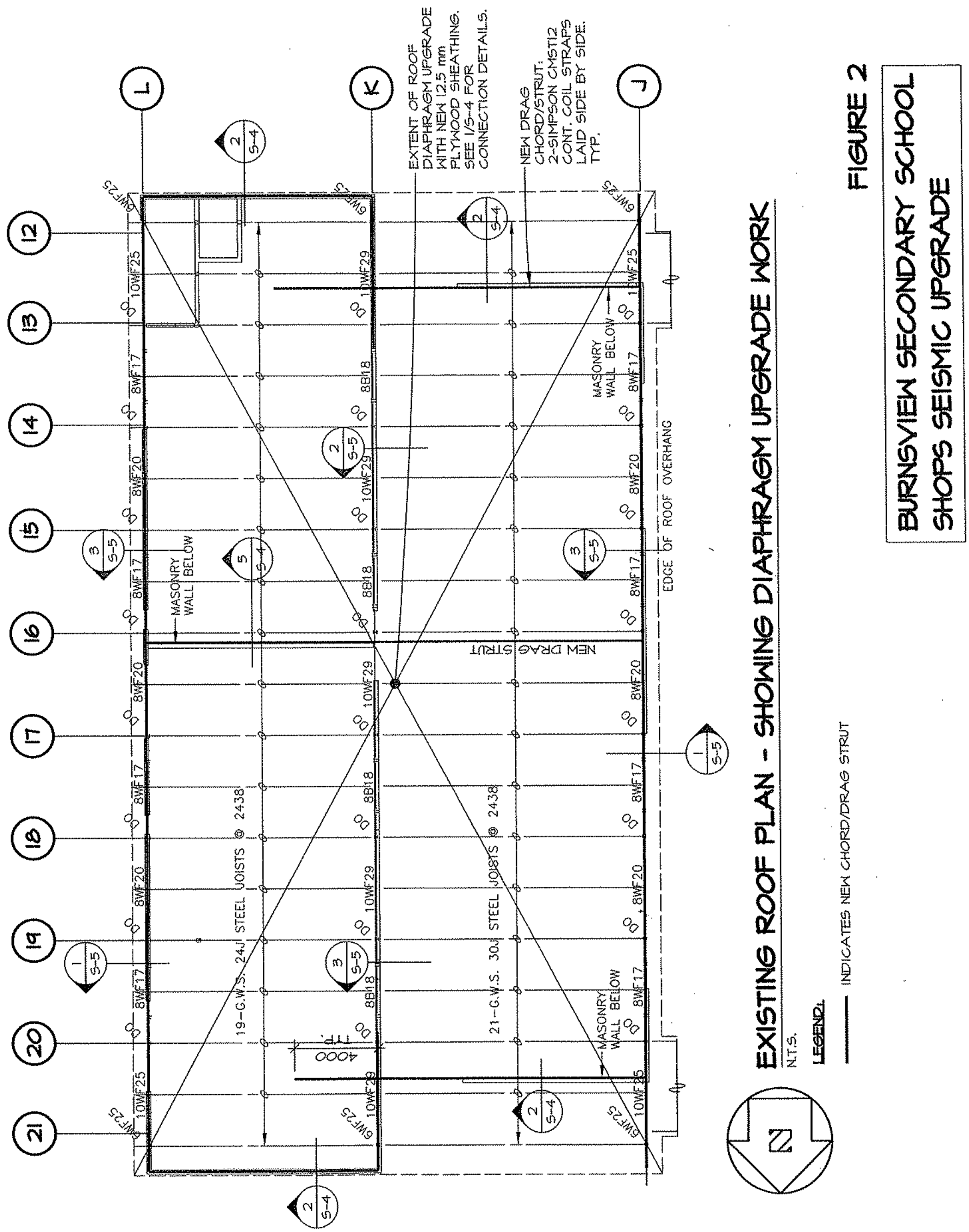
- Required wall reinforcing/grouting to be calculated using CSA S304.1-04 Design of masonry structures.
- Connection of top of wall to wood roof diaphragm per 1.0 m wall for in-plane shear:

  * Perimeter walls = $1.5\,(\underset{kN/m}{2.2})(\underset{R_o}{1.5}) = 5\ \frac{kN}{m}$ <Cl. 10.3>

  * Center wall = $5 \times 2 = 10\ \frac{kN}{m}$
- Connection of wall top for out-of-plane shear:

  According to NBCC 2005 building parts, Cl. 4.1.8.17.
- Wall stability for overturning: According to normal practice.


TYP. @ ALL EXTERIOR & INTERIOR BLOCK WALL AT SHOPS, WITH RESPECT TO WALL REINFORCEMENT & DOWELING, U.N.O.
2
S-4
SIM.
TYP.
11
S-4
L
K
12
13
14
15
16
17
18
19
20
21
305 WIDE CONT STRIP FTG.
WOOD FRAME FLOOR OVER CRAWL SPACE
203 RC WALL
356x254 CONT FTG.
203 RC WALL
610x254 CONT FTG.
203 RC WALL
406x254 CONT FTG.
254 RC WALL
356x254 CONT FTG.
127 mm THK CONC FLOOR SLAB-ON-GRADE
EXISTING FLOOR AND FOUNDATION PLAN
SHOWING EXTENT OF SEISMIC UPGRADE OF MASONRY WALLS
N.T.S.
LEGEND.
INDICATES EXIST. CONC. MASONRY WALL REQUIRING SEISMIC UPGRADE. FOLLOW SECTION MARKS FOR TYPE OF WORK
N
FIGURE 1
BURNSVIEW SECONDARY SCHOOL
SHOPS SEISMIC UPGRADE


EXTENT OF ROOF DIAPHRAGM UPGRADE WITH NEW 12.5 mm PLYWOOD SHEATHING. SEE 1/S-4 FOR CONNECTION DETAILS.
NEW DRAG CHORD/STRUT: 2-SIMPSON CMST12 CONT. COIL STRAPS LAID SIDE BY SIDE. TYP.
NEW DRAG STRUT
MASONRY WALL BELOW
EDGE OF ROOF OVERHANG
19-G.W.S. 24J STEEL JOISTS @ 2438
21-G.W.S. 30J STEEL JOISTS @ 2438
EXISTING ROOF PLAN - SHOWING DIAPHRAGM UPGRADE WORK
N.T.S.
LEGEND:
INDICATES NEW CHORD/DRAG STRUT
FIGURE 2
BURNSVIEW SECONDARY SCHOOL
SHOPS SEISMIC UPGRADE

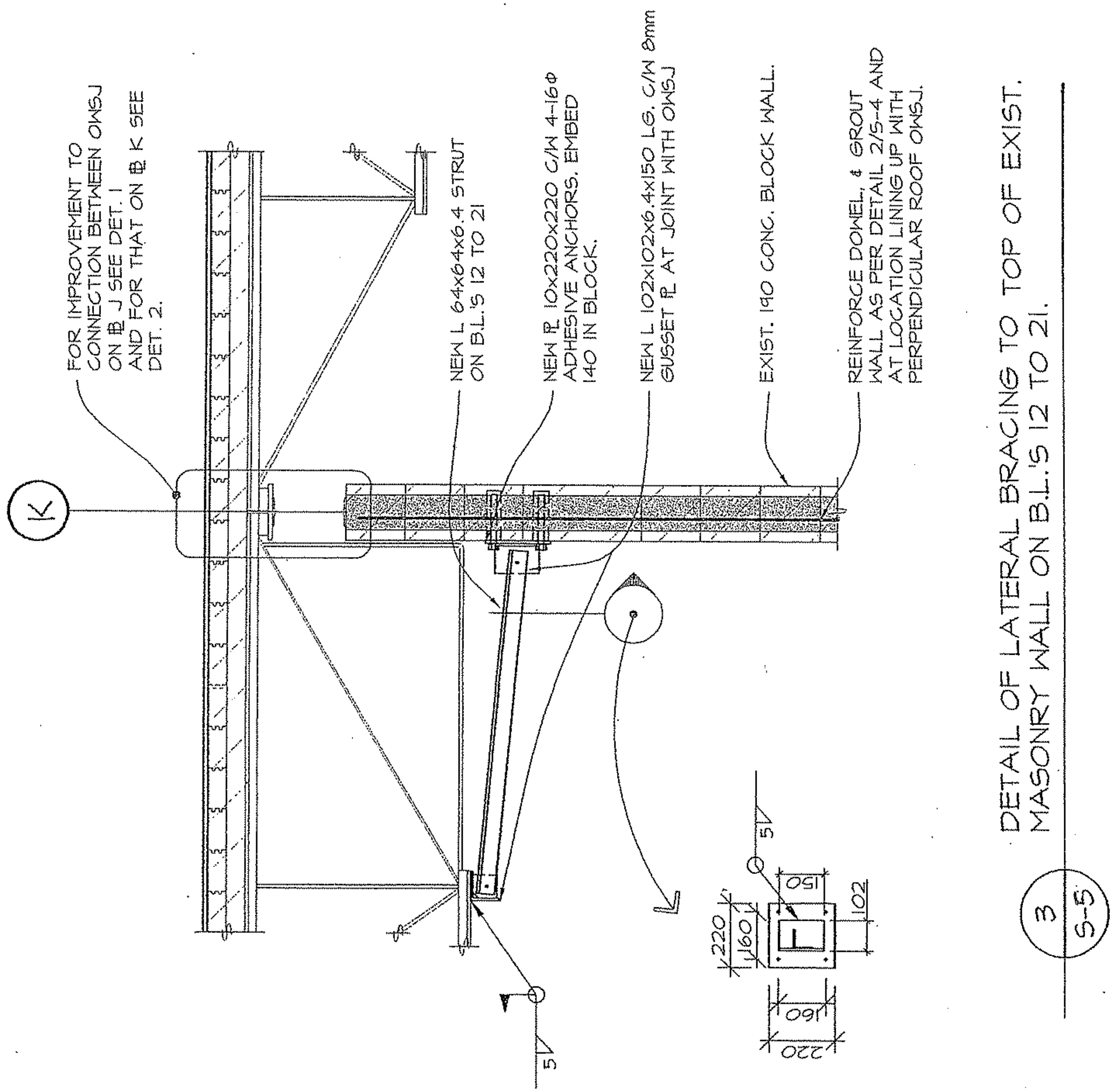

FOR IMPROVEMENT TO CONNECTION BETWEEN OWSJ ON ℄ J SEE DET. 1 AND FOR THAT ON ℄ K SEE DET. 2.
NEW L 64x64x6.4 STRUT ON B.L.'S 12 TO 21
NEW ℙ 10x220x220 C/W 4-16φ ADHESIVE ANCHORS, EMBED 140 IN BLOCK.
NEW L 102x102x6.4x150 LG. C/W 8mm GUSSET ℙ AT JOINT WITH OWSJ
EXIST. 190 CONC. BLOCK WALL.
REINFORCE DOWEL, & GROUT WALL AS PER DETAIL 2/S-4 AND AT LOCATION LINING UP WITH PERPENDICULAR ROOF OWSJ.
K
5
220
160
150
102
DETAIL OF LATERAL BRACING TO TOP OF EXIST. MASONRY WALL ON B.L.'S 12 TO 21.
3
S-5

