Seismic Evaluation and Retrofit of a Precast Concrete Structure

By

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Abstract

A single-storey precast concrete structure constructed in 1963 with masonry-infilled walls was selected for a seismic evaluation. The structure is located in the Montréal region, and is considered a post-disaster emergency building since it houses spare parts for municipal facilities. The warehouse was first evaluated through a visual inspection and identified as being seismically vulnerable. A series of three-dimensional models of the structure were analyzed to evaluate the seismic performance of the structure with respect to the seismic design requirements of the 1995 and 1965 edition of National Building Code of Canada (NBCC). The structural models were analyzed using a linear static analysis accounting for the effects of different diaphragm configurations on the lateral load resisting columns and the effects of masonry-infilled walls. The seismic analyses showed that the warehouse does not satisfy either of the NBCC 1995 and NBCC 1965 requirements. A seismic rehabilitation scheme is proposed comprising the addition of reinforced concrete shear walls, steel braces and an upgraded roof diaphragm. The proposed retrofit satisfies the seismic design requirements of NBCC 1995. Finally, the proposed seismic design provisions of NBCC 2005 are briefly summarized and the impact of the new seismic design provisions on the retrofitted structure is discussed.

Résumé

Un entrepôt de la Ville de Montréal construit en 1963 a été sélectionné pour une évaluation sismique détaillée. Le bâtiment est en béton préfabriqué avec des murs en maçonnerie et est représentatif de plusieurs bâtiments d'urgence construits à la même époque. La première étape du projet a été d'effectuer une visite de l'immeuble et de valider une procédure d'évaluation visuelle de la vulnérabilité sismique. La structure est une structure d'urgence tel que défini dans le Code National du Bâtiment du Canada (CNBC 1995) car elle sert d'entrepôt pour des pièces de rechange pour diverses installations municipales. Une série de modèles structuraux tri-dimensionnels ont été analysés afin d'évaluer la performance sismique de la structure relativement aux exigences du CNBC 1995 (le code en vigueur présentement) et du CNBC 1965 (code en vigueur au moment de la construction). Les analyses effectuées sont du type élastique linéaire et statique. Les modèles considèrent l'effet diaphragme du toit et l'effet des mûrs de maçonnerie. Les analyses démontrent que la structure ne satisfait pas les exigences du CNBC 1995 et 1965. Une réhabilitation sismique est proposée comprenant l'addition de plusieurs mûr de cisaillement en béton armé, des contreventements en acier et un renforcement du toit. Les modifications proposées suffisent pour satisfaire les exigences du CNBC 1995 ainsi que celles de la prochaine édition du code (CNBC 2005).

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The author would like to express his gratitude to Prof. Luc Chouinard for his supervision and support. Gratitude is also extended to Prof. Denis Mitchell for his time and insightful guidance throughout the course of the research program. Without their assistances and patience, the completion of this project would not have been possible.

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William Lim

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a	width of equivalent masonry strut
a_{red}	reduced width of equivalent masonry strut
A_g	gross cross-sectional area of a member
A_n	net cross-sectional grouted area of an infill panel
A_{open}	total area of openings in a masonry-infilled wall
A_{panel}	total area of a masonry-infilled wall panel
A_s	area of steel reinforcement or structural steel member
A_v	area of shear reinforcement
A_{vf}	area of shear-friction reinforcement
b	width of a concrete member
b_w	thickness of a shear wall
c	cohesion stress in shear-friction design
c_c	compression depth of shear wall
C	construction type factor
d_b	diameter of steel reinforcement
d_v	effective shear depth of a concrete member
d_x, d_y	relative distances measure from the centre of rigidity in the specified x and y directions
D	specified dead load
D_n	plan dimension of the building perpendicular to the direction of earthquake loading
D_s	length of lateral load resisting system
e	distance between the centre of mass and centre of rigidity
E	specified earthquake load
E_c	elastic modulus of concrete
E_m	elastic modulus of masonry
E_s	elastic modulus of steel
f'_c	specified compressive strength of concrete
f'_m	specified compressive strength of hollow concrete masonry
f_{ult}	ultimate tensile strength of structural steel
f_y	specified yield strength of steel reinforcement
F	foundation factor
F_z	storey shear at level z

h	total height of a structure
h_m	height of the masonry infill
h_s	storey height in analysis model
H_f	height of confining frame for infill modeling
I	seismic importance factor
I_{col}	moment of inertial of a concrete column
K_x, K_y	member stiffness in the specified x and y directions
l_u	height of shear wall
l_w	length of shear wall
L	specified live load
M_f	factored design moment
M_n	nominal moment resistance
M_p	probable moment resistance
M_r	factored moment resistance
n	number of storey
R	force modification factor
R_1	reduction factor that account for openings in masonry-infilled wall
R_{65}	earthquake zoning factor in 1965 NBCC
R_{crush}	equivalent strut strength associated with crushing failure of masonry-infilled wall
R_{shear}	equivalent strut strength associated with shear failure of masonry-infilled wall
s	spacing of transverse reinforcement in concrete member
S	seismic response factor
t	thickness of masonry-infilled wall
t_{eff}	effective masonry wall thickness excluding the hollow void
T	fundamental period of vibration of a structure
T_f	design axial member force
T_f'	tensile force associated with yielding of braces
T_n	horizontal torsional moment
T_r	factored tensile resistance
U	calibration factor for seismic design
U_2	P-Δ amplification factor for the design of a steel braced frame
V_c	factored shear resistance provided by concrete
V_e	elastic seismic base shear
V_f	design member shear force
V_n	nominal shear resistance
V_m	shear strength of masonry-infilled wall
V_p'	shear force associated with the development of probable moment resistance

V_r	factored shear resistance
V_s	factored shear resistance provided by steel reinforcement
V_D	design seismic base shear
V_T	additional torsion-induced shear force used in the simplified analysis
W	seismic weight of a structure
ϕ_c	resistance factor for concrete
ϕ_s	resistance factor for steel reinforcement and structural steel
γ	importance factor in design load combinations
γ_w	member overstrength factor
λ	concrete density factor
λ_I	parameter reflecting characteristic of an infilled frame
μ	coefficient of friction
θ_{strut}	slope of the infill diagonal to the horizontal in radian
ρ	ratio of steel reinforcement
υ	zonal velocity
Δ	horizontal storey displacement

Chapter 1: Introduction

1.1 Introduction

The remarkable increase in knowledge and experience gained in seismicity and earthquake resistant design has considerably changed the Canadian seismic code requirements and design philosophy in the past 50 years. Most buildings constructed in the 1960s are either not designed for seismic forces or they are designed for a force level that is lower than that can be experienced in an earthquake. As the few Canadian earthquakes that have occurred, including the 1988 Saguenay earthquake (Magnitude 6.0, Québec), have yet to cause obvious destruction and loss of life in urban regions *(Bruneau, 1994)*, the overall awareness and recognition for the need of seismic evaluation and retrofitting often receive unenthusiastic responses. However, recent seismic events including the 1999 Kocaeli earthquake (Magnitude 7.4, Turkey) *(Saatcioglu et al., 2001)* and the 1999 Chi Chi earthquake (Magnitude 7.3, Taiwan) *(Tsai et al., 2000)* have once again reminded engineers that older concrete structures, which have similar non-ductile detailing to those that have collapsed during the earthquakes, urgently need evaluation and upgrading in order to survive an equally destructive earthquake that may occur in Canada. The Turkish earthquake, in particular, has also demonstrated that precast concrete structures without seismic resistant design are extremely vulnerable in earthquakes. Currently, many seismically deficient precast concrete structures are located in seismically active regions of Canada including Montréal. These precast concrete structures, especially for those being post-disaster buildings, should be evaluated and strengthened.

1.2 Overview of Precast Concrete Structures

1.2.1 Current Design Approach

Single-story industrial buildings represent one of the most common forms of precast concrete construction in Canada. These single-storey precast framing structures consist mainly of precast beam-column framing and an integrated double-tee roof system. Frames located at the building perimeter are usually infilled with unreinforced masonry walls. Currently, the seismic design of new precast concrete structures in the North Americas is guided by the Canadian Prestressed Concrete Institute Metric Design Manual *(CPCI, 1996)* and the Precast/Prestressed Concrete Institute Design Handbook *(PCI, 1999)*. The seismic resistance of new precast structures is often ensured by the presence of a robust lateral load resisting elements and a securely tied roof diaphragm for the transfer of the inertial forces (Figure 1-1). Lateral drifts of the structures are limited by the lateral load resisting system so as to satisfy the displacement criteria stipulated by the current National Building Code of Canada *(NBCC, 1995)*. Structural members that are not part of the lateral load resisting system are designed to maintain their gravity-load-carrying capacity during an earthquake. More importantly, capacity design principles are implemented in the design of the precast member connections, roof diaphragms and other non-lateral load resisting elements, so that they can function elastically while the lateral load resisting elements dissipate seismic energy through a predefined inelastic mechanism.

Older precast concrete structures were typically designed with a lack of awareness of capacity design. In addition to the absence of appropriate lateral load resisting systems, members of these existing precast concrete structures are also deficient in strength, stiffness and ductility. Poor seismic performance of older precast structures is also due to inadequate connections between the precast elements. Connections in older precast structures are mostly designed only for shear forces due to gravity loads and possess little resistance against cyclic loading induced by earthquakes. Adding to the problem of weak connections, these precast concrete structures lack alternate load paths and suffer significant structural discontinuities at their joints *(Collins & Mitchell, 1987)* and hence, progressive collapses of these structures due to a lack of structural integrity are likely in a major earthquake.

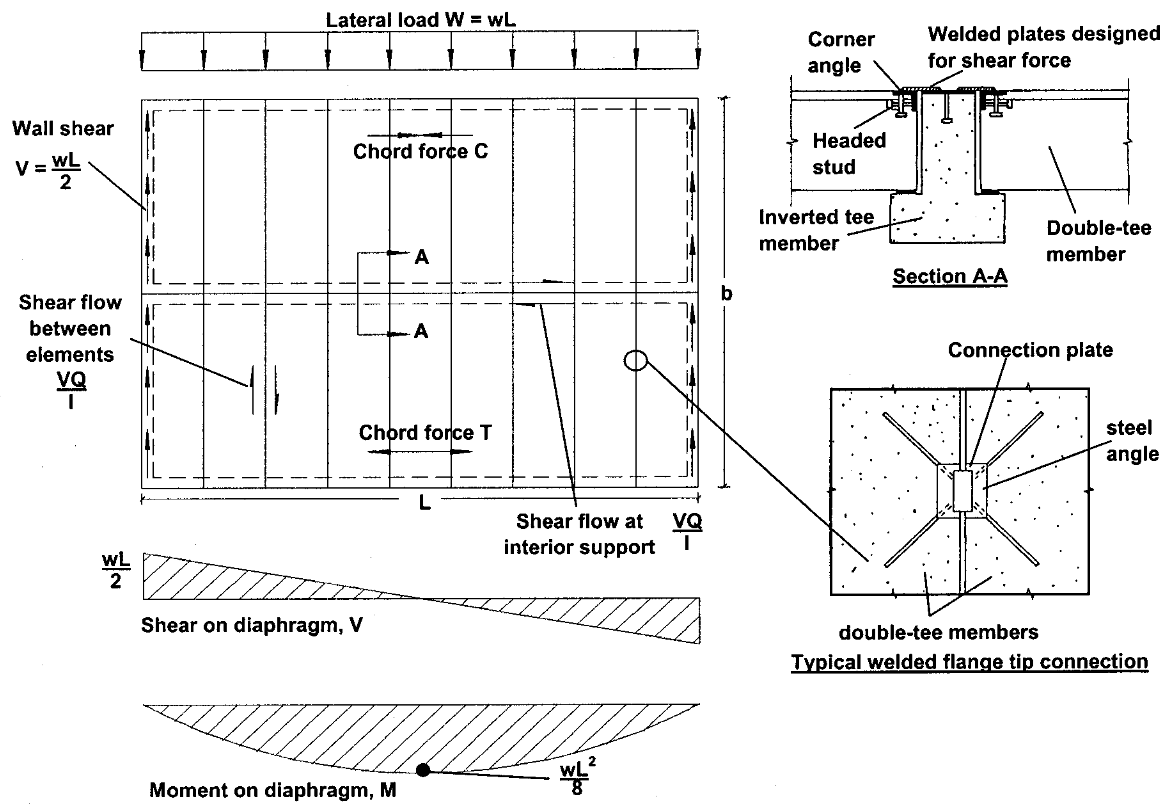


Figure 1-1: Design of precast concrete roof diaphragm *(CPCI, 1996).*

1.2.2 Observed Damages in Recent Earthquakes

Structural damage to precast reinforced concrete structures observed in recent major earthquakes are briefly discussed in this section to provide qualitative insights on the damage anticipated for some non-ductile precast concrete structures. Collapsed or heavily damaged precast concrete structures were widely observed during the 1999 Kocaeli earthquake in Turkey. Most of the structural deficiencies that were commonly found in damaged buildings were similar to those reported previously during the 1985 Mexican earthquake *(Mitchell et al., 1986, 1987)*. It should be noted that the typical precast concrete structures in Turkey are mostly single-storey portal frames that rely only on their cantilever columns to provide lateral load resisting strength and lateral stiffness (Figure 1-2). During the earthquake, these precast frames were severely damaged due to inadequate column confinement and inadequate transverse shear reinforcement (Figure 1-3(a)). Brittle shear failures of the columns, pullout of precast column anchorages (Figure 1-3(b)) and significant plastic hinging at column bases (Figure 1-3(c)) occurred as a result of significant storey drift and flexible roof diaphragms *(Saatcioglu et al., 2001)*. Some precast columns have also undergone brittle failures due to the interfering masonry-infilled walls (Figure 1-4). These relatively rigid infilled walls have posted significant shear demands on their bounding frames by creating short column effects near openings. The irregularly infilled masonry walls could also have amplified the torsional effects on these relatively flexible structures. Some of the precast concrete structures have also sustained considerable impact damage due to pounding of roof elements (Figure 1-5 (a)). Concrete fames that have incorporated inadequate beam-to-columns connections have even experienced catastrophic collapses as shown in Figure 1-5 (b).

Seismic deficiencies associated with precast concrete diaphragms were widely reported in the 1994 Northridge earthquake. Several precast concrete parking structures that contained undamaged shear walls have experienced partial or compete collapse of their roof diaphragm (Figure 1-6 (a)) *(Hawkins et al, 2000)*. The failures of concrete diaphragms prior to that of the shear walls were due to the lack of capacity design, where the precast roof diaphragms were the weakest links in the structures. Precast concrete diaphragms that survived the earthquake were exposed to considerable damage such as buckling of diaphragm chord reinforcement (Figure 1-6 (b)) and concentrated cracks on the cast-in-place topping slab along the column lines (Figure 1-6 (c)). These crack patterns were reported to be considerably different from those that have occurred in monolithic reinforced concrete diaphragms *(Hawkins et al., 2000)*. It was believed that temperature and shrinkage cracks prior to the earthquake have significantly contributed to the disintegration of many concrete diaphragms.

Other damage to precast reinforced concrete structures include rocking of foundations and the effect of soft soil amplification. Inadequate or deteriorated material properties, such as the brittle longitudinal steel reinforcement, will also degrade the seismic performance of existing reinforced concrete structures. It should be noted that observed failures of precast concrete structures during earthquakes are not solely due to the seismic vulnerability of precast concrete structure and the lack of seismic detailing, but also due to unregulated and varying construction practices. Precast concrete structures that have been properly designed and detailed for earthquake resistance have performed well in recent seismic events. In summary, some of the lessons learned from the aforementioned disasters include:

- A majority of out-dated precast concrete structures in Canada possess similar design and detailing deficiencies (Figure 1-3 (c)) as structures that were damaged in recent earthquakes.
- Flexible precast concrete frame structures that exhibit large lateral displacement should be stiffened.
- Precast structural members should be adequately tied together to ensure structural integrity and the effects of masonry-infilled walls should not be neglected.
- The role of composite diaphragm (i.e., the precast double-tees and the cast-in-place concrete topping) within the lateral load resisting system of precast structures should be clearly addressed.

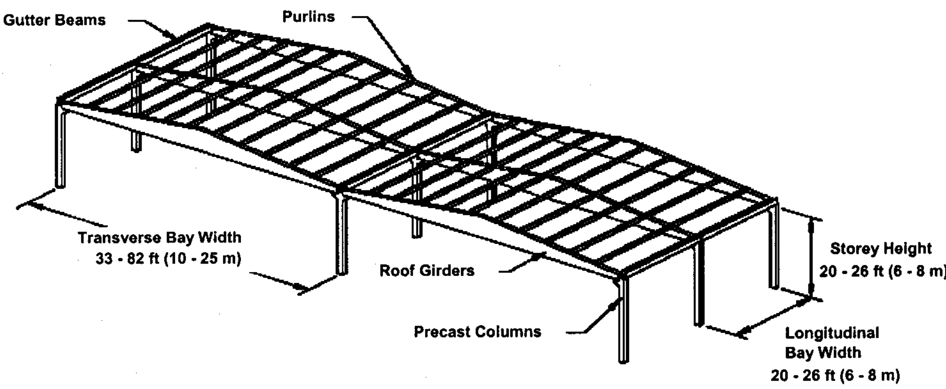


Figure 1-2: Typical single-storey precast concrete structures in Turkey *(Posada & Wood, 2002).*

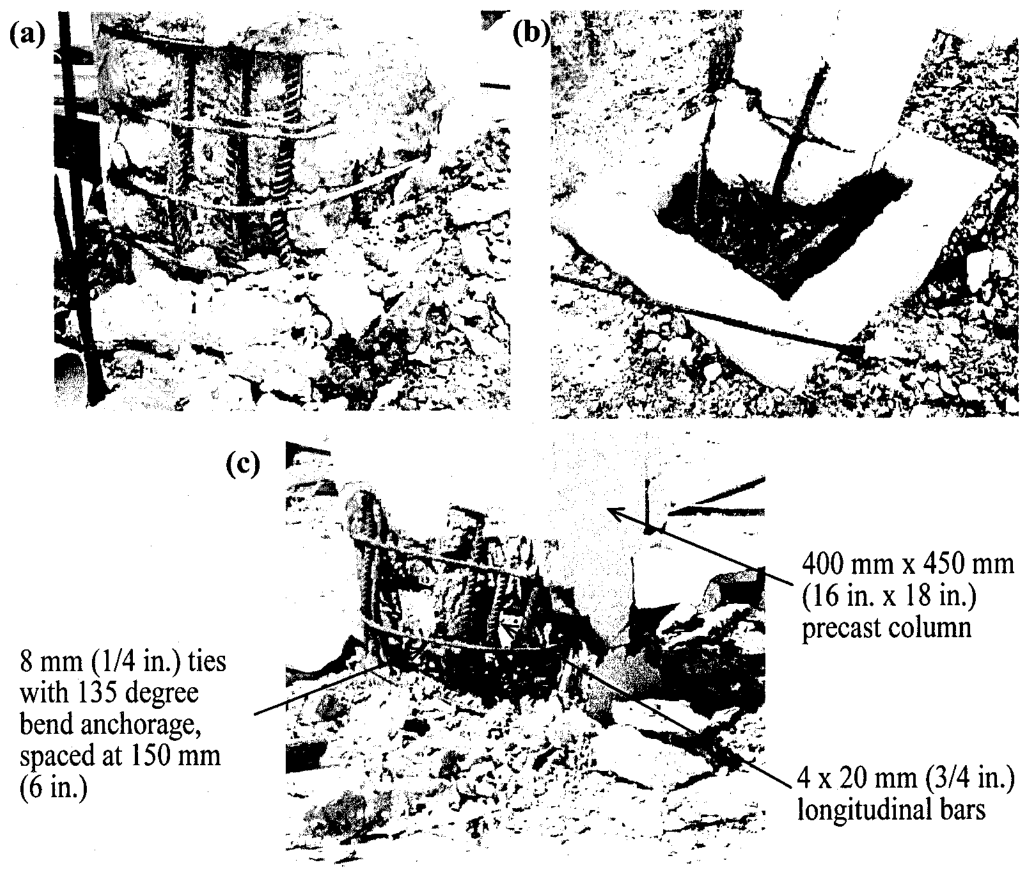


Figure 1-3: Failures at column bases during the 1999 Kocaeli earthquake: (a) lack of confinement and shear reinforcement; (b) pullout of column anchorages; (c) significant plastic hinging at column base *(Saatcioglu et al., 2001).*

Figure 1-4: Shear failure of columns due to the influence of masonry-infilled walls *(Saatcioglu et al., 2001)*.

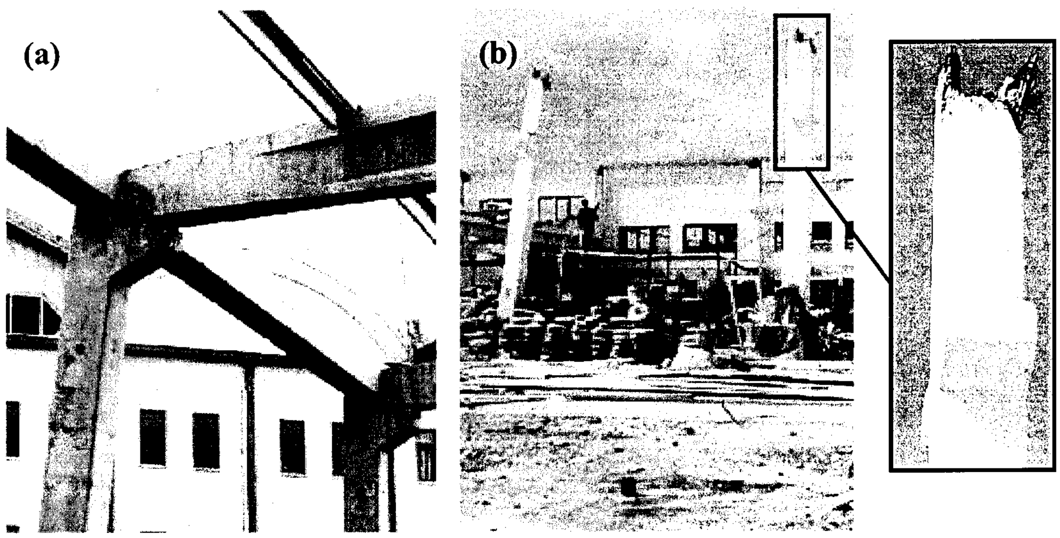


Figure 1-5: Member distress at the roof level: (a) pounding of precast elements *(Posada & Wood, 2002)*; (b) inadequate beam-to-column connections *(Saatcioglu et al., 2001)*.

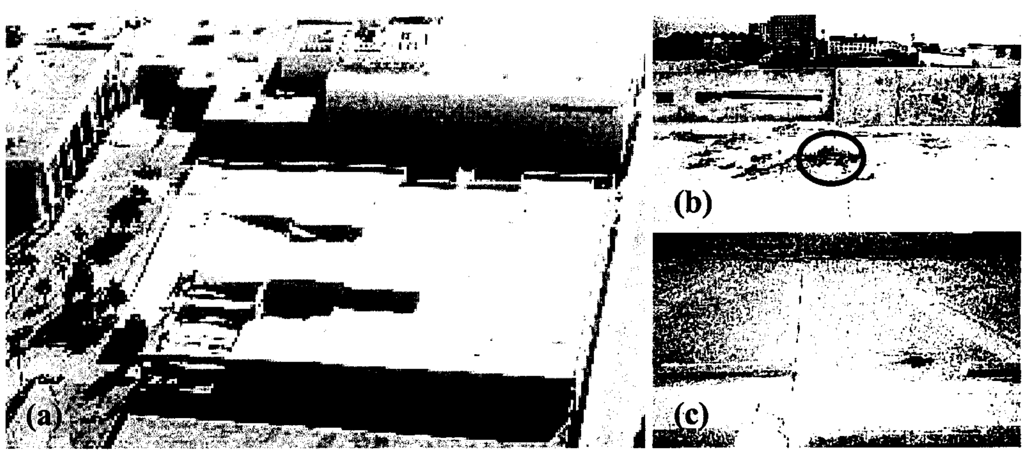
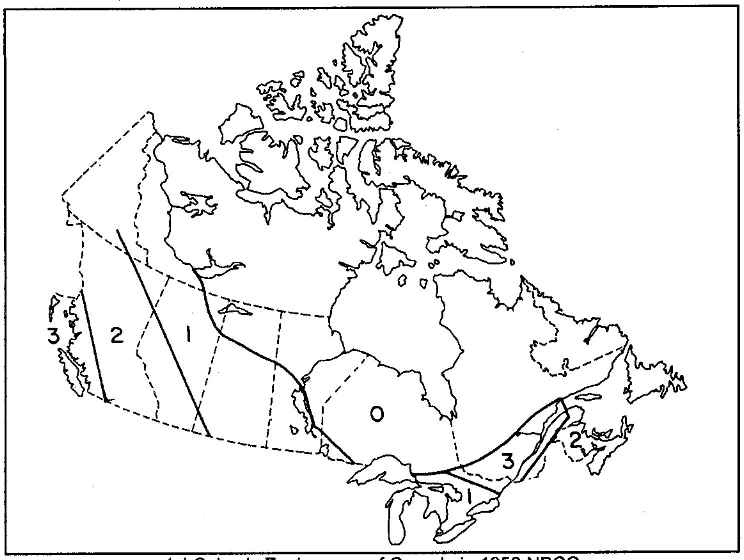


Figure 1-6: Failures of precast concrete diaphragms during the 1994 Northridge earthquake: (a) partial collapse of roof diaphragm; (b) buckling of chord reinforcement; (c) diaphragm cracks along the columns lines *(Hawkins et al., 2000)*.

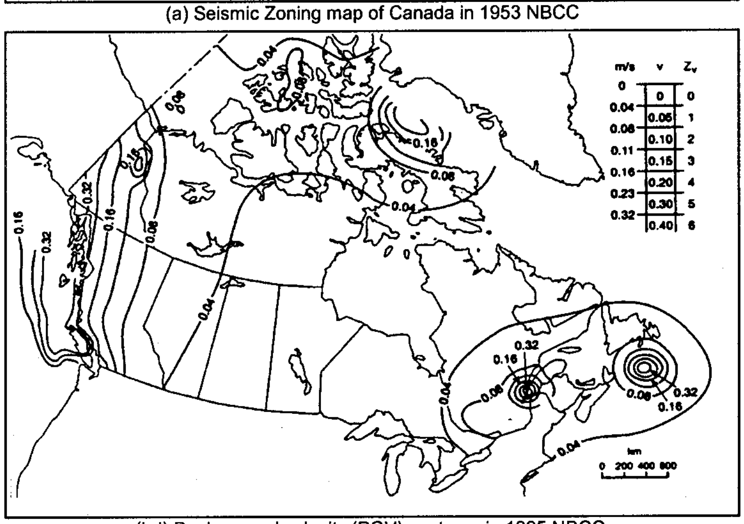
1.3 Review of NBCC Seismic Design Provisions

The review herein will centre its attention mainly on the few editions of seismic design provisions pertinent to the seismic evaluation carried out in the thesis, namely the 1953, 1965 and 1995 versions of NBCC. More complete summaries on the previous editions of NBCC seismic design provisions are available in Heidebrecht *(2003)* and Uzumeri *et al. (1978)*, with the latter primarily focused on provisions prior to 1978.

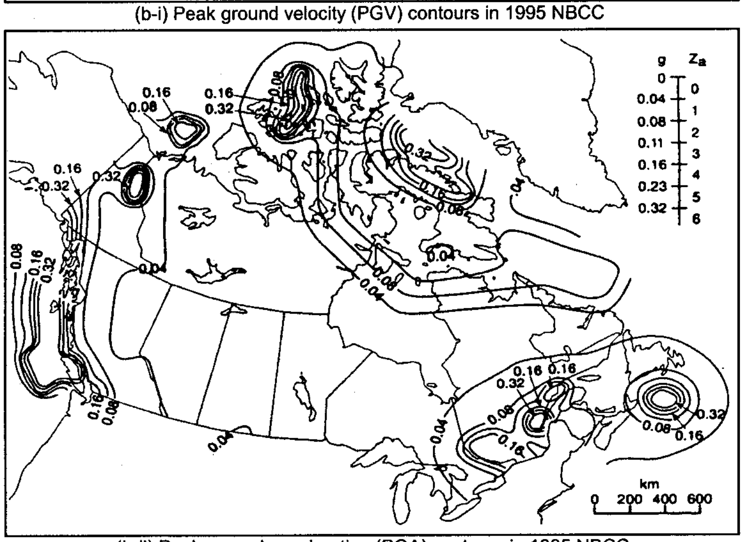
The level of changes and the amount of complexity that the seismic design provisions have gained over the years is reflected by the comparison of the seismic zoning map first published in 1953 to that being used in the current building code (Figure 1-7). The first Canadian seismic zoning map, which remained effective until 1970 (Figure 1-7a), divides Canada into 4 zones ranging from 0 for the least to 3 for the most seismic active zone. The third generation of the Canadian seismic zoning map currently in use (Figure 1-7b) characterizes the seismic hazard in the country with contours of peak ground velocity (PGV) and acceleration (PGA) due to earthquake with a 10% probability of exceedance in 50 years (a return period of 475 years). Unlike the zoning maps in the 1953 NBCC which affect the base shear computation through the use of zonal factors, the current seismic zoning maps affect the base shear computation explicitly with the zonal velocity, υ, and implicitly through the computation of seismic response factor, S, which depends on the ratio of the ground motion parameters, Z_a/Z_v, and the structural period of the building.



(a) Seismic Zoning map of Canada in 1953 NBCC



(b-i) Peak ground velocity (PGV) contours in 1995 NBCC



(b-ii) Peak ground acceleration (PGA) contours in 1995 NBCC

Figure 1-7: Evolution of the Canadian seismic zoning map: (a) Seismic zoning map of 1953 and 1965 NBCC *(Uzumeri et al., 1978)*; (b) ground motions contour in current NBCC *(1995)*.

The NBCC base shear formulation has also received considerable adjustments over the years as shown in Table 1-1. The relatively straight forward base shear equation defined in the 1953 NBCC has only accounted for building flexibility with a "horizontal force factor" specifically defined for each seismic zone. In the 1965 edition of NBCC, the seismic response factors for buildings in all zones are represented by a single curve, "S", for all structural periods and the effect of the seismic zoning is accounted separately with a factor, "R_{65}". New factors are also introduced to improve the seismic base shear formulation: the ductility of the construction type is accounted with a factor "C"; the foundation properties and importance of the structure are addressed with factors "F" and "I" respectively. Torsional and dynamics analyses are also covered for the first time in the seismic design provisions of NBCC.

After 1965, the variables in the base shear formula have been revised in subsequent editions of NBCC based on construction experience and lessons learned from major and moderate earthquakes. In the 1990 NBCC, which is practically the same as the current edition of NBCC *(1995)*, the seismic design provisions received a major restructuring with the introduction of the force modification factor, R, which indicates the ability of a lateral load resisting system to exhibit ductility and hysteretic energy dissipation. The R factor approach is a more rational approach to describe the ductility of a structure than the construction coefficients employed by previous editions of the codes *(Romano, 1990)*. An accompanying calibration factor U of value 0.6 was also introduced to maintain the level of design base shear associated with inelastic response to the same level of protection as the base shear calculated based on previous codes. A comparison of the base shear coefficients calculated based on the aforementioned versions of NBCC for a conventionally constructed structure is shown in Figure 1-8. This comparison shows that the seismic design base shear level has been raised significantly in the current standard, especially for low-rise structures which are susceptible to significant damage even in a moderate seismic event. This is also an indication that buildings designed according to previous editions of seismic codes are no longer in compliance with the current standards, and typically have non-ductile design and detailing, and therefore require seismic evaluation and upgrading.

<table>
<tr><th>NBCC</th><th>Base Shear Formula</th><th>Definition of Variables</th></tr>
<tr><td>1953
to
1960</td><td>$F = C \cdot W$</td><td>C : Horizontal force factor:
$\text{Zone 1: } C = \frac{0.15}{(N+4.5)} \quad \text{Zone 2: } C = \frac{0.3}{(N+4.5)} \quad \text{Zone 3: } C = \frac{0.6}{(N+4.5)}$
W : Seismic weight tributary to the point under coinsideration
(dead load plus 25% snow load)</td></tr>
<tr><td>1965</td><td>$V_D = R_{65} \cdot C \cdot I \cdot F \cdot S \cdot W$</td><td>R : Seismic regionalization factor
(0, 1, 2, and 4 for seismic zone 0, 1, 2, and 3 respectively)
C : Type of construction (ductility) factor
(values ranges from 1.25 for non-ductile structure to 0.75 for moment frames)
I : Seismic Importance factor (1.0 or 1.3)
F : Foundation factor (1.0 or 1.5)
S : Structural flexibility factor
$S = \frac{0.25}{(N+9)}$
W : Total seismic weight (dead load plus 25% snow load)</td></tr>
<tr><td>1995</td><td>$V_D = \frac{V_e \cdot U}{R}$
$= \frac{(v \cdot S \cdot I \cdot F \cdot W) \cdot U}{R}$</td><td>v : Zonal velocity (from seismic zoning map of 1995 NBCC)
S : Seismic response factor:
<table>
<tr><th>Period T</th><th>Z_a/Z_v</th><th>S</th></tr>
<tr><td rowspan="3">≤ 0.25</td><td>> 1.0</td><td>4.2</td></tr>
<tr><td>1.0</td><td>3</td></tr>
<tr><td>< 1.0</td><td>2.1</td></tr>
<tr><td rowspan="3">> 0.25 but < 0.5</td><td>> 1.0</td><td>4.2 - 8.4(T-0.25)</td></tr>
<tr><td>1.0</td><td>3.0 - 3.6(T-0.25)</td></tr>
<tr><td>< 1.0</td><td>2.1</td></tr>
<tr><td>≥ 0.5</td><td>All values</td><td>$1.5/T^{1/2}$</td></tr>
</table>
<table>
<tr><th>Period T Computation</th></tr>
<tr><td>$T = \frac{0.09 \cdot h_s}{\sqrt{D_s}}$
or
$T = 0.1 \cdot N$</td></tr>
</table>
I : Seismic Importance factor (1.0, 1.3, or 1.5)
F : Foundation factor (values ranges from 1.0 for hard rock to 2.0 for soft soils)
W : Total seismic weight (dead load plus 25% snow load)
U : Calibration factor reflecting level of protection (0.6)
R : Force modification (ductility) factor
(values ranges from 1.5 for non-ductile structures to 4.0 for ductile steel structure)</td></tr>
</table>

Table 1-1: Seismic base shear formulas used in various editions of NBCC.

