Seismic Retrofit of a Four-Storey Building in British Columbia

Consolidation vis-à-vis des séismes d'un bâtiment de quatre étages

Erdbebensanierung eines vierstöckigen Gebäudes in British Columbia

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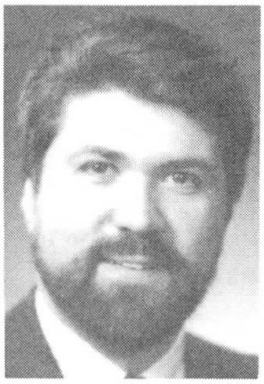

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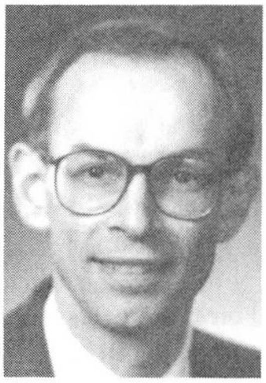

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SUMMARY

The externally mounted steel frame for the seismic retrofit of the BCIT SW1 Main Building presented unique design constraints. The main structure is part of a four-building complex arranged in a rectangular pattern forming a central courtyard. The retrofit of these struc-tures used the courtyard as the optimum location for strengthening. Minimising 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.

RÉSUMÉ

La charpente extérieure métallique pour la consolidation du bâtiment principal de BCIT SW1, a présenté des difficultés très spéciales. La construction principale fait partie d'un complexe de quatre immeubles placés autour d'une cour centrale. Cette cour a été utilisée comme le lieu optimal pour la consolidation des structures. Minimisant l'obstruction des fenêtres et utilisant une consolidation discrète, le système a été réalisé en trois mois d'été. Il satisfait les exigences pour les éléments et les connections.

ZUSAMMENFASSUNG

Der an der Aussenfassade angebrachte Stahlrahmen zur Erdbebensanierung des Hauptgebäudes SW1 des British Columbia Institute for Technology bot einzigartige Entwurfsbeschränkungen. Die Hauptstruktur ist Teil eines aus vier Gebäuden bestehenden Komplexes, welche symmetrisch um einen rechtwinkligen Innenhof angeordnet sind. Der Innenhof wurde als optimal für die Verstärkung der Struktur bestimmt. Sowohl der Entwurf wie die Detaillierung der Rahmenverbindungen wurden im Ingenieurbüro angefertigt, da besondere Bestimmungen in Bezug auf die Duktilität von Rahmenteilen sowie für Verbindungen nach der kanadischen Norm einzuhalten waren. Ferner mussten Richtlinien betreffend minimaler Fensterverdeckung und akzeptabler Struktur berücksichtigt werden, sowie die besonders kurze Konstruktionsdauer während drei Sommermonate.

1. INTRODUCTION

Since 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 part of a four-building complex surrounding a central court-yard area (Fig. 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.

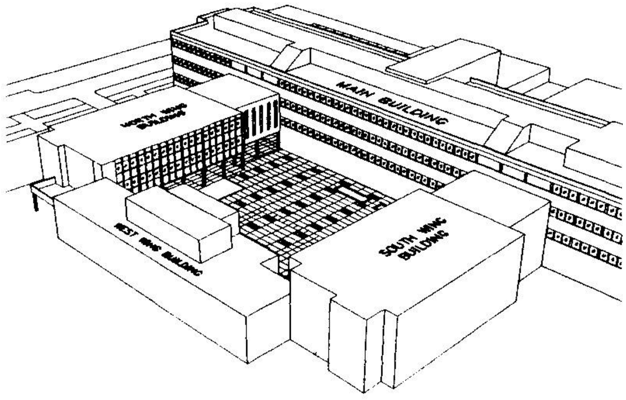

Figure 1 - SW1 Complex

2. EXISTING BUILDING

The Main Building (1962) is a four-storey structure constructed of 125 mm and 150 mm lightweight concrete floor slabs supported on steel beams and concrete encased steel columns. Conventional spread footings are founded on dense non-liquefiable soil. The existing lateral load resisting system consisted of four nominally reinforced concrete stairwells distributed along the building's length. The elastic capacity of various structural elements ranged from 20% to 40% relative to current code requirements.