Figure 3

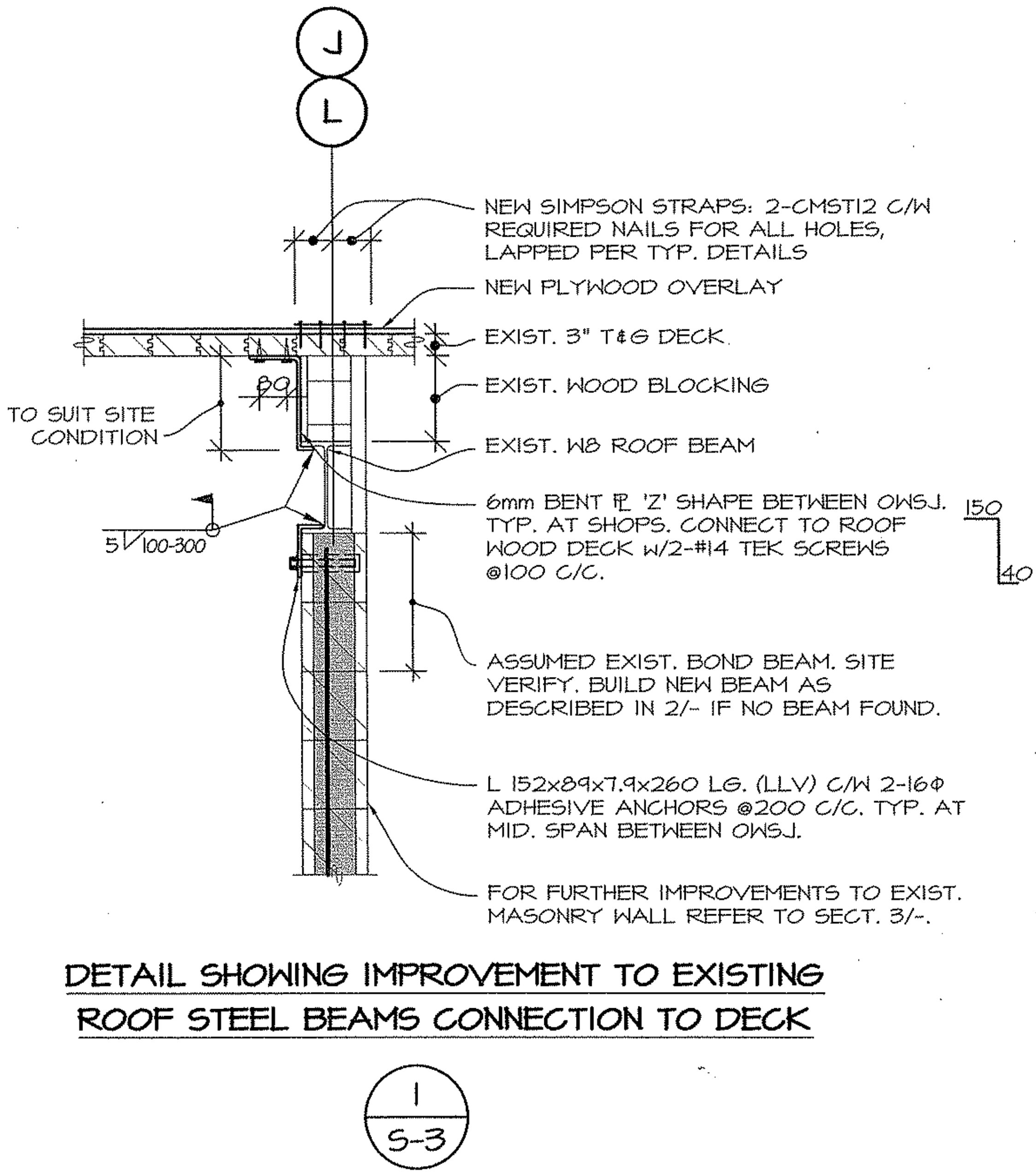

J
L
NEW SIMPSON STRAPS: 2-CMST12 C/W REQUIRED NAILS FOR ALL HOLES, LAPPED PER TYP. DETAILS
NEW PLYWOOD OVERLAY
EXIST. 3" T&G DECK
EXIST. WOOD BLOCKING
TO SUIT SITE CONDITION
80
EXIST. W8 ROOF BEAM
6mm BENT PL 'Z' SHAPE BETWEEN OWSJ. TYP. AT SHOPS. CONNECT TO ROOF WOOD DECK w/2-#14 TEK SCREWS @100 C/C.
150
40
5 100-300
ASSUMED EXIST. BOND BEAM. SITE VERIFY. BUILD NEW BEAM AS DESCRIBED IN 2/- IF NO BEAM FOUND.
L 152x89x7.9x260 LG. (LLV) C/W 2-16Φ ADHESIVE ANCHORS @200 C/C. TYP. AT MID. SPAN BETWEEN OWSJ.
FOR FURTHER IMPROVEMENTS TO EXIST. MASONRY WALL REFER TO SECT. 3/-.
DETAIL SHOWING IMPROVEMENT TO EXISTING ROOF STEEL BEAMS CONNECTION TO DECK
1
S-3

Figure 4

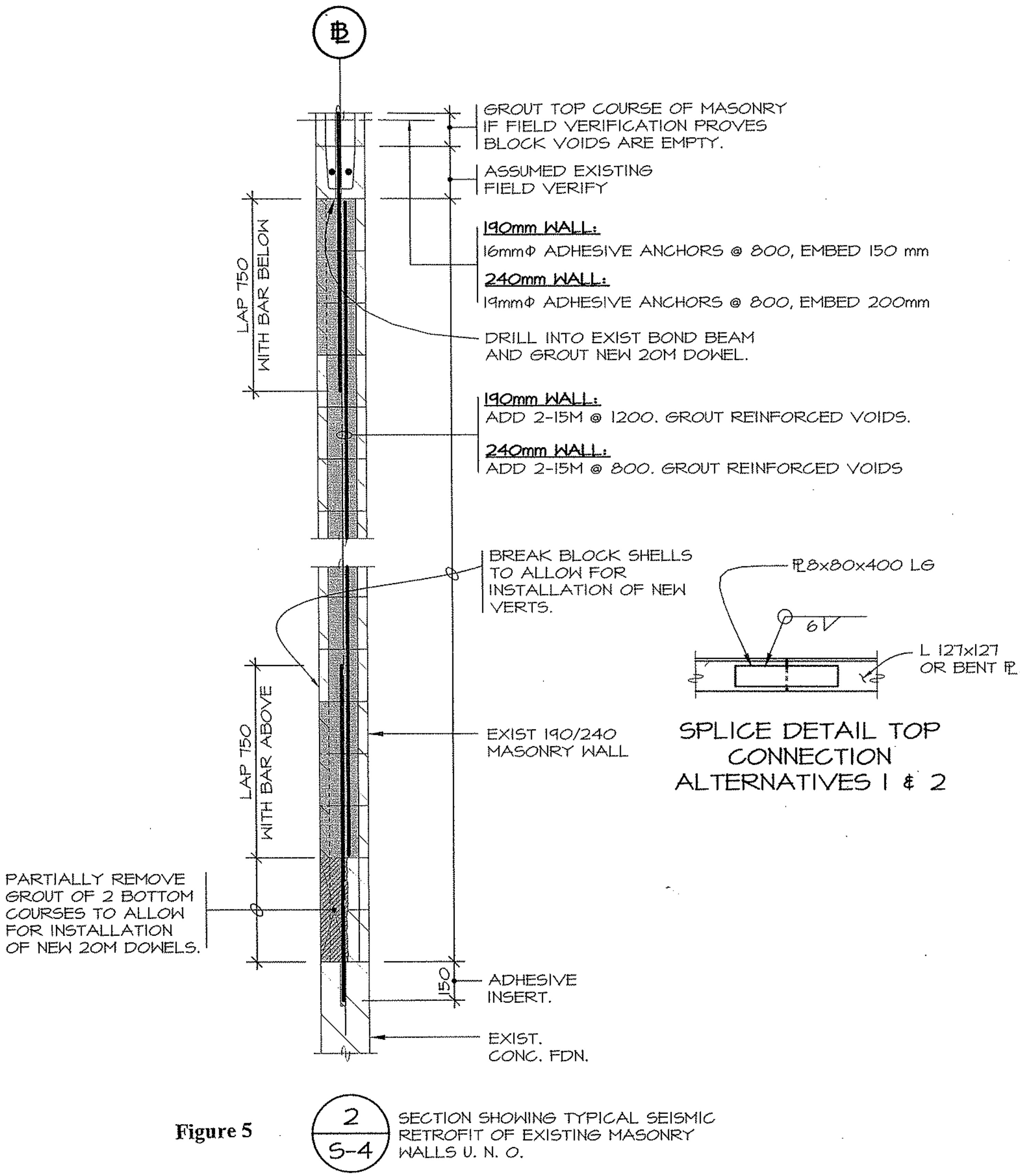

GROUT TOP COURSE OF MASONRY
IF FIELD VERIFICATION PROVES
BLOCK VOIDS ARE EMPTY.
ASSUMED EXISTING
FIELD VERIFY
190mm WALL:
16mmΦ ADHESIVE ANCHORS @ 800, EMBED 150 mm
240mm WALL:
19mmΦ ADHESIVE ANCHORS @ 800, EMBED 200mm
DRILL INTO EXIST BOND BEAM
AND GROUT NEW 20M DOWEL.
LAP 750
WITH BAR BELOW
190mm WALL:
ADD 2-15M @ 1200. GROUT REINFORCED VOIDS.
240mm WALL:
ADD 2-15M @ 800. GROUT REINFORCED VOIDS
BREAK BLOCK SHELLS
TO ALLOW FOR
INSTALLATION OF NEW
VERTS.
PL8x80x400 LG
6
L 127x127
OR BENT PL.
SPLICE DETAIL TOP
CONNECTION
ALTERNATIVES 1 & 2
LAP 750
WITH BAR ABOVE
EXIST 190/240
MASONRY WALL
PARTIALLY REMOVE
GROUT OF 2 BOTTOM
COURSES TO ALLOW
FOR INSTALLATION
OF NEW 20M DOWELS.
150
ADHESIVE
INSERT.
EXIST.
CONC. FDN.
2
S-4
SECTION SHOWING TYPICAL SEISMIC
RETROFIT OF EXISTING MASONRY
WALLS U. N. O.

Figure 5

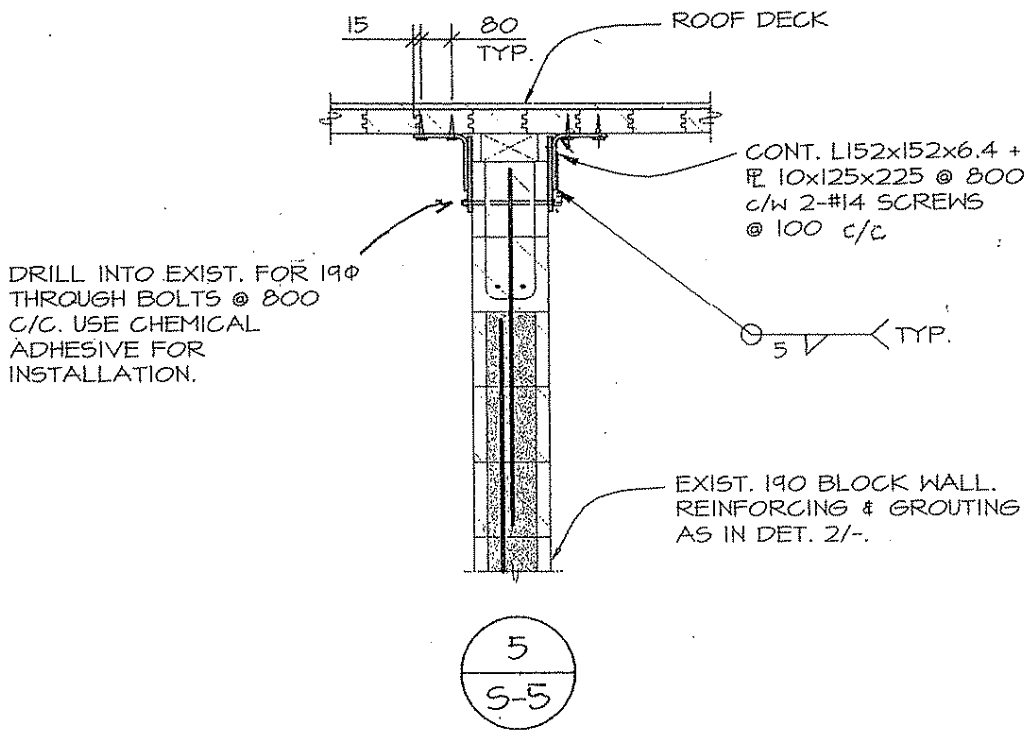

15
80
TYP.
ROOF DECK
CONT. L152x152x6.4 +
PL 10x125x225 @ 800
c/w 2-#14 SCREWS
@ 100 c/c
DRILL INTO EXIST. FOR 19Φ
THROUGH BOLTS @ 800
C/C. USE CHEMICAL
ADHESIVE FOR
INSTALLATION.
5
TYP.
EXIST. 190 BLOCK WALL.
REINFORCING & GROUTING
AS IN DET. 2/-.
5
S-5

Figure 6

Ellendale Elementary School Demonstration Project

2nd Edition Bridging Guidelines

(School District No.36 – Surrey)

Prepared by:
David Nairne and Associates Ltd.

March 2007

1.0 EXECUTIVE SUMMARY

1.1 This document is part of a series of reports prepared to demonstrate the seismic retrofitting of provincial schools. This particular report illustrates the seismic upgrading of the roof diaphragm of the wood/masonry gymnasium for Ellendale Elementary School located in Surrey, B.C. The report summarizes the Seismic Feasibility Study carried out for the subject school, describes the proposed seismic upgrading, presents sample calculations and includes site photographs and seismic upgrading details.

1.2 Ellendale Elementary School consists of a 1,521sm single storey wood frame structure built over a crawl space. Originally constructed in 1968 as a 6-classroom primary school, a Gymnasium was added 1971 followed by a three classroom addition in 1993. A tall concrete masonry wall separates the Gymnasium from the classrooms along with several shorter concrete masonry walls enclosing the mechanical room. Both the roof and floor framing consists of tongue and groove wood decking over glulam or timber beams.

1.3 The 2nd edition Bridging Guidelines provides a rational and state of the art approach in developing cost effective seismic upgrading solution for this project based on drift limits and taking into account the capacities of all existing lateral deformation resisting systems.

1.4 Application of the 2nd edition Bridging Guidelines to the seismic upgrading of Ellendale Elementary school resulted in significant cost savings over the 1st editions Bridging Guidelines in the following areas;

a) Allowing a mixed LDRS systems in the Gymnasium to behave independently
b) Not having to seismic upgrading the existing wood deck diaphragm in the Gymnasium
c) Reducing the design connection forces in the diaphragm, and
d) Delaying the seismic upgrading of the roof diaphragm over the Classroom block until the re-roofing of the Classroom Block
e) Delaying the seismic upgrading of moderate height unreinforced concrete masonry walls until re-roofing of the Classroom Block

1.5 The cost estimate for the Seismic Upgrading "Option B" selected for Ellendale Elementary School developed under the 2nd edition Bridging Guidelines is $ 595,100.00. Our preliminary cost estimate for the seismic upgrading of Ellendale Elementary School if developed under the current requirements of the 2006 BCBC would be in the order of $ 850,000 to $ 950,000.

1projects/4060/demonstrationproject/demo project ellendale.doc

2.0 PROJECT DESCRIPTION

2.1 Introduction

Ellendale Elementary School consists of a 1,521 sm single storey wood frame structure built over a crawl space. Originally constructed in 1968 as a 6-classroom primary school, a Gymnasium was added 1971 followed by a three classroom addition in 1993. A tall concrete masonry wall separates the Gymnasium from the classrooms along with several shorter concrete masonry walls enclosing the mechanical room. Both the roof and floor framing consists of tongue and groove wood decking over glulam or timber beams.