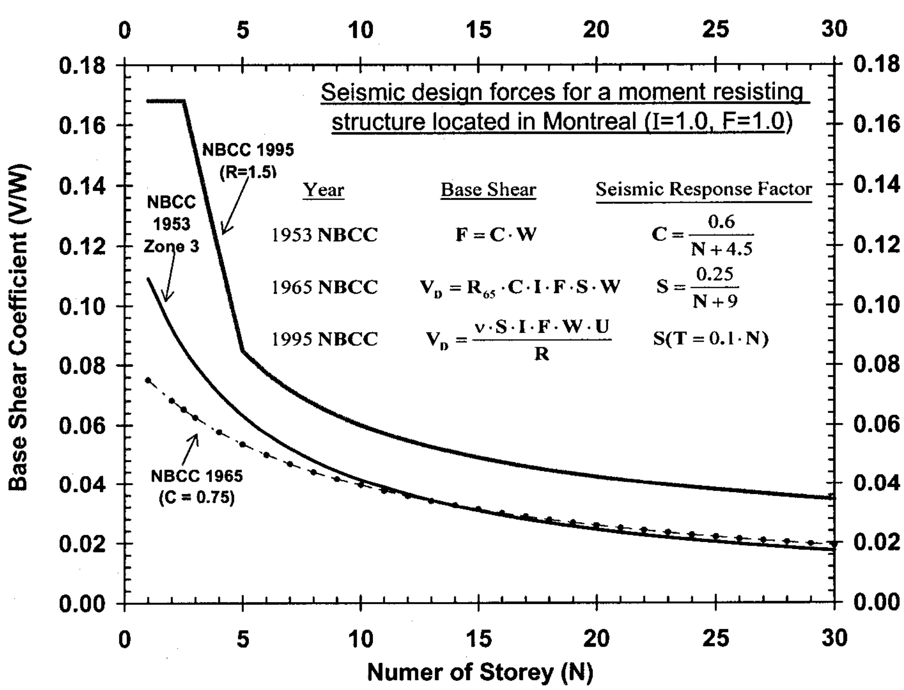


Figure 1-8: Level of seismic design base shear from various editions of NBCC.

1.4 Current Approach of Seismic Evaluation

The goal of conducting seismic evaluations is to identify the seismic vulnerabilities and deficiencies of structures so that they can be retrofitted if deemed necessary. Structural evaluation in the past can be a difficult process especially with the previous versions of the NBCC which adopted an "all or nothing" approach in seismic upgrading of existing structures *(Dandurand, 1988)*. However, guidelines in the Structural Commentaries of 1995 NBCC *(NRC, 1996)* contain a relatively simple approach to evaluate the seismic capacity of an existing structure by specifying that an existing structure shall be granted an acceptable performance if it is capable of resisting 60% of the seismic load computed and analyzed in accordance with the Part 4 (i.e., the Structural Design Section) of 1995 NBCC. The guidelines also explain that structures with unacceptable performance should be retrofitted and designed for the full seismic load as specified by the 1995 NBCC. It is noted that an attempt to implement the Part 4 design requirements in evaluating an existing building may result in impractical and expensive upgrades. Furthermore, in considering the large cost associated with over-conservative structural intervention, and to credit the satisfactory performance displayed so far by some of the existing structures, the "60% criteria" is a reasonably relaxed requirement.

Other guidelines published by National Research Council of Canada (NRC) *(1993a, 1995)* also provide detailed guidance and recommendations on the seismic evaluation and upgrades conducted on Canadian structures. In the case which a large number of buildings are to be evaluated, the screening procedure developed by NRC *(1993b)* can be used to prioritize the order of evaluations based on the seismic risk of each building. Generally, the NRC screening process involves the determination of a Seismic Priority Index (SPI) that reflects the building's performance with respect to the various seismic screening parameters (see Appendix A). The decision of whether the building should be subjected to a more detailed evaluation is based on the criteria shown in Figure 1-9. The NRC screening procedure is currently the only procedure used in

Canada for building screening as part of a seismic evaluation process *(Foo et al., 2001)*. However, changes to the current seismic screening procedure may take place due to changes to the seismic design provisions in the upcoming edition of NBCC *(NBCC, 2004)*.

Apart from Canada, other countries that are subjected to the threat of seismicity also have actively developed various seismic evaluation and retrofit guidelines. In the late 1990s, the Federal Emergency Management Agency (FEMA) of the United States published FEMA 310 *(FEMA, 1998)* and FEMA 356 *(FEMA, 2000)* as a consequence of the professional experiences and lessons gained from the damaging earthquakes that occurred in Mexico, California, and Japan. FEMA 310 outlines a comprehensive seismic evaluation procedure based on a three-tier process which includes a screening phase, an evaluation phase, and a detailed evaluation phase that incorporates a nonlinear analysis of the structure. FEMA 356 is a guide to systematic rehabilitation and intended to be used as a follow up to an evaluation previously conducted. The rehabilitation guideline promotes the use of performance-based design for new seismic upgrades, and engineers are required to select a desired building performance level at a user-defined seismic hazard level listed in Figure 1-10. Despite a few limitations highlighted in D'Ayala & Charleson *(2002)*, the FEMA standards are widely recognized and practiced within and outside the United States. Further discussion and comparisons of the seismic evaluation guidelines of other countries such as the Europe and New Zealand are available in D'Ayala & Charleson *(2002)*.

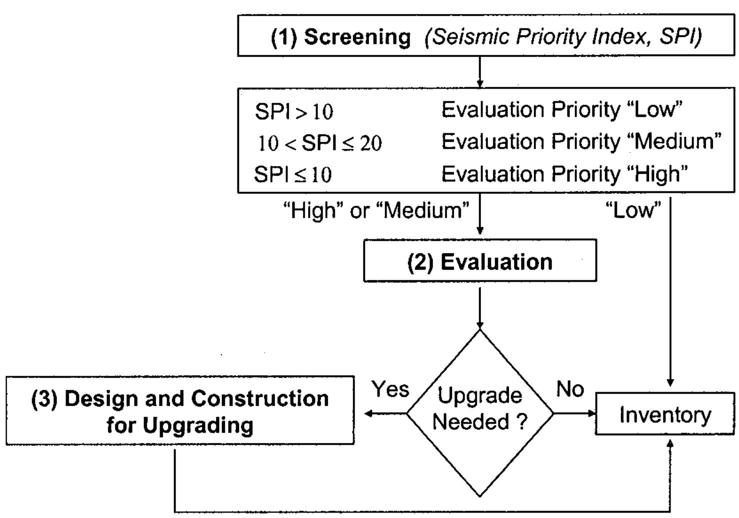
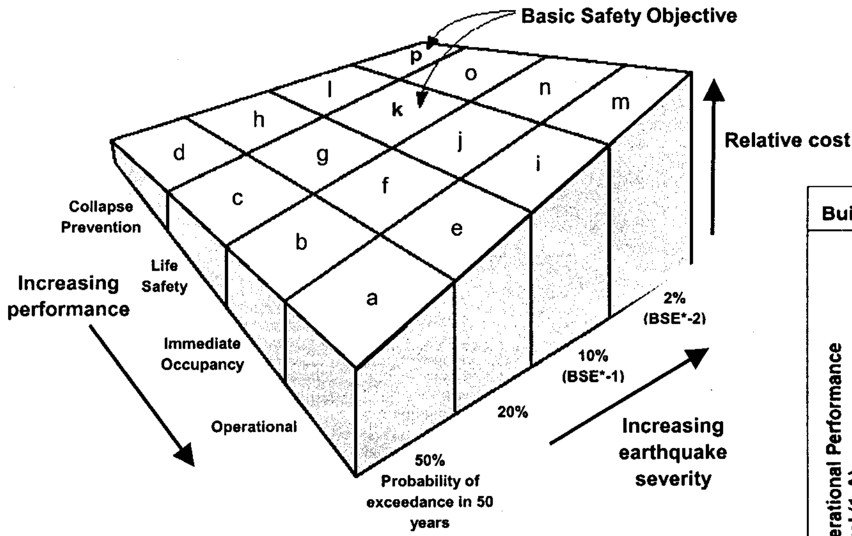


Figure 1-9: The NRC seismic screening procedure *(Foo et al., 2001)*.

Relationship between Rehabilitation Objectives and Program Cost



*BSE = Basic Seismic Earthquake

Earthquake Hazard Level		Building Performance Levels: Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~10%/50 year)	i	j	k	l
	BSE-2 (~2%/50 year)	m	n	o	p

Performance Levels	Descriptions	Drift
Immediate Occupancy	- Limited damage to both structural and non-structural elements during the design earthquake - Vertical and Lateral force resisting systems retain nearly all of their pre-earthquake strength and stiffness - Level of risk for life-threatening injury is low - The building is fully habitable after a design earthquake - Minor post-earthquake repair may be necessary	1% Transient or negligible permanent
Life safety	- Significant damage to both structural and non-structural elements during the design earthquake - Partial or total collapse is prevented - Injuries may occur but the level of risk for life-threatening injury and entrapment is low	2% Transient or 1% permanent
Collapse Prevention	- Hazard to life safety may result due to damaged non-structural elements	4% Transient or permanent

Figure 1-10: A surface matrix showing the relationships between costs, rehabilitation objectives, seismic hazard levels, and the performance levels emphasized in FEMA 356 *(FEMA, 2000).*

1.5 Summary of Previous Investigations

1.5.1 Seismic Evaluation and Strengthening of Concrete Structures

Dandurand *(1988)* and Mitchell *et al. (1988)* have presented an inventory of repairs and strengthening techniques in Mexico City after the 1985 earthquake which have offered some qualitative guidance to the seismic upgrading of existing reinforced concrete structures in Canada. Dandurand has also presented a number of case studies on structural upgrades performed in Canada, and addressed some considerations encountered during the design of seismic upgrades such as the treatment of existing structural elements and the design of infilled shear walls.

Mitchell *et al. (1990)* have inspected, evaluated and retrofitted a single-storey precast concrete industrial structure located in Montréal. The adequacy of the structure was verified according to the static earthquake loading specified in 1965 NBCC, and then retrofitted to satisfy the requirements of 1985 NBCC. The single-storey warehouse was retrofitted by encasing a group of its existing columns with reinforced concrete sleeving. The seismic upgrade has substantially strengthened the structure for lateral loading and it is a technique recommended for similar precast concrete structures in which seismic retrofits are needed.

After the 1999 Turkish earthquake, a number of seismic evaluations and studies on the damaged concrete structures were performed. In 2002, Posada and Wood *(2002)* studied the many precast concrete frames that collapsed during the Turkish earthquake, by comparing the yield displacement capacities of the precast columns with the stiff-soil and soft-soil ground motion records taken during the earthquake. The study showed that the precast frame structures are more likely to collapse when subjected to soft-soil ground motion than when founded on stiff soil, and this phenomenon is insensitive to the amount of longitudinal reinforcements in the columns. The study concluded that the damage to the structures can be reduced only by increasing the dimensions of the precast columns.

Thompson *et al. (2002)* have evaluated and developed a conceptual strengthening scheme for a significantly damaged four-storey concrete frame building with clay-infilled walls during the Turkish earthquake in accordance with the FEMA 310 *(FEMA, 1998)* and FEMA 356 *(FEMA, 2000)* guidelines. The damaged structure was strengthened by inserting new shear walls in its principal directions, and the increase in the lateral load resistance of the building was demonstrated by a pushover analysis. The authors concluded that the structure strengthened according to the FEMA 356 can achieve a Life-safety performance at a base shear level lower than that specified by the Turkish building code. D'Ayala and Charlesons *(2002)* have also conducted a seismic evaluation on a Turkish six-storey moment resisting frame building with masonry-infilled walls according to both FEMA 310 and the New Zealand seismic evaluation guidelines. A discussion on the strengthening strategies applicable to this building was also presented in the report.

Hawkins *et al. (2000)* studied a series of precast concrete diaphragms that were damaged during the 1994 Northridge earthquake. The analyses concluded that a shear-friction design method along with the temperature and shrinkage design requirement is appropriate for the design of concrete topping reinforcement. Moreover, the analysis showed that the induced shears and moments in the diaphragms increase with greater span, and thus, the lateral load resisting system should be located evenly on the building plan to reduce the seismic resistance demand on the diaphragm.

Assaad *(2000)* performed a seismic evaluation of the Sécurité Civile Building in Montréal based on the seismic design provisions of 1985 and 1995 NBCC. The three-storey post-disaster concrete structure constructed in 1990 was reported to be satisfactory under 1985 NBCC seismic codes, provided minor strengthening on the structure is conducted. However, the structure was considered inadequate according to the more stringent requirements imposed by 1995 NBCC. It was concluded that an additional seismic evaluation based on the upcoming edition of the NBCC seismic design provisions should be carried out so as to justify a major seismic upgrade of the structure.

Foo *et al. (2001)* discussed the seismic design provisions of the current National Building Code of Canada *(NBCC, 1995)*, in particular to its application to the seismic evaluation of existing structures. The document has reviewed the variance in seismic evaluation regulations in different regions of Canada and has briefly summarized the current seismic screening methodology in the country. This report also discussed recent seismic upgrading research. Similarly, Moehle *(2000)* summarized the issues pertinent to the seismic vulnerability of existing concrete structures and presented a comprehensive summary on recent research advances in seismic rehabilitation techniques in the United States.

1.5.2 Masonry-Infilled Walls

Different analytical models have been proposed to simulate the behaviour of structures containing masonry infills. The most common approach is the use of diagonal compressive struts. Stafford-Smith and Carter *(1969)* compiled their findings from previous analytical and experimental investigations *(Stafford-Smith, 1962, 1966)* on mortar infilled frames and presented a guide to model equivalent masonry diagonal strut. This report has included guidance on computation of equivalent strut widths and strengths associated with different failure modes of an infilled frame, based on a set of empirical curves related to a dimensionless parameter that expresses the relative stiffness of the infill to the frame. However, the strut width obtained based on this method was considered to be too large and produces unrealistic strut capacities in comparison with the experimental results. Mainstone *(1971)* proposed an alternative strut width formulation based on the relative stiffness parameter and resulted in a smaller and more realistic strut width.

Paulay and Priestley *(1992)* explained that only after infill separation at about 50% to 70% of the ideal shear capacity of the infilled frame, could the system then be modeled as a diagonally braced frame. The authors also proposed a conservative constant strut width that equals to a quarter of the diagonal length of an infilled frame. The authors outlined a few analytical models to evaluate the in-plane resistances of an infilled frame for various failure modes such as tension failure, sliding shear failure (Figure 1-11 (a)) and diagonal strut compression failure. The authors also stated that an adequately connected infill wall can develop a compression membrane resistance (Figure 1-11 (b)) that is generally sufficient to resist out-of-plane deformation.

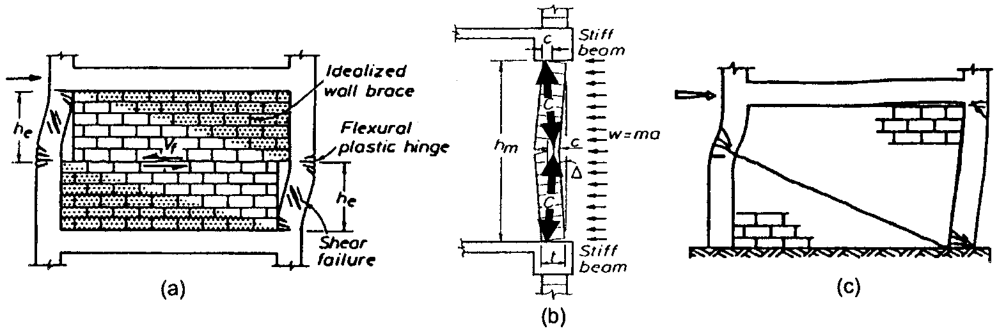
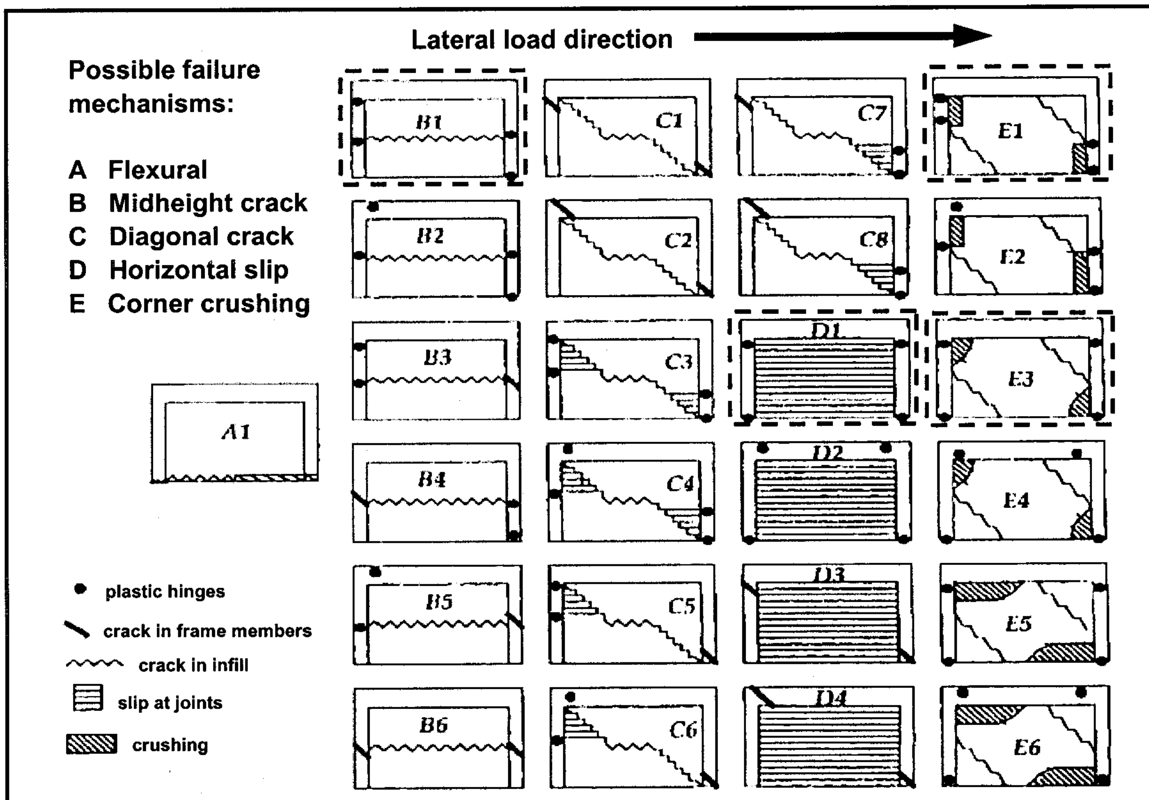


Figure 1-11: Masonry-infilled frame investigations discussed by various researchers: (a) Sliding shear failure mode and; (b) compression membrane resistance discussed by Paulay and Priestley *(1992)*; (c) diagonal tensile crack failure investigated by Tomaževič *(1999)*.

Tomaževič *(1999)* discussed pertinent issues regarding the seismic behavior of masonry infill frames on the basis of European design applications. He further outlined an analytical model that uses concentric and eccentric struts to evaluate the stiffness and strength of a fully infilled frame at different stages of a diagonal tensile crack failure (Figure 1-11 (c)). Crisafulli *et al.* *(2002)* proposed an analytical approach to estimate the strength of the equivalent compressive strut based on the finite element analyses and a failure theory of unreinforced masonry previously published by the authors.

Al-Chaar *(2002)* published a comprehensive procedure on the structural evaluation of unreinforced masonry-infilled structures. This procedure is thorough and similar to that adopted in FEMA 356 for the analysis of masonry-infilled frames. The evaluation involves a pushover analysis of the structure, and the pushover load-displacement curve is approximated by a bilinear relationship to estimate the in-plane stiffness of the structure. This report has also included an out-of-plane analysis procedure. The strut width expressions derived by Stafford-Smith and Carter *(1969)* and Mainstone *(1971)* were adopted in this report to predict an infilled frame behaviour. Eccentric strut and plastic hinges are assigned on the strut-and-frame model to capture realistic failure mechanisms of infilled frames. A method to account for openings in infill panel is also explained in the report. Although the procedure is mainly designed for a non-linear static procedure, the procedure may be modified to suit other analysis platforms, such as linear static or dynamic analysis.

Mehrabi and Shing *(2002)* summarized experimental and analytical research on both in-plane and out-of-plane behaviour of masonry-infilled frames. The authors concluded that the failure mechanisms of infilled frames, which depend largely on the frame-infill interactions, are complicated and cannot be completely represented only by the use of a strut model. Mehrabi and Shing described that there is no single analytical model that can capture all possible load resisting mechanisms of an infilled frame. The authors proposed limit analysis models for the five most probable load resistance mechanisms of reinforced concrete infilled frames (4 of which are highlighted in Table 1-2). The authors also have concluded that out-of-plane failure of a properly connected and fully infilled frame is well resisted by an arching mechanism between the bounding frame and the masonry panel.



Failure Modes	Experimental Observations
(A) Flexural	- The Infill and the frame behave as one flexural element - Separation of the frame and infill does not occur
(B) Midheight crack	- Horizontal Sliding at the mid-height of the infill frame - Short-column shear failure may occur in the column - Plastic hinges may occur at the column mid-height
(C) Diagonal crack	- The infill can develop diagonal strut mechanism - Corner crushing may occur - Diagonal cracks will propagate from the loaded corner - Frame member may fail in plastic hinging or shear failure
(D) Horizontal slip	- Infill slidings will occur at bed-joints - Failure mechanism usually occurs in infills with weak mortar joints - A fairly ductile behaviour can be achieved (provided that shear failure is avoided)
(E) Corner crushing	- Diagonal struts mechanism occur with two distinct parallel cracks - Corner crushing can occur at the corners of the infill

Table 1-2: Failure modes of masonry-infilled frames summarized from Mehrabi and Shing *(2002)*.

1.6 Research Objective and Scope

This research project was conducted in collaboration with the Department of Building Services of the City of Montréal as a part of a wider study on seismic hazards for the City of Montréal. As emergency response buildings require a high degree of reliability during and following an earthquake, a forty-year old single-storey precast reinforced concrete structure with masonry-infilled system was selected for a seismic evaluation. The selected building is typical of several emergency response buildings owned by the City of Montréal and it functions as a warehouse for stocking of emergency supplies.

The research program will demonstrate an evaluation approach and a retrofitting scheme that is applicable to similar structures. The scope of the thesis includes:

- Report on the building condition based on an on-site inspection.
- Conduct studies on the seismic performance of the building in accordance with the equivalent static provisions of 1995 NBCC and 1965 NBCC.
- Investigate the adequacies of the existing precast columns and the influence of roof diaphragm as well as the masonry-infilled walls with the use of a series of three dimensional computer models.
- Conclude on the seismic analysis results and identify the seismic deficiencies of the structure.
- Summarize issues relevant to seismic rehabilitation of concrete structures and discuss a practical seismic retrofitting scheme for the building. The retrofitting scheme is to be analyzed and designed according to the present seismic design standards.

Chapter 2: Description of the Structure

2.1 General

In order to conduct a seismic evaluation for an existing structure, sufficient and accurate as-built information of the structure is required. As-built information includes layout of the structural system, material properties, and member detailing specified at the time of construction. A common problem encountered in the evaluation of an older structure is that complete documentation of the as-built structure is either lost or inaccessible. The information provided in the following sections are comprised of figures obtained from the available plans of the structure under evaluation; and rational assumptions made by referring to the building code that was published near the time of original construction *(NBCC, 1965)*. References containing abbreviated history of construction materials *(CRSI, 2001)* and common construction techniques at the time of construction were also considered in order to define a set of appropriate input for the evaluation of the structure.

2.2 The Structural Layout

2.2.1 Building Plan

The building under investigation is a single-storey warehouse managed by Services des achats et magasins located in Centre Louvain, Montréal. The structure stocks emergency supplies for disastrous events and it has to remain serviceable after a major earthquake and thus, it is conservatively categorized as a post-disaster building. The warehouse is situated in a relatively open area and it is linked to a single-storey office building situated on its east side through a passage way. The building plans were prepared in 1963 and drawings relevant to the member detailing are unavailable.

The building is approximately 401 ft (122.3 m) long and 241 ft (73.5 m) wide with a plan area of approximately 96641 ft^2 (8989 m^2) (Figure 2-1). It has 10 bays along its length and 5 bays along its width. The column lines are regularly spaced, with a centre-to-centre spacing of 40 ft (12.2 m) lengthwise and 48 ft (14.6 m) along its width. The office area is mainly concentrated in the area bounded by column lines A, B, and 4 to 8. Two truck unloading areas are located between column line 1 to 4 and 9 to 11 along column line A. The rest of the floor area is used for storage purpose. The structure has a general clear height of 13 ft (4 m) (Figure 2-2). Due to the presences of the truck unloading ramps, the grade elevation of column line A on column lines 1 to 4 and 9 to 11, are lower by 4 ft (1.2 m). The precast reinforced concrete structure has beam-to-column framing and the roof consists of precast double-tee slabs. The roof is topped with a 2 in. (0.05 m) thick insulation and roofing. On area C-D-5-6, an 11 ft (3.4 m) high penthouse is present at the roof level for the housing of ventilation equipment. The building is founded on soil with an allowable compressive strength of 6 kips/ft^2 (287 kPa) and its perimeter is infilled with masonry walls.

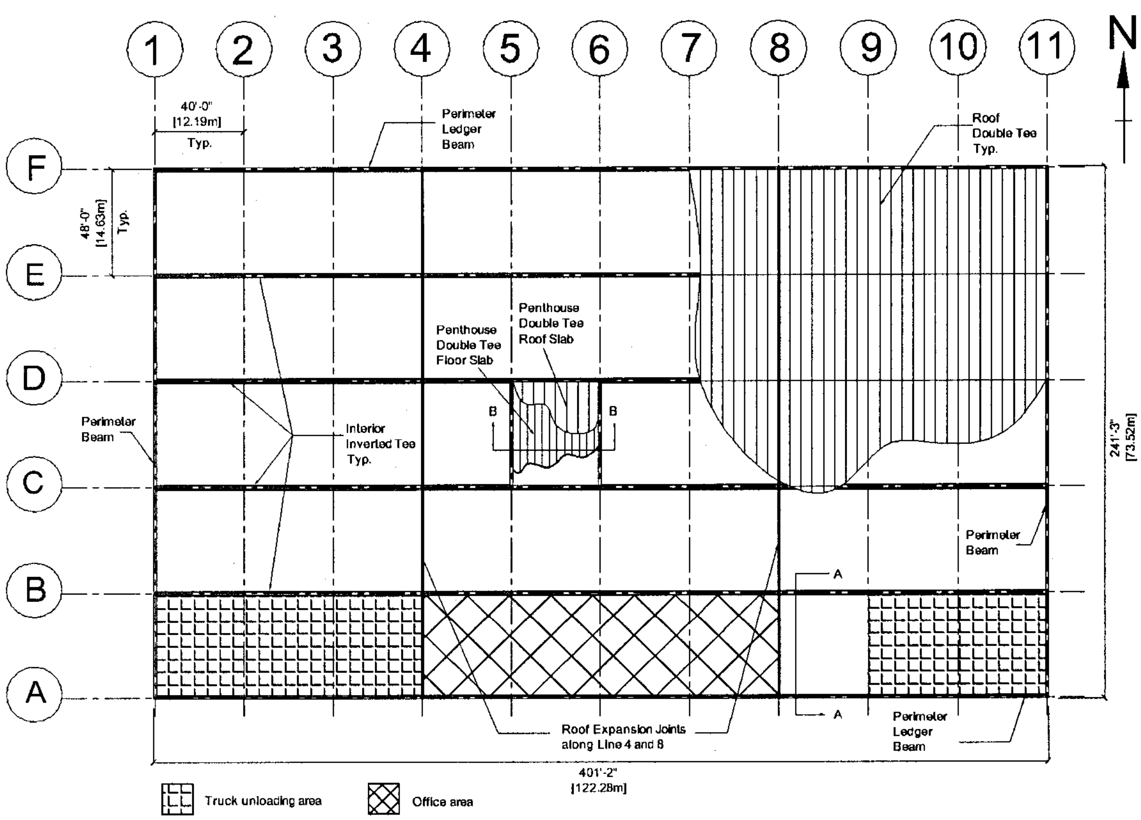


Figure 2-1: Plan view of the warehouse.

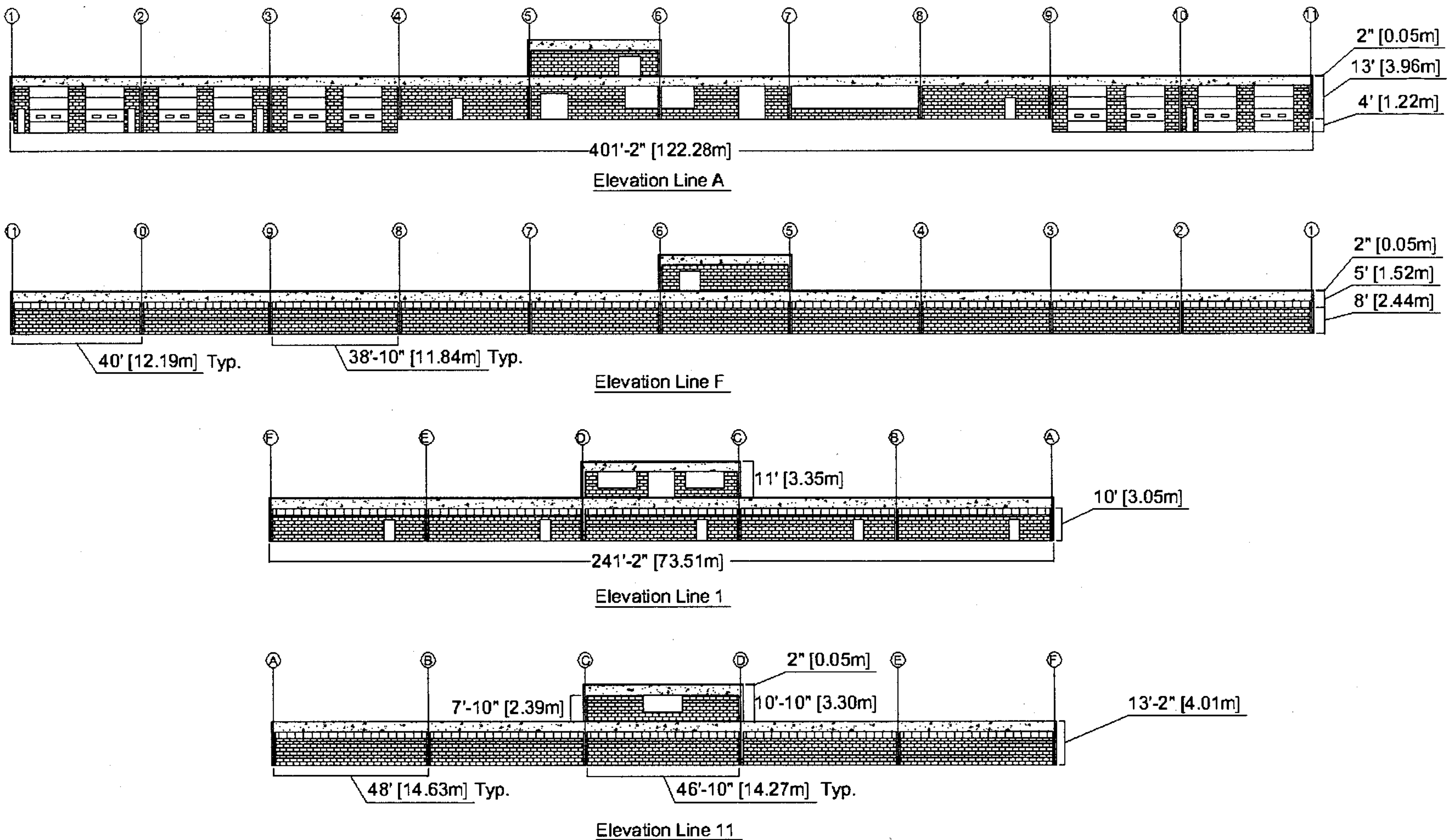


Figure 2-2: Elevation views of the warehouse.

