

Externally Mounted Ductile Frame, Seismic Retrofit for the British Columbia Institute of Technology SW1 Main Building

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ABSTRACT

The ductile braced, externally mounted steel frame for the seismic retrofit of the BCIT SW1 Main Building presented unique design constraints for the consultant. The new seismic provisions of CAN/CSA-S16.1 1989, evoked on the cross-bracing system, which incorporated HSS diagonals and wide flange beams and columns, provided a member capacity limited frame. As the original structure had significant lateral resistance deficiencies, the retrofit limited the deflections of the existing facilities to ensure non-brittle performance by absorbing the full effects of credible earthquake forces. The main structure is part of a four-building complex arranged in a rectangular pattern forming a central courtyard. The retrofit of these structures used the courtyard as the optimum location for strengthening. Minimizing window coverage and providing a visually unobtrusive retrofit and erectable system within three summer months while meeting the specific ductility requirements for the members and connections necessitated both the design and detailing of all the connections within the frame and to the existing structures by the consultant. Increased owner benefits through consultant responsibility for connection design outweighed the marginal fees for upfront detailing costs.

INTRODUCTION

Since the Loma Prieta earthquake of 1989, British Columbia has developed an awareness for the evaluation and upgrading of the province's vulnerable structures. The Ministry of Advanced Education funded the seismic upgrade of the primary laboratory teaching facility at the British Columbia Institute of Technology (BCIT), located in Burnaby, B.C. The facility is identified as the "SW1 Main Building" and is part of a four-building complex surrounding a central courtyard area (see Figure 1). The intent of the project was the upgrading of the building to 100% of current code requirements as specified in the British Columbia Building Code (BCBC) 1992; an extension of the National Building Code of Canada (NBC) 1990.

EXISTING BUILDING

The entire SW1 Complex was constructed in 1962 when the 1960 NBC would have been in effect, however, whether the original design followed NBC guidelines is unknown. The Main Building is a four-storey structure constructed of 5" and 6" lightweight concrete floor slabs supported on steel beams and concrete encased steel columns. Conventional spread footings are founded on dense non-liquefiable soil. A light steel framed penthouse covers approximately 50% of the roof plan area. The existing lateral load resisting system consisted of four nominally reinforced concrete stairwells distributed along the building's length.

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Evaluation of the structural details indicated a structure with nominal capacity because of a lack of specific detailing for ductile behaviour. The elastic capacity of various elements ranged from 20% to 40% relative to current code requirements.

PROJECT CONCEPT DEVELOPMENT

A previous seismic retrofit report for the Main Building recommended incorporation of some 20 new internal reinforced concrete shear walls. The scheme, while sound in strengthening the structure, would have required phased construction over a two or three summer period. The estimate for this upgrade scheme of \$3.3M CDN did not include any non-structural seismic restraint or other building improvements.

The project's conceptual focus was to meet the following objectives: 1, minimizing the project capital cost; 2, completing the project in one three-month period; 3, minimizing disruption to mechanical and electrical systems since remaining buildings of the complex were to remain operational; 4, ensuring that the teaching laboratories would resume classes following summer recess; and 5, maximizing the aesthetic value of the completed project.

Five options were evaluated during the conceptual design stage: 1, strengthening the existing concrete stairwells and foundations; 2, providing new interior concrete shear walls; 3, incorporating new interior steel bracing; 4, using a mix of interior and exterior steel bracing; and 5, employing a combination of exterior steel bracing with exterior concrete shear walls.

After evaluation of all the options, the combination of external steel-bracing with external shear walls was selected (see Figure 2). This option best fulfilled all objectives at approximately 50% of the cost of the preliminary 20 internal shear wall retrofit concept. A benefit from the external retrofit scheme of the Main Building in Phase I would be the provision of some of the structural system upgrading required for the remaining three buildings of the SW1 complex proposed for Phase II. The entire upgrade system would efficiently provide lateral resistance and minimized internal modification so that instructional time to students would not be lost.