Front of Ellendale Elementary School Looking at the Gymnasium.

3.0 2004 SEISMIC ASSESSMENT

3.1 Seismic Assessment as per 2005 NBC

A structural assessment of Ellendale School was carried by the Ministry of Education in 2004 with respect to the earthquake design requirements of the 2005 NBC. A summary of the 2004 Seismic Assessment is presented in Table A.

Table A
2004 Seismic Assessment Summary

Building Block	Gross Floor Area m^2	Major Seismic Deficiencies	Base Shear Demand	Seismic Risk	Total Project Costs Estimate
Block No.1 Gymnasium	276	West HCB wall is unreinforced No anchorage of HCB to foundations Lack of roof to wall connections No diaphragm chords Weak diaphragm capacity	87% W	High	$ 128,620
Block No.2 1968 Classroom	935	Weak diaphragm Lack of chords Weak out of plane capacity of unreinforced HCB Foundations lack capacity locally	87 %	Moderate/ High	$ 440,390
Block No.3 1993 Classroom	310	None identified	17%	Low	n/a
Block No.4 Portable	90	None identified	26%	Low/ Moderate	n/a
Total	**1,611**				**$ 569,010**

Notes:
1. School on Site Class C.
2. Cost estimate by Ministry of Education

4.0 SEISMIC MITIGATION FEASIBILITY STUDY

4.1 Seismic Mitigation Feasibility Study

The Seismic Mitigation Feasibility Study for Ellendale Elementary School was carried out for School District No. 36 in two phases in accordance with the Ministry of Education's (MOE's) Feasibility Study Guidelines. The first stage tested the project assumptions and confirmed that the school continued to pose a medium to high seismic risk. This involved a review of previous seismic assessments, including the re-assessment of the findings of a Structural Assessment carried out by the Ministry of Education in 2004, using a set of "Bridging Guidelines" base on new performance based seismic assessment tool, the UBC 100. The second stage, which would proceed only if supported by the conclusions of the first stage, required a more detailed evaluation of the seismic deficiencies and the preparation of seismic upgrading options, along with project schedules, implementation strategy and cost estimates

4.1 **Phase I Feasibility Study**

The Phase I Feasibility Study was completed in December 2005 using the 1st Edition Bridging Guidelines. The findings of this study are summarized in Table B.

Table B
Phase I Feasibility Study Summary

Seismic Hazard	Issues Identified	Findings
Structural	Construction	The building structure generally conforms with the structural drawings
	Previous Seismic	None of the seismic deficiencies identified in the 2000 Seismic Assessment nor the 2004 Structural Assessment have been addressed.
	Upgrading Gymnasium Block	The Gymnasium block is seismically deficient in terms of lack of well defined with adequate strength load path and connections, weak roof diaphragm, weak shear walls, lack of out-of-plane restraint of the hollow concrete block wall, weak floor diaphragm, lack of lateral stability in post and beam floor framing
	1968 Classroom	The Classroom Block is seismically deficient in terms of lack of well defined with adequate strength load path and connections, weak roof diaphragm, weak shear walls, weak floor diaphragm, lack of lateral stability in post and beam floor framing
Non-Structural	Unreinforced hollow block walls	The non structural hazard identified are unreinforced hollow concrete block partition walls

Geologic & Site		No potential geologic/site hazards were identified in the 2004 Assessment-we confirm this finding.
Adjacency	Falling Hazards	No potential falling hazards were identified in the 2004 Assessment-we confirm this finding.

The study confirmed the findings of the 2004 Seismic Assessment and concluded that the Gymnasium Block continued to pose a high seismic risk and that the 1968 Classroom Block continued to pose a medium high risk.

4.2 Phase II Feasibility Study

The Phase II Feasibility Study was completed in April 2006 using the 1st Edition Bridging Guidelines. Two different seismic upgrading options were developed as summarized below.

Table C
Seismic Upgrading Summary

Area	Option A Seismic Upgrading	Option B Seismic Upgrading
Strategy	Strengthen roof diaphragm and shear walls with plywood in locations that will minimize alterations required to exterior of the school	Strengthen roof diaphragm and shear walls with plywood in locations that will minimize alterations required to interior of the school.
Upgrading to 1968 Classrooms	Add plywood sheathing to 834 lineal m of exterior and interior and walls and replace sections of exterior glazing with new plywood shear walls Add plywood sheathing to new pony walls in crawl space and corresponding to new interior shear walls. Install perimeter connection between diaphragm and wall. Brace between main floor columns in crawl space and install new connections at top and bottom of columns	Add plywood sheathing to 413 lineal m of interior and exterior walls and replace sections of exterior glazing with new plywood shear walls or steel cross bracing. Add plywood sheathing to new pony walls in crawl space and corresponding to new interior shear walls. Install perimeter connection between diaphragm and wall Brace between main floor columns in crawl space and install new connections at top and bottom of columns Install plywood to top of wood roof decking over classroom and offices

Upgrading to Gymnasium	Install 248 m^2 plywood to underside of wood roof decking. Add plywood sheathing to interior face of perimeter walls. Reinforce and grout concrete block walls	Install 248 m^2 plywood to top of wood roof decking. Add plywood sheathing to exterior or interior face of perimeter walls. Reinforce and grout concrete block walls
Level of Disruption	Major disruption to interior spaces and finishes	Limited disruption to interior spaces and finishes
Flexibility in Schedule	Upgrading work limited to when school is unoccupied	Much of work can be carried out during school hours
Construction Schedule	Construction in two phases during July/August summer break	Construction in two phases during July/August summer break
Construction Cost Estimate	$ 885, 391.00 Adjusted for construction in Summer 2006	$ 761,996.00 Adjusted for construction in Summer 2006

4.4 Option B Selected

District No.36 decided to proceed with Seismic Upgrading Option B due to lower construction costs and less disruption to the building. Drawings and specifications were completed for Option B in November 2005 for Construction Tender.

4.5 2nd Edition Bridging Guidelines

The 2nd Edition of the Bridging Guidelines was released in November 2006. Option B was reviewed in detail with respect to the changes in the 2nd Edition for potential cost savings. Further site investigation carried out by Levelton Engineering in February 2007 to determine details of the construction of the roof diaphragms and concrete masonry walls. A comparison of the seismic design parameters between the two editions is summarized in Table D.

Table D
Comparison Seismic Design Parameters
1st and 2nd Edition of Bridging Guidelines

Seismic Parameter	Description	1st edition	2nd edition
Site Class	As per geotechnical report	C	C
Seismic Zone		4	4
LDRS Instability Drift Limit	W-2 Unblocked wood wall M-2 Unreinforced masonry	4% 1.5 %	4 % 1.5%
Minimum Factored Resistance	W-2 unblocked wood wall M-2 Unreinforced masonry	12% W 18% W	8 % W 21 % W
Diaphragm	D-2 Unblocked wood	n/a	Ro =1.7

Application of the 2nd edition of the Bridging Guidelines resulted in several changes to Option B including;

a) No seismic upgrading of the existing Gymnasium diaphragm required

b) Seismic upgrading of the Classroom block roof diaphragm delayed until replacement of the roof

c) Reduction in number and size of connections between roof diaphragm and LDRS

d) Delay in reinforcing concrete masonry walls around mechanical room unit replacement of the roof.

5.0 PROPOSED SEISMIC UPGRADING

The following provides commentary on some of the highlights of the seismic upgrading of the Gymnasium roof diaphragm with respect to the application of the 2nd edition of the Bridging Guidelines. Sample calculations are contained in Appendix A, seismic details in Appendix B and site photographs in Appendix C.

5.1 Roof Diaphragms

The existing roof diaphragm over the 1968 Classroom block consist of 40 x 130 tongue and groove wood decking fastened only to the glulam roof beams spaced at 2438 mm o.c. with spikes. The roof diaphragm has insufficient shear strength and requires upgrading with a new layer of plywood sheathing laid on top of the existing roof decking. As the Classroom block is entirely of wood frame construction, the timing of the upgrading of the existing roof diaphragm will be delayed to coincide with the replacement of the roof so that the diaphragm upgrading can be carried out in a more cost-effective manner (refer to BG-2 Section 10.8.3.)

The roof diaphragm over the 1971 Gymnasium consists of 64 x 130 tongue and grove decking fastened with 5.4mm diameter by 200 mm long spikes driven horizontally between each plank at 765 mm o.c. +/- 253 mm. No upgrading is required to the existing Gymnasium roof diaphragm itself as the construction of the roof decking meets the criteria set forth in the BG-2 section 10.8 Sentence 1 and 2. New chord members, new shear connections and new out-of-plane connections are required to complete the load path from the diaphragm to the LDRS.

5.2 LDRS

5.2.1 Mixed LDRS

The existing Gymnasium roof diaphragm is supported on four sides by LDRS's with two wood framed LDRS's in the EW direction and one concrete masonry LDRS and one wood framed LDRS in the NW direction. The selection of the governing drift is typically limited by the system with the lowest drift limits. In the case of the Gymnasium with a "mixed LDRS" in the NS direction, the governing drift limit would normally be 1.5% as governed by the concrete masonry shear wall as a M-2 LDRS with an ISDL @ 1.5% versus the wood shear wall as a W-1 LDRS with a ISDL @ 4%. However, the BG-2 allows mixed LDRS systems with significant separation and flexible diaphragms to behave substantially independent of each other provided the diaphragm distortion limits are not exceeded. The Gymnasium diaphragm meets these distortion limits and the concrete masonry wall and the wood frame were designed independently with their respective ISDL.

5.2.2 Concrete Masonry Walls

The existing concrete masonry walls are running bond with some horizontal joint reinforcement but no vertical reinforcement.

The NS 6.3 m high concrete masonry divides the 1971Gymnasium from the 1968 Classroom Block. The wall is in contact along its length at the 3.75 m level with the roof diaphragm of the 1968 Classroom Block. The upper portion of this wall requires minimum vertical reinforcement as per BG-2 Section 6.5. Although it could be argued that the lower portion of this wall may not require vertical reinforcement under the out-of-plane exemptions listed in BC-2 Section 6.5 Sentence 3, the delayed upgrading of the roof diaphragm of the Classroom Block led the lower portion of the to be reinforced vertically as well.

The three sections of the 3.75 m high concrete masonry walls in the 1968 Classroom Block adjacent the Gym wall major wall encloses a Mechanical Room and abuts two washrooms. Due to the low occupancy hazard of the rooms in and around these concrete masonry walls, it was judged reasonable to delay the reinforcement of these concrete masonry walls to coincide with the re-roofing and upgrading of the Classroom Block diaphragm.

6.0 COST BENEFIT ANALYSIS

6.1 The cost estimate for seismic upgrading “Option B” selected for Ellendale developed under the 2nd edition Bridging Guidelines is $ 595,100.00.

6.2 No detailed design was carried out for seismically upgrading Ellendale elementary School under the provisions of the current 2006 BCBC. However, we believe that such upgrading will require;

a) The Immediate upgrading of the roof diaphragm of the Classroom and Block and Gymnasium
b) Reinforcement of all of the concrete masonry work
c) Installation of new foundations for new shear walls

6.3 We estimate that the cost estimate to seismically upgrading Ellendale Elementary School to 2006 BCBC would be in the order of $ 850,000 to $ 950,000.

7.0 FINDINGS AND RECOMMENDATIONS

7.1 The 2nd edition Bridging Guidelines provides a rational and state of the art approach in developing cost effective seismic upgrading solution for this project.

7.2 Specific provisions in the Guidelines resulted in significant cost savings in

a) Allowing a mixed LDRS systems in the Gymnasium to behave independently
b) Not having to seismic upgrading the existing wood deck diaphragm in the Gymnasium
c) Reducing the design connection forces in the diaphragm
d) Delaying the seismic upgrading of the roof diaphragm over the Classroom block

7.3 Some of the concepts and provisions presented in the Guideline are not obvious or intuitive and careful consideration and engineering judgement is recommended.