2.2.2 Roof Members

The roof of each bay is covered by eight 5 ft wide by 18 in. deep (1524 mm by 457 mm) precast double-tees that span in the North-South direction (Figure 2-3). Under the penthouse on area C-D-5-6, double-tee floor members of dimensions 3 ft - 4 in. wide by 18 in. deep (1016 mm by 457 mm), and together with two 14 in. wide by 32 in. deep (356 mm by 819 mm) beams provide the roof framing (Figure 2-4). The double-tee roof members are supported by the 36 in. (914 mm) deep perimeter ledger beams along column lines A and F, and internally by the 36 in (914 mm) deep inverted-tee beams along column lines B to E. The two ledger beams are framed into rectangular beams of dimensions 14 in. wide by 36 in. deep (356 mm by 914 mm) at column lines 1 and 11. All precast double-tees and beams are simply supported and have simple connections to the precast columns. The detailed dimensioning and properties of all prefabricated members are given in Table 2-1. Information on interior reinforcement and the connections between the double-tees are not available. The geometric properties of a typical double-tee roof member were taken from a similar double-tee section in PCI *(1971)*; and they are modified accordingly to determine the assumed sectional properties of the smaller double-tee members supporting the penthouse.

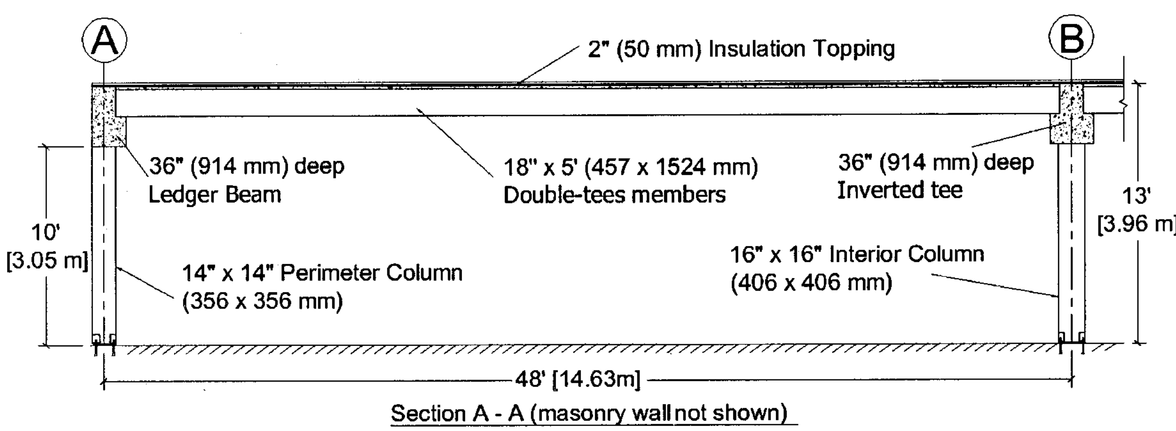


Figure 2-3: Elevation view of Section A-A.

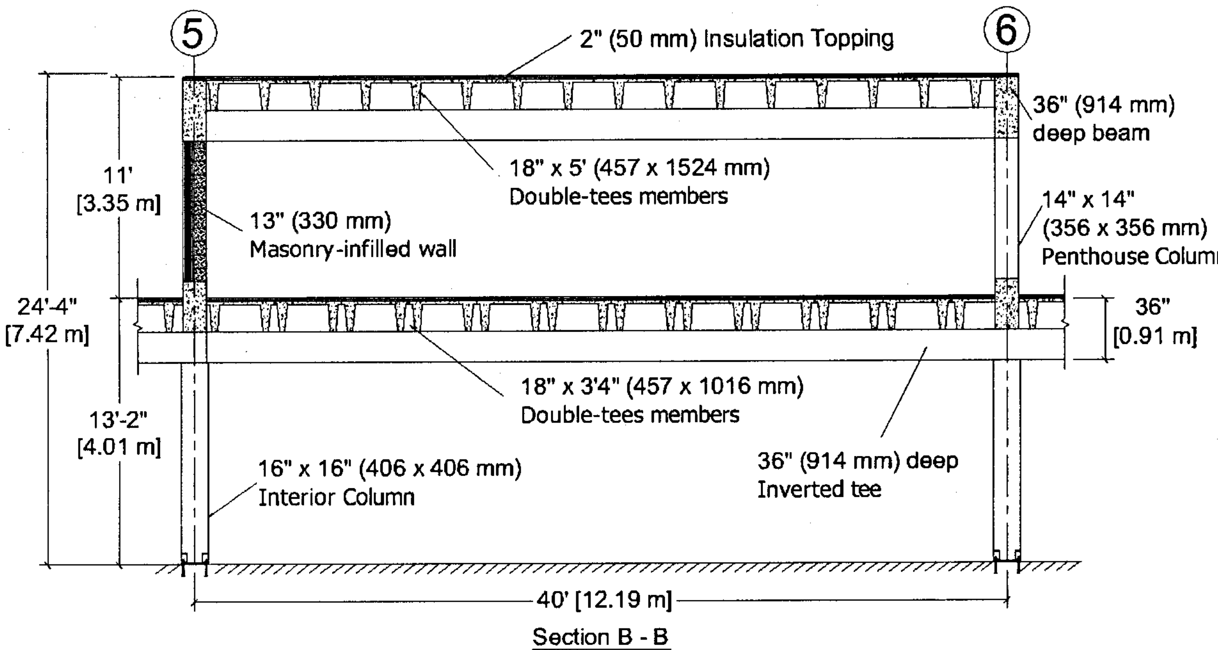
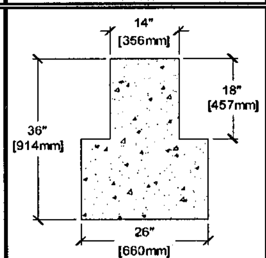
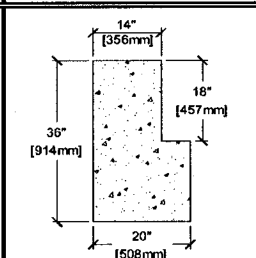
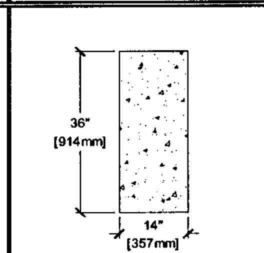
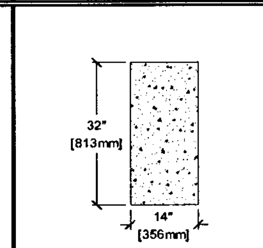


Figure 2-4: Elevation view of Section B-B.

	Inverted-Tee Beam		Ledger Beam		Perimeter Beam		Interior Beam	
Section								
Area	720 in^2	0.465 m^2	612 in^2	0.395 m^2	504 in^2	0.325 m^2	448 in^2	0.289 m^2
yb	15.3 in.	388.6 mm	16.4 in.	416.9 mm	18 in.	457.2 mm	16 in	406.4 mm
Selfweight	0.75 kips/ft	10.95 kN/m	0.64 kips/ft	9.31 kN/m	0.53 kips/ft	7.66 kN/m	0.47 kips/ft	6.81 kN/m

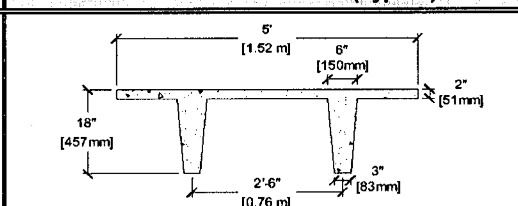
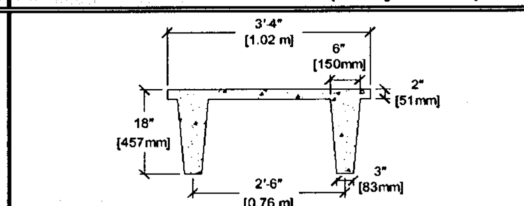
	Double-Tee Roof Member (Typical)		Double-Tee Floor Member (at bay C-D-5-6)	
Section				
Area	268 in^2	0.173 mm^2	228 in^2	0.147 mm^2
I	7546 in^4	3.24×10^9 mm^4	6403 in^4	3.00×10^9 mm^4
yb	12.48 in	317 mm	11.69 in	297 mm
Selfweight	278 lb/ft	4.05 kN/m	236 lb/ft	3.45 kN/m

Table 2-1: Sectional properties of precast members.

2.2.3 Columns

The typical interior and perimeter (and penthouse) columns are of dimension 16 in. by 16 in. (406 mm by 406 mm) and 14 in. by 14 in. (356 mm by 356 mm), respectively. These square columns have a typical clear height of 10 ft (3 m) except for columns A2, A3, and A10 which are 14 ft (4.3m) in height. The as-built details of various column base connections are shown in Figure 2-5. Each precast column is bolted to the pedestals with four anchor bolts through its preinstalled pockets and is grouted to form a moment connection. Unlike the grout detail for an interior column, the perimeter columns have their outer grout surface being offset a distance of 1 in. (25.4 mm) away from the outside column surface (see Figure 2-5). The direction of offset depends on the column location on the plan. For example, the perimeter grout section in Figure 2-5 represents a typical section located on column line 1. The corner grout sections are eccentric to the column bases in its both principal directions.

The single-storey warehouse relies entirely on its cantilever columns for its lateral load resistance, thus, the reinforcing details are important in determining the seismic performance of the columns. Three possible reinforcing schemes (A, B and C) for the columns were assumed. It was first assumed that at least eight dowel bars were welded to the steel angle pockets to form the minimum reinforcement at level A-A as shown in Table 2-2. Scheme A assumes that a 4 bars reinforcing cage is slipped over the dowel bars over a lap splice length. Scheme B assumes that the dowel bars are extended throughout the entire height of the column. Finally, scheme C combines the previous two schemes and forms a more ductile reinforcement. In all three schemes, #5 and #3 bars are used as longitudinal and transverse reinforcement respectively.

In accordance with 1965 NBCC, all sections shall have a minimum clear cover of 1.5 in. (38 mm). The column ties used have "standard hooks" defined in 1965 NBCC which includes a 90-degree bend plus an extension of at least 6 bar diameters giving a distance of 3.8 in. (97 mm). The 1965 NBCC specified that column tie spacing is given as the smallest of: (i) 16 longitudinal bar diameters; (ii) 48 tie diameters; (iii) the least dimension of the column. Calculations showed that the tie spacing of all columns is governed by a distance of 16 bar diameters which is 10 in. (254 mm), and this tie spacing was assumed to be halved immediately above the pockets. For scheme A and C, the tension lap splice length specified by 1965 NBCC is founded to be 24 bar diameter for bars with a yield strength of 40 ksi (276 MPa) but not less than 12 in (305 mm). Calculations showed that a minimum tension lap splice length of 15 in. (381 mm) is needed.

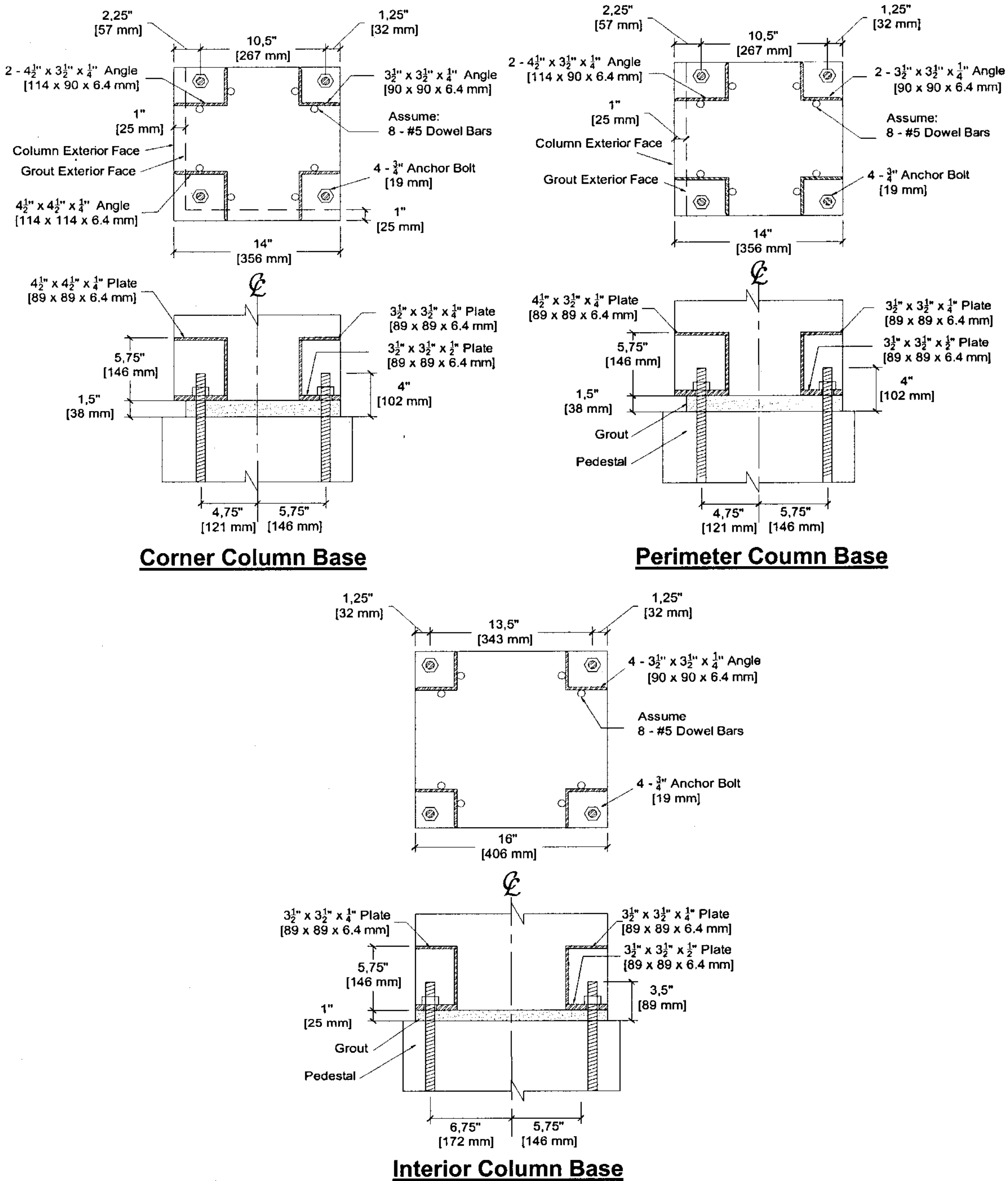


Figure 2-5: Column base connections.

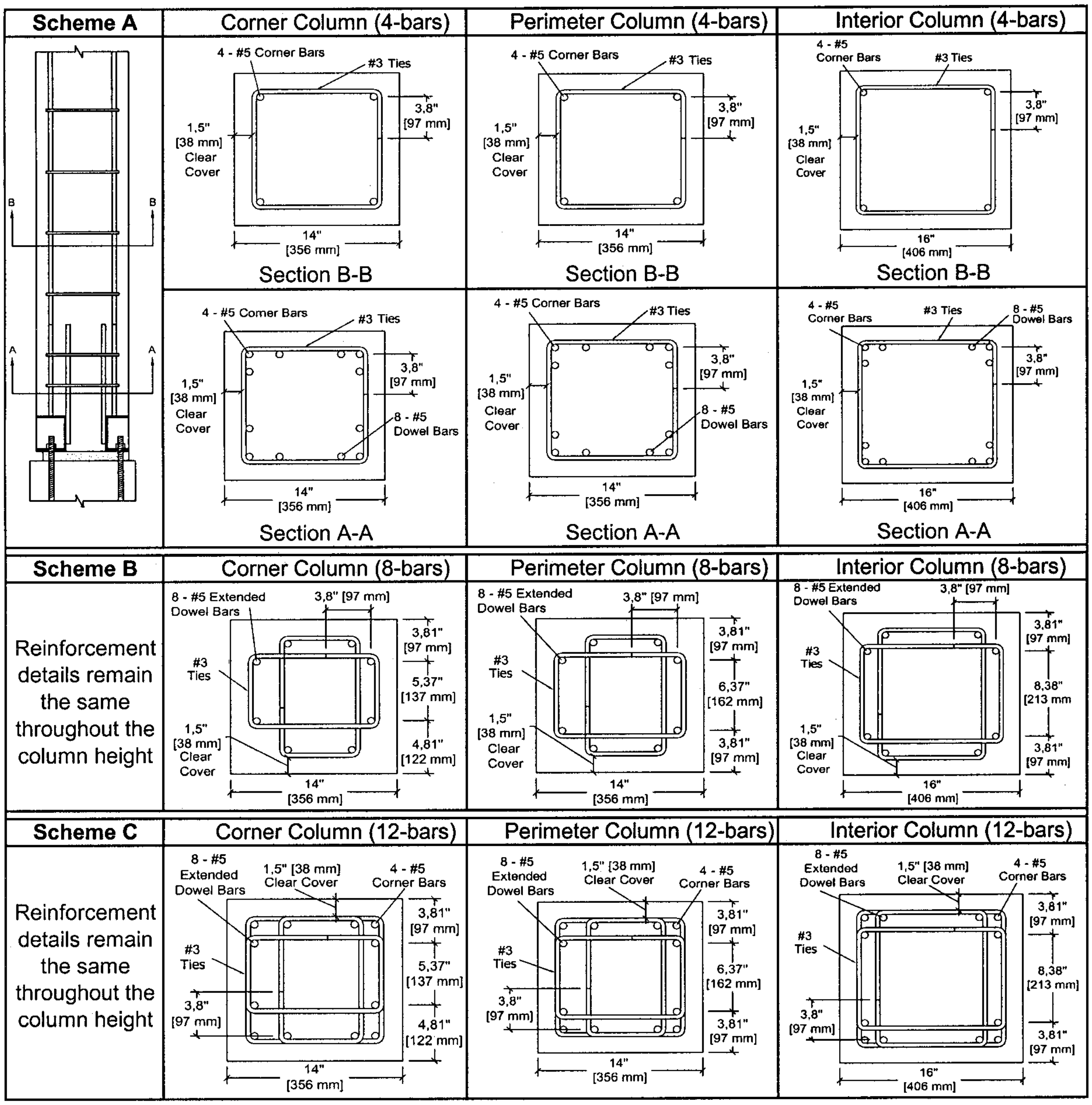


Table 2-2: Assumed reinforcing details for columns in the warehouse structure.

2.2.4 Masonry Walls

Masonry-infilled walls are used as environmental separators along the building perimeter. The 13 in. (330mm) thick masonry-infilled wall is made up of a layer of clay brick veneer and a layer of hollow concrete blocks which are filled with "Zonolite" insulation (Figure 2-6). Both layers of the infilled wall are unreinforced, and they are not structurally tied to the bounding frames. Frames along column lines 1, 11, and F are partially infilled to an uniform height of 8 ft (2.4 m) to allow room for installation of windows. Due to the presence of garage doors and a series of window openings, the perimeter concrete frames along column line A are irregularly infilled to various extents (Figure 2-2).

Besides being used as an infill material, concrete masonry blocks are also used for interior partitions. It is believed that the interior masonry partition walls were installed after the construction of the main structural since there is no evident connection installed between the roof members and the partition walls.

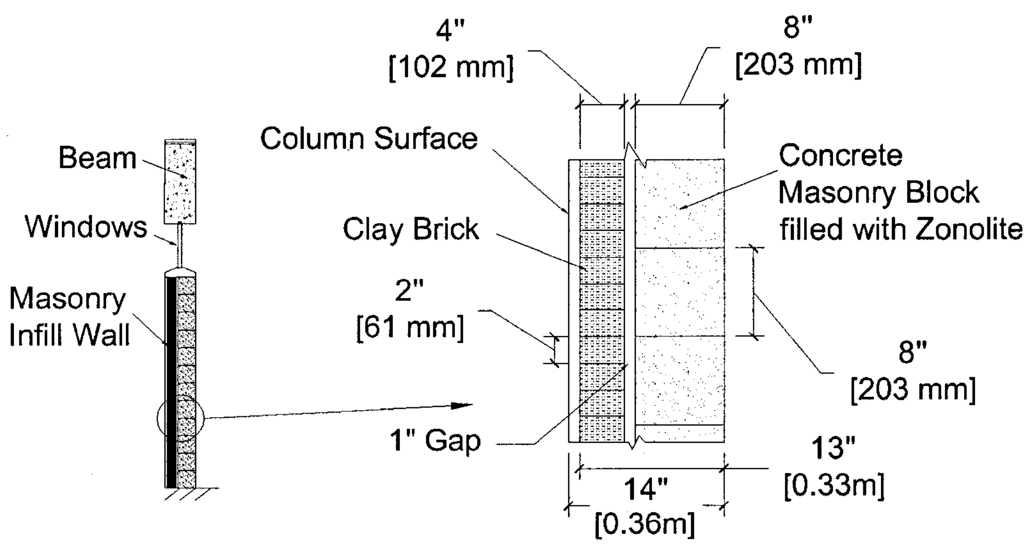


Figure 2-6: Details of masonry-infilled walls.

2.3 Materials

2.3.1 Concrete

The existing precast reinforced concrete elements are assumed to be fabricated with normal density concrete with a compressive strength, $f_c^{'}$, of 5000 psi (35 MPa). The elastic modulus of the concrete E_c is calculated according to Equation 2-1 given by the CSA A23.3-94 Standard *(CSA, 1994)*:

$$E_c = 4500 \cdot \sqrt{f_c^{'}} \qquad \textbf{Equation 2-1}$$

Both E_c and $f_c^{'}$ in Equation 2-1 are in MPa units, and E_c is calculated to be to be 3832 ksi (26422 MPa). The specified material properties of the grout are conservatively assumed to be identical to the concrete with a compressive strength of 5000 psi (34.5 MPa).

2.3.2 Steel

The steel reinforcement in the precast elements and including the deformed bars, tie reinforcement, and anchor bolts, are assumed to be of intermediate grade having a minimum tensile yield strength of 40 ksi (276 MPa) and a modulus of elasticity E_s of 29000 ksi (200000 MPa). The ultimate tensile strength of the reinforcing steel is taken as 70 ksi (480 MPa). The calculated material properties that are used in the subsequent analysis are summarized in Table 2-3.

Concrete / Grout Material Porperties		
Density γ_c	150 lb/ft^3	2403 kg/m^3
Compressive Strength f'_c	5000 psi	34.5 MPa
Modulus of Elasticity E_c	3832 ksi	26422 MPa
Steel Reinforcement / Anchor Bolt Material Poperties		
Yield Strength f_y	40 psi	276 MPa
Ultimate Tensile Strenght f_{ult}	70 ksi	483 MPa
Modulus of Elasticity E_s	29000 ksi	200000 MPa

Table 2-3: Various material properties assumed for the structure.

2.3.3 Masonry Units

Based on the wall details illustrated in the as-built plans (Figure 2-6), the concrete and clay masonry units are assumed to be similar to those shown in Table 2-4. Since the clay tiles will be neglected in the subsequent analysis, their compressive strength is not investigated herein. For the concrete masonry blocks, their bed-joint material is assumed to be "Type S" mortar which is capable of achieving high bonding and lateral strength under minimum exposure *(Cutler, 1965)*. The minimum compressive strength of the "Type S" mortar is taken as 1800 psi (12.41 MPa) *(NBCC, 1965)*. The hollow concrete masonry unit has a void ratio of about 45%, and its density is approximately 90 lb/ft^3 (1441 kg/m^3) *(Salin, 1971)*. The compressive strength of each concrete masonry unit is conservatively taken as 2031 psi (14 MPa) *(Bruneau, 1994)*. However, due to the fact that a masonry assemblage has a lower compressive strength than an individual unit, the compressive strength of the overall concrete masonry assemblage, f'_m, is found by interpolating the compressive strength values tabulated for concrete blocks jointed with "Type S" mortar (*Bruneau, 1994)* and a value of 1326 psi (9.14 MPa) is obtained. The shear strength, V_m, and the modulus of elasticity, E_m, of the unreinforced hollow concrete masonry blocks, are defined by Equation 2-2 and Equation 2-3 *(NBCC, 1965)*, and are calculated to be 26.5 psi (183 kPa) and 1326 ksi (9140 MPa) respectively.

$$V_m = 0.02 \cdot f_m' \quad \leq 50 \text{ psi (345 MPa)}$$ **Equation 2-2**

$$E_m = 1000 \cdot f'_m \quad \leq 3000 \text{ ksi (20684 MPa)}$$ **Equation 2-3**

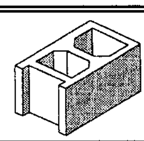
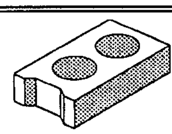
Hollow Concrete Masonry Block			
	Length	16 in.	406 mm
	Width	8 in.	203 mm
	Height	8 in.	203 mm
	Density γ_m	90 lb/ft^3	1442 kg/m^3
Compressive Strength f'_m	1326 psi		9.14 MPa
Modulus of Elasticity E_m	1326 ksi		9140 MPa
Shear Strength V_m	26.52 psi		183 kPa
Hollow Clay Brick			
	Length	8 in.	203 mm
	Width	4 in.	102 mm
	Height	2.7 in.	68 mm
	Density γ_m	125 lb/ft^3	2000 kg/m^3

Table 2-4: Properties of masonry units assumed for the structutre.

2.4 Building Inspection

The warehouse (Figure 2-7) was visually inspected to locate existing signs of distress and deterioration that can potentially affect its aseismic ability. No destructive testing was carried out during the site investigation. Despite the lack of a specific lateral load resisting system, the structure has performed well historically but has never been heavily solicited as it would during a major earthquake. Minor deterioration was observed at a few exterior beam-to-column joints (Figure 2-8) where concrete spalling was noticed and the embedded steel connection was exposed. However, no cracks were apparent on the masonry-infilled walls. The interior gravity load supporting system was generally in fair condition with minor deterioration due to aging (Figure 2-9). Observation suggested that the double-tee roof members might have been interconnected by typical welded-clip connections. However, the distribution of these connections discontinues at some of the double-tee interfaces (Figure 2-9), and in addition to the absence of a cast-in-place concrete topping, the ability of the roof to function as an effective roof diaphragm is uncertain. The inspection also revealed that some of the double-tee members sitting on the perimeter ledger beams have experienced significant shear cracks at their supporting stems (Figure 2-10). Concern is also expressed as to the significant displacement of the structure in the East-West direction, where the masonry-infilled walls and the connected window panels experienced severe out-of-plane bowing and separation from the precast framing. Prior to the time of inspection, a series of bolted "steel straps" were installed in order to secure the displacing perimeter infilled frame to the double-tee stems. However, this remedy was ineffective as most of the bolts are eventually loosened or have fallen out of place (Figure 2-11). The structure is also reported to have experienced minor vibration from trains passing on a nearby railway track.

Based on the observed building performance and the available as-built information, a number of potential seismic hazards associate with the building are identified. A major deficiency that may lead to a partial collapse of the building is the inadequacy of the existing columns. The cantilever columns are the primary aseismic components of the structure which dissipate seismic energy through flexural hinging at their bases. The strength and ductility of the existing precast columns, however, are limited by the non-ductile reinforcement. The structure also lacked a properly connected roof assemblage that allows the development of diaphragm action. The structural integrity of the roof members may also be jeopardized by the two roof dilation joints (on column line 4 and 8) where potential separation of the roof may take place. The overall structure has a symmetrical and regular plan layout. The column spacing is large to maximize the space usage and all frame members are required to span over a large distance. The structural framing of the warehouse is somewhat flexible and with a low structural redundancy, and it is susceptible to torsional effects. The perimeter masonry-infilled walls are not isolated from the precast framing and they may introduce high shear forces on the surrounding columns locally during an earthquake.

Based on the information gathered on the precast concrete structure, a NRC Seismic Priority Index (SPI) was calculated for the warehouse structure. A high seismic risk score of SPI = 29.6 indicates that the structure requires a detailed structural evaluation.

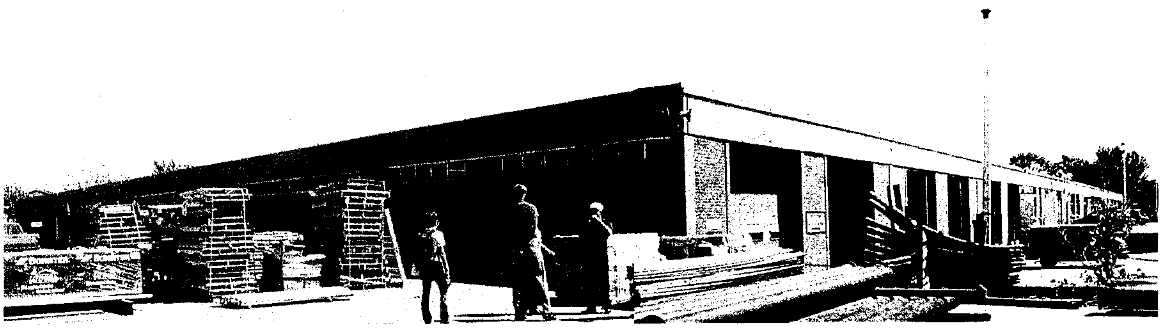


Figure 2-7: The single-storey precast concrete structure under investigation.

Figure 2-8: Exterior condition of the building and deterioration at an exterior beam-to-column support.

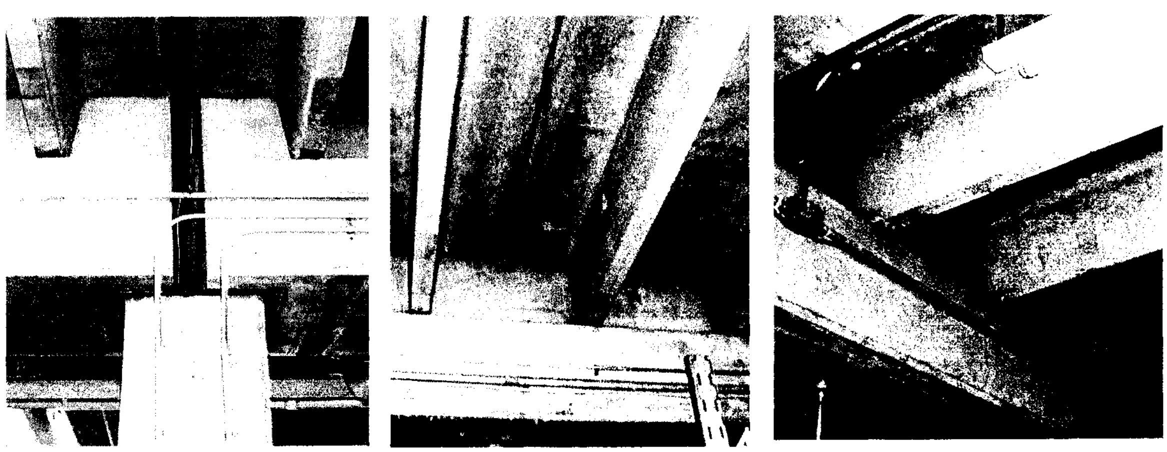


Figure 2-9: Interior framing and connections between the precast elements.

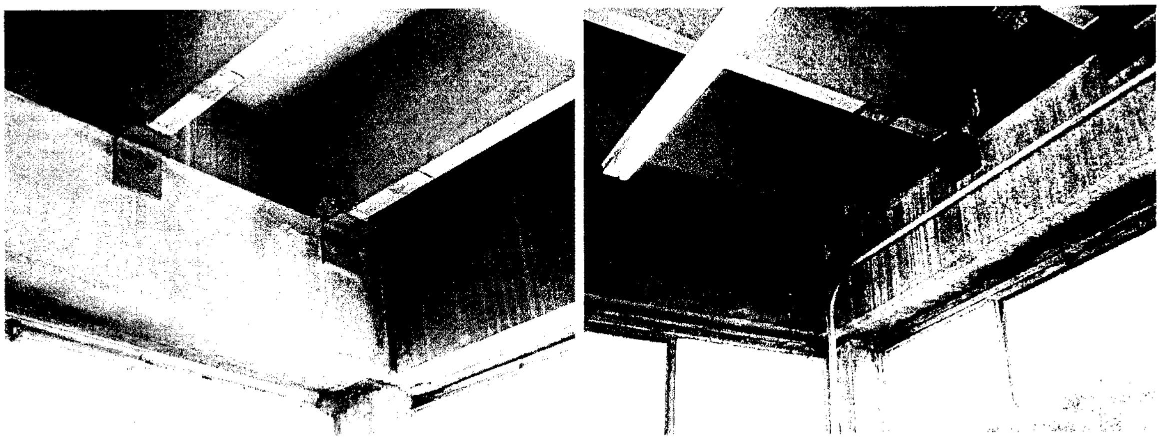


Figure 2-10: Shear cracks at the double-tee supported ends.