3. PROJECT CONCEPT DEVELOPMENT

A previous seismic retrofit report for the 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; 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 (Fig. 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.

Phase I's scope 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. 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. 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.

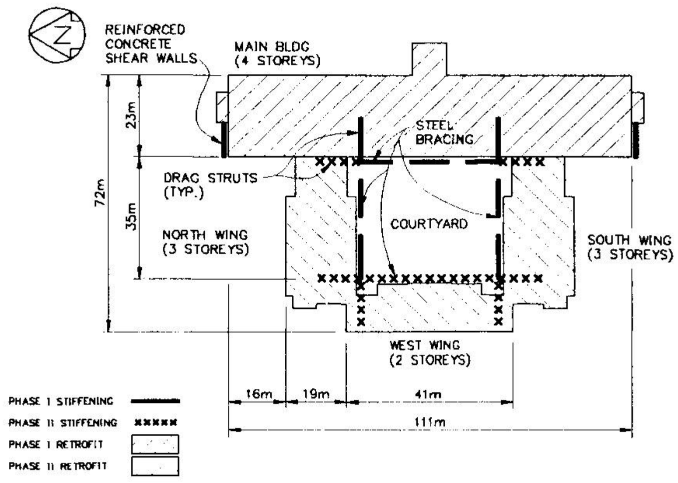

Figure 2 - SW1 Complex Retrofit Plan

4. DUCTILE BRACED FRAME DESIGN

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. 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.

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 305 x 305 x 13 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. 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; and 4, completing the project within one summer recess. From these reasons, 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 the code. Responsibility for the connection design could ensure complete control of aesthetic uniformity and avoid any industrial type connection appearance. 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 otherwise compromise the aesthetic qualities of the project and more significantly, the project's schedule.

5. BRACING CONNECTION DESIGN

Selection of framing connections was contingent on the erection sequence of the members and minimization of field welding. End-plate connections on beams with bracing gusset hardware were selected 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. 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, (Fig 3). This plastic region on the gusset reported to be achieved [1] 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%. 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 and foundation designs.

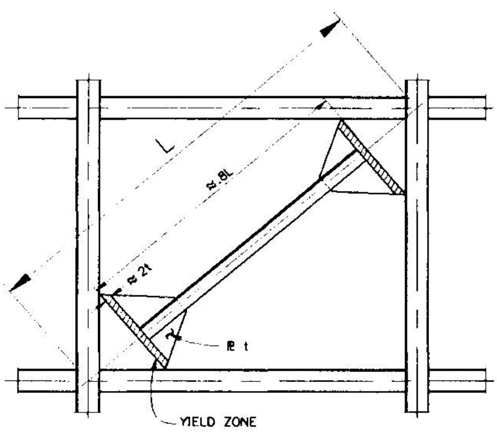

Figure 3 - Researched
Out-of-Plane Yield Zones in Gussets

Design of a complete joint, i.e., top and bottom bracing connections on either side of the column centreline, was not straightforward. Each multiple joint condition was examined for the worst design case. The governing brace force per side of 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 minimizing connection sizes led to the selection of M24 diameter A490M 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.

DETAILING SCHEDULE				
GENERIC BRACE JOINT	JOINT #3			
	LEVEL	2	℄	J. L.
	L	R	T	B
	✓	✓	✓	✓
DIMENSIONS				
A	675			
B	762			
C	892			
D	664			
E	250			
F	520			
G	6			
H	450			
I	220			
J	100			
K	75			
L	65			
WELDS				
M1	10			
M2	8			
M3	8			
M4	10			
N	10			
O	5			
P	6			
Q	6			
R	CP			
PLATE THK.				
S	28			
T	12			
U	6			
MISC. INFO				
DOUBLER ℞'s (V)	EACH			
Θ (DEG.) (W)	40.364			
ERECTION L (X)	L100x100x10			
BOLT # (Y)	25			
BOLT GRADE (Z)	A490M			