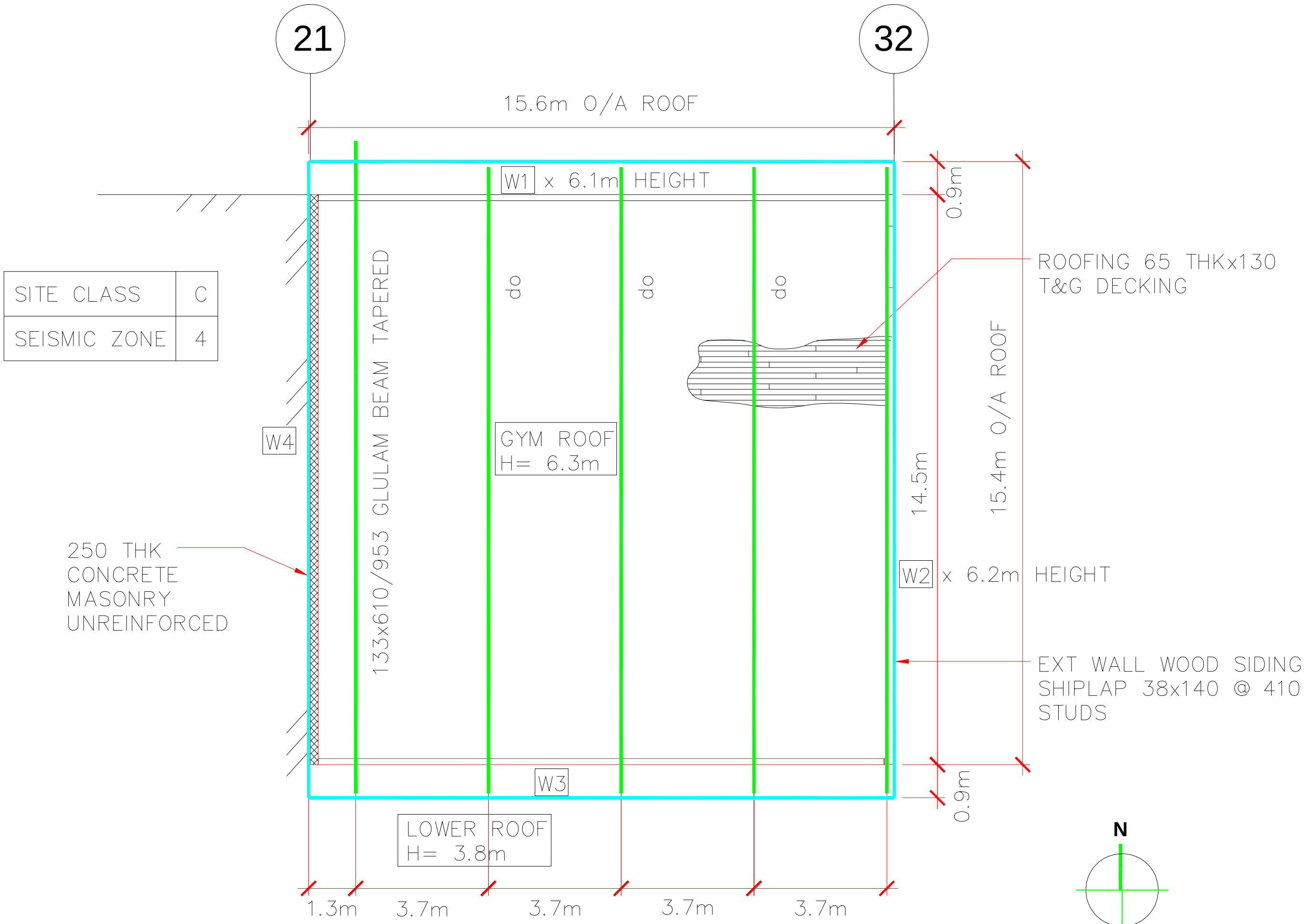
7.4 Application of the 2nd edition Guidelines to the seismic upgrading of Ellendale Elementary school resulted in significant cost savings in

a) Allowing a mixed LDRS systems in the Gymnasium to behave independently
b) Not having to seismic upgrading the existing wood deck diaphragm in the Gymnasium
c) Reducing the design connection forces in the diaphragm, and
d) Delaying the seismic upgrading of the roof diaphragm over the Classroom block until the re-roofing of the Classroom Block
e) Delaying the seismic upgrading of moderate height unreinforced concrete masonry walls until re-roofing of the Classroom Block

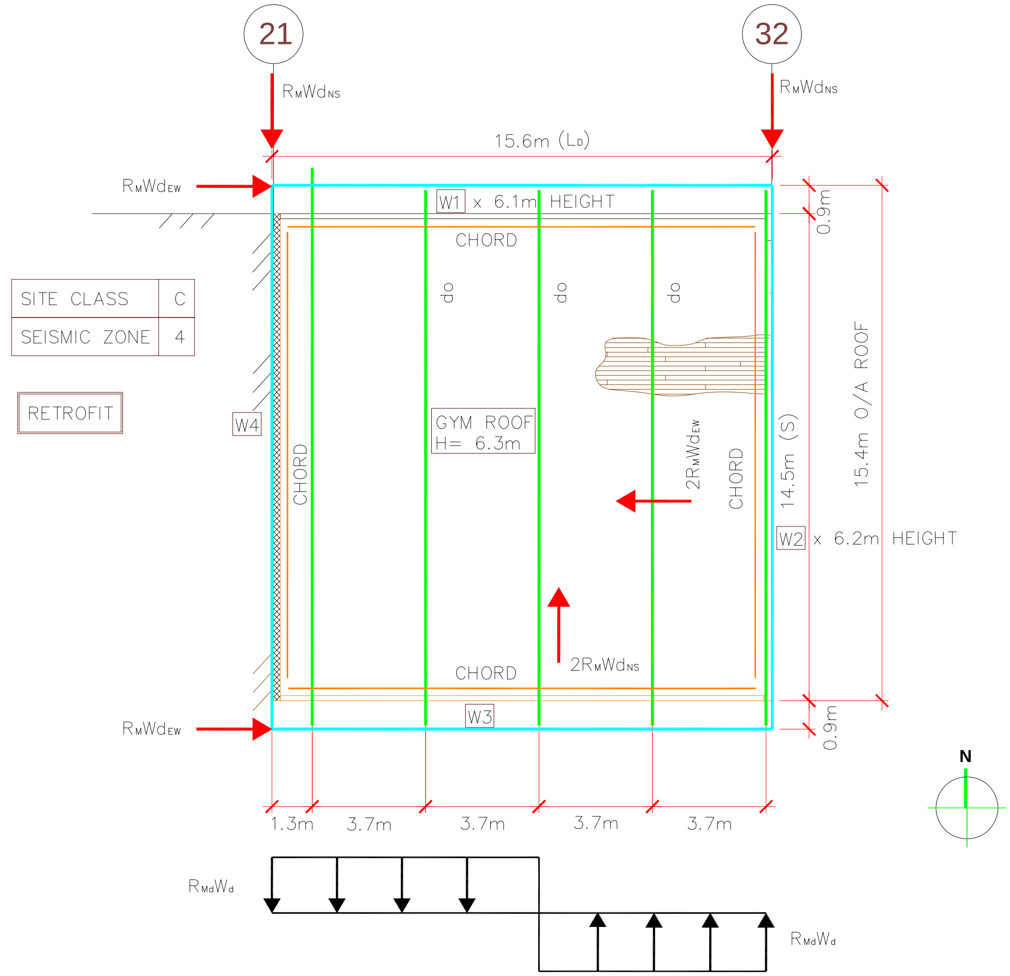
7.5 The cost estimate for seismic upgrading “Option B” selected for Ellendale developed under the 2nd edition Bridging Guidelines is $ 595,100.00. Our preliminary cost estimate for the seismic upgrading of Ellendale Elementary School if developed under the current requirements of the 2006 BCBC would be in the order of $ 850,000 to $ 950,000.

APPENDIX A

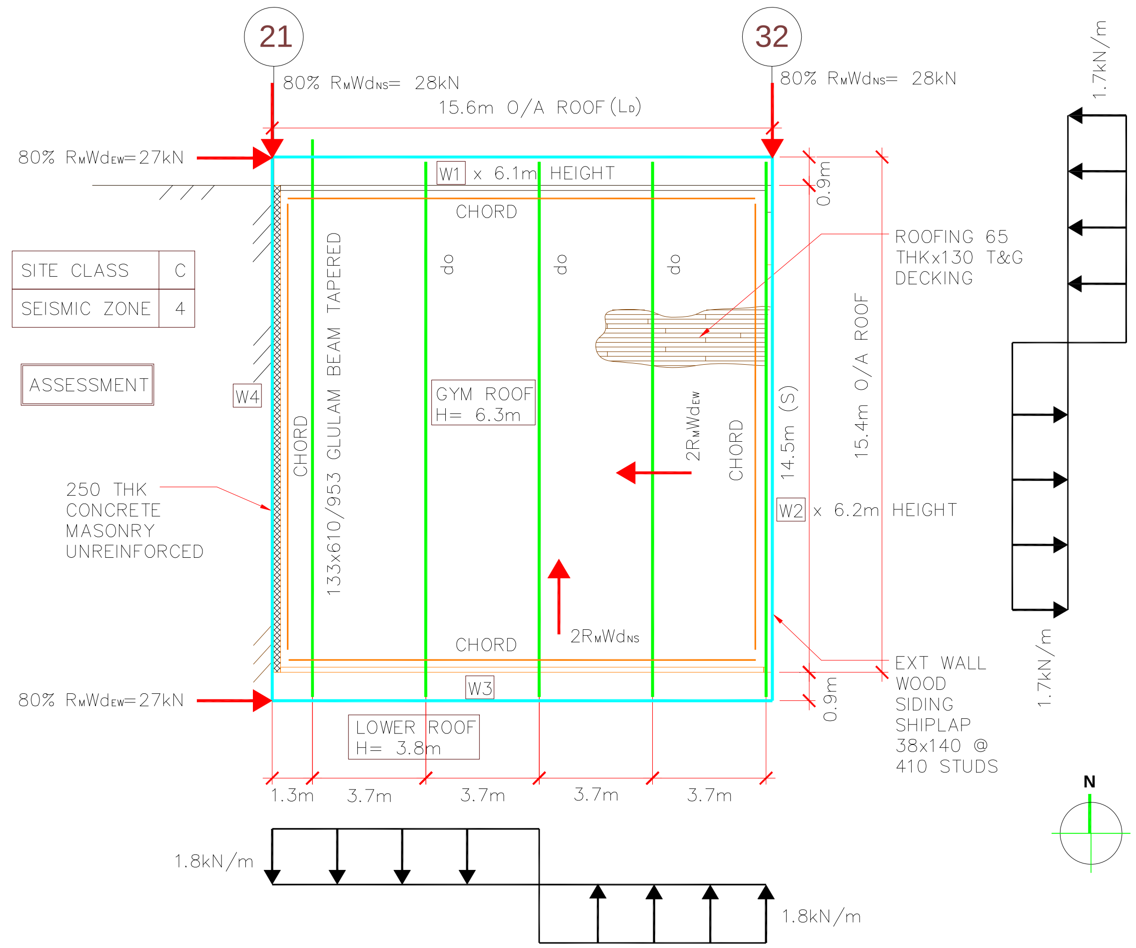
SAMPLE CALCULATIONS



BG-2 Reference	Comments	Results
	Assembly Weights	
	Roof Area = (6.1m)*(17.5m)	282 m^2
	Roof dead load	0.72 kPa
	Roof snow load x 25%	0.56 kPa
	Dead load typical exterior Gym walls	0.48 kPa
	Dead load concrete masonry Gym wall	2.90 kPa
	Roof Diaphragm Weights	
10.2 (1)	Based on roof dead + 25% snow + tributary dead load of walls perpendicular to earthquake direction	
	Diaphragm weight for NS earthquake	W_{dNS} =434 kN
	Diaphragm weight for EW earthquake	W_{dEW} =420 kN
	Wall Weights (full height)	
	W_1 = (0.72 + 0.56)(282)	W_1= 361 kN
	W_2 = 0.48(16.1)(6.1)	W_2 = 47 kN
	W_3 = 0.48(16.1)(6.1)	W_3 = 47 kN
	W_4 = 2.9(15.7)(6.2)	W_4 = 282 kN

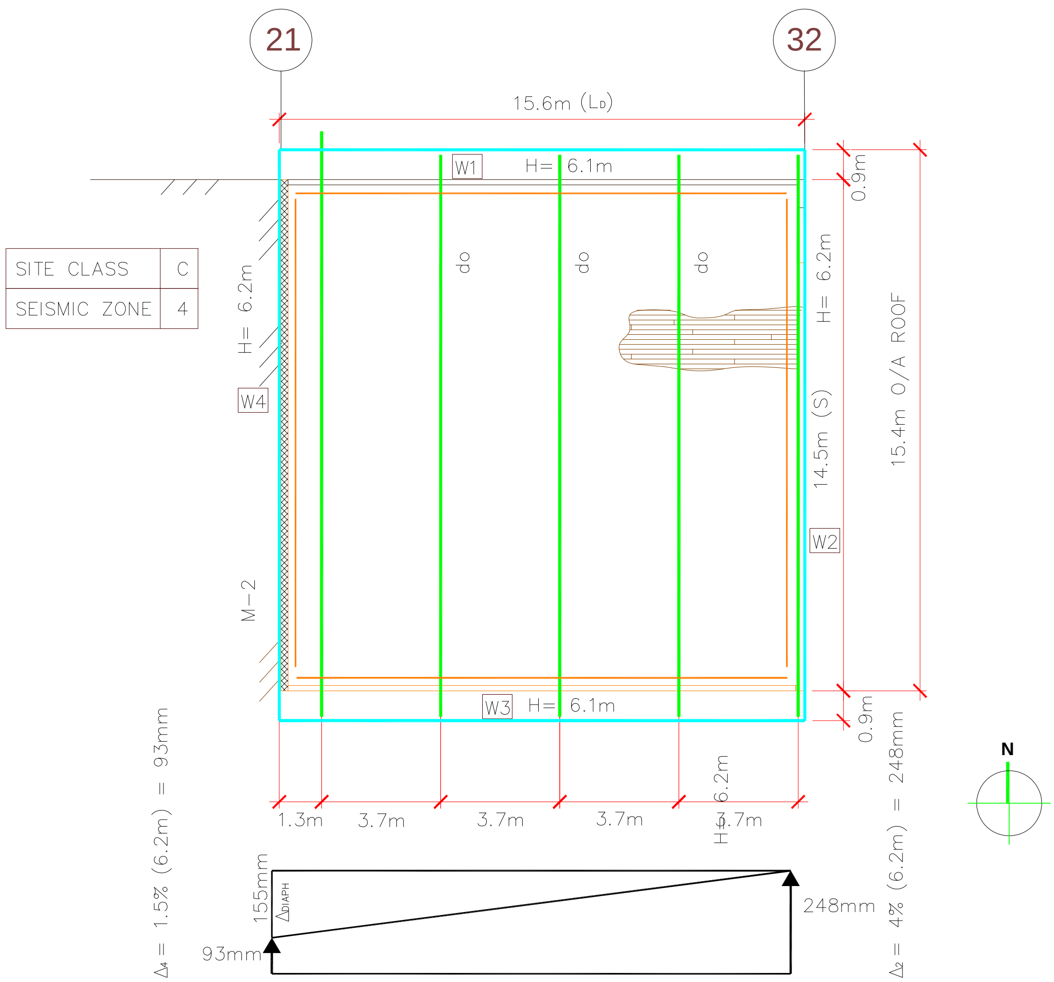


BG-2 Reference	Comments	Results
	Diaphragm chord design	
Connection retrofit presentation pg 8	Dynamic analysis indicates that for a flexible wood diaphragm the shear distribution is closer to a rectangular shape rather than a triangular shape. This results in double the chord force compared to a triangular shaped shear distribution.	
	Diaphragm Bending Moment, $M = \frac{PL_d}{4} = \frac{2R_mW_dL_d}{4}$	
	Diaphragm Chord Force, T=C	
	$T = C = \frac{M_d}{S} = \frac{2R_mW_dL_d}{4*S}$	
	NS Direction $T=C = \frac{2(35kN)(16.1m)}{4(15.7m)} =$	18.0kN
	EW Direction $T=C = \frac{2(35kN)(16.1m)}{4(15.7m)} =$	16.5kN
	Provide new diaphragm chord member using new steel angle, steel plate and wood depending on connection conditions.	