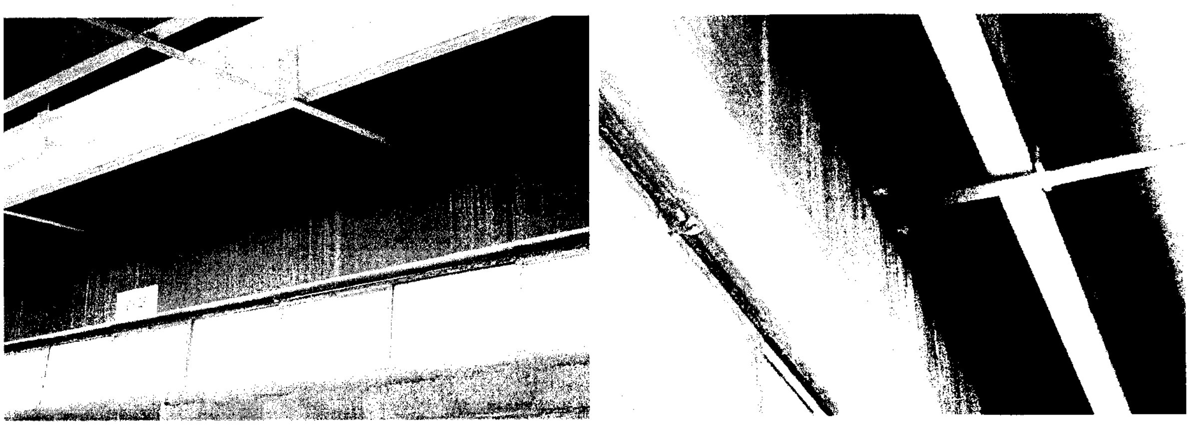


Figure 2-11: Steel straps installed previously to minimize out-of-plane bowing of the perimeter wall.

Chapter 3: Structural Modelling and Analyses

3.1 Introduction

The objective of this research program is to carry out an assessment of the seismic adequacy of the existing precast concrete structure according to the current building code requirements *(NBCC, 1995)*. Structural and non-structural vulnerabilities identified in the visual screening process are addressed in the seismic analysis. A seismic analysis can be conducted using a variety of procedures, which include linear or non-linear and static or dynamics analyses (Table 3-1). It should be noted that results computed by various analysis procedures may differ in considering the limitations of the respective procedures *(FEMA, 1998)*. The choice of an appropriate analysis for a given seismic evaluation depends on: the desired level of analysis; the building type and geometry; and regulations stipulated by the building codes or seismic evaluation manuals.

In recognizing that the Canadian building code ensures buildings constructed in Canada are of an adequate level of protection, the single-storey precast concrete warehouse was evaluated according to the 1995 NBCC evaluation criteria for existing buildings (i.e., the 60% criteria described in Chapter 1). The NBCC equivalent static procedure, which is essentially a conservative linear elastic procedure, was used to assess the strength and stiffness of the structure. For the comparison purpose, an analysis of the warehouse based on the seismic design provisions of 1965 NBCC was also conducted to benchmark the expected performance of the structure at the time of design and construction. Since the seismic resistance demand required by the current building codes is usually much higher than the capacities of existing buildings, it was decided to first analyze the structure with a normal seismic importance. If the structure demonstrates a satisfactory performance under the prescribed loading, the added requirements for a post-disaster building can be considered by introducing the appropriate seismic importance factors into the analysis.

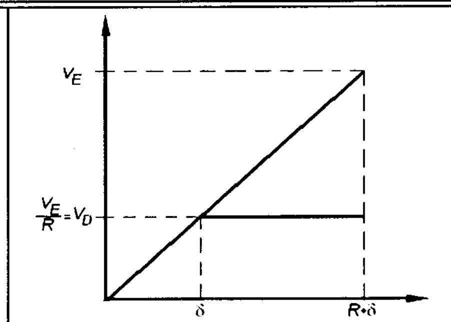
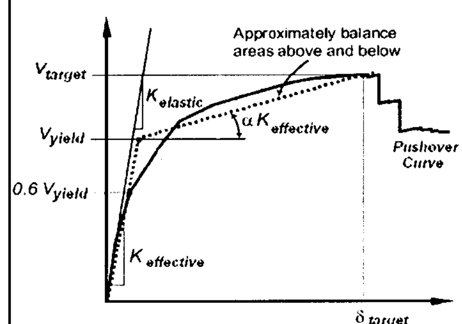
Analysis Methods	Comments	
Linear Static Analysis	- Elastic material properties are assumed - An equivalent design base shear is statically applied on the structure - Elastic member forces and storey displacements are computed. - Horizontal torsional effects are accounted by a specified eccentricity - A force reduction factor is used to account for ductility for force-base design (e.g., "R" in 1995 NBCC seismic provisions) - Stiffness and strength degradation of members are not accounted for - Not recommented for buildings with irregular mass and stiffness distribution - P-Δ effects may be included	 Idea force-displacement relationship of static analysis method of 1995 NBCC
Non-linear Static (Pushover) Analysis	- Inelastic material responses are accounted for - The analysis model is monotonically loaded in one direction - Equivalent elastic or plastic deformation and internal forces in each member are directly accounted for in the model - Maximum strength and global displacement capacities of the structure are determined - Effective global stiffness of the structure may be estimated by a bilinear load-displacement model - The load-displacement curve is often used in a displacement-based design procedure (e.g., FEMA publications) - Dynamics response after member yielding or degradation may not be accurately accounted for - P-Δ effects may be included	 Force-displacement curve of a pushover analysis given in FEMA 356
Response Spectrum and Modal Analysis	- Linear material properties are assumed to determine the maximum modal responses - Approopriate number of modes should be included to ensure sufficient particiapting mass are accounted for (e.g., 1995 NBCC requires at least 90% of the total mass are accounted in the analysis) - Appropriate combination method is selected to combine the modal responses (e.g., SRSS) - Suitable for buildings with irregular horizontal or vertical configurations - P-Δ effects may be included	
Non-linear Dynamic Time-history Analysis	- Non-linear material characteristics are accounted in the analysis. - A time-step integration procedure is used to determine the most realistic and complete seismic response for a specific earthquake record - Analysis greatly depends on a careful selection of input ground motion records - A set of selected accelerograms are usually used to generate an analysis "envelope" or an averaged repsonse - Suitable for buildings with irregular horizontal or vertical configurations - P-Δ effects may be included	

Table 3-1: Various analysis methods used in seismic evaluation.

3.2 Lateral loading on the Structure

3.2.1 1995 NBCC

The minimum static design base shear, V_D, for the design of a new building according to 1995 NBCC is given by:

$$V_D = \frac{V_e \cdot U}{R} = \frac{v \cdot S \cdot I \cdot F \cdot W \cdot U}{R} \qquad \textbf{Equation 3-1}$$

where the term V_e is the base shear corresponding to elastic response; υ is the zonal velocity (taken as 0.1 for Montréal area); F is the foundation factor (taken as 1.0 for stiff soil); I is the importance factor (taken as 1.0 for normal structure); U is a calibration factor with a value of 0.6; and R is the force modification factor (taken as 1.5 for a precast concrete structure with low ductility). The seismic response factor, S, which is dependent on the fundamental period of the structure, T, may be taken as 0.1 times the number of storeys, n, for a moment resisting structure, or alternatively computed by the following empirical equation:

$$T = \frac{0.09 \cdot h}{\sqrt{D_s}} \qquad \textbf{Equation 3-2}$$

where h is the height of the structure; and D_s is the length of the lateral load resisting system in a direction parallel to the design earthquake force. D_s may also be taken as the dimension of the building parallel to the applied force if the structure does not have a well defined lateral load resisting system. In taking the effective storey height of the warehouse as 10 ft (3 m) (i.e., the height of the main structure), calculations according to Equation 3-2 give structural periods of 0.10 seconds and 0.13 seconds in the North-South (N-S) and East-West (E-W) directions respectively. With the seismic response factor of the warehouse in both N-S and E-W directions being 4.2 (see Table 1-1), the seismic design base shear forces in both directions are computed to be 0.17·W. The effective seismic weight, W, is taken as the sum of total dead load plus 25% of the snow load and 50% of the live loads due to use and occupancy listed in Table 4.1.6.3 of 1995 NBCC.

The computed design base shear is distributed over the height of the structure in proportion to the seismic weight at each level according to the following equation:

$$F_z = V_D \cdot \frac{W_z \cdot h_z}{\sum_{i=1}^{n} W_i \cdot h_i} \qquad \textbf{Equation 3-3}$$

in which subscript z denotes the storey level; F_z is the storey shear force; and h_z is the storey height. The equivalent static procedure of 1995 NBCC further states that the horizontal torsional moment, T_n , at each storey associated with the following cases should also be included in a seismic analysis:

$$T_{nz} = F_z \cdot (1.5 \cdot e_z \pm 0.1 \cdot D_{nz}) \qquad \textbf{Equation 3-4 (a)}$$

$$T_{nz} = F_z \cdot (0.5 \cdot e_z \pm 0.1 \cdot D_{nz}) \qquad \textbf{Equation 3-4 (b)}$$

where the term e_z is the offset distance between the centre of mass and centre of rigidity in the direction perpendicular to the design earthquake force at level z; and the term $0.1 \cdot D_{nz}$ represents an accidental eccentricity that is introduced by the code to account for the differences in structural properties between the actual structure and its analytical model.

According to the current limit state design standard, for all occupancies other than storage or assembly, the effects of specified dead loads, D, and specified live loads, L, and earthquake load, E, are combined according to the following equations:

$$1.0 \cdot D + \gamma \cdot (1.0 \cdot E) \qquad \textbf{Equation 3-5 (a)}$$

$$1.0 \cdot D + \gamma \cdot (0.5L + 1.0 \cdot E) \qquad \textbf{Equation 3-5 (b)}$$

where the term γ stands for an importance factor (taken as 1.0 for most structures); and the live loads in Equation 3-5 (b) include the effects of specified gravity live loads due to use and occupancy as well as the snow load on the structure. It should be noted that all loads in the above combinations are taken as their specified values to reflect earthquake as an accidental event. The seismic resistance design should be clearly distinguished from the design for other factored service loads. Furthermore, the inter-storey drift due to the seismic design forces should be limited to 2% and 1% of the storey height for a normal and post-disaster building respectively. P-Δ effects produced by the combined gravity loads acting on the deformed the building should also be included in the analysis.

3.2.2 1965 NBCC

For seismic design based on a static analysis, the minimum static base shear, V_D, specified by 1965 NBCC is given by:

$$V_D = R_{65} \cdot C \cdot I \cdot F \cdot S \cdot W \qquad \textbf{Equation 3-6}$$

where R_{65} is the earthquake zoning factor (taken as 4.0 for Montréal area); C is the construction type factor (taken as 0.75 for reinforced concrete framing); I is the importance factor (taken as 1.0 for normal structure); F is the foundation factor (taken as 1.0 for stiff soil); and S is the seismic response factor which is computed by the following equation:

$$S = \frac{0.25}{9+n} \qquad \textbf{Equation 3-7}$$

where the variable n stands for the number of storeys in the building. Since the penthouse roof is included in the seismic analysis and is considered as a second storey, the seismic response factor, S, is calculated to be 0.023. Thus, based on Equation 3-6, the 1965 NBCC seismic design base shear forces acting on the structure in both E-W and N-S directions are calculated to be 0.068·W. The seismic weight, W, is taken as the sum of total dead load plus 25% of the snow load specified by 1965 NBCC. The distribution of the lateral base shear over the height of the structure follows Equation 3-3. The horizontal torsional response at each storey z is also investigated according to a specified torsional moment given by:

$$T_{nz} = F_z \cdot (1.5 \cdot e_z \pm 0.05 \cdot D_{nz}) \qquad \textbf{Equation 3-8}$$

Furthermore, 1965 NBCC specifies that gravity and lateral loadings in a seismic analysis should be combined in the following manner:

$$1.35 \cdot D + 1.35 \cdot (L + E) \qquad \textbf{Equation 3-9 (a)}$$

$$0.9 \cdot D + 1.35 \cdot E \qquad \textbf{Equation 3-9 (b)}$$

It should be noted that Equation 3-9 (b) is for wind load analysis. However, considering that the load combination given by Equation 3-9 (b) yields a more conservative overturning effect on a laterally loaded structure, it is appropriate to use the load combination in a seismic analysis.

3.3 Structural Modelling

3.3.1 Modeling of Precast Concrete Framing

A three-dimensional computer model of the single-storey structure was developed using ETABS version 8 *(CSI, 2003)*. Each structural column is modeled as a frame element with a "pinned" connection at the top and rigid connection at the base. To account for cracking in the concrete member, the gross elastic sectional stiffness of each column is multiplied by a factor of 0.7 as suggested by the CSA A23.3-94 Standard *(CSA, 1994)*. In addition, the "cracked" elastic column stiffness is further divided by the force modification factor R (i.e., 1.5) so that the elastic lateral displacement output is amplified to an "inelastic" displacement value as required by 1995 NBCC. To further account for the sway effects associated with the inelastic displacement and the combined gravity loads (i.e., P-Δ effects), the built-in iterative P-Δ analysis option in ETABS was activated. The precast concrete roof members of the warehouse are modeled as an equivalent concrete slab with its thickness and weight obtained by averaging the weight of precast members and masonry walls evenly over the roof area (Figure 3-1). To ensure that overturning effects due to seismic forces are correctly modeled, the original column height is extended from 10 ft (3 m) to 12.4 ft (3.8 m) so that the centroid of the equivalent roof slab coincides with the centre of mass of the precast roof structure.

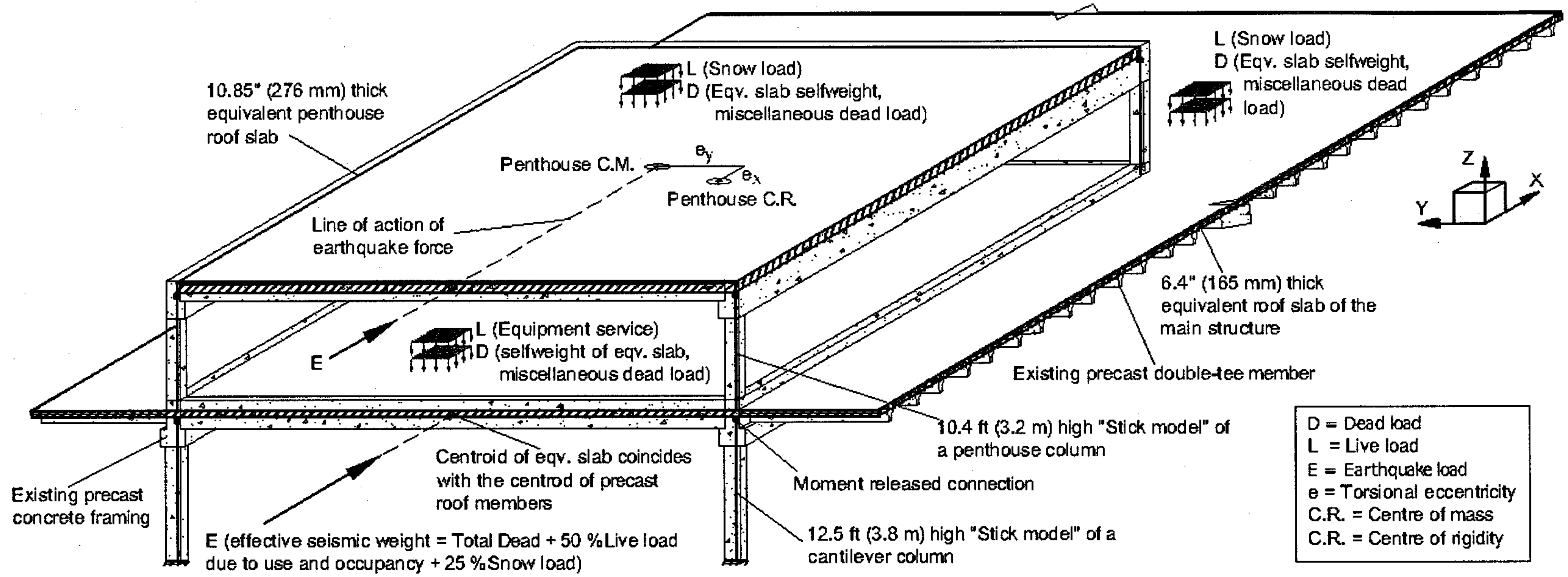


Figure 3-1: Applications of design loads and modeling of the precast framing (masonry-infilled walls not shown).

Uniform distributed specified load on the main roof						
Eqv. slab selfweight	80 psf	(3.84 kPa)	Roof members	7217 kips	(32104 kN)	From both penthouse and the main storey
Miscellaneous dead load	21 psf	(1.00 kPa)	50% Masonry walls	459 kips	(2041 kN)	
Equipment dead load	25 psf	(1.20 kPa)	50% Columns	+ 82 kips	(364 kN)	
Equipment live load	75 psf	(3.60 kPa)	Eqv. slab selfweight	7758 kips	34509 kN	
NBCC 1995 snow load	48 psf	(2.32 kPa)				
NBCC 1965 snow load	43 psf	(2.07 kPa)	Divide by main roof area = 96748 ft^2 (8988 m^2)			
Uniform distributed specified load on the penthouse roof			Roof Members	205 kips	(913 kN)	
Eqv. slab selfweight	136 psf	(6.49 kPa)	50% Masonry walls	52 kips	(230 kN)	
Other dead load	21 psf	(1.00 kPa)	50% Columns	+ 3 kips	(15 kN)	
NBCC 1995 snow load	48 psf	(2.32 kPa)	Eqv. slab selfweight	260 kips	1158 kN	
NBCC 1965 snow load	43 psf	(2.07 kPa)	Divide by penthouse roof area = 1920 ft^2 (178)			

Table 3-2: Specified loads in the analysis.

The penthouse, which has a similar precast framing system to the main structure, is included in the analysis model. The penthouse structure is modeled in a similar way as the main structure. However, since the penthouse columns are discontinuous at the main roof level, the base of each penthouse column is modeled as a pinned-connection. In order to maintain the penthouse stability during the lateral load analysis, a set of equivalent masonry struts are added to the penthouse bays as compression-only diagonal braces. All of the specified loads used in the structural analysis are summarized in Table 3-2.

3.3.2 Modeling of Masonry-Infilled Walls

Although the masonry-infilled walls of the warehouse are assumed to have little lateral load resisting contribution due to out-of-plane displacements and irregular infilling pattern, they are included in the analysis to investigate their in-plane stiffening effects on the structure. The infilled walls are modeled as a set of diagonal compressive struts (Figure 3-2) using the formulation of Al-Chaar *(2002).* The thickness of the equivalent strut is taken as the width of the concrete masonry-infilled wall, and the strut width is estimated by first calculating a dimensionless parameter $H_f \cdot \lambda_I$ that expresses the relative stiffness of the infilled wall to the surrounding frame:

$$H_f \cdot \lambda_I = H_f \cdot \sqrt[4]{\frac{E_m \cdot t \cdot \sin(2 \cdot \theta_{strut})}{4 \cdot E_c \cdot I_{col} \cdot h_m}} \qquad \textbf{Equation 3-10}$$

where λ_I is a characteristic value of an infilled frame; H_f is height of the confining frame, E_m, t and h_m are the elastic modulus, thickness and height of the concrete masonry infill respectively; E_c and I_{col} are the elastic modulus and moment of inertial of the concrete columns; and θ_{strut} is the slope of the infill diagonal to the horizontal in radians. Based on the calculated parameter $H_f \cdot \lambda_I$, the equivalent strut width of the masonry-infilled wall in the elastic range, a, is computed as:

$$a = 0.175 \cdot D \cdot (\lambda_I \cdot H_f)^{0.4} \qquad \textbf{Equation 3-11}$$

where D is the infill diagonal length (Figure 3-2 (a)). To account for openings, the strut width given by Equation 3-11 may be reduced by an empirical reduction factor R_1

computed as:

$$a_{red} = a \cdot (R_1) = a \cdot \left(0.6 \left(\frac{A_{open}}{A_{panel}} \right)^2 - 1.6 \left(\frac{A_{open}}{A_{panel}} \right) + 1 \right) \qquad \textbf{Equation 3-12}$$

where a_{red} is the reduced strut width of a perforated infill panel; A_{open} and A_{panel} are the areas of the openings and the infill panel respectively. If the total area of openings is greater than 60% of the infill panel area, the effect of the compressive strut may be neglected. In the case of a partially infilled frame, the height of the infill, h_m, in Equation 3-10 may be taken as the height of the partial infill (Figure 3-2 (b)). Except for the truck unloading bays and the penthouse bay located on column line 5 which are significantly perforated, all infilled walls in the buildings are modeled in the analysis and their effective strut width are computed as shown in Table 3-3.

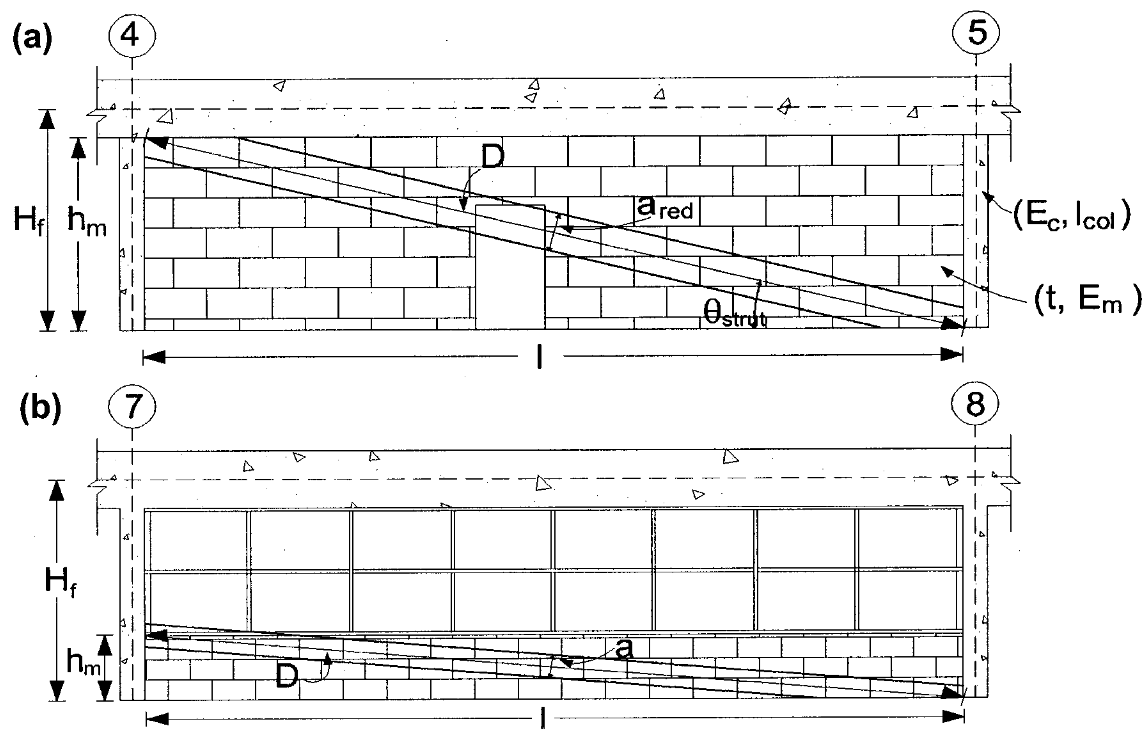


Figure 3-2: Modeling of the masonry-infilled walls: (a) an infilled frame with an opening; (b) a partially infilled frame.

Column Line	Strut Location	Equivalent Strut Width (a)		Perforation (%)	Opening Reduction Factor (R_1)	Reduced Strut Width (a_{red})	
		(in.)	(mm)			(in.)	(mm)
A	4-5, 8-9	47.4	1203	5	0.91	43.3	1100
	5-6, 6-7	47.4	1203	17	0.74	35.1	892
	7-8	45.8	1163	-	1.00	45.8	1163
F	All bays	46.7	1187	-	1.00	46.7	1187
C & D	Penthouse 5-6	47.6	1208	12	0.81	38.7	983
1	All bays	57.0	1447	6	0.91	51.9	1319
11	All bays	57.0	1447	-	1.00	57.0	1447
6	Penthouse C-D	58.0	1473	16	0.76	43.8	1112

Table 3-3: Equivalent strut widths used in analysis.

In the structural model of the building, all equivalent struts are modeled as axial force-only elements with moment releases at both ends. Since the diagonal struts resist only axial compressive forces, their orientation should be treated as a function of the direction of applied earthquake forces and torsional moments. The elastic lateral stiffness of each compressive strut is divided by R=1.5 to be consistent with the reduced column stiffness and to account for the inelastic displacements of the structure. For the position of the equivalent struts, Al-Chaar *(2002)* proposed that both ends of the struts are to be eccentrically placed with rigid links at a distance away from the beam surface or the column base, so that the infill forces are transferred directly to the columns for maximum shear effects. However, since all columns of the building are weak and "short column" shear failure of the perimeter columns is most likely to take place due to the presence of the partially infilled walls, the equivalent struts of partially infilled walls are connected directly to the columns at the levels of the partially infilled walls and with the other end connected directly to the opposite columns bases. For the few perforated fully-infilled frames located on column line A, the infilled walls are modeled by equivalent masonry diagonal struts that are concentrically connected within the bays.

3.3.3 Structural Analysis

The seismic performance of the structure is assessed according to the methodology described in Figure 3-3 which involves: a rigid diaphragm analysis (RD); a series of sub-structure analyses (SUB) and a simplified tributary-area based analysis (SA). The rigid diaphragm analysis investigates the lateral load resistances of the existing columns by idealizing the entire roof structure as one rigid diaphragm that distributes the earthquake forces to the columns in proportion to their lateral stiffness and their distances from the centre of rigidity of the structure. To further account for roof separation along the two roof dilation joints on column lines 4 and 8, a series of individual "sub-structures" with rigid roof diaphragms are also analyzed. The rigid diaphragm model and sub-structures models are studied by means of a set of three-dimensional analysis models developed with ETABS *(CSI, 2003)*. Furthermore, all ETABS models are divided into two groups: a set of column-only models which assumed that the masonry infilled walls will fall out-of-plane at the beginning of an earthquake; and a set of models with equivalent masonry struts to account for the effects of the masonry-infilled walls if they remain in place during an earthquake.

In all ETABS models, the effects of earthquake forces in the two principal directions of the building and their associated maximum horizontal torsions are investigated. The earthquake forces and horizontal torsions on the ETABS models are applied in a way similar to Figure 3-4, in which the earthquake force acting through the centre of mass of the roof diaphragm is replaced by a statically equivalent combination of earthquake force and a torsional moment at the centre of rigidity of the structure. Eccentricities due to variation of lateral stiffness across the building are automatically calculated by ETABS for all three-dimensional models. These actual eccentricities are amplified and combined with the accidental eccentricities in accordance with the torsional provisions given in Equations 3-4 and Equation 3-8 to yield the most unfavorable torsional effects on each structural model.

Aside from the three-dimensional ETABS analyses, a conservative simplified analysis is also conducted to investigate if the building is capable of resisting the applied earthquake forces if the roof structure fails to function as an integral unit. In this simplified analysis, the lateral force acting on each column is assumed to be 1.15 times of the seismic force associated with its tributary area *(Redwood, 1998)* (Figure 3-5). In order to conservatively account for the possible torsional effects on the structure, an additional torsion-induced shear force, V_T, acting on each column parallel to the earthquake force is approximated by Equation 3-13 (a). In a y-direction earthquake, the additional torsion-induced shear force acting on a column located on coordinate (i,j) is computed as:

$$V_{T\,yi} = \frac{T_n \cdot K_{yj} \cdot d_{xj}}{\sum_i K_{yi} \cdot d_{xi}^{\;2} + \sum_i K_{xi} \cdot d_{yi}^{\;2}}$$ **Equation 3-13 (a)**

in which T_n is the applied torsional moment; d_x and d_y are the distances from the centre of rigidity to the column at coordinate (i,j) in the x and y directions; K_x and K_y are the column stiffnesses in the x and y directions respectively; coordinate (i,j) is the building grid which ranges from 1 to 11 for i; and A to F for j. Similarly, torsion-induced shear acting on a column located at coordinate (i,j) in a direction parallel to an x-direction earthquake is calculated by:

$$V_{T\,xi} = \frac{T_n \cdot K_{xi} \cdot d_{yi}}{\sum_j K_{yj} \cdot d_{xj}^{\;2} + \sum_j K_{xj} \cdot d_{yj}^{\;2}}$$ **Equation 3-13(b)**

It should be noted that although the above equations are developed on the basis of the rigid diaphragm analogy, they provide a rational means to account for the torsional effects on the structure in the simplified analysis. The torsion-induced shear forces acting on each column in a direction perpendicular to an earthquake can be calculated with a slight modification to the above equations. However, for simplicity, only additional shear forces parallel to the applied earthquake loadings are considered in the analysis (i.e., a two-dimensional analysis). In addition, P-Δ effects associated with the combined gravity load and inelastic lateral displacements of the columns are also accounted for in the simplified analysis. The effects of masonry-infilled walls, however, are neglected in the analysis.

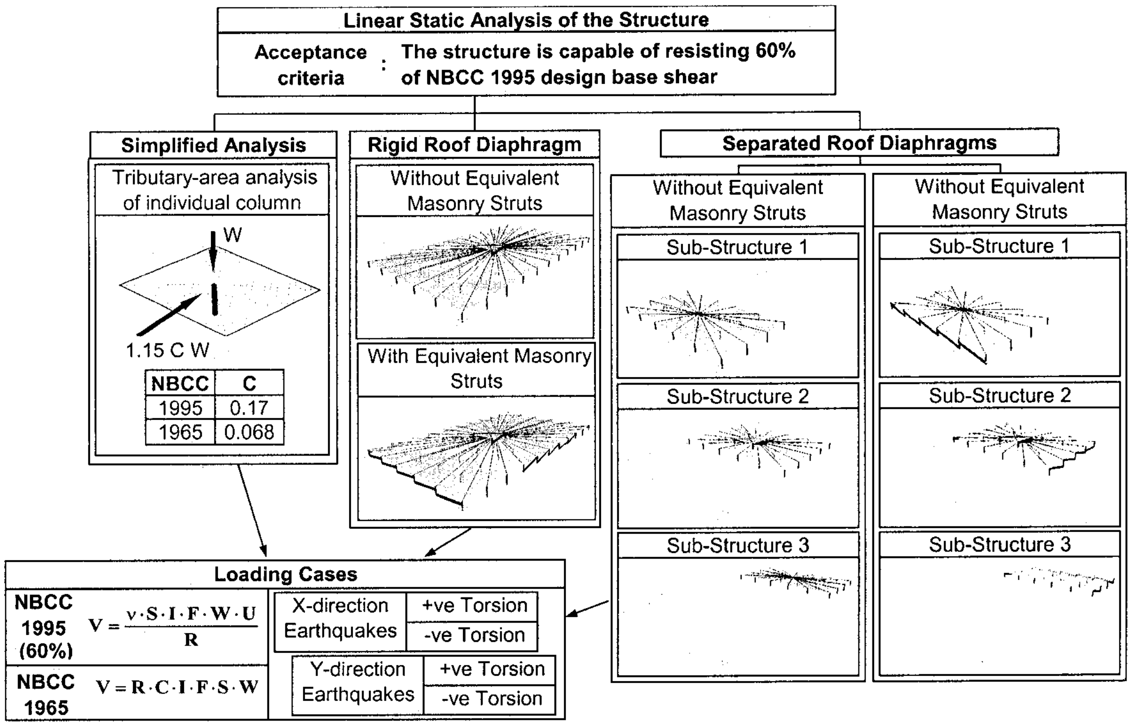


Figure 3-3: Analysis methodology used in seismic evaluation of the warehouse.

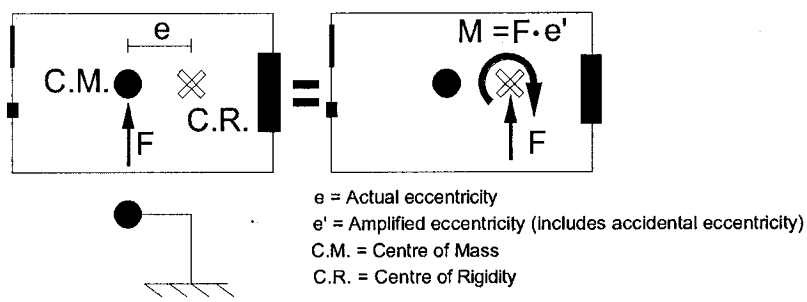


Figure 3-4: Schematic diagram showing the application of earthquake forces on a single-storey structure with a rigid diaphragm.

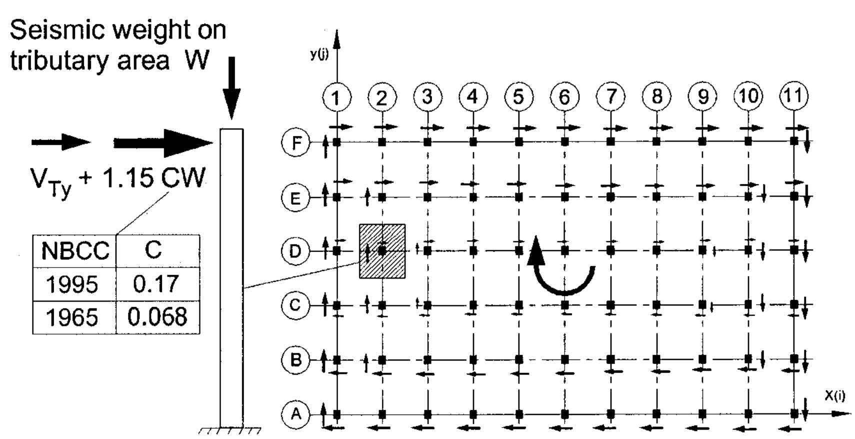


Figure 3-5: The simplified analysis and the additional torsion-induced shear accounted for in the analysis.