The scope of retrofit in Phase I was to raise the level of structural resistance of only the Main Building to current building code standards but would also include restraint for all non-structural, mechanical, and electrical items. A future retrofit plan, Phase II, would continue the seismic upgrade of the remaining three buildings. For the full seismic restraint requirement of Phase I, a three-sided steel bracing system together with external concrete end shear walls would be required. North-south resistance would be provided entirely by the steel bracing along the Main Building's west side, however, code requirements including ground torsional effects, ultimately produced an effective eccentricity of approximately 65 ft. for an earthquake oriented in this direction. The new concrete shear walls and the two frames of steel bracing oriented east-west along the north and south wings would therefore provide the necessary torsional resistance. East-west resistance could simply be drawn from the stiffness of the reinforced concrete shear walls at the ends of the Main Building and the braced frames connected to the north and south wing structures.

Since aesthetics were important to the success of this project, attention to detail required the services of an architect. Incorporation of repetitive visual softening into the details of the braced frame,

in particular the gusset plate connections, challenged the designer and ensured the delivery of a non-industrial type bracing system appearance.

EXTERNAL BRACING SYSTEM RESPONSE CRITERIA

A ductile-braced frame scheme with an R value of 3.0 was selected for several reasons. The ductility rating of this system would approach equivalence with that of the proposed reinforced concrete end walls having an R=3.5 and overall member force levels would be reduced significantly; 33%, over nominally ductile and 50% over non-ductile steel-framing schemes. However, greater demand in the design and detailing of connections would be required to ensure the necessary ductile behaviour. Reducing the force levels was the primary consideration because the irregularly long north-south plan dimension created a large torsional component.

DUCTILE BRACED FRAME DESIGN

A wide flange beam and column grillage incorporating hollow structural steel (HSS) bracing was selected as the preferred system. Several bracing arrangements were studied for the building faces requiring stiffening. Full concentric braced systems were compared to partial concentric braced systems. For the partially braced options force levels at the base would require HSS 12"x12"x1/2" braces for tension and compression resistance, however, the stringent width-thickness ratios for ductile braced frames and the extreme demand that would be placed on the foundations precluded their use. A more manageable pattern of forces would be transferred into the foundations by the full bracing options.

Design of exposed steel retrofits requires increased consideration of appearance. In some cases the development of details initiates the selection of the main framing members. Of particular importance to this project were: 1, minimizing window coverage from connection hardware; 2, providing an unobtrusive appearance; 3, restricting erection to a small mobile crane due to a 9' - 0" vertical access limitation; and 4, completing the project within one summer recess. From these reasons, while adherence to delivery of a well-engineered and executed project to the client was maintained, the non-traditional consideration of full in-house detailed engineering of the connections evolved. It was soon apparent that other benefits would follow from such an engineering approach.

As most local steel detailers would be relatively unexposed to the concept of ductility provisions for connection design, the lack of pertinent experience would lead to an intolerable prolonged shop drawing approval process. Furthermore a general design philosophy capable of producing connection symmetry and uniformity would require all the restraints recognized by the consulting design engineer. By providing a set of connection design loads on the contract set, as is the standard industry practice, the design concept conformance would not be ensured without the lead and commitment of the bracing systems engineer-of-record. It became clear that the design engineers' responsibility would require extension beyond the usual provision of connection design loads and specification of connection type and would have to include full details to comply with the intents of S16.1, clause 27.1. Responsibility for the connection design could ensure complete control of aesthetic uniformity and avoid any industrial type (architecturally unacceptable) connection appearance. In summary, regardless of whether or not the actual connection design would be performed by the fabricator, the completion of the project necessitated a higher degree of participation by the consulting design engineer than traditional design

projects warrant. Failure to provide this degree of commitment would compromise the aesthetic qualities of the project and more significantly, the project's schedule.

BRACING CONNECTION DESIGN

Selection of framing connections was contingent on the erection sequence of the members and on minimization of field welding for quality control measures. End-plate connections on beams with bracing gusset hardware were the engineering connection system of choice since they allowed manageable erection for both the frame connections and the connections to the buildings. The gusset plates, which would receive the slotted HSS braces prepared with angle end clips, would enable quick erection bolting followed by the brace-to-gusset fillet field welding.

The project architect, having reviewed the size implications of the connection arrangement, requested a modification which incorporated the use of scalloped gusset plates; the detailed radii of which were relative to each braced connection coinciding at the joint and other local joints. This architectural consideration would impose additional constraints on the engineering design of the project; not only were all the connections to be detailed, but a common geometric relationship between all joints for aesthetics was to be incorporated.