B-G Reference	Comments	Results
	Existing Diaphragm Assessment	
P-2	Existing Gym roof diaphragm consists of 64mm THK x 130 T&G wood decking max span 3.7m over glulam roof beams. Decking is side spiked with 5.4mm diameter x 200 LG spikes for assessment purposes. The gym roof diaphragm can best be classified under prototype D-2 unblocked diaphragm as a very flexible wood diaphragm.	Diaphragm Prototype D-2 Unblocked Sheathing
10.3	Min. required lateral factored resistance Rmd for risk assessment would be taken as 80% of Rmd from FIG 10-2(c).	
Figure 10-2(c)	Fig 10-2(c) D2 zone 4 site class C Rmd = 8% for approximate 15m diaphragm span.	Rmd = 8%
	Minimum required retrofit factored resistance is 80% of Rmd. ie. 80% (8%) W_d = 6.4 % Wd	6.4%Wd
	Ignoring torsion, minimum factored retrofit diaphragm resistance required:	
	NS earthquake 6.4% W_{dNS} = 6.4%(434kN)	28kN
	EW earthquake 6.4% W_{dEW} = 6.4%(420kN)	27kN

	The existing wood deck roof diaphragm is considered to have acceptable strength if the following conditions are met:	
	a) Bldg with site class C	Yes
	b) LDRS on four sides of diaphragm	Yes
	c) Max diaphragm span 15m	Max 15.4 m diaphragm clear span-close enough
	d) Deck is at least 64mm THK	Yes
	e) Deck spikes confirmed at max. 1000mm o/c	Yes, decking side spiked at max 765 mm +/- 253 mm o/c confirmed by pachometer testing
	Roof diaphragm shears:	
	NS direction $V_{NS} = \frac{28kN}{15.7m} =$	1.8kN/m
	EW direction $V_{EW} = \frac{27kN}{16.1m} =$	1.7kN/m
	Check factored resistance of horizontal diaphragms with 64mm thick spiked wood decking as listed in the 2004 BC School Seismic Assessment 2004 reference sheet	
	$V_r = 2.0kN/m > V_{NS} = 1.8kN$ $> V_{EW} = 1.7kN$	OK OK

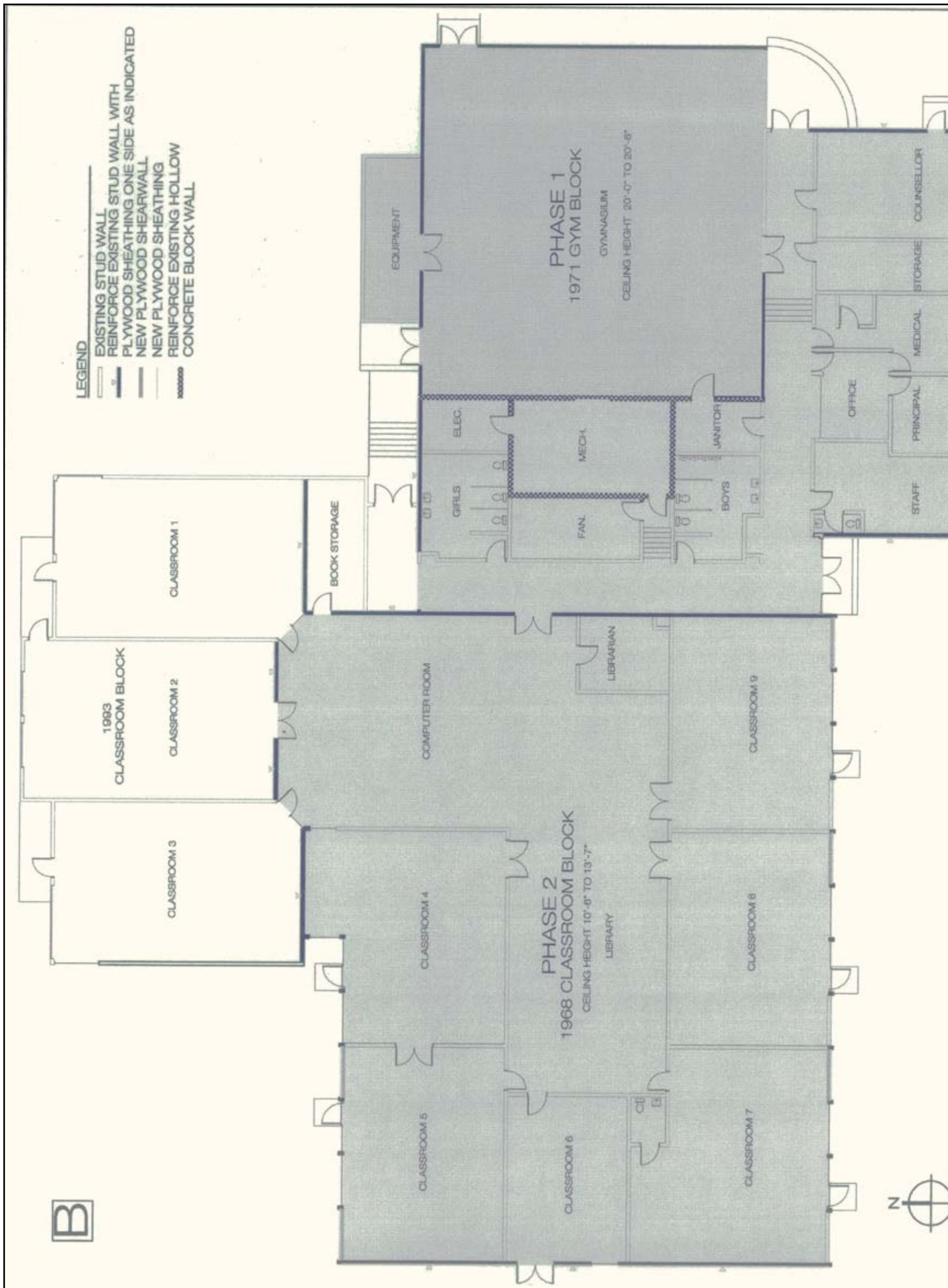


BG-2 Reference	Comments	Results
10.9	**Mixed LDRS system**	
	In the case of mixed LDRS systems, governing drift is typically limited by the system with the lowest drift limit in NBC 2005.	
	In NS direction of gym governing drift limit would be 1.5% due to W4 being a masonry M-1 LDRS with an ISDL of 1.5% VS wood W-1 being a wood W-2 LDRS with an ISDL of 4%.	
	BG-2 allows a flexible diaphragm to distort to an inelastic strain limit of 1% allowing different LDRS to behave independently of each other with different drifts.	
Section 11.3 Errata March 2007	**Diaphragm Connections** Minimum factored resistance of a connection loaded in shear (diaphragm to LDRS shear) or tension (out-of-plane) in the wood diaphragm is Rmc = Red/nc where Red is resistance of diaphragm nc is number of connections.	
	Red for existing Gym roof deck diaphragm = 2.0 kN/m.	

	Therefore, for in-plane shear connection between roof diaphragm and LDRS, Rmc = Red = 2.0kN/m	
	For out-off-plane connection, in NS direction, one half of the diaphragm will resist the out-of-plane forces. Red = 2.0kN/m (5.6 m/2)(2 sides) = 11.2 kN Therefore, Rmc = 11.2 kN/15.7 m long wall = 0.7 kN/m. In plane shear connection force governs.	Rmc = 2.0 kN/m
	Use nailed connections using 4" common nails (4.88 mm diameter) Nr = 1.44 kN(J_D=1.3) = 1.87 kN	