3.4 Discussion of Results

Upon reviewing the results from various analyses, critical cases that yield the most unfavourable effects on the structure were selected in Table 3-4. Column forces in the column-only models (i.e., cases without equivalent masonry struts) and nominal member resistances (i.e., material resistance factors ϕ_c and $\phi_s = 1.0$) for various column sections (see Section 2.2.3) are compared. Next, the analyses with equivalent masonry struts are discussed, followed by a comparison of storey drifts with the limit defined in the current building code *(NBCC, 1995)*.

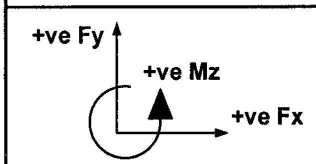
3.4.1 Column-only Models

3.4.1.1 Moment Resistance of Columns

In column-only models, each column is laterally loaded as a cantilever member in which the maximum bending moment occurs at the column base and the shear force is uniform along the column height (Figure 3-6 and Figure 3-7). Therefore, it is assumed that the grout section and the bolt-pocket regions of the precast columns are the critical zones where flexural hinging may take place, provided premature shear failure does not occur in the columns. In order to assess the adequacy of the precast concrete columns, the overturning moments in the direction of the applied earthquake force and the axial compressive force of selected columns were plotted with the nominal axial force-moment interaction diagrams for various column sections (Figure 3-8 through Figure 3-13). The nominal axial force-moment interaction diagrams were generated using RESPONSE-2000 *(Bentz & Collins, 2000)* and the sectional responses of the grout layers were computed by considering the anchor bolts as steel reinforcement of the sections.

	Analysis Scheme	Applied Storey shear			Eccentrictiy		* Torsional Moment		
Without Equivalent Masonry Struts	**Rigid Roof Diaphragm (RD)**	**Storey**	**Y-direction (kips)**	**Y-direction (kN)**	**X-direction (ft)**	**X-direction (m)**	**Z-direction (kips-ft)**	**Z-direction (kNm)**	** **Case**
	1995 NBCC	Penthouse	98	438	-34	-10	-3385	-4589	RD-YN95
		Main roof	1834	8157	-42	-13	-76273	-103412	
	1965 NBCC	Penthouse	40	176	-32	-10	-1281	-1737	RD-YN65
		Main roof	731	3251	-22	-7	-15751	-21356	
	Sub-Structure 1 (SUB)	**Storey**	**X-direction (kips)**	**X-direction (kN)**	**Y-direction (ft)**	**Y-direction (m)**	**Z-direction (kips-ft)**	**Z-direction (kNm)**	**Case**
	1995 NBCC	Main roof	551	2450	31	9	16873	22877	SUB-XP95
	1965 NBCC	Main roof	243	1081	19	6	4516	6123	SUB-XP65
	Simplified Analysis (SA)	**Storey**	**Y-direction (kips)**	**Y-direction (kN)**	**X-direction (ft)**	**X-direction (m)**	**Z-direction (kips-ft)**	**Z-direction (kNm)**	**Case**
	1995 NBCC	Penthouse	98	438	-4	-1	-394	-534	SA-YN95
		Main roof	1834	8157	-40	-12	-74212	-100617	
	1965 NBCC	Penthouse	40	176	-4	-1	-158	-215	SA-YN65
		Main roof	731	3251	-40	-12	-29575	-40098	
With Equivalent Masonry	**Rigid Roof Diaphragm (RD)**	**Storey**	**Y-direction (kips)**	**Y-direction (kN)**	**X-direction (ft)**	**X-direction (m)**	**Z-direction (kips-ft)**	**Z-direction (kNm)**	**Case**
	1995 NBCC	Penthouse	98	438	-34	-10	-3384	-4589	RD-YN95S
		Main roof	1834	8157	-42	-13	-77489	-105061	
		Penthouse	98	438	-6	-2	-576	-781	RD-XN95S
		Main roof	1834	8157	-130	-40	-238733	-323679	
	Sub-Structure 1 (SUB)	**Storey**	**X-direction (kips)**	**X-direction (kN)**	**Y-direction (ft)**	**Y-direction (m)**	**Z-direction (kips-ft)**	**Z-direction (kNm)**	**Case**
	1995 NBCC	Main roof	551	2450	96	29	53091	71982	SUB-XP95S

** Positive Sign Convention for Torsional Moment*



*** Format of Case Label*

RD - Y N 95 S
1 2 3 4 5

1	*Type of analysis*
2	*Direction of earthquake force: X or Y*
3	*Direction of torsion: P = positive, N = negative*
4	*Version of NBCC considered*
5	*With or without masonry struts*

Table 3-4: Most critical loading cases for various analysis models.

Cases RD-YN95 and RD-YN65

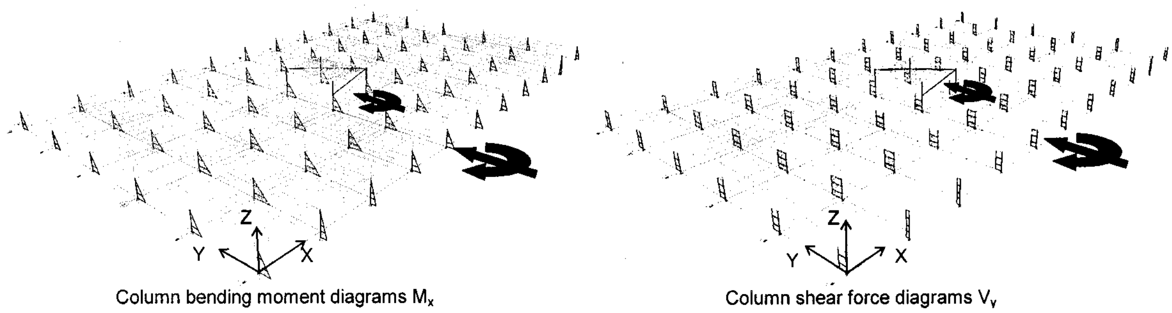


Figure 3-6: Bending moment and shear force distribution in a rigid diaphragm model.

Cases SUB-YN95 and SUB-YN65

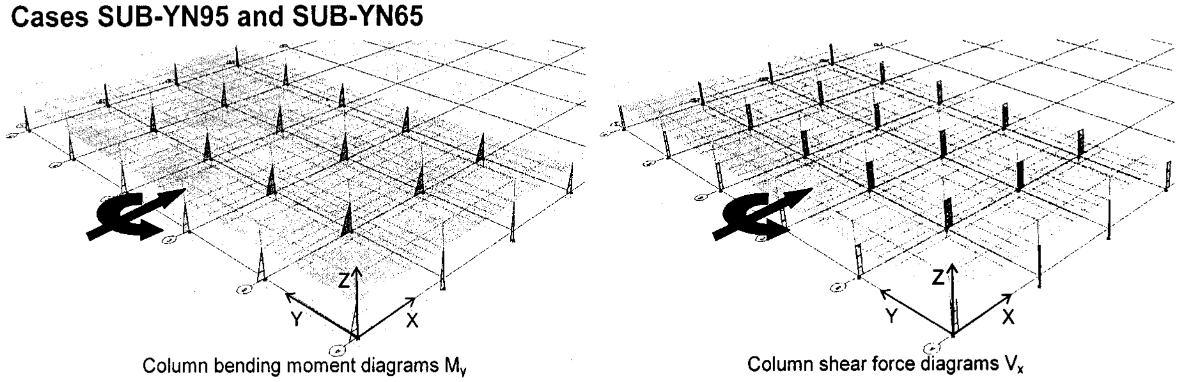


Figure 3-7: Bending moment and shear force distribution in sub-structure 1 model.

Generally, all axial force-moment interaction diagrams showed that the flexural resistance provided by the columns are grossly insufficient to satisfy the 60% evaluation criteria and this deficiency can be much more severe if the structure was evaluated for the full 1995 NBCC seismic design force. These plots have indicated that column failure due to insufficient flexural resistances at a low level of axial compressive load is likely to occur in the structure. The overall deficiency in column moment resistance is insensitive to the type of roof configuration adopted in the analysis. However, the strength deficiency is shown to be the most critical for the interior columns for the simplified analysis models and for exterior columns for the rigid diaphragm models.

Considering the results relative to the seismic design provisions of 1965 NBCC on all plots, it is clear that most of the columns in the analyses do not meet the 1965 NBCC code requirements. In order to verify the lateral load design of the columns at the time of

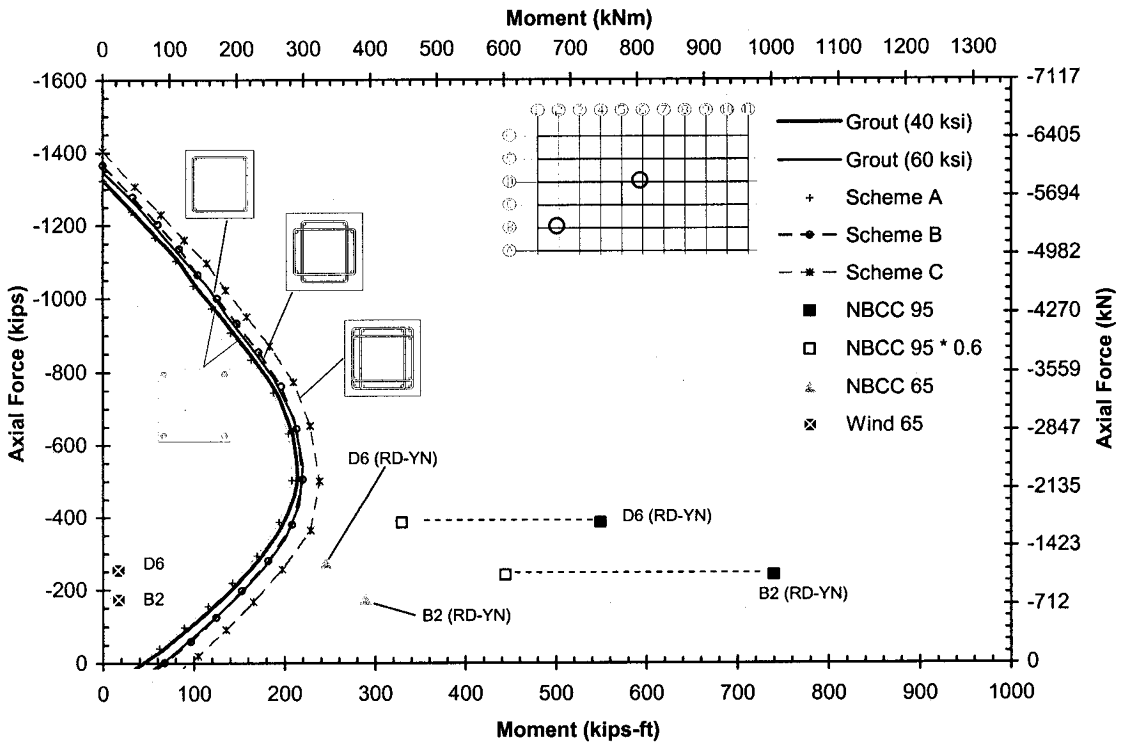


Figure 3-8: Selected rigid diaphragm analysis results plotted with the nominal sectional resistances of various assumed interior column sections.

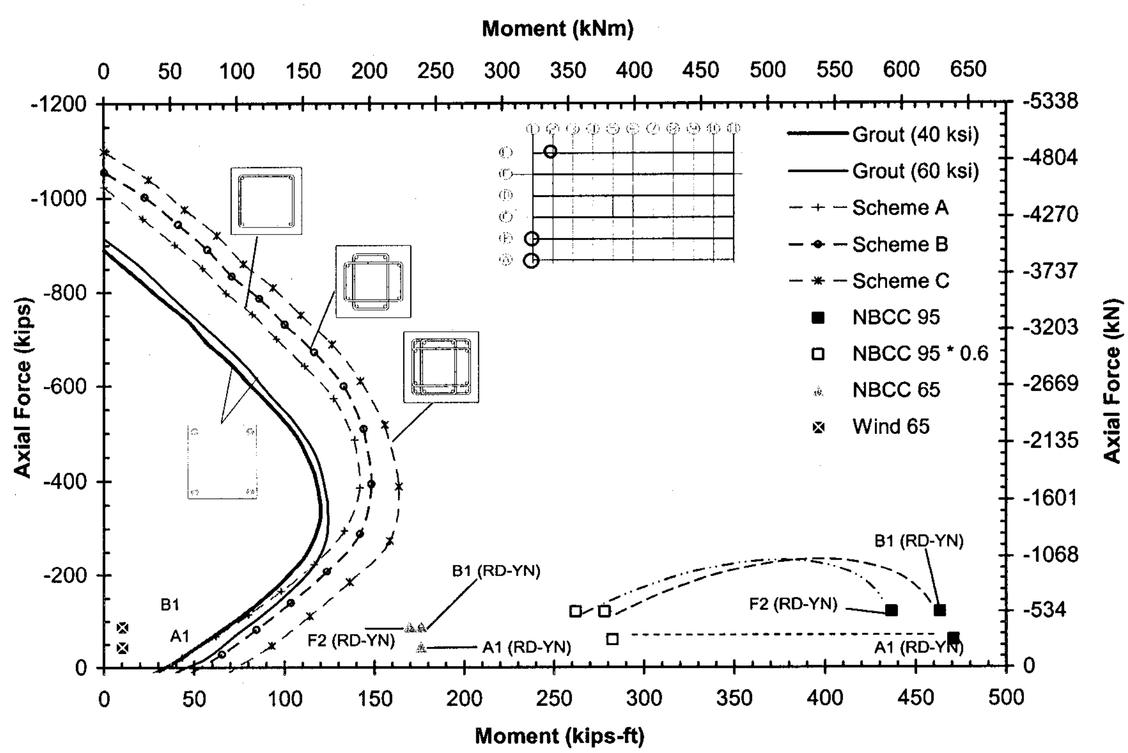


Figure 3-9: Selected rigid diaphragm analysis results plotted with the nominal sectional resistances of various assumed exterior column sections.

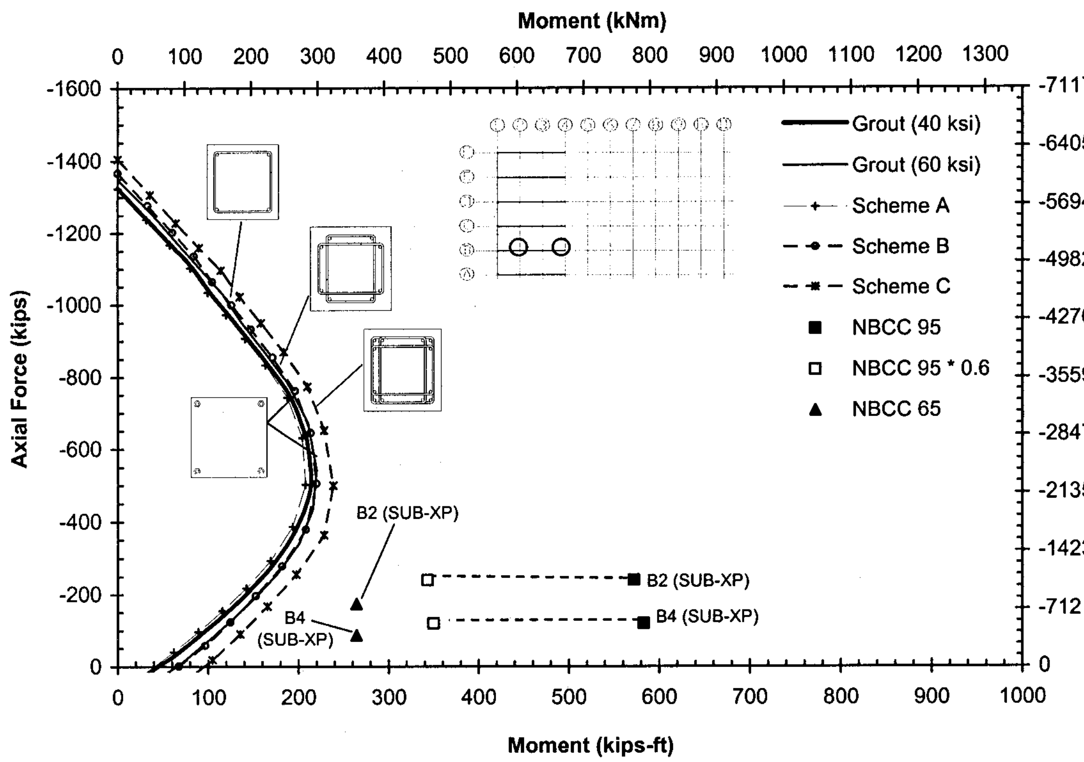


Figure 3-10: Selected analysis results of sub-structure 1 plotted with the nominal sectional resistances of various assumed interior columns sections.

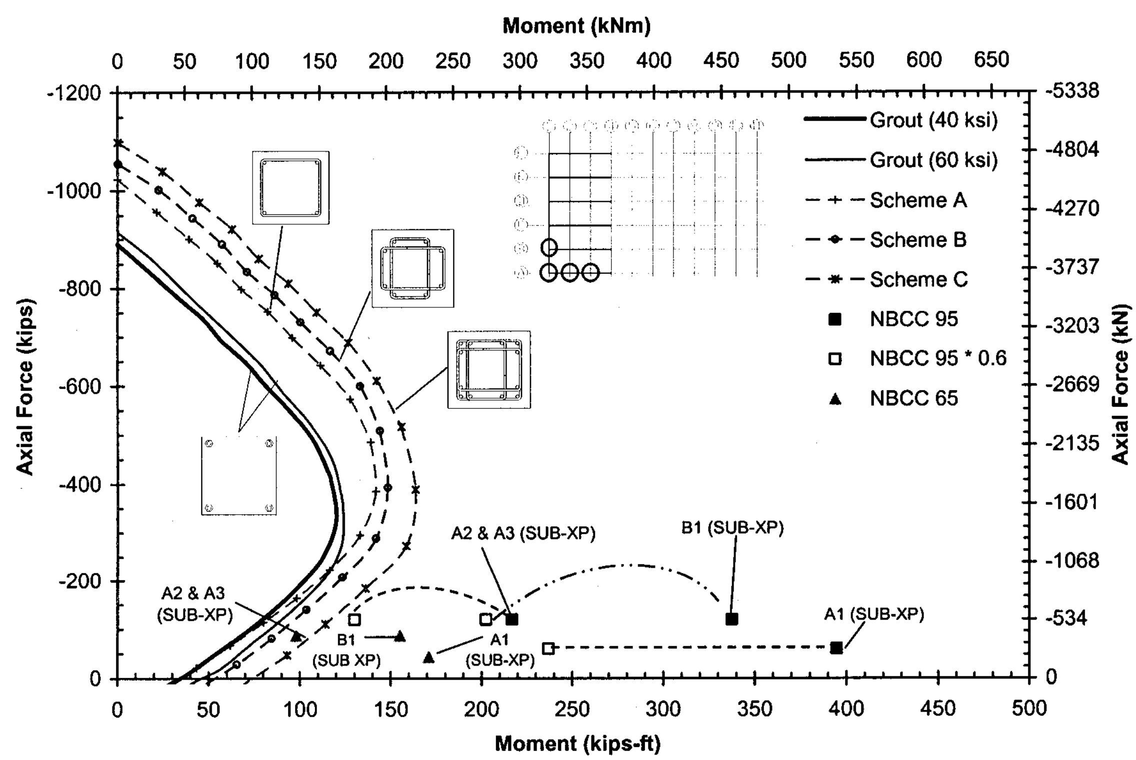


Figure 3-11: Selected analysis results of sub-structure 1 plotted with the nominal sectional resistances of various assumed exterior column sections.

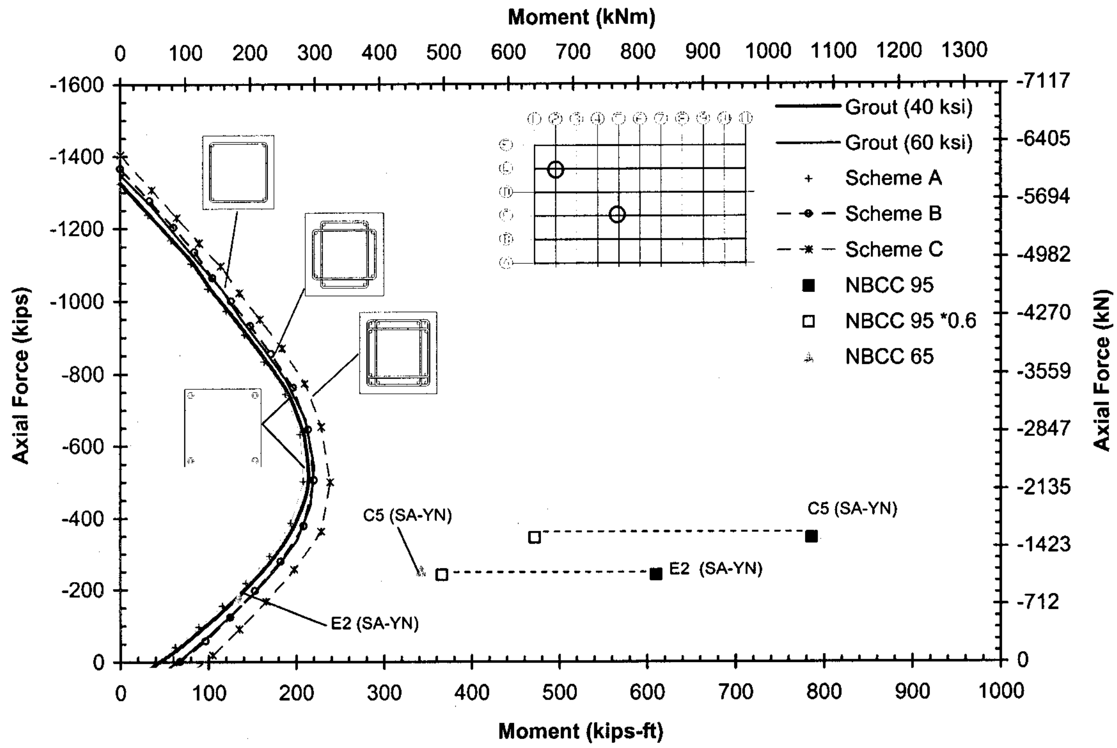


Figure 3-12: Selected results of simplified analysis plotted with the nominal sectional resistances of various assumed interior column sections.

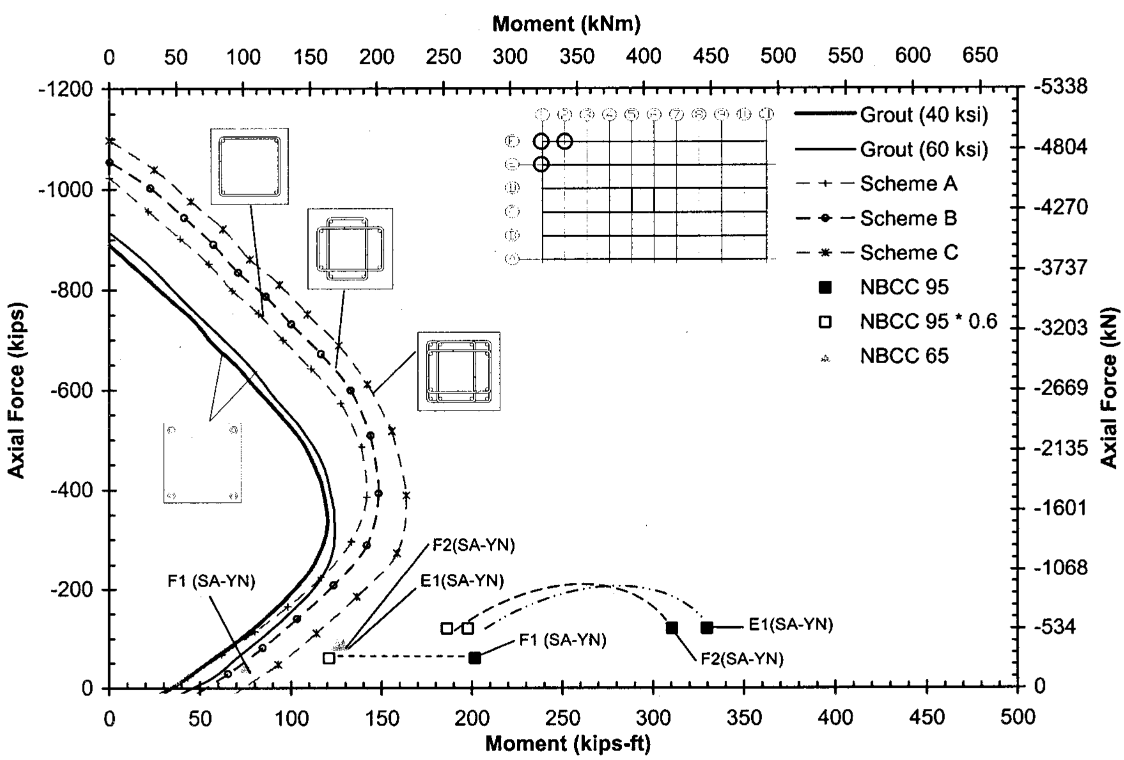


Figure 3-13: Selected results of simplified analysis plotted with the nominal sectional resistances of various assumed exterior column sections.

construction, a lateral wind load analysis based on 1965 NBCC (Figure 3-8 and Figure 3-9) was also conducted. The 1965 NBCC wind loads are significantly lower than the seismic design forces, and the columns satisfy the 1965 NBCC wind load requirements. Therefore, the analysis indicates that seismic resistance design was not considered at the time of design of the structure.

3.4.1.2 Shear Resistances of Columns

Column shear forces are compared to the nominal shear resistances, V_n, calculated according to the shear design guidelines of the current CSA A23.3-94 Standard *(CSA, 1994)*:

$$V_n = V_s + V_c \qquad \textbf{Equation 3-14}$$

in which V_s and V_c are the shear resistances of the ties and the concrete respectively, and V_s is computed according to the following equation:

$$V_s = \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v}{s} \qquad \textbf{Equation 3-17}$$

in which ϕ_s is the steel reinforcement resistance factor (taken as 1.0); A_v is the area of the transverse reinforcement; f_y is the yield strength of the transverse reinforcement in MPa; d_v is effective shear depth of the member in millimetres; and s is the spacing of the ties in millimetres. The concrete shear strength, V_c , is computed according to the following equation:

$$V_c = 0.2 \cdot \lambda \cdot \phi_c \sqrt{f'_c} \cdot b \cdot d_v \qquad \textbf{Equation 3-17}$$

in which λ is the concrete density factor (taken as 1.0); ϕ_c is the concrete resistance factor (taken as 1.0); f'_c is the concrete compressive strength in MPa; b and d_v are the width and effective shear depth of the member in millimetres. The predicted shear resistances of the reinforced column sections are shown in Table 3-5. The shear strength at the grout interface is taken as the anchor bolt shear strength (7.2 kips/bolt (32 kN/bolt)) *(PCI, 1999)*.

Reinforcing Details	Interior Column Shear Strength				Exterior Column Shear Strength				Shear Strength of Anchor Bolts	
	V_s		$V_n=V_s+V_c$		V_s		$V_n=V_s+V_c$			
	(kips)	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)	(kN)
Scheme A	12	54	50	221	10	46	39	171	29	128
Scheme B	11	51	47	208	9	41	34	152		
Scheme C	23	104	60	265	19	86	45	202		

Table 3-5: Shear resistances of various reinforced concrete column sections.

In evaluating the shear resistance of the columns, the contribution of the concrete may be conservatively neglected since they are deteriorated, and concrete cover is spalled on some members as observed during the site inspection. In order to determine if the shear resistance of columns governs their ultimate failure mode, the shear resistance, V_s , computed in Table 3-5 are compared with the shear force associated with the development of nominal moment resistance, M_n. The nominal moment resistance of each reinforced column section (Table 3-6) was obtained by performing a section analysis using RESPONSE-2000 with an axial compression force equal to the combined dead and live loads acting on the column. Figure 3-14 shows that the nominal shear resistances provided by the various assumed column sections are generally greater than the shear strength needed for the development of nominal moment resistances of the columns. This comparison concludes that nominal flexural hinging will be the primary failure mode of the columns.

Although it was demonstrated in the previous assessment that most of the columns in the structure will fail in flexure at force levels lower than 60% of the design forces given by 1995 NBCC, the interior and exterior columns that were previously evaluated for their moment resistance are further evaluated for their shear resistances. Figure 3-15 shows that the nominal shear resistances V_s of selected interior columns are generally lower than 60% of the 1995 NBCC requirements. Similarly, Figure 3-16 has shown that selected exterior columns reinforced with schemes A and B (column ties with 2 stirrup legs) are inadequate in terms of their shear resistances; and for columns that are reinforced with scheme C (column ties with 4 stirrup legs), their shear resistance are considered to be acceptable. Based on these comparisons, it was concluded that the precast columns are grossly inadequate in terms of their shear resistances.

Columns	Column Axial Compression		Nominal Moment Resistance M_n					
			Scheme A		SchemeB		Scheme C	
	(kips)	(kNm)	(kips-ft)	(kNm)	(kips-ft)	(kNm)	(kips-ft)	(kNm)
Interior	240	1068	152	206	170	230	193	262
Perimeter	120	535	83	112	98	133	118	160
Corner	60	268	60	81	78	105	98	133

Table 3-6: Nominal moment resistances of various reinforced sections.

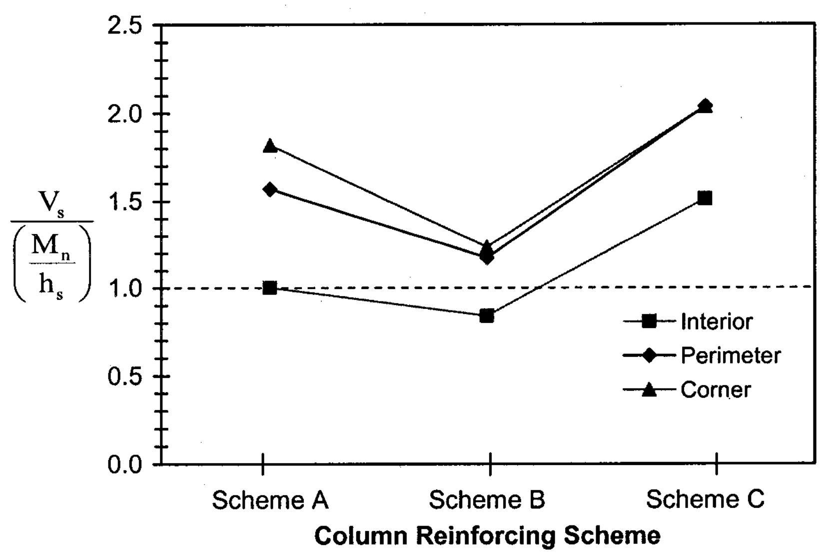


Figure 3-14: Ratio of nominal shear resistances to the shear forces associated with the development of nominal moment resistances for various reinforced column sections.

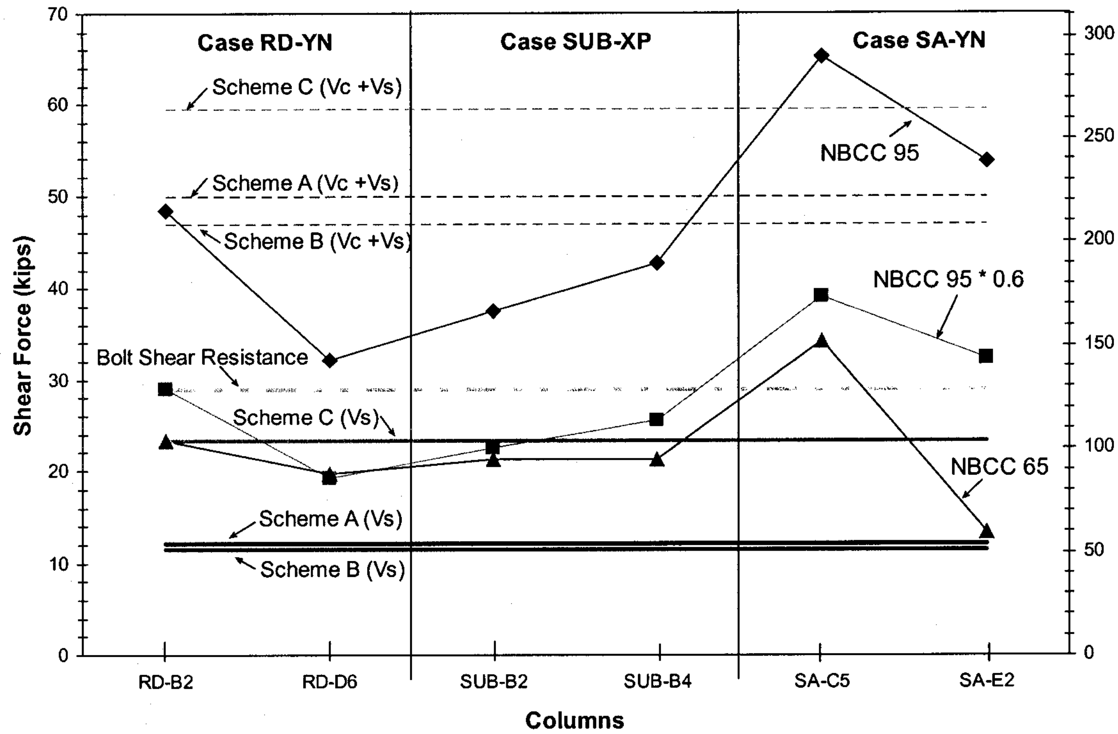


Figure 3-15: Comaprison between shear forces and nominal shear resistances of selected interior columns.