Intrinsic in the connection development was the provision of an out-of-plane yield zone to satisfy the codes stipulation of avoiding brittle failures on gussets through hinge formation upon brace buckling, (see Figure 3). This plastic region on the gusset reported to be achieved (Astaneh et al. 1986) maintains the brace end back approximately two times the gusset plate thickness from a plane created by a line connecting the gusset plate vertical and horizontal extremities. Because of the predetermined hinge locations at either end of a brace, its effective length could be modified to approximately 80% of the work point dimension between joint centreline intersections. The slenderness reduction would be essential to design acceptance of smaller braces and to overstrength provisions of the code so that, consequential lower force levels would result for the connection designs.

Design of a complete joint, i.e., top and bottom bracing connections on either side of the column centreline, was not a simple condition of the largest brace design force selection for all the gussets. Each multiple joint condition was examined for the worst design case. The governing brace force per side of the connection established the geometry of the entire connection so that symmetry could be approached. Overall joint force equilibrium was an essential component of the design of each element of the connection for the earthquake load condition.

For conformance of connection appearance, some controls on gusset geometry were established. Large shear and pass-through forces combined with the necessity of minimizing the connection sizes led to the general selection of 1" diameter A490 bolts. The traditional 30° angle straight line distribution, for force dissipation into the gusset plate from the brace end connection was modified. The angle produced from the leading tip of the gusset plate within the slotted region of the brace would be checked to lie within a range of +/- 3° from 15°. Having established the basic required geometry, the trigonometric relationships necessary for calculation of the radii for the scallops could be solved.

To allow for manageable repetition of the design procedure and to summarize the detail information required, a spread sheet program was developed. Iterative flexibility and geometric parameter sensitivity conditions were incorporated to assist the engineer during the interactive design

session. The final gusset plate geometry, plate and weld sizes, bolt layout, etc., necessary for complete production of fabrication drawings were tabled and referenced to a generic connection detail (see Figure 4).

FRAME-TO-BUILDING CONNECTION DESIGN

Connection design for brace members within a ductile frame is dealt with adequately for the design of new structures in S16.1. For the retrofit of existing buildings, in which new frames are incorporated into the overall structural system, specific design criteria for connection force levels between new and existing structural components is left to the discretion of the seismic designer. For this project, the overstrength provision, a crucial consideration in the detailing, ensured full resistance of the frames could be achieved. Connection design force levels of the frames to the buildings for each floor level under consideration, were first limited to twice the calculated earthquake shear. When this force presented an unreasonable number of anchors and their clustering interfered with their efficiency, a total floor shear force calculated on the buckling capacity of storey braces was substituted. Because of the optimization of the bracing sizes this overstrength limitation ranged between 1.3 and 1.7 of the design earthquake shears.

The location of the beams of each frame facilitated connection directly to the floor diaphragms. The Main Building was finished with an exposed aggregate stucco. A band full length of the connection zones was sawcut and lightly jack-hammered to exposed the formed concrete surface. A 3/16" thick industrial grade neoprene strip installed behind the flanges of these connector units accommodated some of the finish variations from the original forming.

DRAG-STRUT DESIGN

Horizontal reactions from the slabs at each floor level and the roof were transferred through drag struts to the frames along the north and south wings of the complex. Openings cut in the Main Buildings' walls allowed the fabricated struts to be installed from the courtyard. The top flange of each drag strut was bolted with adhesive anchors to the underside of the slab. For accommodation of variations in the soffit elevations, grout dry packed between the connecting flange and the slab ensured pure shear transfer in the anchors. A similar design philosophy as for the shear connections of the frames to the buildings was used for maximum connection force criteria selection.

Very high shear forces within the drag struts (668 k max), were found to be most efficiently transferred through this arrangement although not without connection difficulties. High moments occurring from the eccentricity between the drag strut, and frame beam centrelines, combined with the axial forces from the braced frame action precipitated local heavy stiffening of the columns. The effective wide flange shape of the drag strut, while promoting the shear shift from the slab soffit to its centreline, allowed equalized tension forces to be developed in the end-plate connection bolts. Because of the cramped connection region, bolt sizes reached 1 1/4" diameter in the A490 grade.