APPENDIX B

SEISMIC UPGRADING OPTION B
CONCEPT PLANS



Preliminary Seismic Upgrading Option B - Main Floor Plan

APPENDIX C

SEISMIC UPGRADING DETAILS

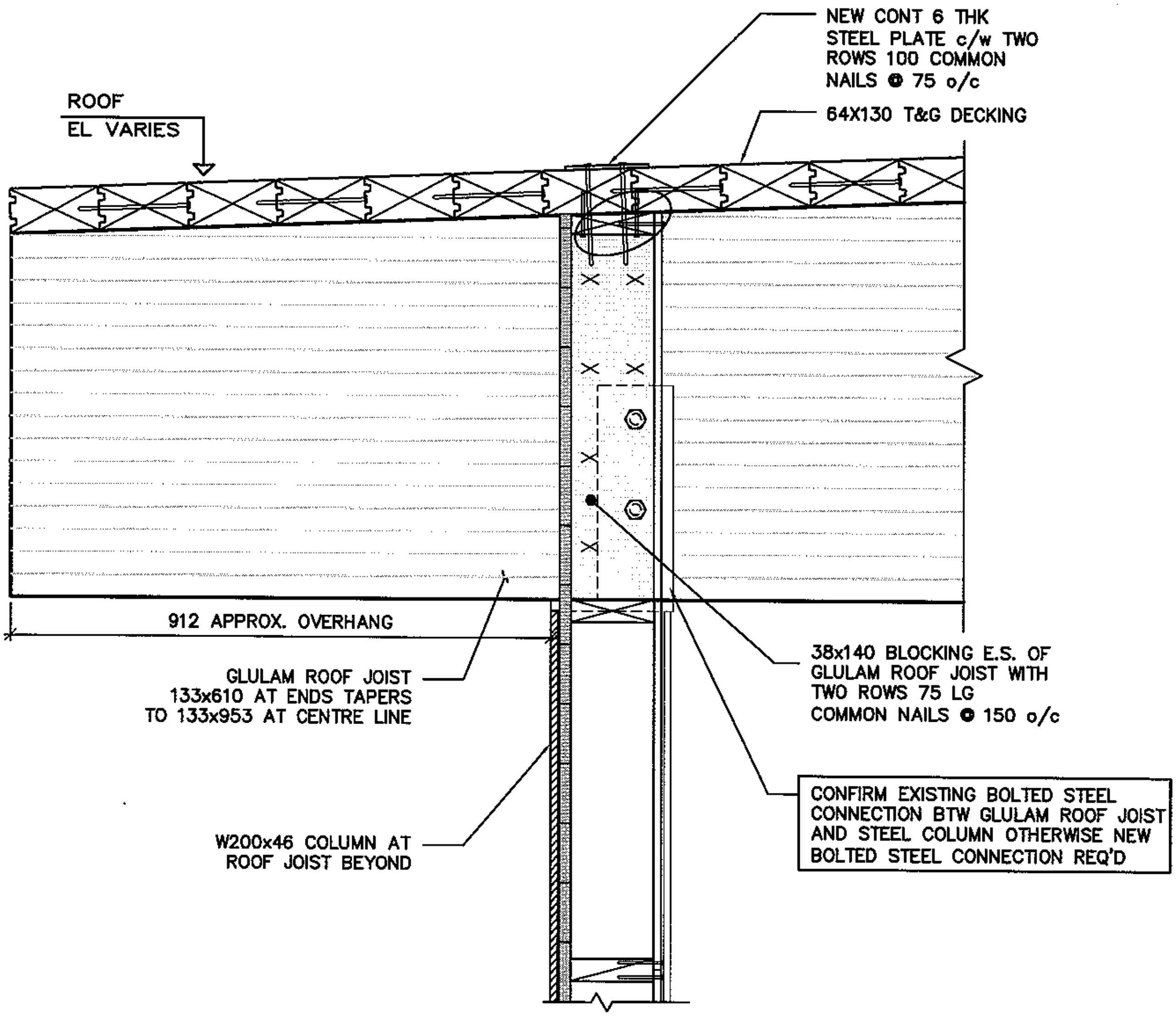


Figure 1 Gym Wall Perpendicular to Roof Joists

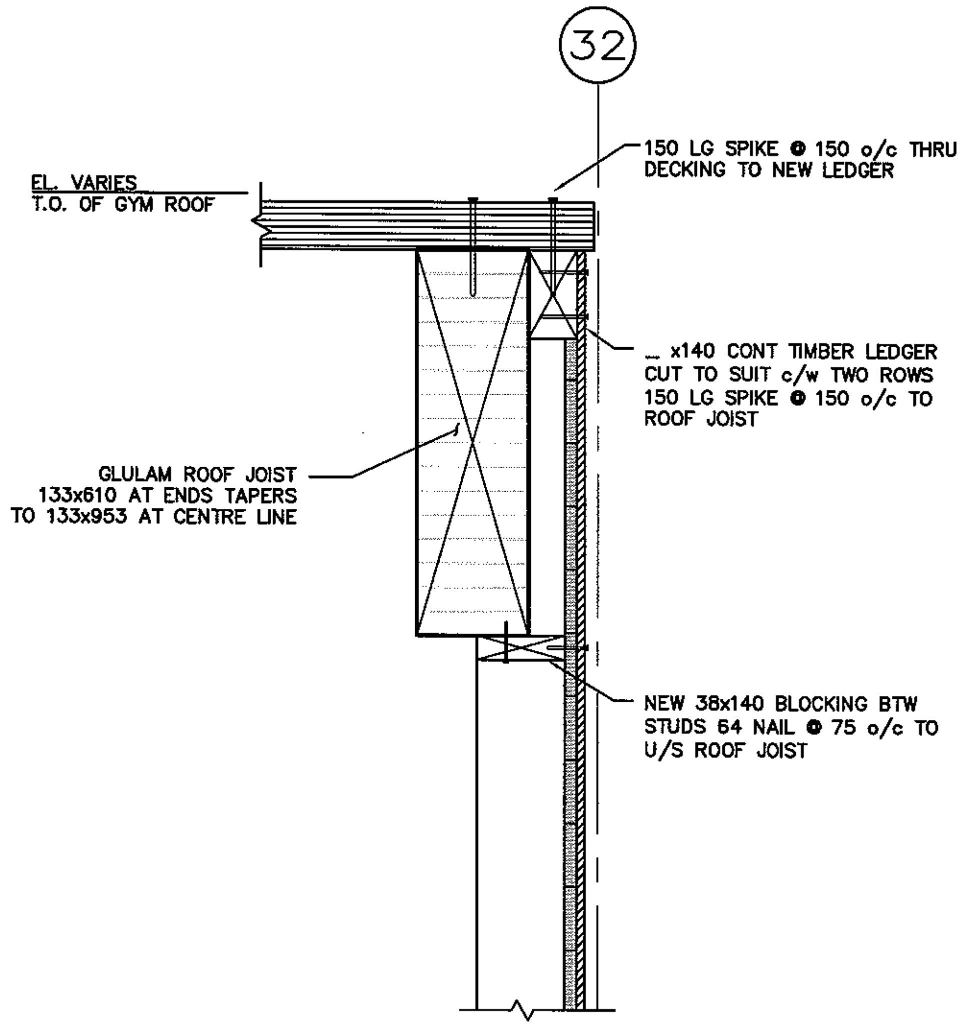


Figure 2 Gym Wall Parallel to Roof Joists

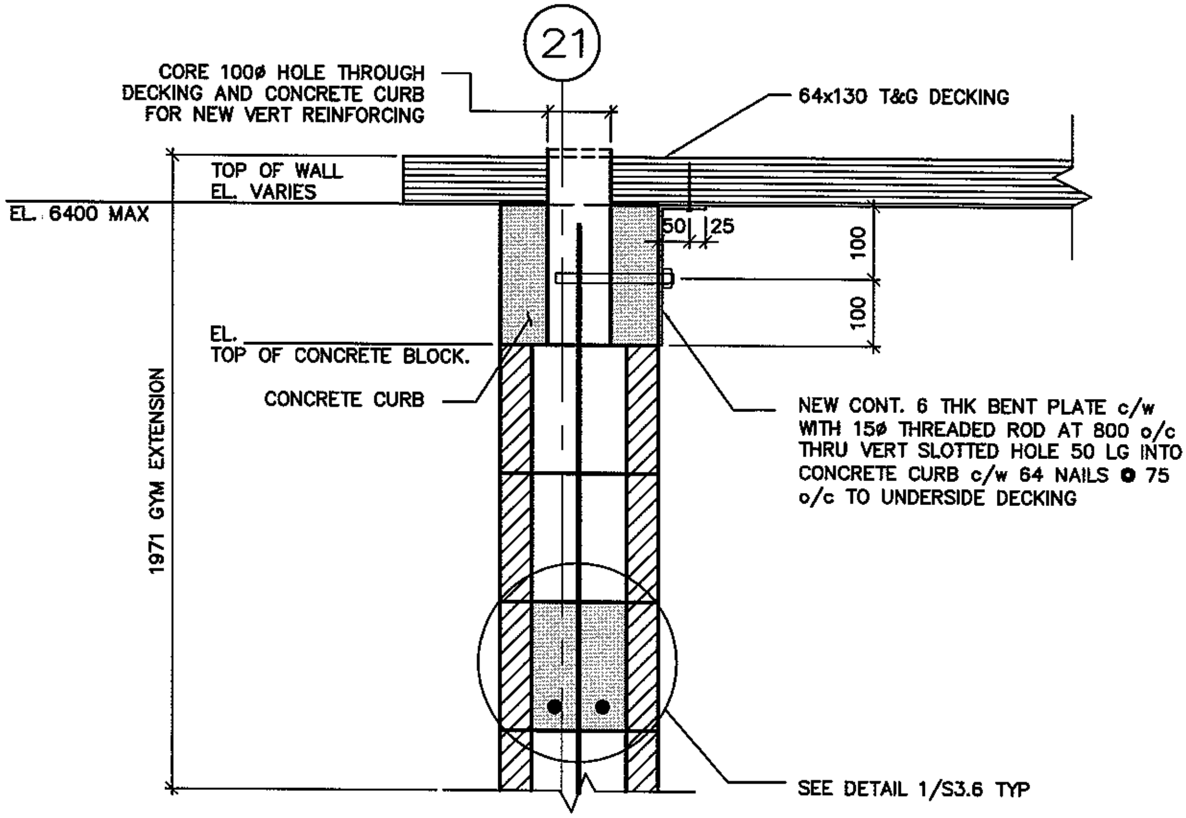


Figure 3 Concrete Block Wall

APPENDIX D

SITE PHOTOGRAPHS

Figure 1 Exterior of Gymnasium

Picture 2 Exterior View of Roof

Figure 3 Elevation of concrete masonry wall in Gymnasium

Figure 4 Hole in concrete foundation wall supporting concrete masonry wall in Gym

Picture 5 Crawlspace under Classroom Block

Bridging Guidelines
Second Edition

Demonstration Project No. 6

for

SEISMIC RETROFIT OF HEAVY PARTITION WALLS FOR MT. DOUGLAS SENIOR SECONDARY SCHOOL

(School District No. 61 - Greater Victoria School District)

Prepared by:
TBG Seismic Consultants Ltd.

March, 2007

1.0 EXECUTIVE SUMMARY

This document is one in a series of reports that have been prepared by British Columbia consulting engineering firms engaged in the seismic retrofitting of provincial schools as part of the British Columbia Ministry of Education's $1.5 billion seismic mitigation program.

This report describes the proposed seismic retrofitting of heavy partition walls in the Mt. Douglas Senior Secondary School in Saanich (Greater Victoria School District). Highlights of this report include:

(1) Elimination of the need for wall reinforcement for the out-of-plane behaviour of unreinforced concrete masonry walls up to 4.1 metres in height.

(2) Minimizing the disruption to wall-mounted teaching aides

(3) Maximizing off-site fabrication to minimize on-site installation so that work can be completed in two summer periods

(4) 60% reduction in construction costs when compared with costs for code-based retrofit designs

2.0 PROJECT DESCRIPTION

2.1 Introduction

Mt. Douglas Secondary is a large secondary school in Saanich, British Columbia, and is one of the flagship schools in the Greater Victoria School District (School District No. 61). The school comprises a two storey classroom building with additional one storey classrooms and a gymnasium. The total floor area of the school is approximately 9,400 m2. The school was built in 1960 and has had at least eight major additions over the 1965-1993 period.

This report describes the proposed seismic retrofitting of the heavy unreinforced concrete masonry partition walls in the two storey classroom block of the school. This block was built in 1960, has a floor area of 6,620 m2 and has seven additions constructed in 1965, 1966, 1968, 1969, 1972, 1990 and 1993.

The primary lateral structural system of the two storey classroom block is comprised of a non-ductile concrete frame, a tongue-and-groove wood roof and unreinforced concrete masonry parition walls.

The purpose of this demonstration project is to contribute to the building of a library of school retrofit projects that can be used by engineer practitioners as a valuable reference resource in future school retrofit projects.

2.2 Block Photographs

Photographs of the two storey classroom block (Block #49-1) exterior and interior are given in Figures 1-3.

2.3 Heavy Partition Walls

Sketches of the heavy partition walls are given in Figures 4-8.

A basic description of the heavy partition walls is as follows:

(a) Unreinforced concrete masonry first constructed in 1960 and built of 140 mm thick and 190 mm thick masonry units

(b) Five basic types of partition walls (refer to Figure 4):

- (i) First storey infill walls (2946 mm high)
- (ii) First storey corridor walls (3505 mm high)
- (iii) Second storey partial height walls (2438 mm high, 140 mm thick)
- (iv) Second storey corridor walls (4115 mm high)

(v) First and second storey exterior spandrel walls (914 mm high)

2.4 Retrofit Schedule

The overall schedule for the seismic retrofit of the heavy partition walls in Mt. Douglas Secondary School is as follows:

(1) Completion of Stage 1 report - November, 2005
(2) Completion of Stage 2 report - October, 2006
(3) Start of construction - June, 2007
(4) Completion of construction - August, 2009

3.0 SEISMIC ASSESSMENT

3.1 Stage 1 Feasibility Study

The primary conclusions of the Stage One feasibitity report were as follows:

(1) Level of Risk

Block #49-1 was assessed as a very high risk by virtue of no lateral support for unreinforced concrete masonry cantilever partition walls that are 2.4-4.1 metres in height.

(2) On-going Educational Instruction

GVSD confirmed that this block of the school would be required for on-going educational instruction.

(3) Stand-alone Seismic Project

The seismic upgrade of the school and this block in particular were confirmed to be less than 70% of the replacement cost.

(4) Municipal Approval

Consultation with the Municipality of Saanich is on-goingto obtain municipal cooperation with this voluntary seismic upgrade.

3.2 Stage 2 Feasibility Study

The highest seismic risk components in Block #49-1 are the unreinforced concrete masonry cantilever partition walls.

A combination of the following three retrofit options were considered for upgrading the unreinforced concrete masonry partition walls:

(1) Grouting at the top of in-fill walls to ensure adequate confinement
(2) Installation of horizontal steel support beams close to the top of cantilever walls
(3) Reinforcement or external steel straps for improved out-of-plane performance

Installation of vertical bracing elements was considered impracticable because of the large number of cupboards, counter tops and teaching aides attached to the partition walls.

3.3 Geological/Site Hazards

There are no significant geologic/site hazards (low risk of liquefaction or land slide) at this school site.

3.4 Site Response Analysis

The Mt. Douglas Secondary site has been assessed as a site with soil profile characteristics on the boundary between Site Class C and Site Class D site classifications. Therefore, this site is a candidate for a site response analysis. At the time of preparation of this demonstration project, preliminary site response analysis indicated a Site Class C response. A formal site response analysis will be completed upon commencement of the final design.

The retrofit measures proposed in this report for the heavy partition walls are relatively independent of the site classification (Site Class C/D).

3.5 Hazardous Materials

The classrooms and small offices of Block #49-1 have asbestos-containing material in the drywall joint filler and in the floor finish. Our proposed retrofit has been designed to minimize disturbance of these hazardous materials.

3.6 Seismic Risk

As noted in Section 3.1(1), our assigned level of risk for the unreinforced concrete masonry partition walls is very high risk by virtue of no lateral support of the walls and no wall reinforcing.

4.0 PROPOSED SOLUTION

4.1 Retrofit Philosophy and Strategy

The sole focus of this proposed seismic upgrading is collapse prevention. After the design earthquake, heavy damage may well be inflicted on the heavy partition walls. The retrofit design is acceptable if the threat to life safety is substantially reduced and if egress from the building is not unduly impeded.

Conceptually, seismic retrofits have been developed to minimize interference to educational operations of the school. Restraint of the interior masonry walls comprises mostly off-site shop fabrication (steelwork) with fast, piece-small on-site installation. This approach eliminates the need for swing space during the construction period (summer construction only).

4.2 Retrofit Concept

The proposed seismic upgrade concepts for the five different types of heavy partition walls listed in Section 2.3 are as follows:

(1) First Storey Infill Walls

(a) Wall Description: Refer to Figure 4 for location of the first storey infill walls. Walls are comprised of unreinforced masonry 2946 mm high and 190 mm thick.

(b) Retrofit Concept: (i) Grout top of walls at interface with concrete beam (ii) Install restraint plates at top of wall (iii) Remove two top corner blocks and replace with compressible material

(c) Retrofit Strategy: Take advantage of stiff confining concrete construction at top and bottom of walls. Grouting at the top of the walls will ensure confinement will be generated at the onset of out-of-plane rocking of the walls. The restraint plates provide additional lateral support for the top of the wall. Removal of the top two corner masonry units will prevent development of high strength compression struts that could fail the columns or could cause hazardous heavy local damage in the corner masonry units.

(2) First Storey Corridor Walls

(a) Wall Description: Refer to Figure 4 for location of the first storey corridor walls. Walls are comprised of unreinforced masonry 3505 mm high and 190 mm thick.

(b) Retrofit Concept (refer to Figure 5): (i) Grout top of walls at interface with second floor slab (ii) Install one-sided clip angles to support top of wall (iii) Install exterior steel straps either side of one door

(c) Retrofit Strategy: Grouting at the top of the walls will provide reasonable assurance that confinement will be generated at the onset of out-of-plane rocking of the walls. The restraint plates provide additional lateral support at the top of the wall. The external steel straps will enhance egress by mitigating the severity of masonry damage either side of the exit door.