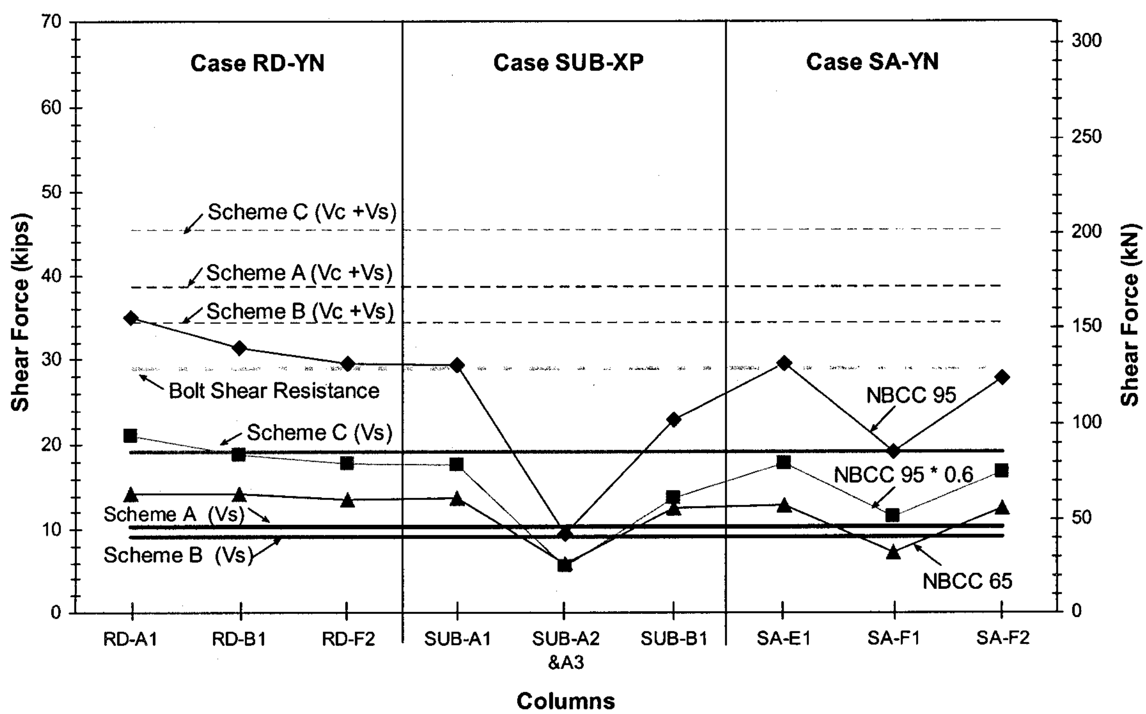


Figure 3-16: Comaprison between shear forces and nominal shear resistances of selected exterior columns.

3.4.2 Significance of Masonry-Infilled Walls

The addition of equivalent diagonal struts into the models revealed the detrimental effects associated with the masonry-infilled walls. Besides increasing the torsional effects by introducing stiffness irregularities into the structure; the equivalent diagonal struts of the partially infilled wall have also promoted the "short column" effects by transferring their axial forces directly to the already strength deficient exterior columns at the levels of the partial infills. As it is shown in Figure 3-17 and Figure 3-18, the equivalent struts of the partially infilled walls have generated a series of column shortening on the exterior columns along line F, and along lines 1 and 11 when the rigid diaphragm model is loaded in the x and y directions. The shear force and the bending moment recorded on the internal force diagrams of the critical exterior columns have exceeded their nominal shear and moment resistances computed previously in Table 3-5 and Table 3-6, respectively. Upon separation of the roof structure along the roof dilation joints, the struts-and-columns model of sub-structure 1 is the most critical case due to the stiffness irregularity associated with the equivalent struts on column lines 1 and F (Figure 3-19). As the structure is subjected to an x-direction earthquake force and its associated torsional moment, the columns located at the relatively flexible side of the structure (column line A) are subjected to significant overturning moment and shear forces.

Table 3-7 compares the strut forces computed in the linear static analysis of cases RD-YN95S and SUB-XP95S with the approximated strut resistances given by the following equations *(Al-Chaar, 2002)*:

$$R_{crush} = a_{red} \cdot t_{eff} \cdot f'_m \qquad \textbf{Equation 3-15 (a)}$$

$$R_{shear} = \frac{A_n \cdot V_m \cdot R_1}{\cos\theta_{strut}} \qquad \textbf{Equation 3-17 (b)}$$

where R_{crush} and R_{shear} are the equivalent strut strengths associated with crushing and shear failure modes of the masonry-infilled wall respectively; t_{eff} is the effective masonry wall thickness excluding the hollow area; and A_n is the net cross-sectional grouted area of the infill panel along the wall length. Table 3-7 shows that the governing "shear strength" of the equivalent struts are significantly lower than the strut forces computed in the analyses for both of the full and reduced 1995 NBCC seismic loading. However, considering the fact that linear static analysis neglects material non-linearity and the redistribution of internal forces, it probably overestimates the actual strength of the masonry-infilled walls by considering the equivalent struts as infinitely elastic members. The analysis has also led to relatively conservative estimates of the strut effects on the columns. In reality, the masonry-infilled walls could have disintegrated or failed before reaching the strut forces computed using a linear elastic analysis. Despite the pitfall in the linear static analysis, the column shear resistances computed in Table 3-5 are compared with the maximum shearing strength available by the masonry wall (i.e., $R_{shear} \cdot \cos\theta_{strut}$), and it is shown that the shear resistances of the columns remained inadequate to resist the forces associated with the ultimate shear capacity of the masonry-infilled walls.

The simulation of the effects of masonry-infilled walls on their bounding frame by means of a set of simplified compressive struts has shown that the partially infilled walls will potentially have a negative impact on the already strength deficient precast columns. It is strongly suggested that the masonry-infilled walls to be separated from the bounding concrete frame to eliminate the adverse effects of the non-structural elements.

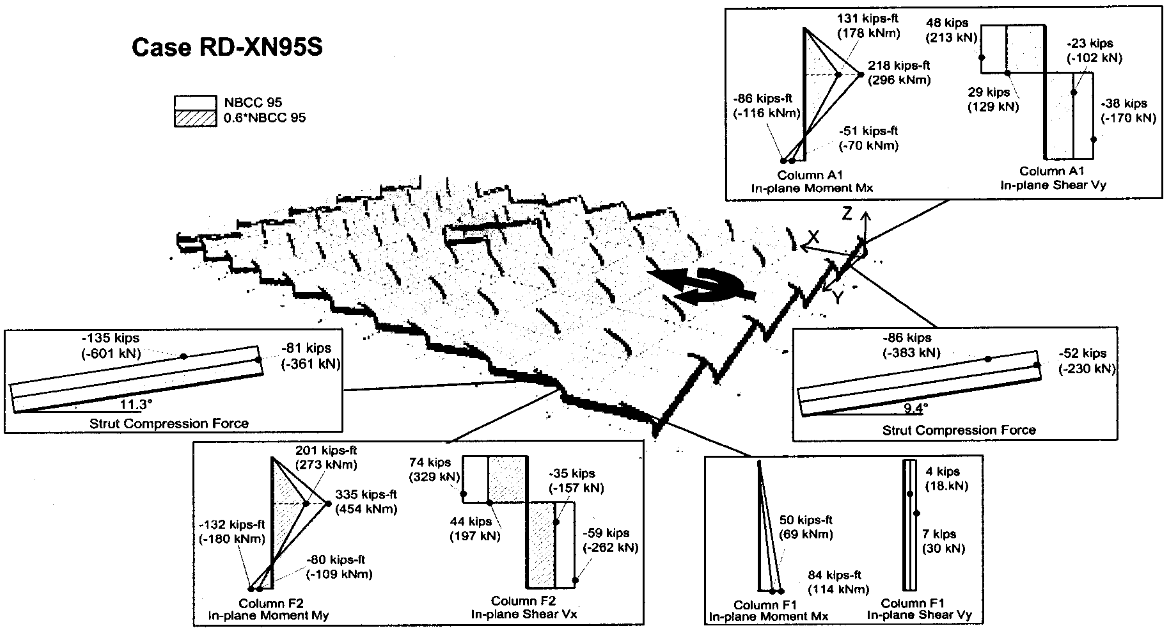


Figure 3-17: Selected internal force diagrams of members affected by the partially infilled walls in a rigid diaphragm model subjected to x-direction earthquake forces.

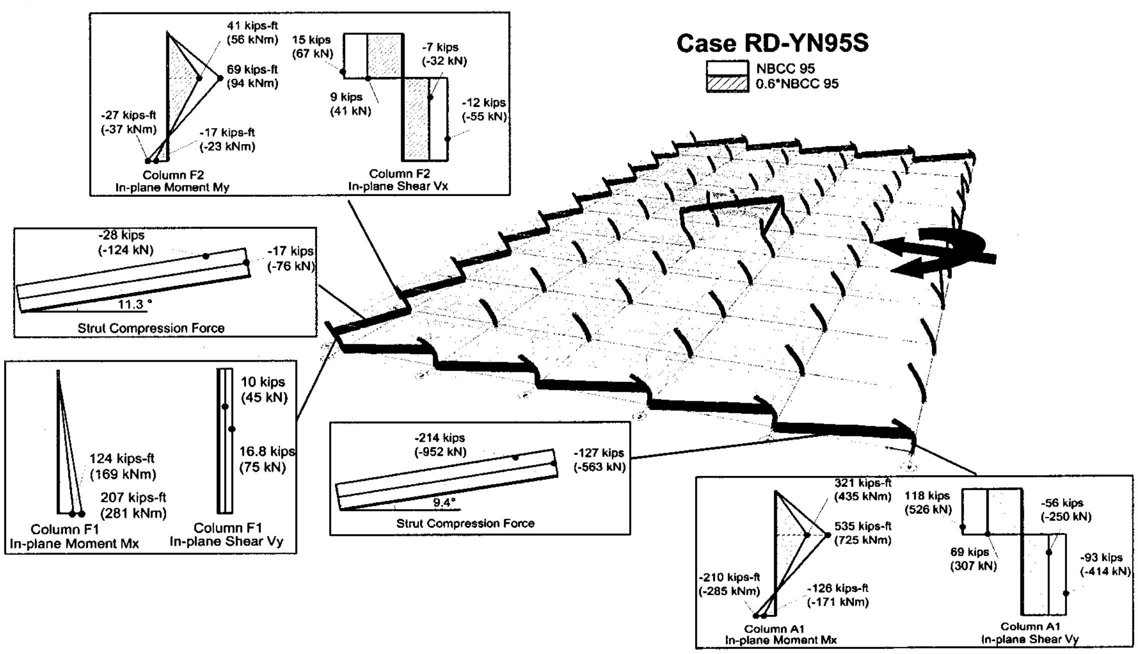


Figure 3-18: Selected internal force diagrams of members affected by the partially infilled walls in a rigid diaphragm model subjected to y-direction earthquake forces.

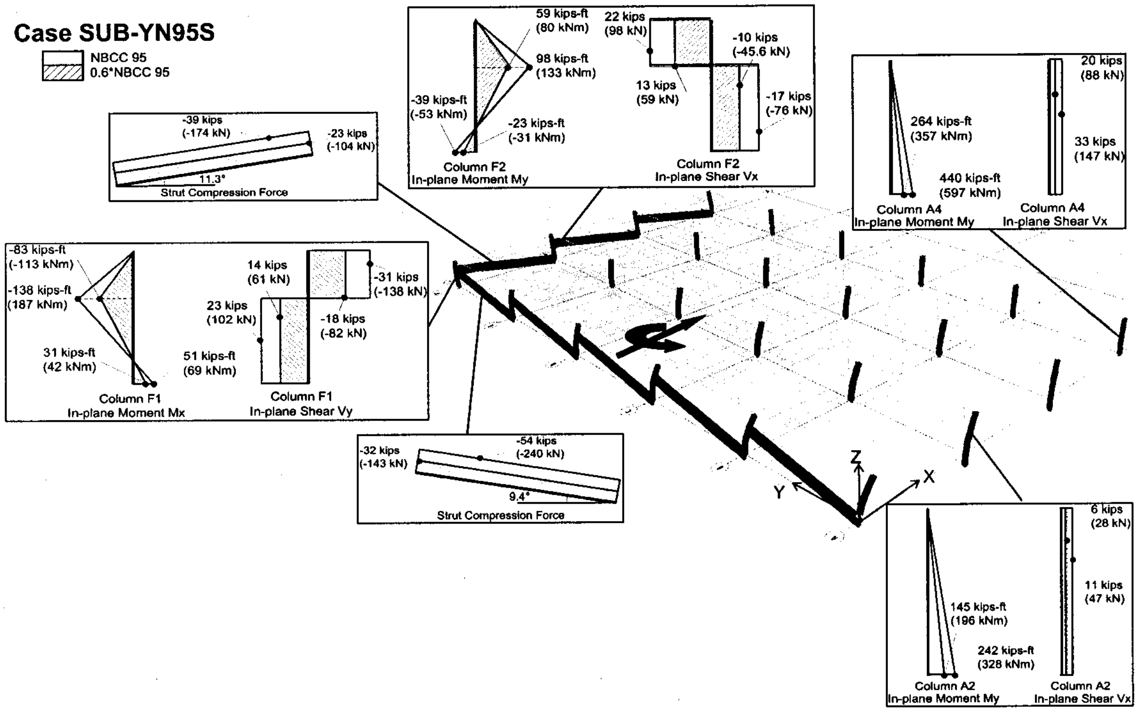


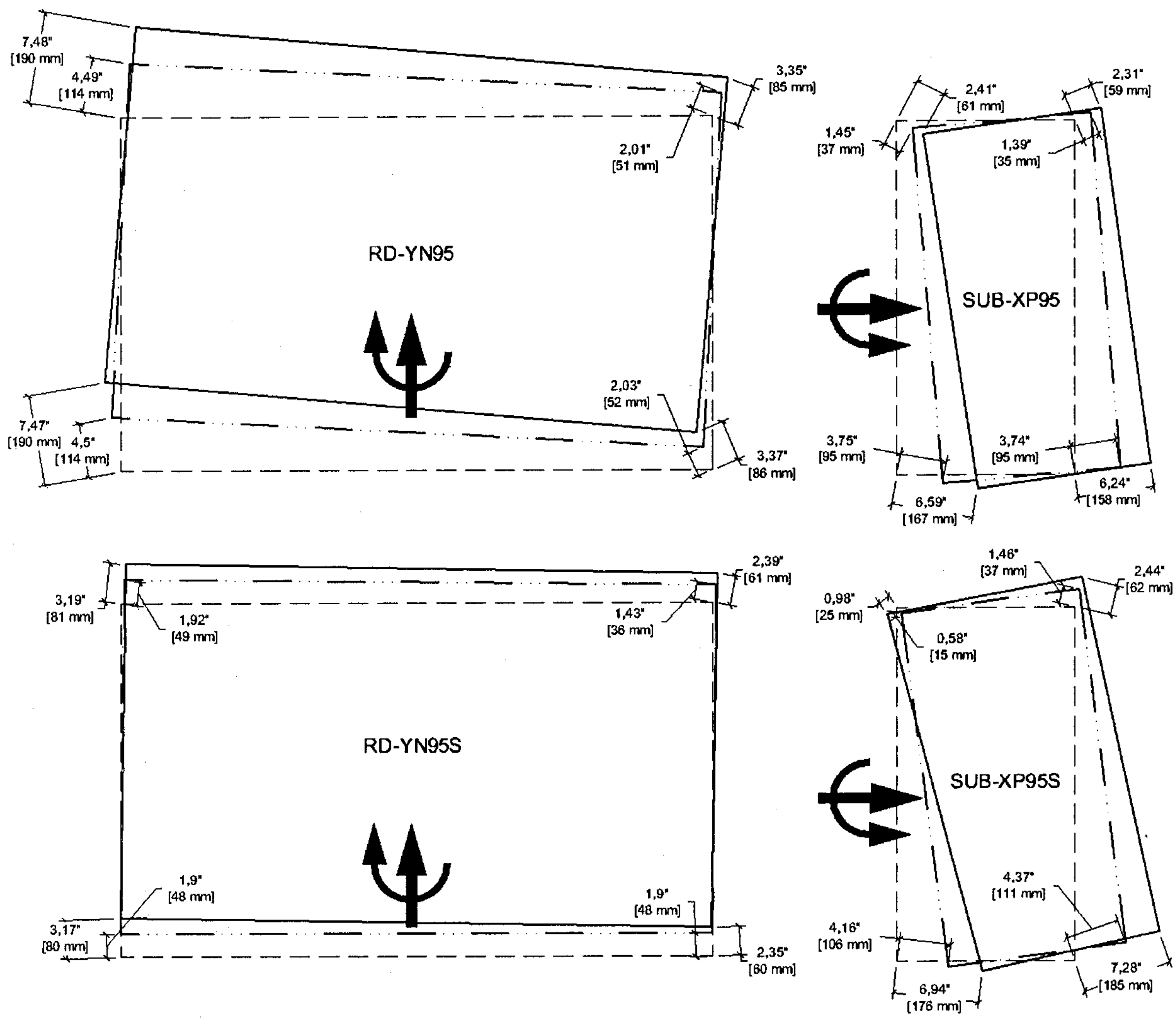
Figure 3-19: Selected internal force diagrams of members affected by the partially infilled walls in a sub-structure 1 model subjected to x-direction earthquake forces.

Column Line	Strut Location	Masonry Strut Strength: Crushing (R_{crush}) (kips)	Crushing (R_{crush}) (kN)	"Shear" (R_{shear}) (kips)	"Shear" (R_{shear}) (kN)	Strut Forces from Analysis Cases: RD-YN95S Full Loading (kips)	RD-YN95S Full Loading (kN)	RD-YN95S 60% Loading (kips)	RD-YN95S 60% Loading (kN)	SUB-XP95S Full Loading (kips)	SUB-XP95S Full Loading (kN)	SUB-XP95S 60% Loading (kips)	SUB-XP95S 60% Loading (kN)
A	4-5, 8-9	172	766	**35**	**156**	-55	-243	-33	-146	-	-	-	-
	5-6, 6-7	140	621	**28**	**126**	-46	-205	-28	-123	-	-	-	-
	7-8	182	810	**37**	**165**	-3	-12	-2	-7	-	-	-	-
F	All bays	186	827	**38**	**168**	-28	-123	-17	-74	-39	-174	-23	-105
C & D	Penthouse 5-6	154	685	**31**	**137**	-73	-323	-44	-194	-	-	-	-
1	All bays	206	919	**41**	**184**	-211	-939	-127	-563	-54	-239	-32	-143
11	All bays	227	1008	**45**	**202**	-163	-723	-98	-434	-	-	-	-
6	Penthouse C-D	174	775	**34**	**152**	-102	-454	-61	-272	-	-	-	-

Table 3-7: Comparisons of equivalent strut strength to strut forces computed in analysis case RD-YN95S and SUB-XP95S.

3.4.3 Story Drift

Although the warehouse is considered to be a post-disaster building, the maximum storey drift limit is taken as 0.02 times the storey height in order to be consistent with the seismic design forces used in the analysis (i.e, for normal structures). The storey height, h_s , of the structure is taken as the effective column height of the structural models which is 12.5 ft (3.8 m). The maximum storey drift of a rigid roof diaphragm is determined at each corner of the structure as the resultant of the storey drifts in both x and y directions. Storey drifts in the simplified analysis models are determined by calculating the free end displacement of the most severely loaded cantilever columns in the structure. The displaced configurations of roof diaphragm for critical analysis cases are shown in Figure 3-20. In the case of the rigid diaphragm models, the relatively flexible columns, which are mainly designed for gravity loadings, are insufficient to restrain the structure laterally. The problem of significant storey drift persists after the roof structure has separated as shown in the case of sub-structures 1. This situation is more severe when an imbalance of lateral stiffness distribution is introduced by the addition of equivalent masonry struts at the building perimeter. Table 3-8 summarizes the maximum inelastic inter-storey drifts recorded from the selected analysis cases, and it has shown that the storey drifts are generally unacceptable with respect to the limit defined by the current building code *(NBCC, 1995)* and thus, it is concluded that additional lateral load resisting elements are needed to stiffen the existing structure.



Deflection ($R\cdot\Delta_{elastic}$) due to 1995 NBCC design base shear

Deflection ($R\cdot\Delta_{elastic}$) due to 60% of 1995 NBCC design base shear

Figure 3-20: Inelastic storey displacements of various analysis cases.

Case	Maximum Storey Drift				Storey Drift Limit 2%h_s	
	Full loading		60% Loading			
	(in.)	(mm)	(in.)	(mm)	(in.)	(mm)
RD-YN95	7.5	190	4.5	114	3.0	76.2
RD-YN95S	3.2	81	1.9	48		
SUB-XP95	6.6	167	3.8	95		
SUB-XP95S	6.9	176	4.4	111		
SA-YN95 Interior column	4.6	118	2.8	71		
SA-YN95 Perimeter column	3.3	84	2.0	51		

Table 3-8: Comparisons of storey drifts and the drift limit specified by 1995 NBCC.

3.5 Conclusions on Seismic Evaluation

The linear elastic analyses of various structural models with different roof configurations have shown that both the nominal moment and shear resistances of the columns are insufficient to resist 60% of the design earthquake forces specified by 1995 NBCC and the seismic design forces given by 1965 NBCC. Without a well defined lateral load resisting system, the flexible structure has experienced significant storey drifts. Analyses using struts-and-columns have further demonstrated that the partial masonry-infilled walls at the building perimeter can further degrade the seismic performance of the precast columns by introducing high shear forces at undesired locations on the columns. The series of analyses has also shown that a rigid diaphragm can systematically deliver the storey shear force to the lateral force resisting elements based on their stiffness. Effort should be undertaken to provide the necessary connections between the roof members so that a rigid diaphragm can be achieved. Other roof structure configurations, such as the separated roof diaphragms which may induce pounding effects due to significant lateral drifts of individual sub-structure; and the present roofing which is composed of discrete precast members, should all be avoided.

Based on the observations from the site inspections as well as the results from structural analyses performed according to 1995 NBCC seismic evaluation procedures, the warehouse is considered to be grossly deficient in its aseismic capacity. It should be noted that the seismic analysis has been conducted only on the basis for a normal building and the strength deficiency of the members could have been amplified to approximately 1.5 times if the structure was analyzed as a post-disaster building. In considering the seismic vulnerability of the existing structure, and especially with it being a post-disaster building, a seismic upgrade of the structure is recommended to improve its seismic performance. Some of the important issues to be addressed in the design of seismic upgrades include:

- A set of well defined lateral load resisting system should be added to the structure.
- Masonry-infilled walls should be separated from the columns and they should be securely tied to the adjacent framing to avoid out-of-plane failure.

- The precast double-tee roof members should be tied together so that the roof diaphragm can behave as an integrated unit.
- The flexible structure should be stiffened by adding vertical lateral load resisting elements to the existing structure.
- Additional lateral load resisting elements should be installed at the penthouse.

Chapter 4: Seismic Rehabilitation

4.1 Introduction

Seismic rehabilitation (or retrofitting) is an effective means to mitigate seismic risk of buildings with deficient lateral load resisting systems. However, seismic rehabilitation projects have often received some resistance even for key post-disaster structures as building owners often worry that engineers will overdesign the seismic upgrades based solely on conservative analytical assumptions *(Bruneau, 1994)*. Thus, engineers should strive for realistic, economical and less disruptive retrofitting schemes in which the performance objectives of the retrofitting should be discussed and agreed by the building owner. Building owners should also be informed about consequences relevant to retrofitting such as the anticipated damage to the lateral force resisting elements and the various retrofit alternatives available for the structure.

4.1.1 Seismic Retrofitting Techniques

The design of the seismic rehabilitation should strive to improve the structural redundancy and the aseismic capability of a building by: removing existing structural irregularities; adding adequate lateral load resisting elements; or upgrading existing structural members. Depending on the severity of the seismic inadequacies concluded from a seismic evaluation, seismic rehabilitation of an existing building may be performed locally, globally (Table 4-1) or with a combination of both methods *(Moehle, 2000)*. Seismic retrofitting at the global level, however, offers a more effective means to strengthen and stiffen a structure for lateral loads and torsional effects, and they are commonly practiced in both aftermath rehabilitation and seismic mitigation projects. As a general review, the following sections will briefly summarize some of the seismic retrofitting techniques applicable to existing reinforced concrete structures.

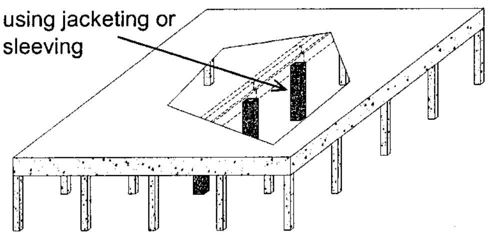
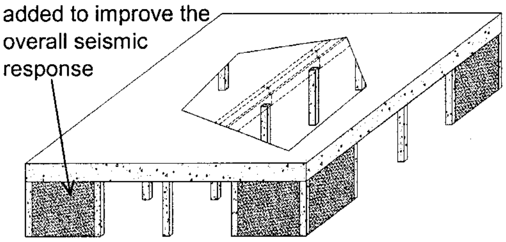
Seismic Upgrade at Local Level	Comments	Examples
Columns retrofitted using jacketing or sleeving 	- Existing structure possess a reasonable aseismic capability - Original lateral force resisting system in the building is preseverved - Only a few inadeuqate structural elements are strengthened - Strength and ductility of selected component are improved - Often economically viable	- Concrete column sleeving - Steel or Fibre Reinforced Polymer (FPR) column jacketing - External stirrups or steel plates reinforcements for concrete beams
Seismic Upgrade at Global Level	**Comments**	**Examples**
Shear walls are added to improve the overall seismic response 	- Seismic resistance of the structure is largely inadequate - Additional and well defined lateral load resisting elements are required - Original structural layout is modified often with removals of building irregularities - Flexibility of original structure are usually reduced by the prescence of new vertical elements	- Reinforced concrete shear walls - Concentric or eccentric steel braces - Moment resisting steel frames - Seismic or base isolation

Table 4-1: Seismic upgrades at local or global levels.

4.1.1.1 Addition of Concrete Shear Walls

An addition of reinforced concrete shear walls is a conventional method to limit storey drift and to reduce the damage to the structural and non-structural elements for a flexible frame structure. New shear walls are usually added to the building perimeter to enhance the torsional resistance of the building, as well as to minimize the disruptions to the interior working space. Significant modification to the existing structural plan is often not necessary. Shear walls may be constructed within the existing concrete framing by inserting either a single or a series of inter-connected precast concrete panels *(Dandurand, 1988)*. More effectively, an infilled concrete wall incorporating existing columns as end piers can be cast within a concrete frame (Figure 4-1) *(Elenas et al., 2002)*. Potential lap-splices in the existing columns should be located and reinforced by welded overlapping bars to avoid tension failure of the boundary elements *(Moehle, 2000)*. The horizontal reinforcement of the infilled shear wall may be anchored around the existing columns with the use of a struts-and-ties model as shown in Figure 4-2. In lieu of infilled shear walls, stand alone shear walls with new footings may be constructed just inside the building walls so that the existing foundation is not disturbed. At locations where an additional shear wall can be an obstruction to existing openings or exits, an addition of coupled shear walls may be considered (Figure 4-3) *(Miller & Reaveley, 1996)*. In all added shear walls, continuity between the newly constructed walls and the lateral distributing system (e.g., a rigid diaphragm) should be ensured by appropriate dowel connections or shear connectors. These connections should be designed to account for inelastic effects produced by the shear wall, such that yielding of the lateral load distributing diaphragm is prohibited.

Existing concrete shear walls with non-ductile or inadequate detailing may be retrofitted by reinforced concrete jacketing (Figure 4-4) *(Elenas et al., 2002)* or shortcreting. However, differential shrinkage and creep at the interface of the new and existing concrete will significantly affect the capacity of the retrofitted wall and should be carefully addressed by a load history analysis. Existing reinforced concrete coupling walls located in a moderate seismic region (e.g., Montréal) may be retrofitted by bolting a thin steel plate to an accessible side of a coupling beam (Figure 4-5) *(Mitchell et al., 1996)*. A steel plate with a length longer than the coupling beam with the bolts extended into the confined core of the coupling beam is recommended for enhanced energy absorption and plate buckling resistance.

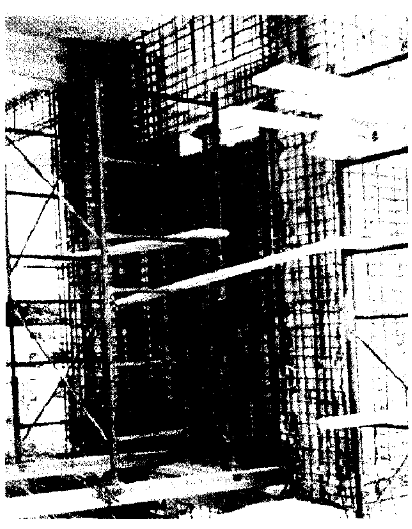


Figure 4-1: Adding a shear wall within an existing concrete frame *(Elenas et al., 2002)*.

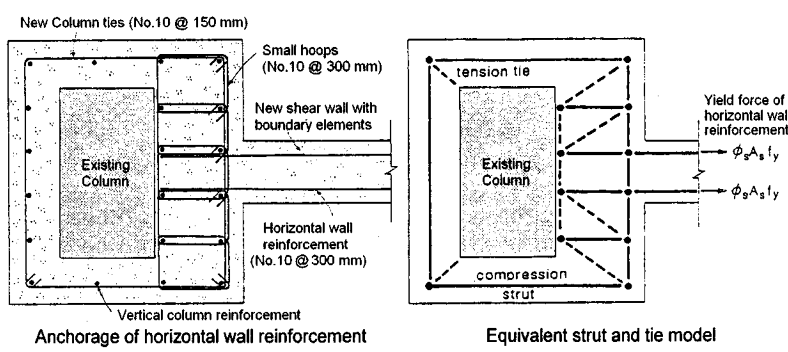


Figure 4-2: Anchorage of horizontal reinforcement for an infilled shear wall with boundary elements *(Dandurand, 1988)*.

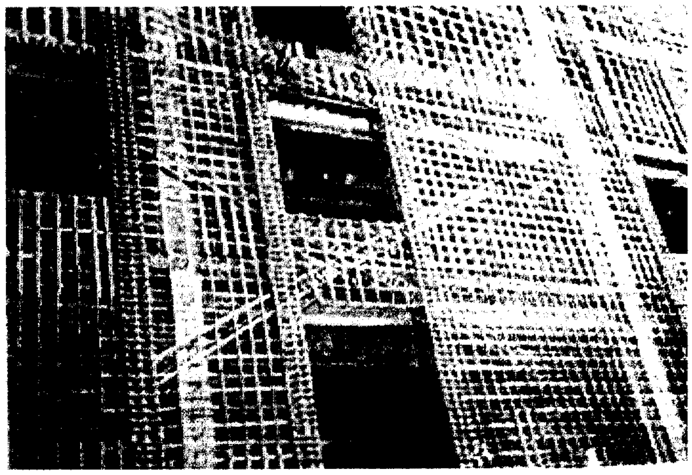


Figure 4-3: Addition of coupled shear walls outside a structure ***(Miller & Reaveley, 1996).***

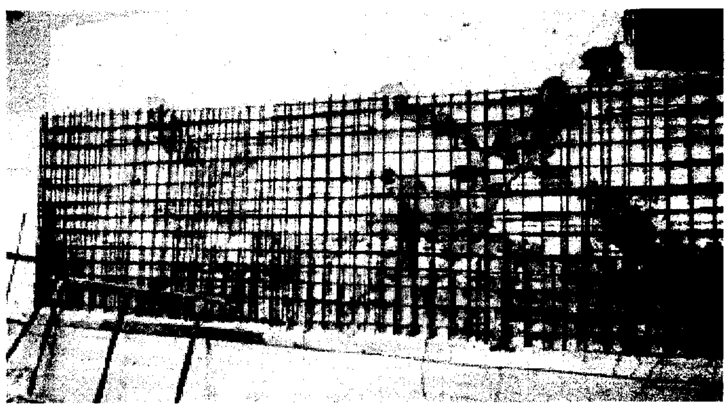


Figure 4-4: Shear walls repaired by conrete jaketing ***(Elenas et al., 2002).***

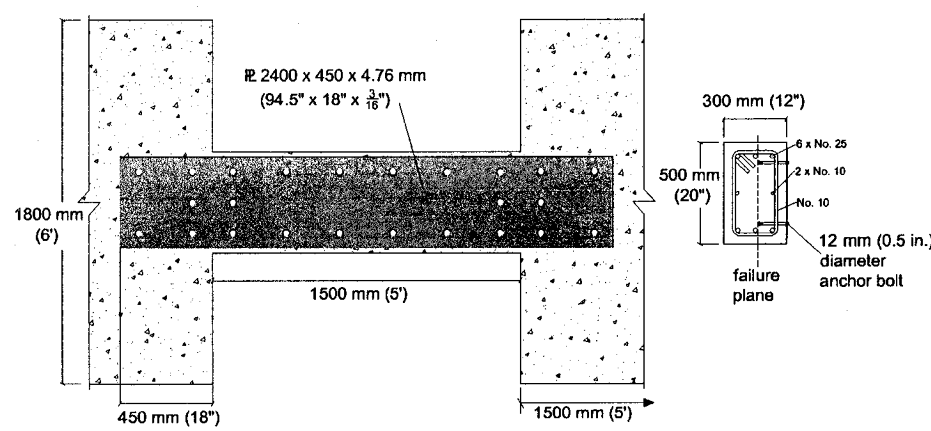


Figure 4-5: A method of repairing coupling beam using bolted steel plates investigated by Mitchell ***et al., (1996).***

4.1.1.2 Steel Braces

Seismic upgrading using concentric or eccentric steel cross-braces is a light weight retrofitting technique and permits rapid installation that causes minimal disruption to the building. Modifications to the existing column foundations at the brace bent are usually necessary to account for the additional shear force introduced by the braces. The design and detailing of the brace connections and the steel to concrete anchorages are critical in such retrofitting methods. The connection design should be approached by the capacity design principle where all connections are detailed to elastically sustain the expected yield force resulting from the inelastic actions of the braces during an earthquake. The sizing of the brace members is often governed by the slenderness ratio and flat-width ratio limits associated with inelastic and local buckling of the brace sections. These limits, however, are less stringent for tension-only braces in low-rise structures where brace buckling is a less important design factor. Although the added braces are designed to resist the full lateral seismic load acting on a structure, the lateral load resistance of the existing non-ductile concrete framing should not be neglected. As failures of the existing weak columns will lead to excessive drifting and the loss of gravity load carrying capacity of the structure *(Badoux & Jirsa, 1990)*, the non-ductile columns in the brace bent should also be retrofitted to accommodate forces arise from the lateral displacement of the braced structure.