GENERAL FOUNDATION DESIGN

Overstrength upper bounds of the frames' shear capacity were used in the design of each foundation. Capacity factors bounding one and a half times the design base shears and overturning

moments were typical. As the steel braces were optimized, minimum overstrength foundation design forces resulted and reasonably sized foundations were achieved.

Along the Main Building, because the shears were so large, a unique deep foundation mat was used. For the required shear resistance in the soil, a grid of trenches was excavated within a plan area approximately 20 ft x 140 ft. The foundation resembled a large inverted ice-cube tray cast into the soil. Clustered arrangements of tension only soil anchors were located in the end regions of each frame to resist the large uplift forces from overturning effects.

CONCLUDING REMARKS

The project was substantially completed the day before classes reconvened. Well detailed and documented contract drawings and specifications led to few issues requiring resolution. The drawing thoroughness in particular, maintained the fabrication schedule.

For this project, the decision of full in-house design of the connections provided the owner's solution within the time frame necessary. Control of aesthetics and integrity in the ductility provisions were maintained by this approach (see Figure 5).

From the designers' perspective, other implications resulting from this project become issues worthy of discussion. Since full responsibility for the design including every aspect of connection is carried by the engineer-of-record, a more thorough review of shop drawings than the traditional process of screening for general design conformity is required and appropriate time for this review should be allowed for.

Ductility design requires a clear understanding of system, member and connection performance. The existing practice of identifying maximum force levels on the drawing for use by detailers in the design of connections is becoming more difficult if not impossible to convey accurately without misinterpretation especially in establishing equilibrium conditions for correct joint design for complex ductile structures. A significant portion of the success of the SW1 Main Building Seismic Upgrade is attributed to the "take-charge" attitude regarding the connection design. The result of this approach wholly complied with the intent of the codes, that is, the engineer-of-record being fully conversant with the performance specification of the connections carried their design through to the proper production of shop drawings by the fabricator. Responsibility for the project's elements was not separated from the engineer at a critical point in the design process.

REFERENCES

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2. British Columbia Building Code, 1992. Building Standards Branch, Ministry of Municipal Affairs, Recreation and Housing.
3. CAN/CSA-S16.1-M89, Limit States Design of Steel Structures, Fifth Edition. National Standard of Canada, Canadian Standards Association, 1989.
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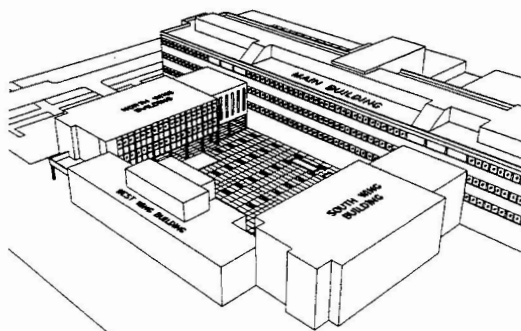