(3) Second Storey Partial Height Walls

(a) Wall Description: Refer to Figure 4 for location of the second storey partial height partition walls. Walls are comprised of unreinforced masonry 2438 mm high and 140 mm thick.

(b) Retrofit Concept (refer to Figure 6): (i) Install horizontal steel restraint beam close to top of wall.

(c) Retrofit Strategy: Provide stiff lateral support at top of wall to ensure out-of-plane stability.

(4) Second Storey Corridor Walls

(a) Wall Description: Refer to Figure 4 for location of the second storey corridor walls. Walls are comprised of unreinforced masonry 4115 mm high and 190 mm thick.

(b) Retrofit Concept (refer to Figure 7): (i) Install horizontal steel restraint beam 900 mm below top of wall (ii) Install steel strap to provide out-of-plane stability of 600 mm high cantilever wall above steel restraint beam

(c) Retrofit Strategy: Provide stiff lateral support close to top of wall to ensure out-of-plane stability. Only a steel plate is required to provide out-of-plane rocking stability for the cantilever portion of the wall above the restraint beam.

(5) First and Second Storey Exterior Spandrel Walls

(a) Wall Description: Refer to Figure 4 for location of the exterior spandrel walls on the first and second storeys. These cantilever walls are comprised of unreinforced masonry 914 mm high and 190 mm thick.

(b) Retrofit Concept (refer to Figure 8): (i) Install exterior horizontal steel strap close to top of wall for second storey walls only

(c) Retrofit Strategy: Only a steel strap is required to provide out-of-plane rocking stability for the relatively low height exterior cantilever walls above the concrete floor slab.

4.3 Retrofit Details

The preliminary retrofit design of the seismic retrofit of the heavy partition walls is as follows:

(1) First Storey Infill Walls

If the first storey infill walls are to crack at mid-height and rock out-of-plane along the mid-height crack, the top edge of the top masonry units will need to rise 24 mm vertically. The grouting of any space between the top of the masonry wall and the underside of the concrete beam will ensure that only nominal out-of-plane rocking will occur before large confinement forces are generated to provide out-of-plane stability (arching action will prevent out-of-plane failure).

In the unlikely event that a concrete column might fail, the infill masonry wall acts as a secondary vertical supporting element. The restraint plates are installed to provide added lateral support at the top of the wall to further enhance the ability of the damaged wall to
provide vertical support when necessary.

(2) First Storey Corridor Walls

The second floor concrete slab does not provide as stiff a confinement as the concrete beams above the first storey infill walls. However, it is anticipated that confinement will be sufficient to provide out-of-plane stability. The one-sided clip angles are intended to provide additional lateral support.

The purpose of the vertical steel straps either side of the door is to provide some added integrity to the masonry encasing the doorway, thereby reducing the probability of egress obstruction.

(3) Second Storey Partial Height Walls

The steel restraint beams only need to generate sufficient restraint force to prevent cantilever rocking.

For a cantilever wall 2438 mm high and 140 thick, the horizontal restraint force in the horizontal steel restraint beam 2138 mm above the floor
$= 1.7 \text{ kPa} \times 2.438 \times 0.07 \div 2.138$
$= 0.14 \text{ kN/m length}$

Therefore, the maximum lateral bending moment in HSS restraint beam up to 8.5m long
$= 0.14 \times 8.5 \times 8.5 \div 8$
$= 1.3 \text{ kNm}$

The yield moment capacity of the HSS 102x102x4.8

$= 0.9 \times 350000 \times 0.00004$

$= 12.6$ kNm >> 1.3 kNm - OK

(4) Second Storey Corridor Walls

The second storey corridor walls are restrained by a horizontal HSS beam approximately 3.5 m above the floor. A horizontal plate is provided at the second top masonry unit level to ensure stability of the portion of the masonry wall above the HSS.

The size of the HSS beam is oversized to provided the equivalent of a surcharge on top of the wall equal to at least 50% of the weight of the wall below the HSS. This equivalent surcharge provides an added degree of stability for the wall rocking out-of-plane. The HSS beam has sufficient strength to carry a vertical load equal to a 1,75m height of the masonry wall (including torsional effects).

Refer to the exterior spandrel beam details for the design check of the restraint plate 75x6.

(5) First and Second Storey Exterior Spandrel Walls

The horizontal force required to restraint a exterior spandrel wall (0.9 m high) against rocking with the restraint force applied 700 mm above the second floor slab

$= 0.9 \times 2.1$ kPa $\times\, 0.095 \div 0.7$

$= 0.26$ kN/m

The steel restraint plate 75x6 relies on catenary action to provide lateral restraint. Assume the restraint plate has constant curvature. If the restraint plate deflects 10 mm midway between columns (4877 mm apart), the maximum lateral restraint force generated by the restraint plate is 0.35 kN/m. Therefore, the spandrel wall will rock no more than 10 mm laterally.

4.4 Laboratory Testing

CSA S304.1-04 does not permit the retrofit methods proposed in Section 4.2 for a school located on Site Class C soils in Victoria on two counts; high seismicity ($Sa(0.2) > 0.75$) and high walls (> 3m).

Out-of-plane analysis indicates that the retrofit methods proposed in Section 4.2 are safe. We propose that UBC conduct full scale tests to verify acceptable performance. The testing protocol will yield test data for seismic zones 3-5.

5.0 COST BENEFIT ANALYSIS

This section provides a cost comparison between code-based retrofits and the retrofits proposed in this report. The code-based retrofits are designed for an Importance Factor of 1.0 and with no reference to the Bridging Guidelines.

(1) First Storey Infill Walls

- Wall area - 300 m2
- Code retrofit- $80/m^2$ for reinforcement
  - $160/m^2$ for removal/replacement of surface-mounted teaching aides
- Proposed retrofit - $20/m^2$.
- Site overheads including construction management (no professional fees) - 20%
- Retrofit cost estimates - $86,000 (code)
  - $8,000 (proposed)

(2) First Storey Corridor Walls

- Wall area - 850 m2
- Code retrofit - $80/m^2$ for reinforcement
  - $80/m^2$ for removal/replacement of surface-mounted teaching aides
- Proposed retrofit - $50/m^2$.
- Site overheads including construction management (no professional fees) - 20%
- Retrofit cost estimates - $163,000 (code)
  - $51,000 (proposed)

(3) Second Storey Partial Height Walls

- Wall area - 500 m^2
- Code retrofit - $80/m^2$ for reinforcement
  - $100/m^2$ for HSS restraint
  - $160/m^2$ for removal/replacement of surface-mounted teaching aides
- Proposed retrofit - $100/m^2$.
- Site overheads including construction management (no professional fees) - 20%
- Retrofit cost estimates - $204,000 (code)
  - $60,000 (proposed)

(4) Second Storey Corridor Walls

- Wall area - 1000 m^2
- Code retrofit - \$80/m^2 for reinforcement
  - \$175/m^2 for HSS restraint
  - \$80/m^2 for removal/replacement of surface-mounted teaching aides
- Proposed retrofit - \$175/m^2.
- Site overheads including construction management (no professional fees) - 20%
- Retrofit cost estimates - \$402,000 (code)
  - \$210,000 (proposed)

(5) First and Second Storey Exterior Spandrel Walls

- Wall area - 600 m^2
- Code retrofit - \$120/m^2 for reinforcement
- Proposed retrofit - \$80/m^2.
- Site overheads including construction management (no professional fees) - 20%
- Retrofit cost estimates - \$86,000 (code)
  - \$52,000 (proposed)

In comparative terms, the cost efficiency of the proposed retrofits results in a minimum construction cost reduction of \$560,000 (\$85/m^2) for the seismic upgrading of the heavy partition walls of the main classroom block. This estimate of the reduction in construction cost excludes design contingency, professional fees and construction cost escalation.

6.0 CONCLUSION AND RECOMMENDATIONS

The construction cost efficiency of the retrofit designs proposed in this report and conforming to the Bridging Guidelines is substantial (60% reduction) for the seismic restraint of heavy partition walls. The major contributors to the construction cost efficiency are as follows:

(1) Minimizing disruption of teaching aides (removal and replacement is expensive)

(2) Elimination of reinforcement for out-of-plane behaviour

We recommend that UBC conduct full scale tests to demonstrate the satisfactory performance of the retrofit methods proposed in Section 4.2.

7.0 ACKNOWLEDGEMENTS

This report has been prepared by TBG Seismic Consultants Ltd. who have been retained by the Greater Victoria School District to design and supervise the seismic upgrading for Mt. Douglas Senior Secondary School.

TBG Seismic Consultants Ltd. wishes to acknowledge the contributions from the following agencies and companies in this school seismic upgrade project; the British Columbia Ministry of Education, the Greater Victoria School District, the Association of Professional Engineers and Geoscientists of British Columbia (APEGBC), the University of British Columbia and the APEGBC Peer Review Committee.