Besides the conventionally designed braces, advanced steel braces incorporating friction and viscous damping devices are gradually being adopted in various seismic rehabilitation projects *(Foo et al., 2001)* (Figure 4-6). Another advanced steel brace retrofitting is the use of buckling restrained braces (Figure 4-7). The buckling restrained brace is laterally supported over its entire length with a mortar tube that can slip in relative to the steel brace such that the steel brace can repeatedly yield in tension and compression without buckling *(Brown et al., 2001)*. This lateral bracing system holds a great deal of promise for both seismic retrofitting and the construction of new seismic resistant structure. The design guidelines for buckling restrained braces are currently being prepared in the United States.

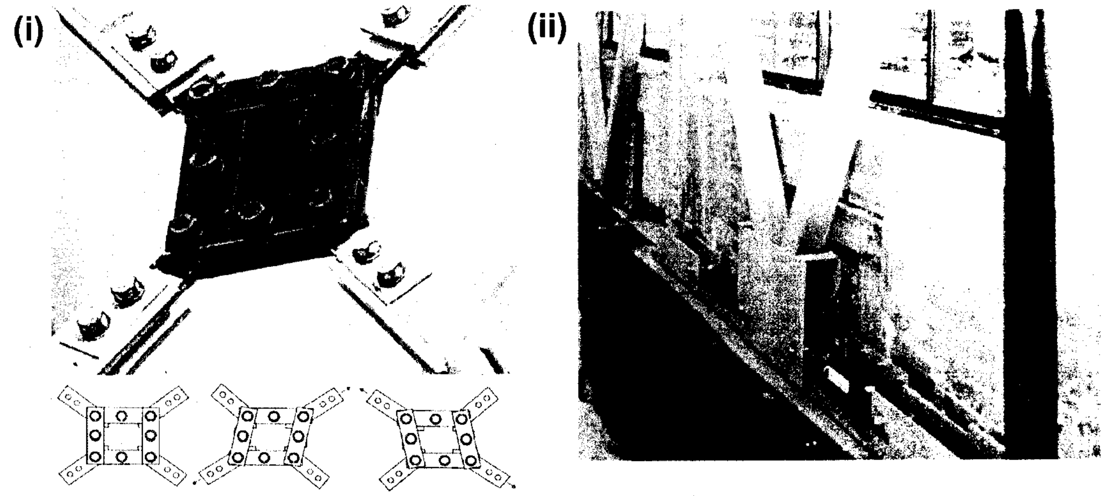


Figure 4-6: Steel brace retrofit incorporating: (i) a friction damper compose of clamped steel plates; (ii) viscous dampers *(Foo et al., 2001).*

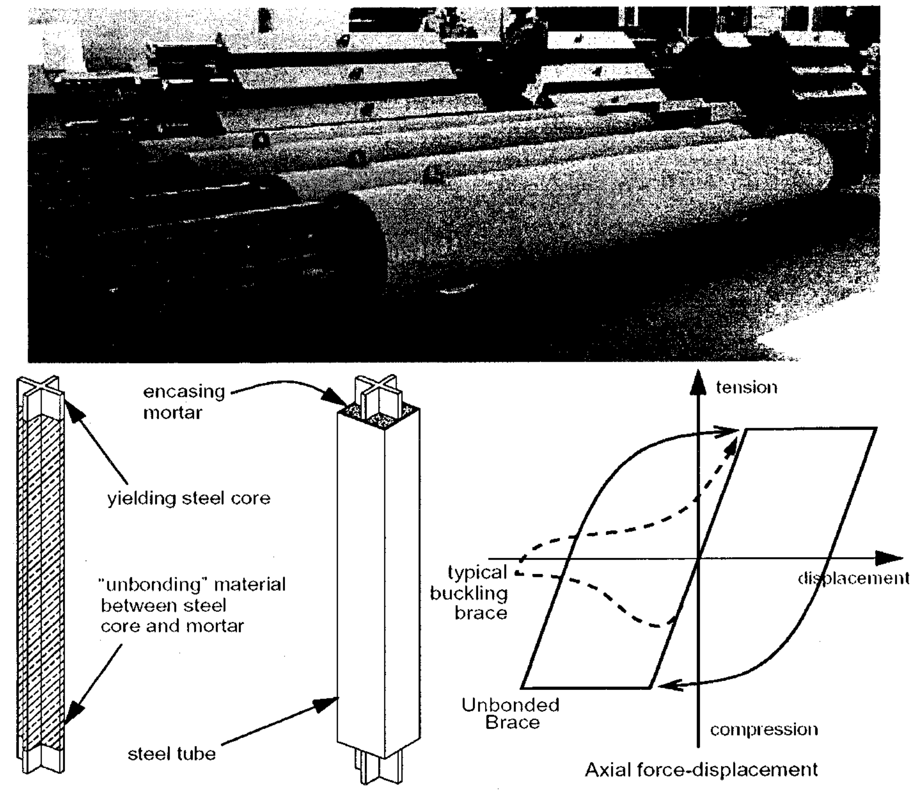


Figure 4-7: Buckling restrained braces *(Brown et al., 2001).*

4.1.1.3 Column Retrofitting

Existing reinforced concrete columns are commonly retrofitted by the addition of reinforced concrete sleeving or steel caging to improve their concrete confinement, as well as their flexural and shear strength. Reinforced concrete sleeving is constructed by casting a layer of new concrete around the existing columns with additional and adequately detailed longitudinal flexural reinforcement and seismic ties. Steel caging is commonly provided by covering the columns with structural steel angles. The corner angles are connected by welded batten plates that are designed and spaced to resist shear force associated with the nominal moment resistance developed by the corner angles *(Poon, 1999)*. The batten plates are capable of not only significantly elevating the shear resistance of the columns, but also provide an effective concrete confinement and buckling restrains to the vertical angles. In the retrofitting methods mentioned above, contact surface between the new and existing concrete should be roughened and cleaned to enable proper contact of the new materials. In the case of steel caging, voids between the existing columns and the added caging should be grouted for continuity.

Recently, seismic retrofitting of concrete columns has utilized the applications of fiber reinforced polymers (FRP). FRP retrofitting involves wrapping the existing or damaged columns with unidirectional FRP laminate, and the orientation of the fibre may vary depending the retrofitting objective (e.g., fibre may be oriented horizontally to increase member column shear resistance). Significant research results have shown that columns retrofitted by FRP jacketing are substantially improved in their level of concrete confinement, shear and flexure resistances, as well as member ductility *(Foo et al., 2001)*. However, attention should be given to the adhesion between the existing concrete and the exterior FRP laminar where debonding may occur especially on the compressive side of the member. The interface between the two materials is often treated or repaired to ensure a proper profile prior to the wrapping process. The use of FRP retrofitting is becoming a competitive seismic rehabilitation solution. Such method is particularly useful for cases where immediate retrofitting of damaged column is needed or where access to the retrofit area is limited *(Triantafillou, 2001)*.

4.1.1.4 Masonry-infilled Walls

In cases where the partial masonry-infilled walls will promote brittle short column failures, the non-structural walls should be separated from their bounding frame. Proper anchorage of the masonry-infilled wall to the structural element should be provided to avoid out-of-plane failure (Figure 4-8) *(Allen, 1993)*. However, in the case where the masonry wall is regularly infilled with no perforation, experimental studies *(Mehrabi et al., 1996; Al-Chaar et al., 2002)* have shown that a masonry-infilled frame can achieve a lateral strength and stiffness greater than those of the bare frame, and only then, retrofitting of existing infilled walls can be an option to improve the seismic performance of an infilled concrete frame. A masonry-infilled wall can be retrofitted by adding a layer of 2 to 4 in. (75 mm to 100mm) thick reinforced shotcrete *(D'Ayala and Charlesons, 2002)* or by reinforcing it with FRP composites for both in-plane and out-of-plane resistances *(Triantafillou, 2001)*. Alternatively, it is suggested that low-rise masonry walls can be retrofitted by bolting diagonal steel strips on both sides of the wall (Figure 4-9) *(Taghdi et al., 1998)*. Such a retrofitting method, which was originally developed to improve the lateral strength, stiffness and ductility of masonry walls, may provide an additional option for the seismic upgrade of masonry-infilled walls. However, it should be noted that during the design of the infill upgrade, the available strength of the bounding frame, such as shear, flexure and tensile resistance of the columns, should also be considered so that premature failure of the frame members prior to the development of the infill resistance is avoided.

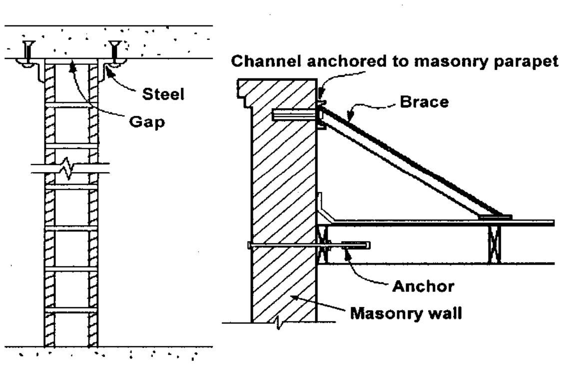


Figure 4-8: Anchorage of masonry wall *(Allen, 1993)*.

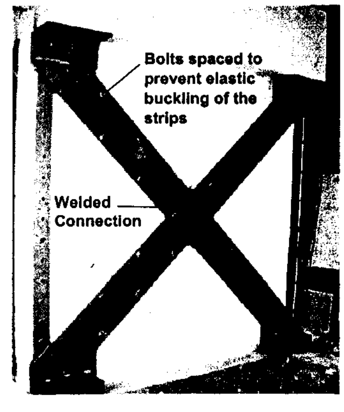


Figure 4-9: Masonry wall retrofitted using bolted steel strips *(Taghdi et al., 1998)*.

4.2 Seismic Retrofitting of the Single-storey Warehouse

4.2.1 Conceptual Strengthening Scheme

Seismic evaluation has indicated that the single storey warehouse severely lacks strength and lateral stiffness to resist a seismic design force computed for an important structure. According to the seismic evaluation procedure of the 1995 NBCC, structures that fail to satisfy 60% of the current seismic design requirements should be retrofitted and designed for the full seismic load specified by the current building code. A retrofitting scheme for the warehouse, which involves adding shear walls, penthouse braces and an upgrade of the roof diaphragm, is proposed and schematically shown in Figure 4-10. The addition of concrete shear walls is an effective solution to limit the lateral drifts and P-Δ effects that are experienced by the flexible concrete frame structure. The added shear walls will be considered as the primary lateral load resisting system of the structure and are conservatively designed to resist all lateral loads acting on the structure. The existing precast columns, which are considered as a set of gravity loads carrying elements, will function only as a secondary lateral load resisting system that enhance the structural redundancy of the building.

Two sets of parallel shear walls of equal size are symmetrically positioned at the building perimeter to maintain a regular structural layout and maximize the torsional resistance provided to the structure. The initial design concept of the walls was to construct them as infilled shear walls within the selected bays as shown in Figure 4-10. However, such concept was not practical due to the obstruction of the ledger beams which prevent the development of a secure connection between the infilled shear walls and the roof diaphragm. Furthermore, a one-bay long shear wall may provide excessive moment resistance which eventually lead to a large shear resistance demand on the wall and its connections. Constructing the shear walls just behind the existing masonry walls is a more practical option which allows the wall-to-diaphragm connection to be passed thought the roof members and permits the development of vertical continuity. Other benefits of locating the wall just inside the building perimeter include: existing architectural finishes of the building can be preserved; disruption to the interior working area is minimized; and lastly, the shear walls are kept in the building interior where the effects of thermal changes on the walls are reduced. The footings of the new shear walls will be constructed from the building interior such that existing column foundations are not disturbed. At the building exterior, the masonry-infilled walls are separated from their confining frames by a 1 in. (25 mm) thick grouted gap to remove their detrimental effects on the vertical load carrying capacity of the columns. Proper lateral supports of the partially infilled masonry walls are provided by anchoring them to their adjacent columns.

In order to achieve a rigid roof diaphragm, a 2 in. (50 mm) thick concrete topping reinforced with welded wire fabric is to be cast above the existing roof structure. Although such method will increase the seismic weight of the structure, it is considered a more effective solution than adding bolted steel straps to the double-tee members. Chord reinforcement resisting the bending moment internal to the roof diaphragm; and drag struts that facilitate the transfer of inertial forces in the diaphragm to the shear walls, are added to the diaphragm edges. The penthouse is retrofitted by adding four sets of tension-only diagonal cross-braces. Tension-only steel braces are installed at the penthouse bays to resist the relatively small earthquake forces acting on the penthouse. The existing penthouse columns are externally reinforced by structural steel angles and welded batten plates. To ensure the existing penthouse roof can behave as an integral unit during earthquake, a set of tension-only cross ties connecting the four comers of the penthouse roof is added.

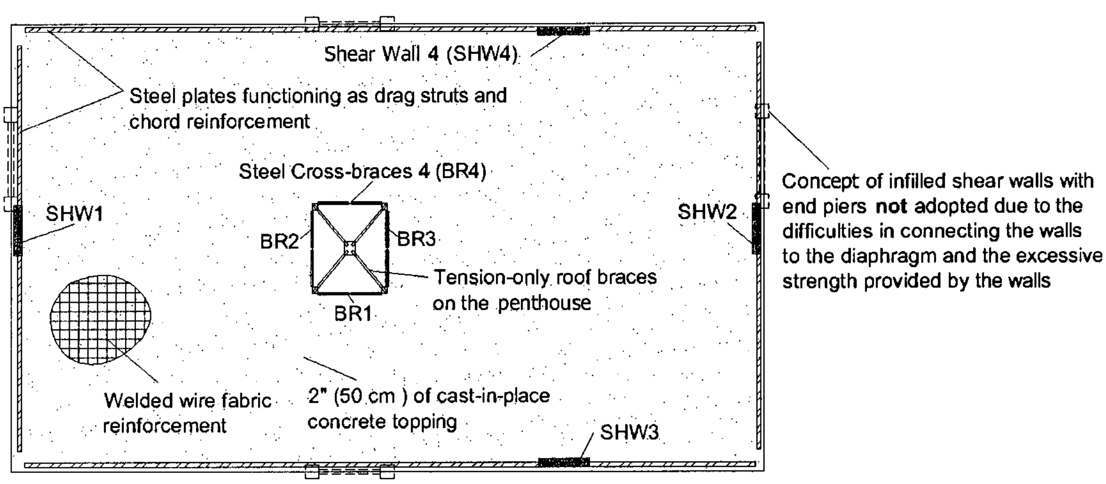


Figure 4-10: Conceptual strengthening scheme for the single-storey warehouse.

4.2.2 Design of the Seismic Retrofit

4.2.2.1 Revision of Seismic Design Forces

The seismic design base shear acting on the retrofitted structure is calculated according to Equation 3-1 with the revision of a few parameters. The empirical period of the retrofitted structure is recomputed by taking D_s in Equation 3-2 as the shear wall length rather than the dimensions of the structure. The resulting seismic response factor, however, remains equal to 4.2 for the short period low-rise structure. Considering that the warehouse has to be in operation during and after disastrous events, the seismic importance factor of the building is conservatively raised to a value of 1.5 so that the retrofitted structure is safely designed with a lower inelastic demand and a tighter drift limit.

As the seismic retrofit of the structure is designed in accordance with the current CSA A23.3-94 Standard for the Design of Concrete Structures *(CSA, 1994)* and CSA S16.1-01 Standards for Limit States Design of Steel Structures *(CSA, 2001)*, the design base shear acting on the structure is qualified for the use of a higher force modification factor, R. In the design of the shear walls, the applicable R values ranges from: the present R = 1.5 for a brittle system; to R = 2 for a stiff system with moderate inelastic demand; and R = 3.5 where the retrofitted structure for a ductile and flexible system (i.e., a structure with larger inelastic displacement) (Figure 4-11). Initial design calculations have shown that the use of R = 2 will result in a relatively heavy reinforcing configuration, such that with additional detailing, the wall may qualify the stringent detailing requirements for a ductile flexure wall (R = 3.5). Thus, it was decided to design the walls using the factor of R = 3.5, and after which, the existing precast columns will be checked if they can sustain displacement induced forces resulting from the use of the higher R value. Based on the above changes, the seismic forces acting on the building computed based on Equation 3-1 is $0.108 \cdot W$, where the seismic weight W includes the added mass due to retrofitting.

The penthouse is relatively small (2% of the size of main structure) and has little effects on the seismic behaviour of the structure. With the existing penthouse columns possessing little deformation capacity and ductility, the penthouse braces were designed with a different force modification factor from the main structure of R = 2. This R factor corresponds to the design of a "Limited-ductility Concentric Braced Frame" given in the CSA S16.1-01 *(2001)* design standard.

4.2.2.2 Structural Modeling

In order to determine the design forces for the retrofitted elements, a linear static analysis accounting for torsional effects was performed. The revised seismic design base shear and torsional moments applied to the analysis model are shown in Table 4-2. The analysis model of the retrofitted structure is constructed by adding four wall elements to the previously constructed rigid diaphragm model. The equivalent masonry compression struts at the penthouse were replaced with a set of diagonal tension-only braces. The equivalent roof slab in the model was also thickened to account for the new cast-in-place topping as well as the change in lumped masses associated with the retrofitting. With the shear walls resisting all the seismic load acting on the structure, the lateral load resistance contributed by the columns are neglected. The shear walls were modeled with wall elements with fixed supports at their bases and pinned connections at the roof diaphragm level. To account for concrete cracking, the stiffness of the walls is taken as 0.7 of the gross EI. This "cracked stiffness is further divided by the value of 3.5 to account for the inelastic displacement of the structure (i.e. $R \cdot \Delta$). The iterative P-Δ option in the program is also turned on although it is expected that it has little influence on the shear wall structure.

As the penthouse braces are designed for R = 2, a separate analysis using the appropriate design forces is performed. In this analysis, the elastic lateral stiffness of the tension-only steel braces are divided by R = 2 to obtain the inelastic storey drift of the penthouse. Computed design forces in the shear walls and braces are shown in Table 4-3.

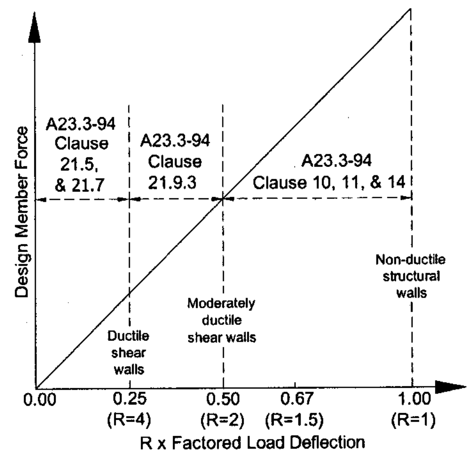


Figure 4-11: Design of reinforced concrete shear walls at various ductility levels (R) according to the CSA A23.3-94 *(1994)*.

EQ. Direction	Level	Storey Shear		Torsional Moment	
		kips	kN	kip-ft	kN
E-W (R=3.5)	Penthouse Roof	61	272	305	414
	Main Roof	1470	6539	35460	48077
	Σ	1531	6811	35766	48491
N-S (R=3.5)	Penthouse Roof	61	272	-254	-345
	Main Roof	1470	6539	-60482	-82001
	Σ	1531	6811	-60736	-82346

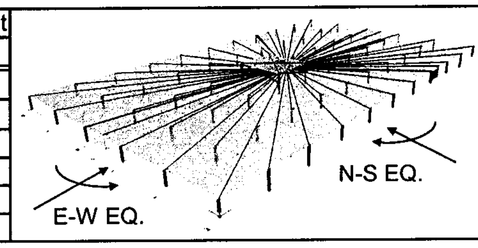
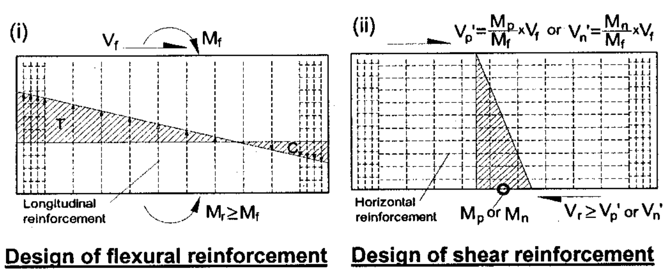


Table 4-2: Seismic design base shear forces and torsional moments (R=3.5) applied on the retrofitted structure.

Shear Walls (R=3.5)	EQ. Direction	Shear Force		Base Moment	
		kips	kN	kip-ft	kNm
SHW1	E-W	-67	-298	-848	-1150
	N-S	**885**	**3937**	**11149**	**15116**
SHW2	E-W	67	298	848	1150
	N-S	652	2900	8213	11135
SHW3	E-W	**807**	**3590**	**10172**	**13791**
	N-S	68	302	848	1150
SHW4	E-W	730	3247	9193	12464
	N-S	68	302	848	1150

Braces (R=2)	EQ. Direction	Axial Force	
		kips	kN
BR1	E-W	**66**	**294**
	N-S	6	27
BR2	E-W	5	22
	N-S	**62**	**276**
BR3	E-W	5	22
	N-S	53	236
BR4	E-W	50	222
	N-S	6	27

Table 4-3: Design forces in shear walls and braces computed by a linear static analysis (values in bold indicate forces used in the design of retrofit elements).



where:
- V_r = Factored shear resistance
- V_f = Design shear force from analysis
- V_p' = Probable design shear force
- V_n' = Nominal design shear force
- M_r = Factored moment resistance
- M_f = Design moment from analysis
- M_p = Probable moment resistance (when R = 3.5)
- M_n = Nominal moment resistance (when R = 2.0)

Figure 4-12: Design concept of moment and shear resistances of shear walls.

4.2.2.3 Design of Shear Walls

The design process of a shear wall may be schematically represented by Figure 4-12. The longitudinal flexure reinforcement is designed to resist the design moment computed from a linear static analysis of the retrofitted structure. Horizontal shear reinforcement and the wall-to-diaphragm dowel connections of each wall are both designed to resist the shear force associated with the formation of plastic hinging at the wall base. For uniformity, shear walls oriented in the N-S and E-W directions are designed for the critical forces in SHW1 and SHW3 respectively. The shear walls are designed and detailed as "Ductile Flexural Walls" in conformance with Clause 21.5 of CSA A23.3-94 *(1994)*. Appropriate material resistance factors (i.e., $\phi_c = 0.6$, $\phi_s = 0.85$ for steel reinforcement, $\phi_s = 0.90$ for structural steel) are applied in the design. Both shear walls are properly dimensioned for stability, and the concentrated and distributed wall reinforcement are designed according to the respective design clauses given in CSA A23.3 *(1994)*. The horizontal shear reinforcement is designed based only on the shear resistance demand due to the development of the probable moment resistance which is assumed to be 1.47 times of the factored moment resistance. The reinforcing detailing of the shear walls are shown in Figure 4-13 through Figure 4-15. Detailed calculations on the design of the shear walls are available in Appendix B.

The dowel connection between the shear walls and the roof diaphragm are designed to resist the "probable shear force" arising from the probable moment resistance based on the shear-friction design method covered in Clause 11.6 of CSA A23.3-94 *(1994)*. The dowel connections are designed to behave elastically even when the shear wall exhibits inelastic behavior. The method of connecting the existing concrete floor diaphragms to the shear walls differ somewhat at different locations in the building depending on the orientation of the double-tee members. Shear walls SHW1 and SHW2 are fully extended to the roof, where holes will be cut locally to allow the No.20 dowels to be inserted through the roof diaphragm. To ensure that the shear walls will not be loaded by gravity forces, a 1 in. (25mm) thick grouted gap is allocated between the roof and the shear walls to make room for the vertical deflection of the double-tee members. In the E-W direction, shear wall SHW3 and SHW4 have a limited "dowelable" area due to the obstruction of the existing double-tees. As an alternative to a congested doweling scheme, it is assumed that the interlocking "shear keys" formed by the double-tee stems and shear walls can assist in the shear transfer, such that a sufficient number of dowels are provided to resist the design shear force acting on the wall and the "shear keys" will provide the remaining resistance needed to resist the probable shear force due to hinging at the wall base.

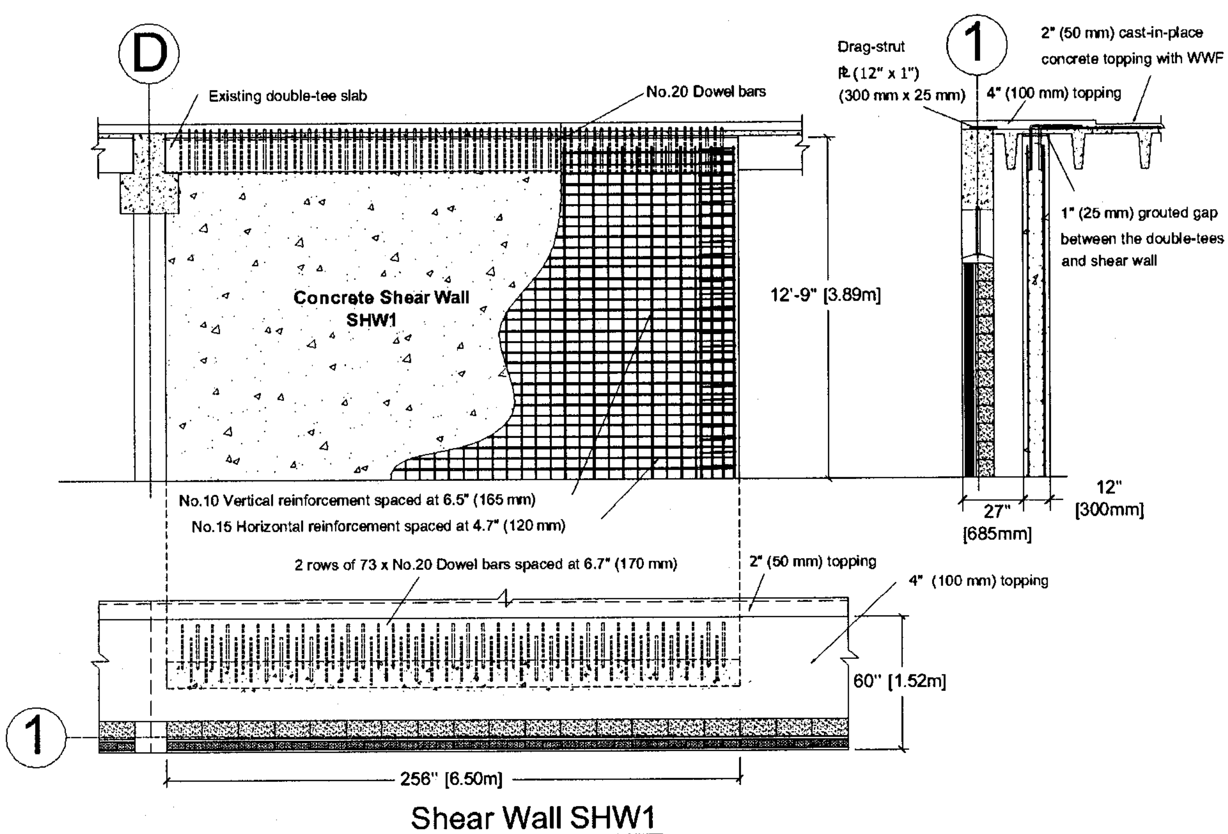


Figure 4-13: Elevation of shear wall SHW1.

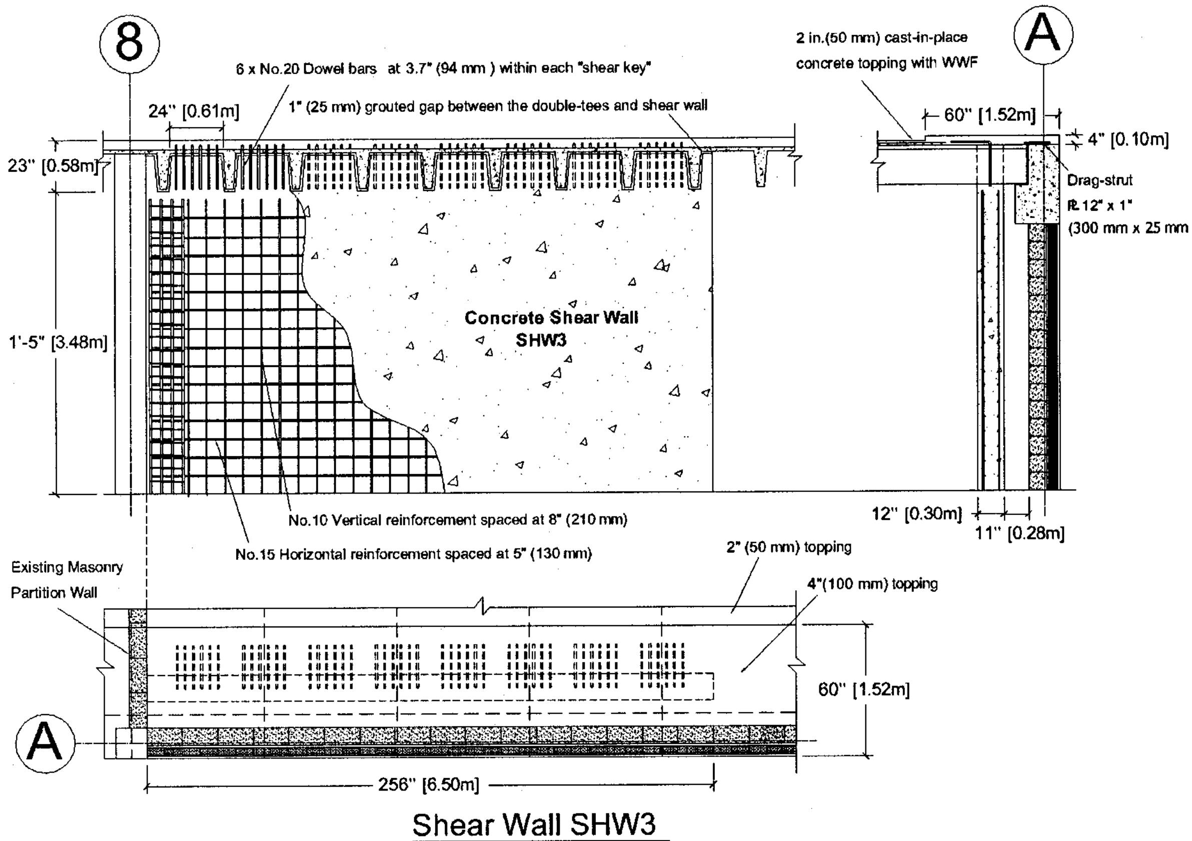


Figure 4-14: Elevation of shear wall SHW3.