Figure 1 - SW1 Complex

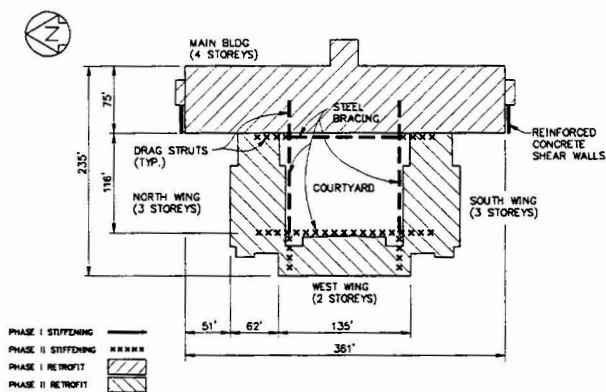


Figure 2 - SW1 Complex Retrofit Plan

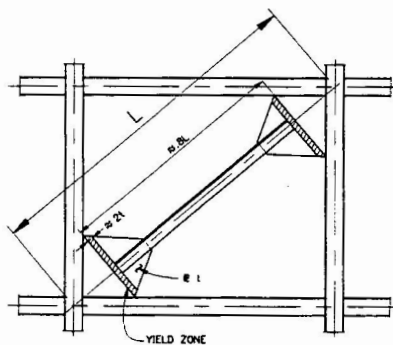


Figure 3 - Researched Out-Of-Plane Yield Zones in Gussets

DETAILING SCHEDULE	
GENERIC BRACE JOINT	JOINT #3
	LEVEL 2 8 L L
	L R T B
	✓ ✓ ✓ ✓
DIMENSIONS	
A	26 1/2" (26.575')
B	30"
C	36 1/2" (36.118')
D	26 5/8" (26.142')
E	10"
F	20 1/2"
G	6"
H	18"
I	24"
J	3 1/2"
K	3"
L	24"
WELDS	
M1	1/2"
M2	1/2"
M3	1/2"
M4	1/2"
N	1/2"
O	1/2"
P	1/2"
Q	1/2"
R	1/2"
PLATE THK	
S	1/4"
T	1/2"
U	1/2"
MISC INFO	
DOUBLER # (V)	EAD1
Ø (DEC) (R)	40.384
ERECTOR L (S)	L4" x 4" x 1/2"
BOLT # (Y)	1"
BOLT GRADE (Z)	A490

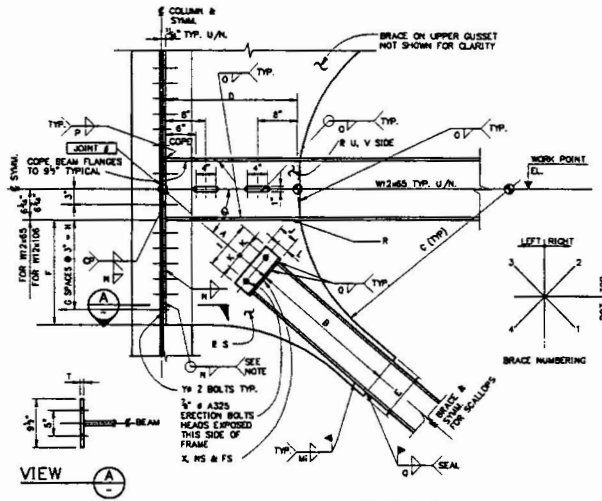


Figure 4 - Generic Bracing-To-Column Connection Details at Beams

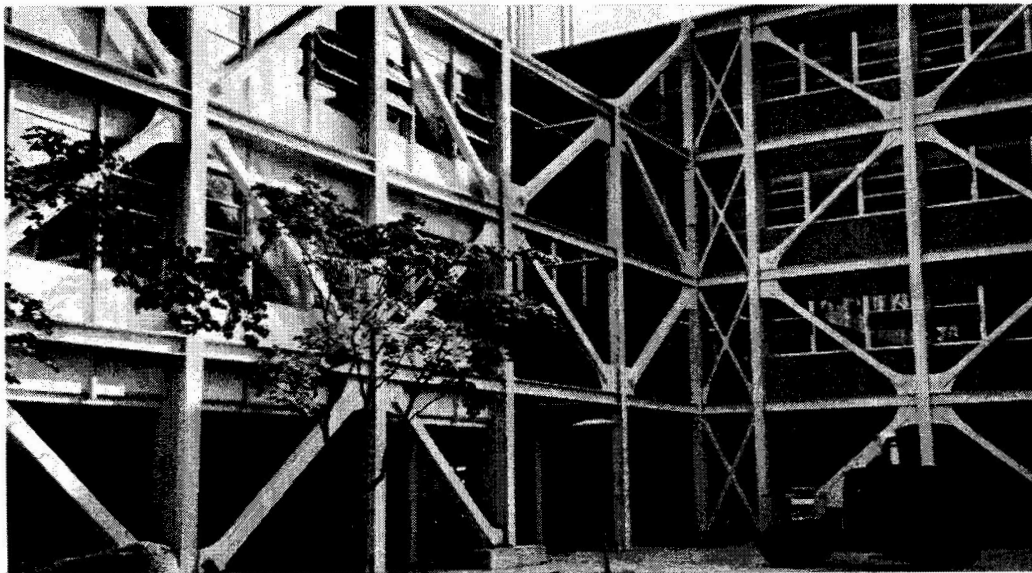


Figure 5 - Courtyard Perspective of Finished Frames