Figure 1 – Main Classroom Block

Figure 2 – Wall-Mounted Teaching Aides for 1st Storey Infill Masonry Wall

Figure 3 – Wall-Mounted Shop Equipment

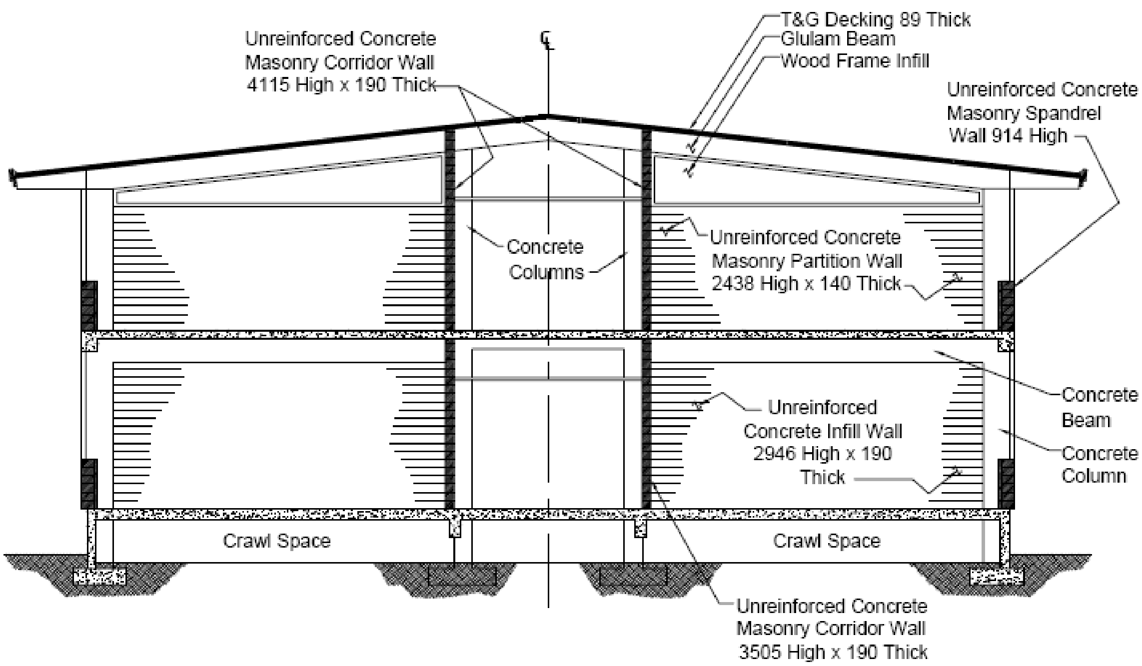


Figure 4 – Typical Section of Main Classroom Block

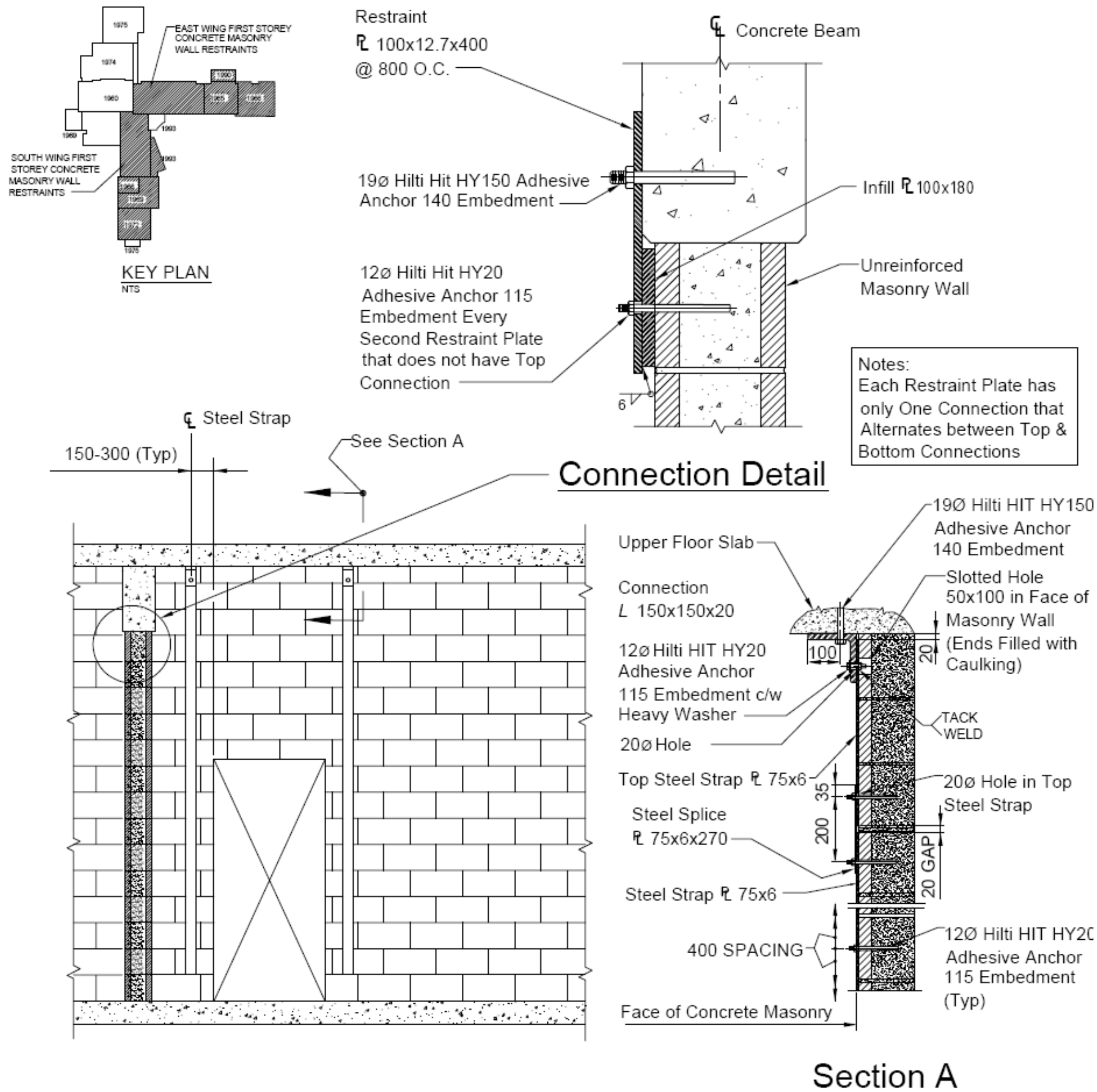


Figure 5 – Retrofit Concept for 1st Storey Corridor Walls

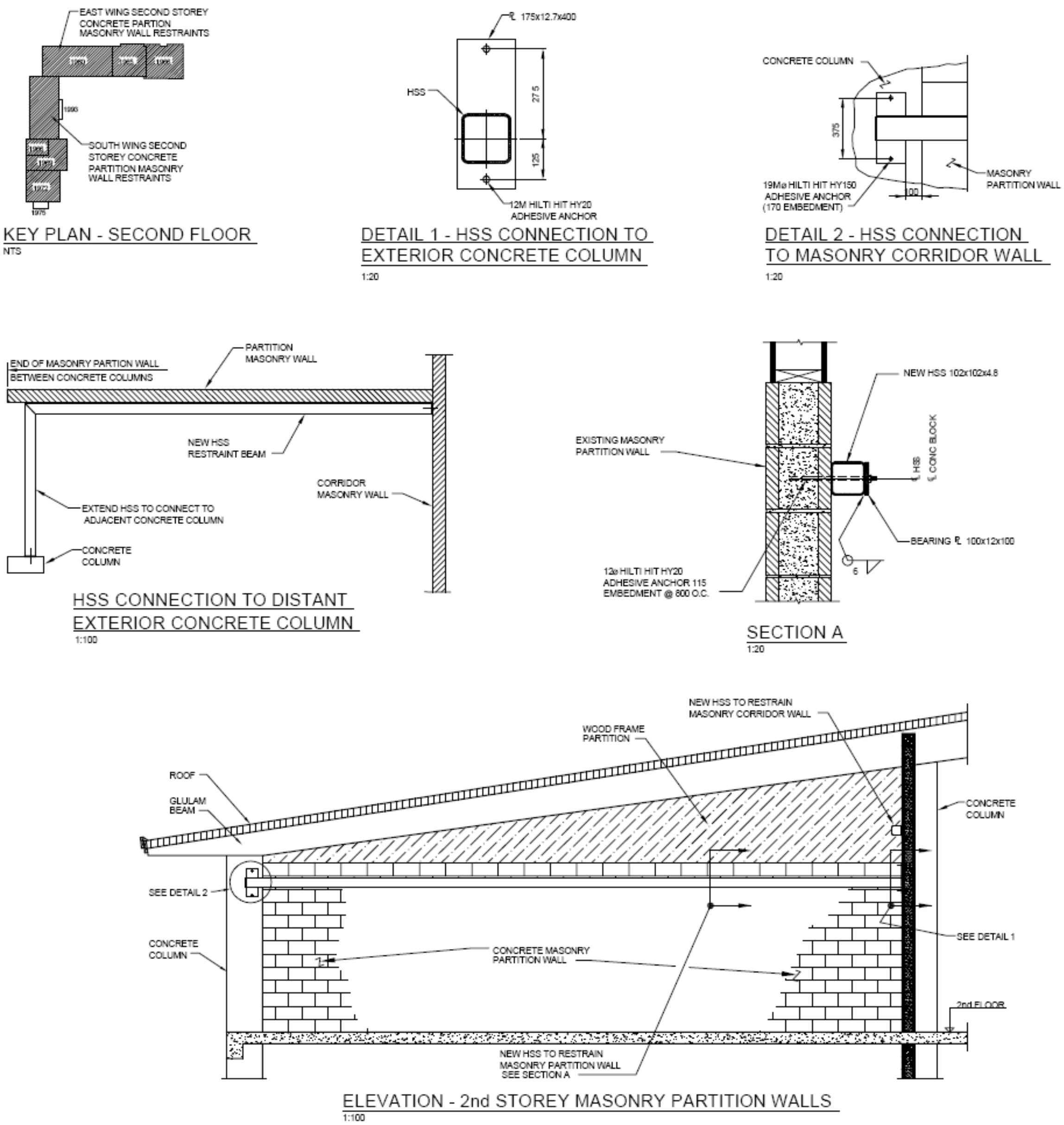


Figure 6 – Retrofit Concept for 2nd Storey Partial Height Partition Walls

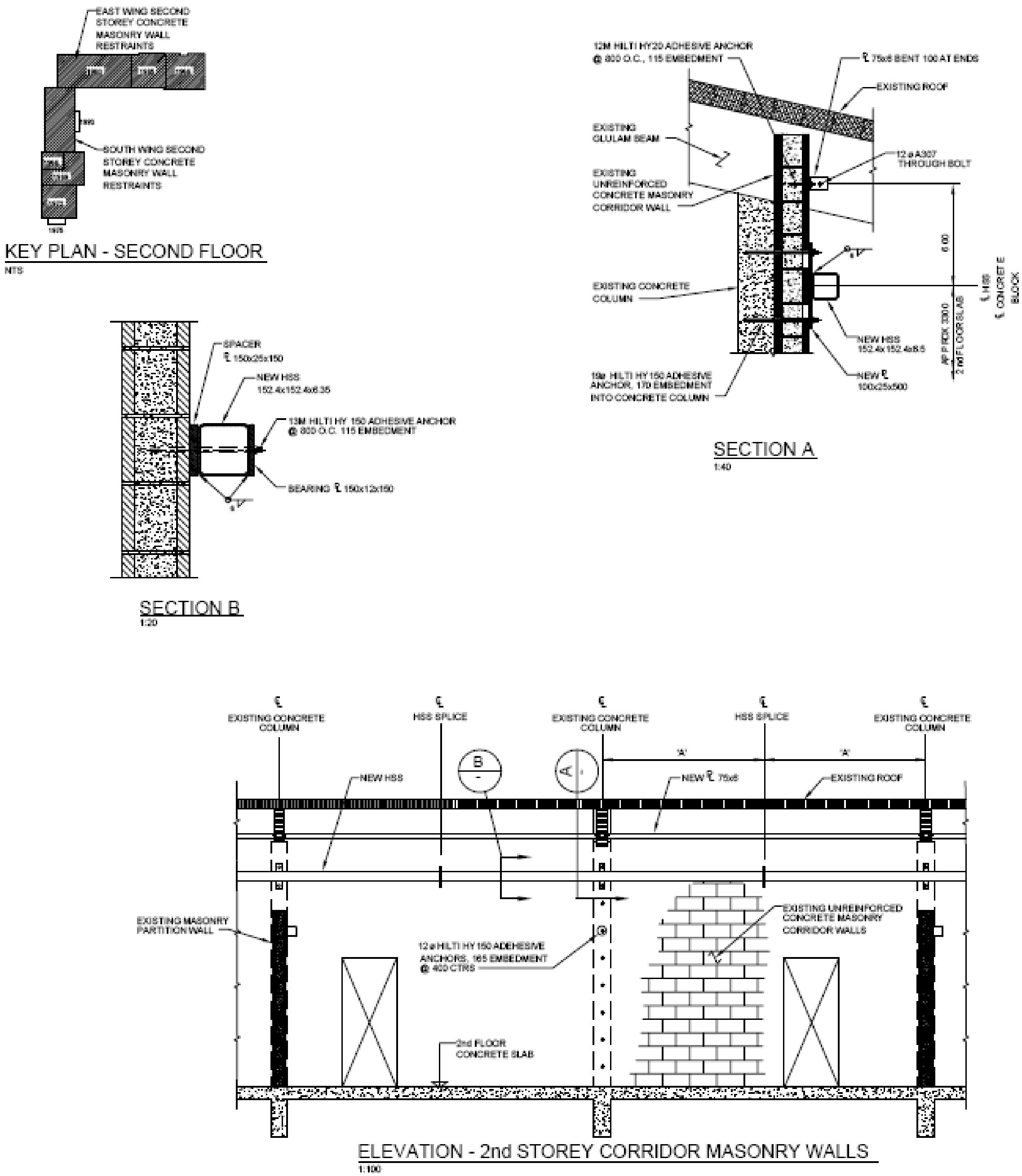


Figure 7 – Retrofit Concept for 2nd Storey Corridor Walls

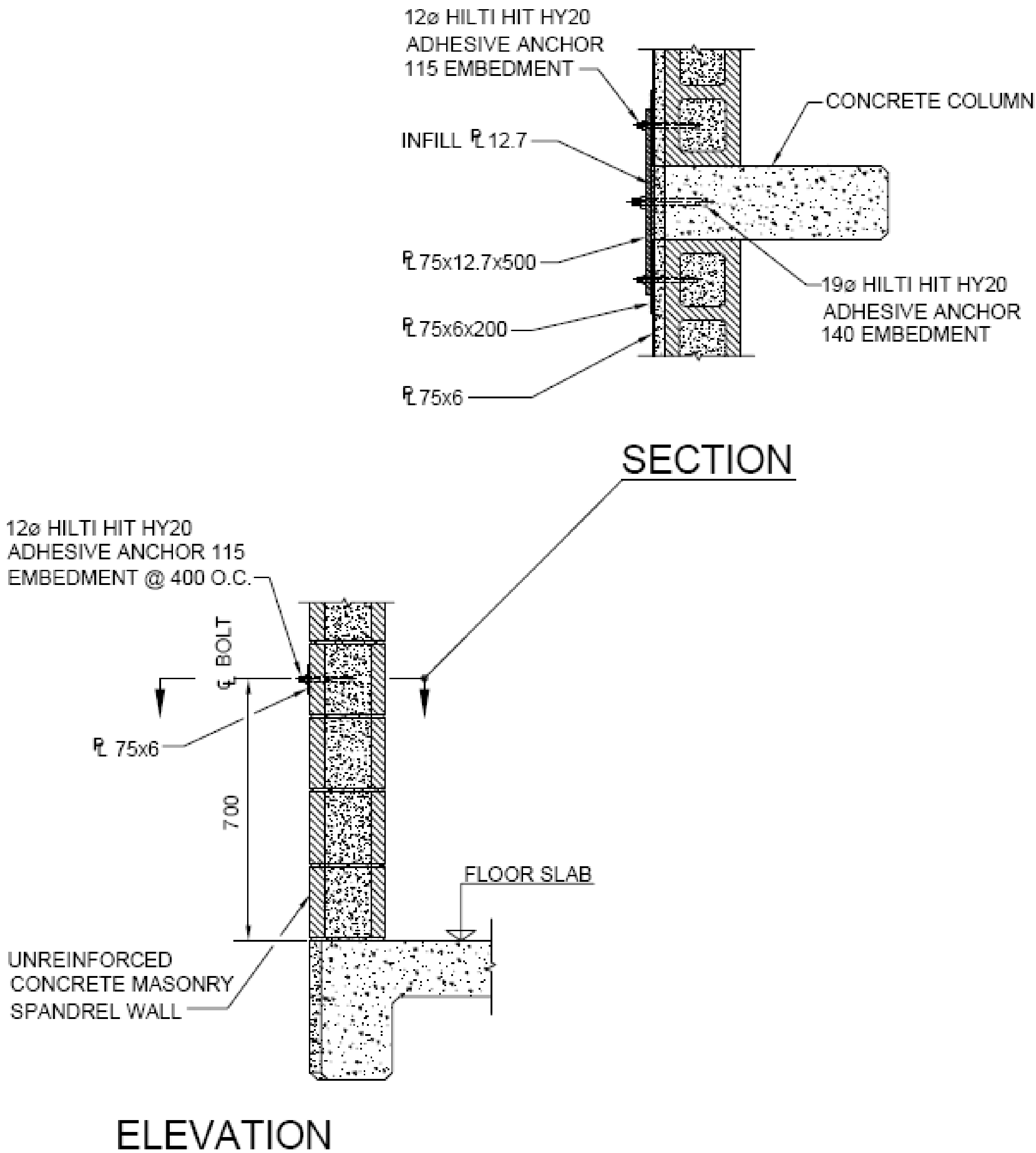


Figure 8 – Retrofit Concept for Exterior Spandrel Walls