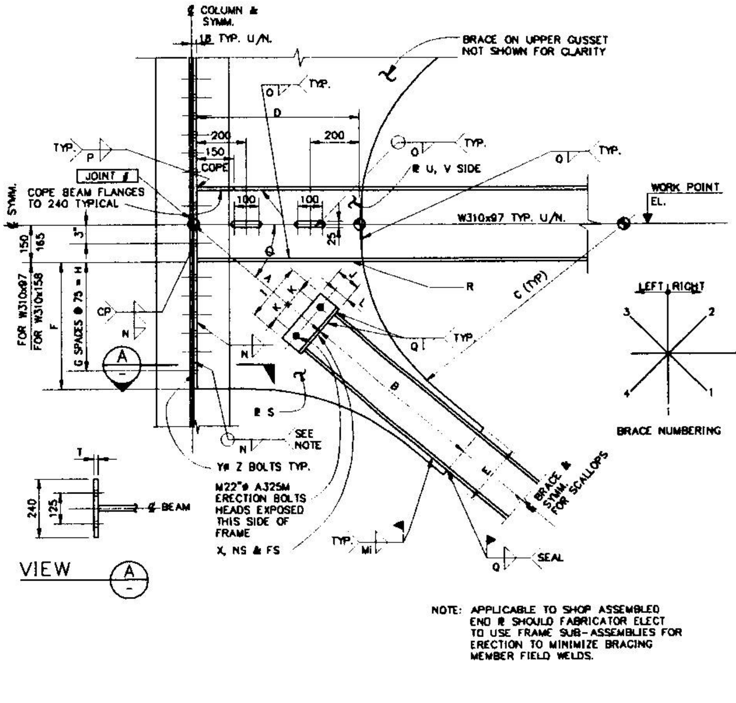

Figure 4 - Generic Bracing to Column Connection Details at Beams

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. 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 (Fig. 4).

6. 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 at the discretion of the 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.

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.

7. CONCLUDING REMARKS

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 (Fig. 5).

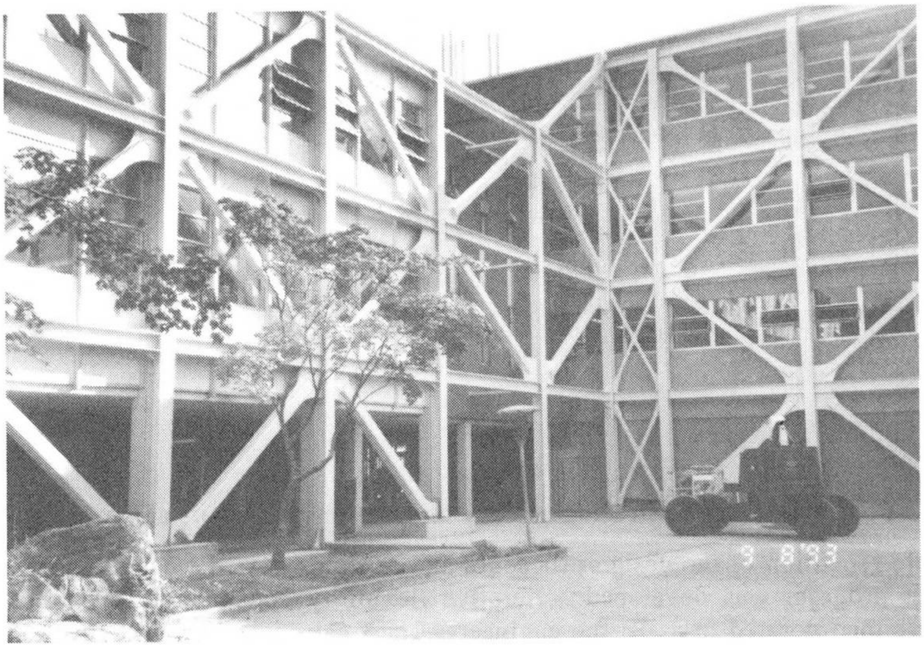

Figure 5 - Courtyard Perspective of Finished Frames

Ductility design requires clear understanding of system, member and connection performance. The existing practice of providing maximum force levels on drawings for detailers on seismic projects is inadequate 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

1. ASTANEH, A., Goel, S.C., and Hanson, R.D. Earthquake Resistant Design of Double-Angle Bracings. Engineering Journal, AISC, 23(4), 133-147, 1986.