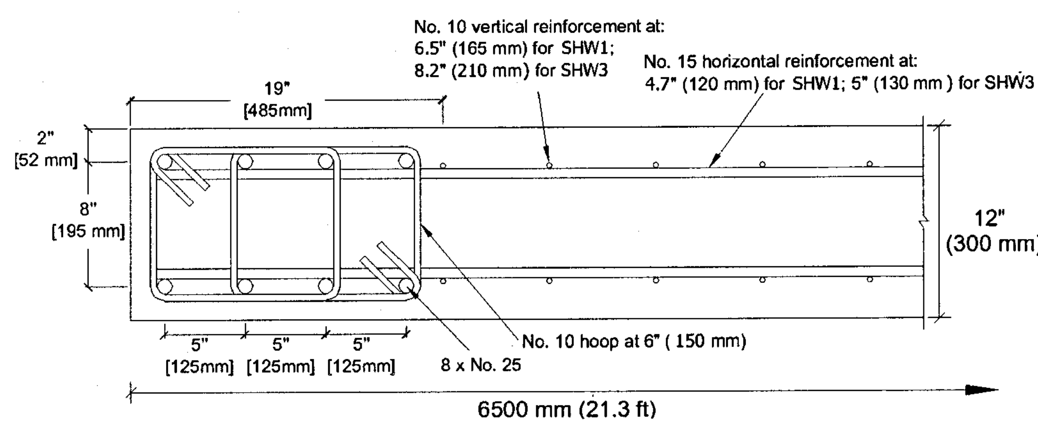


Figure 4-15: Reinforcing details of the shear wall.

4.2.2.4 Design of Roof Diaphragm Upgrades

The upgrading of the diaphragm is designed for the probable shear forces associated with the development of hinges at the wall bases so that the diaphragm will function elastically during an earthquake. In idealizing the diaphragm as a horizontal deep beam supported at the shear walls (Figure 4-16), the bending action will be resisted by the chord reinforcement, and with the diaphragm adequately reinforced for shear by welded wire fabric reinforcement (WWF). The shear forces at the diaphragm ends are transferred to the shear walls by the combination of direct shear on the wall-to-diaphragm connections and axial resistance of the drag-struts and WWF. The WWF in the 2" (50 mm) thick cast-in-place concrete topping was designed according to a shear-friction method. The factored shear resistance of the precast roof diaphragm with cast-in-place topping, V_r, was designed using Equation 4-1 modified from Hawkins *et al. (2000)*:

$$V_r = \phi_s \cdot A_{vf} \cdot f_y \cdot \mu \qquad \textbf{Equation 4-1}$$

in which A_{vf} is the total area of shear-friction reinforcement; f_y is the yield stress of the WWF taken as 400 MPa; μ is the coefficient of friction taken as 1.0; and ϕ_s is the material resistance factor of steel reinforcement taken as 0.85. The sizes and spacing of the WWF reinforcement were chosen such that the reinforcement ratio is sufficient to resist the probable shear force developed in the shear walls. The WWF reinforcement is also checked to have satisfied the minimum shrinkage reinforcement ratio in both directions. Before the casting of the new topping, the existing surface is to be cleaned and roughened to ensure proper contact between the added topping and the existing double-tee roof members.

Drag-struts and flexural chord reinforcement are provided in the form of steel plates and they are to be bolted to the double-tee flanges. In order to accommodate the additional edge reinforcement, the thickness of the concrete topping is increased to 4" (100 mm) near the diaphragm edges. In the case of the more severe N-S direction earthquake, the drag-struts are designed to deliver the inertial force tributary to their strut length as demonstrated in Figure 4-16. Due to the limited space available within the topping layer, the maximum sizes of the drag-struts are limited to a cross-section of 12 in.

wide by 1 in. thick (300 mm by 25 mm). However, drag-struts of such dimensions are not able to deliver their full tributary inertial force. The deficit in the load transfer capacity was compensated by considering the WWF aligned parallel to the drag-strut direction. As the "tributary blocks" (see Figure 4-16) displace as an entity, the inertial force acting in the diaphragm will be transferred to the dowel connections protruding from the shear walls.

On the edges perpendicular to the earthquake, maximum chord forces were determined from the bending moment at diaphragm midspan. As the steel plates provided at the edges are not able to fully resist the computed chord forces, it was decided to engage some of the WWF at the diaphragm midspan to help resist the bending effects. Since the diaphragm shear is minimal in the midspan region, the WWF, previously designed for the shear resistance at the diaphragm ends, are adequate. Detailed calculations on the design of the diaphragm upgrades are available in Appendix B.

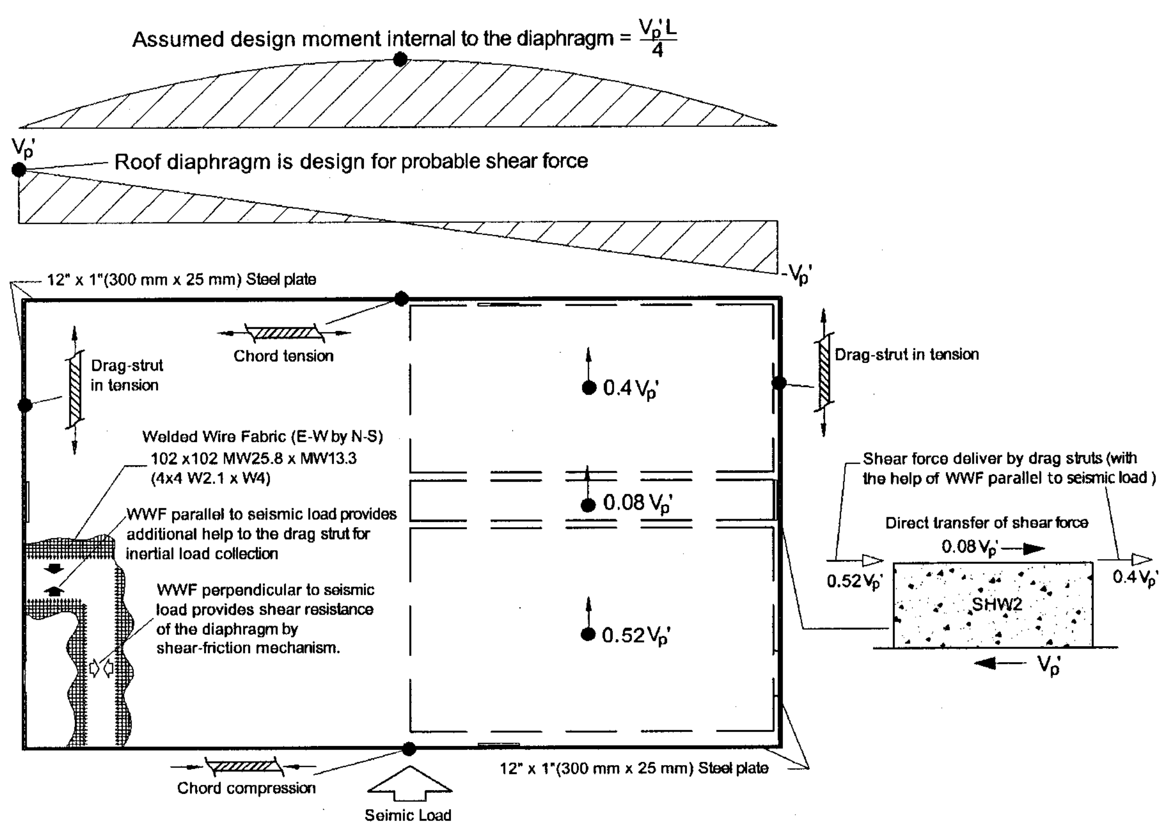


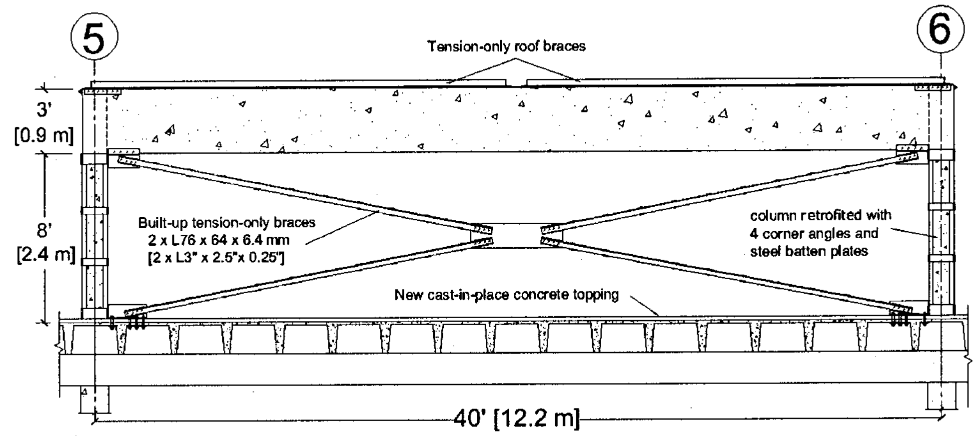
Figure 4-16: Design of the roof diaphragm upgrade.

4.2.2.5 Design of Penthouse Upgrades

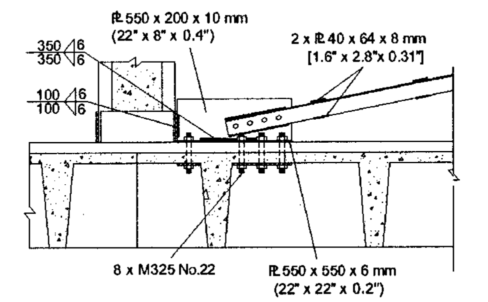
The tension-only steel braces at the penthouse are designed as braces with "limited-ductility" according to CSA S16.1-01 *(2001)*. The braces are sized to resist the maximum design forces computed in the computer model (Table 4-3) and to limit storey drifting of the penthouse. The tension-only braces are designed to be concentrically connected within each bay, but during the actual installation, small eccentricity may occur due to obstruction by the existing concrete members. Each steel brace consists of two Grade W300 L76×6.4×6.4 angles welded together by 0.3 in. (8 mm) thick batten plates at the top and bottom of the built-up section. The braces were selected to meet the specified flat-width ratio limits and the overall slenderness ratio of the steel brace is kept under the 300 limit. At the penthouse base, each brace is connected to a 0.4 in. (10 mm) thick gusset plate welded vertically to the retrofitted columns, and horizontally to a steel plate that is bolted through the roof diaphragm. At the penthouse top, the brace is bolted to a gusset plate that is welded to the retrofitted columns and to a steel plate that is attached to two structural angles, which in turn are bolted to the bottom corners of the ledger beam. All brace connections, as well as the steel-to-concrete anchorages are designed for the anticipated yield force (i.e., 1.1 times f_y but not less than 385 MPa) of the braces. To ensure the ductile behaviour of the braces, additional steel plates are welded to the angles ends to avoid net-section fracture at the bolted connections. All gusset plates and connection elements were checked for bearing resistance as well as net-section fracture to avoid brittle failure of the connections during an earthquake. The conceptual retrofits of the penthouse braces are shown in Figure 4-17 and Figure 4-18. Detailed calculations on the design of the penthouse retrofit are available in Appendix B.

Each penthouse column is retrofitted to sustain forces resulting from the inelastic displacement of the braced penthouse. Four L64×6.4×6.4 structural steel angles are placed at the corner of the penthouse columns with welded steel batten plates (Figure 4-19). The corner angles provide additional flexural resistance for the columns, while the spacing of the horizontal batten plates is adjusted to resist shear force associated with the development of the nominal moment resistance of the angle-reinforced column section.

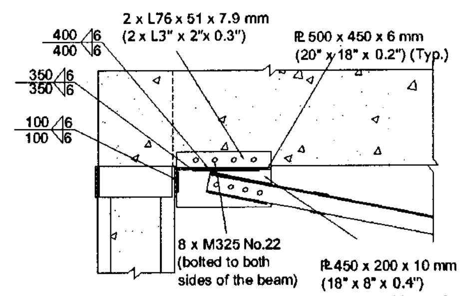
The horizontal tension-only cross-braces added to the penthouse roof are designed for forces experienced by the penthouse structure when the vertical braces undergo inelastic behaviour. The horizontal braces, which consist of Grade W350 WT100×26 structural tee members, are welded to the steel-to-concrete connections at the four corners of the penthouse roof and the gusset plate bolted to the roof centre (Figure 4-20). In order to reduce the slenderness of the roof brace, lateral supports for the braces are provided by welding the braces to a steel plate which is in turn bolted down to the penthouse roof.



Tension-only braces BR1 in p enthouse bay 5-6 (E-W direction)

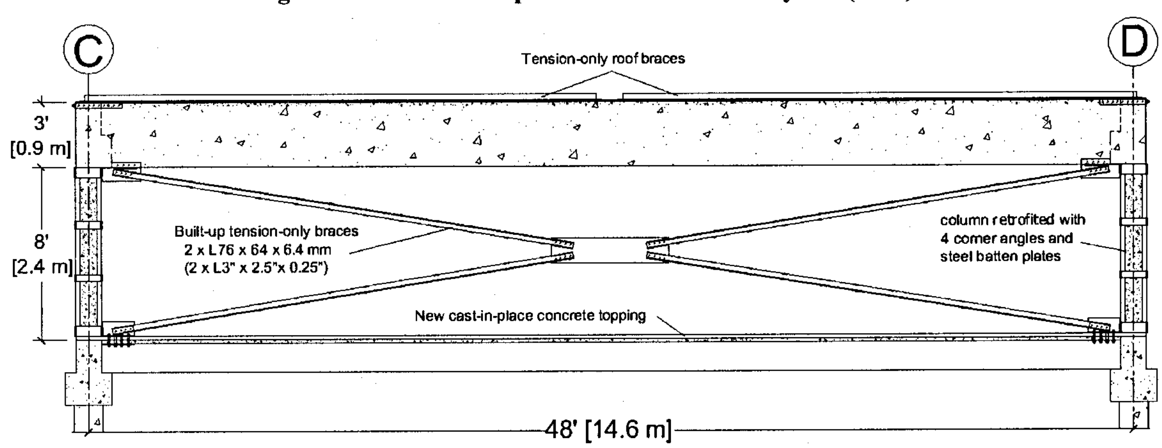


Brace BR1 connection at column base level

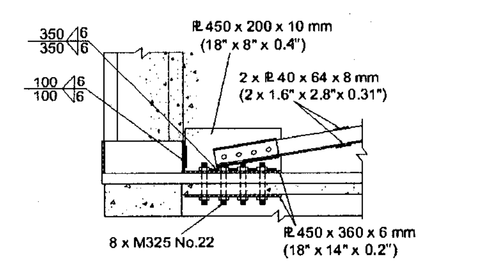


Brace BR1 connection at roof level

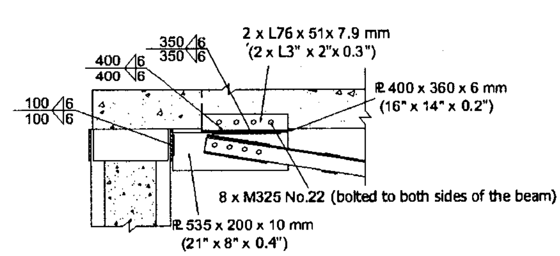
Figure 4-17: Details of penthouse retrofit at bay 5-6 (BR1).



Tension-only braces BR2 in p enthouse bay C-D (N-S direction)



Brace BR2 connection at column base level



Brace BR2 connection at roof level

Figure 4-18: Details of penthouse retrofit at bay C-D (BR2).

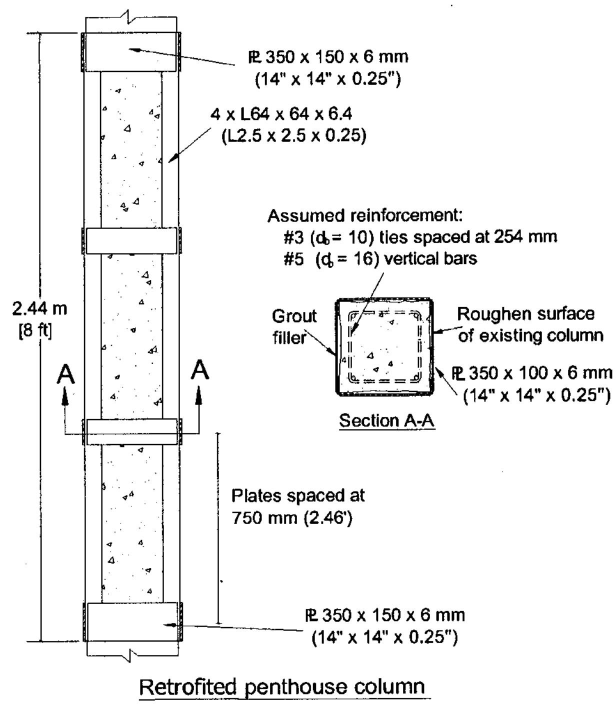


Figure 4-19: Details of penthouse column retrofit.

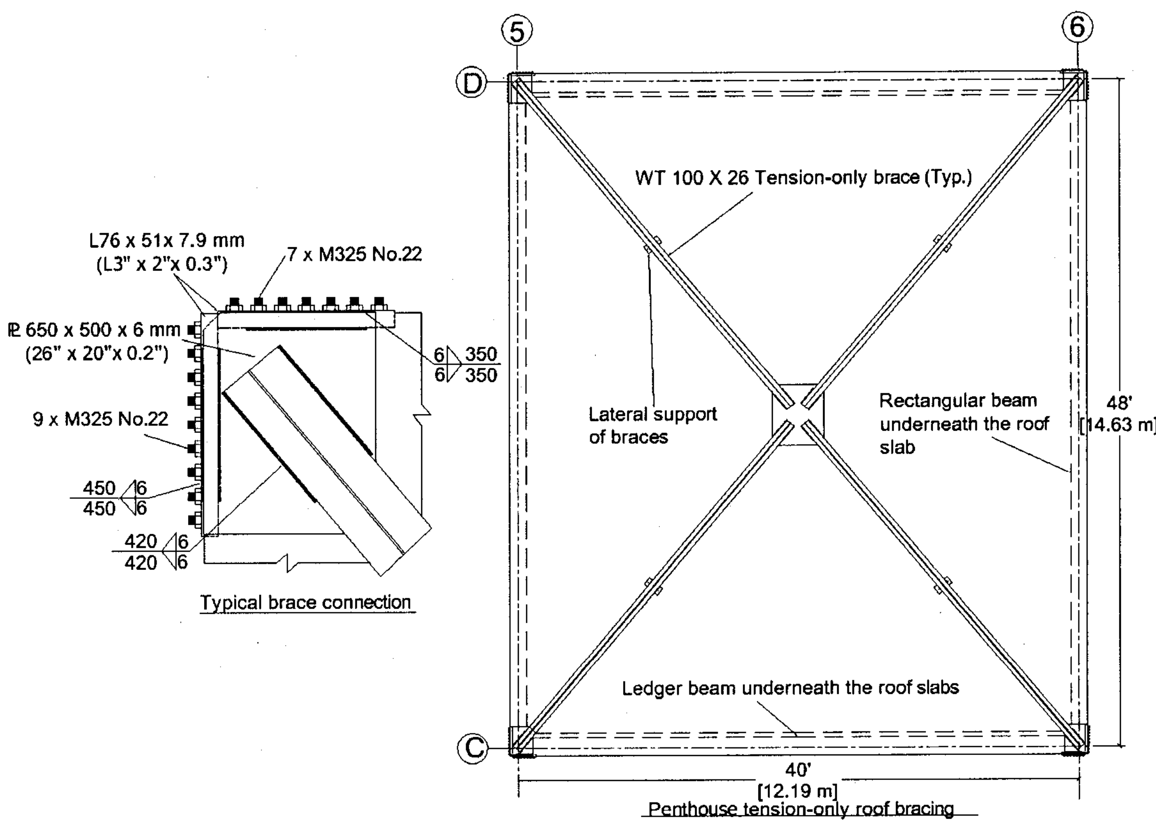


Figure 4-20: Tension-only roof braces on the penthouse.

4.3 Comments on the Retrofitted Single-storey Warehouse

The structure is to be retrofitted for a seismic importance factor of 1.5 and a system ductility level of R = 3.5 (except for the penthouse where R = 2 is used). The presence of a well defined lateral load resisting system has not only greatly increased the lateral strength of the structure, but also has successfully reduced the lateral drift experienced previously by the structure. Figure 4-21 shows that the inelastic inter-storey drifts of the retrofitted structure and the penthouse when they are subjected to both x and y direction earthquakes and the associated torsional moments. The recorded inter-storey drift values for both of the main structure and the penthouse are within the 1% storey drift limit defined for a post-disaster structure. Furthermore, the effects of the displacement induced forces in the existing precast columns are checked as shown in Figure 4-22. The displacement induced moments in the critical interior and perimeter columns are calculated based on the displacements of the column tips recorded in the y-direction earthquake. Both plots have shown that the overturning moment with the combined gravity load acting in the selected columns are well within the nominal resistance envelopes of the various assumed column sections.

As an additional comment, the building owner should be informed that seismic retrofitting designed with the use of the R factor does not guarantee a damage-free building. Instead, damage to the structure will be confined to the ductile lateral load resisting system with the use of capacity design approach and these elements are repairable. Some minor damage to non-structural elements may also occur especially if they lack proper anchorages. Conversely, the use of a low R value such as 1 or 1.5 will result in less structural damage; however, an increase in both direct and indirect cost due to considerable structural intervention will be expected.

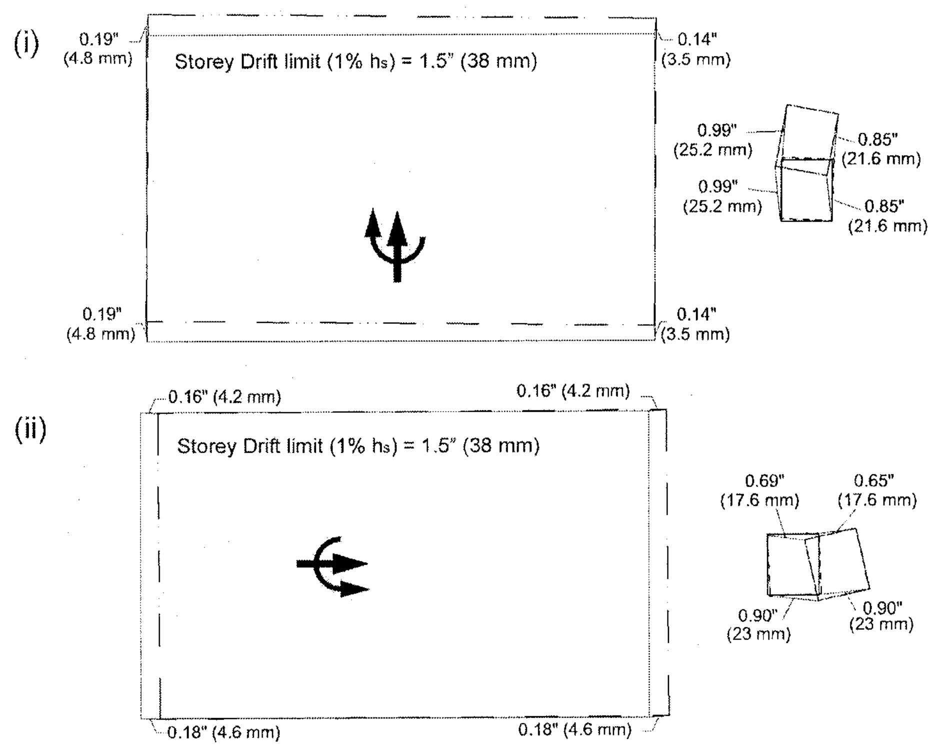


Figure 4-21: The inelastic storey drifts of the retrofitted structure under the full 1995 NBCC design earthquake loads: (i) in the y-direction; and (ii) in the x-direction.

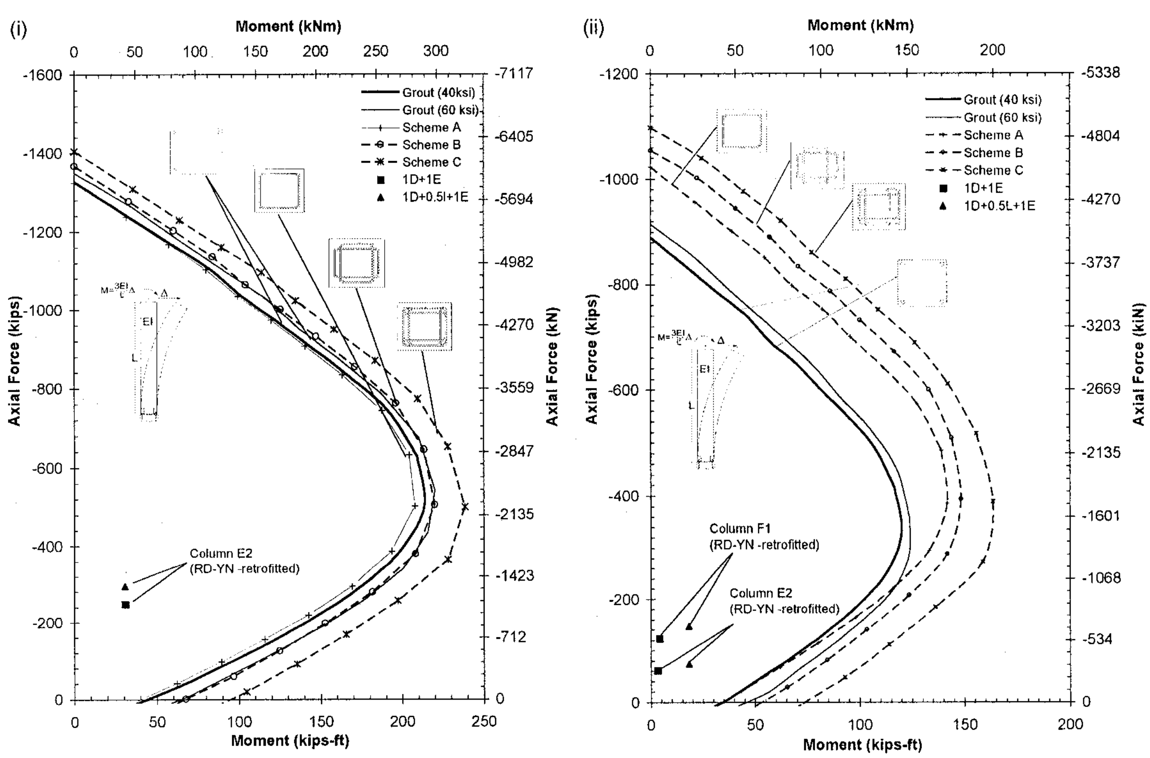


Figure 4-22: Deflection induced moment and gravity load in selected (i) interior columns; and (ii) exterior columns (values of the column deflections correspond to those recorded in Figure 4-21 (i)).

4.4 Impact of the Proposed 2005 NBCC Seismic Design Provisions

A new edition of NBCC seismic design provisions is being proposed at the time of preparation of this report, and it is likely to become available in the upcoming edition of NBCC *(NBCC, 2004)*. Based on the draft copy of the new provisions *(NBCC, 2004)*, the followings sections will briefly summarise the new seismic design provisions and discuss its impact on the retrofitted structure.

4.4.1 Summary of the Proposed Seismic Design Provisions

The equivalent static procedure of the proposed 2005 NBCC seismic design provisions is summarized in Table 4-4. As the use of dynamic analysis has become increasingly significant in the new edition of the seismic code, the use of equivalent static analysis to determine design forces is only permitted for regular structure and some irregular structures with stringent restrictions *(Humar & Mahgoub, 2003)*. The empirical period equations given in the new code are similar to those in the current code *(NBCC, 1995)* except for the case of shear wall structures. Seismic importance factors reflecting the social importance of a structure remained essentially the same as those in the current 1995 NBCC.

The increased sophistication in the proposed provisions occurs in the departure from the use of peak ground acceleration and velocity zoning maps, to the generation of a set of site-specific uniform hazard spectra that corresponds to an earthquake event with a 2% probability of exceedance in 50 years (i.e., a return period of 2500 years). The determination of design acceleration spectrum (Table 4-5) for a given major Canadian city involves the use of uniform hazard spectra provided by the new code as well as a set of period dependent foundation factors. Unlike the present foundation factors in 1995 NBCC which function as an explicit multiplier, the new foundation factors (Table 4-6) will affect the design base shear formula implicitly through amplification or deamplification of the design spectral value at the referenced soil conditions (i.e., class C site) to account for the geological conditions at the building site.

Another important change in the base shear formula is the replacement of the ductility factor, R, and calibration factor, U, with a set of coupling force modification factors namely: R_d, which reflects the ductility of the lateral load resisting system; and R_o, which reflect the considerable reserve of strength due to member overstrength and good design and detailing practice *(Mitchell et al., 2003)*. The new seismic design provisions have also imposed stringent requirements to avoid problems associated with irregular structural configuration. A building with one or more of the eight structural irregularities specified by in the new provisions will be restricted in the type of analysis procedures permitted. Horizontal stiffness irregularity across a structural plan will be reflected by the calculation of a torsional sensitivity ratio, B_x, as shown in Table 4-4. For irregular structures with torsional flexibility greater than 1.7 (i.e., $B_x > 1.7$), the use of dynamic analysis is mandatory.

The proposed inter-storey drift requirements are similar to those in the current seismic design provisions *(NBCC, 1995)*. However, such drift limitations are actually more stringent by considering the seismic design forces in 2005 NBCC, are computed based on a very rare and more damaging earthquake event *(DeVall, 2003)*. Therefore, structures that are designed under the new seismic code requirement are expected to have a greater reserve of lateral strength and stiffness than those designed according to the current 1995 NBCC.

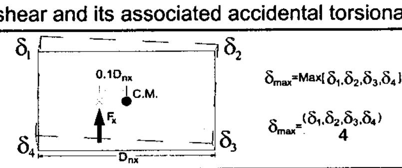
Proposed Seismic Design Provisions of 2005 NBCC	
Static Design Base Shear	**Definition of Variables**
$V_D = \frac{S(T_a) \cdot M_v \cdot I_E \cdot W}{R_d \cdot R_o}$ but not less than $\frac{S(2.0) \cdot M_v \cdot I_E \cdot W}{R_d \cdot R_o}$ for lateral load resisting system with Rd ≥ 1.5, V_D need not be taken greater than: $\frac{2}{3} \cdot \frac{S(0.2) \cdot I_E \cdot W}{R_d \cdot R_o}$	$S(T_a)$: Desigin spectra response acceleration expressed as a ratio of g for period T_a
	T_a : Structural period of the building, may be empircally computed as:
	Concrete frame: $T_a = 0.075 \cdot (h_n)^{3/4}$; Steel frame: $T_a = 0.085 \cdot (h_n)^{3/4}$; Other moment frame: $T_a = 0.1 \cdot N$; Shear wall structure: $T_a = 0.05 \cdot (h_n)^{3/4}$ — where h_n is building height in metre (m)
	M_v : Higer mode effects factor
	I_E : Seimic importance factor defined as: Low 1.0; Normal 1.0; High (e.g., school) 1.3; Post-disaster 1.5
	W : Seismic weight (dead load, 25% snow load, 60% of storage load)
	R_d : Ductility-related force modification factor (ranges from 1.0 to 2.0)
	R_o : Overstrengh-related force modification factor (ranges from 1.0 to 1.7)
Torsional Sensitivity and Analysis	**Definition of Variables**
$B_x = \frac{\delta_{max}}{\delta_{ave}}$	B_x : Ratio of maximum to average storey displacements; values for a single-storey penthouse with a weight less than 10% of the level below need not be considered.
For B ≤ 1.7: Application of static torsional moment is $T_x = F_x \cdot (e_x \pm 0.1 \cdot D_{nx})$	δ_{max} : Maximum storey displacement at extreme points of a structure at level x due to the applied storey shear and its associated accidental torsional moment (i.e., $\pm(0.1 \cdot D_{nx}) \cdot F_x$)
For B > 1.7: Dyanmic analysis is mandatory	δ_{ave} : Average displacement at extreme point of a structure at level x due to the applied storey shear and its associated accidental torsional effects.
	for example: 
Design Lateral Deflection	**Interstorey Drift Limit (%)**
$\Delta_{Design} = \Delta_{analysis} \cdot \frac{R_d \cdot R_o}{I_E}$	Post-disaster: 0.01
	High seismic importance: 0.02
($\Delta_{analysis}$ includes P-Δ & torsions)	Others: 0.025

Table 4-4: A summary of seismic design provisions for the proposed 2005 NBCC ***(NBCC, 2004).***

Computation of Design Spectral Acceleration in NBCC 2005	Design Spectral Value for Montreal	
	Class C	Class D
T ≤ 0.2 : $F_v \cdot S_a \cdot (0.2)$	0.69	0.94
T = 0.5 : Smaller of $\begin{cases} F_v \cdot S_a \cdot (0.5) \\ F_a \cdot S_a \cdot (0.2) \end{cases}$	0.34	0.46
T = 1.0 : $F_v \cdot S_a \cdot (1.0)$	0.14	0.19
T = 2.0 : $F_v \cdot S_a \cdot (2.0)$	0.048	0.065
T ≥ 4.0 : $F_v \cdot S_a \cdot (2.0)/2$	0.024	0.033

Table 4-5: Computation of design spectral acceleration involving the foundation factors F_v and F_a.

Site Class	F_a as a function of $S_a(0.2)$ and soil class		Montreal F_a
	$S_a(0.2)$=0.50	$S_a(0.2)$=0.75	$S_a(0.2)$=0.69
C*	1	1	1
D**	1.2	1.1	1.124

Site Class	F_v as a function of $S_a(1.0)$ and soil class		Montreal F_v
	$S_a(1.0)$≤0.10	$S_a(1.0)$=0.2	$S_a(0.2)$=0.14
C	1	1	1
D	1.4	1.3	1.36

* Class C: very dense Soil and soft rock
** Class D: very stiff soil

Table 4-6: Determination of the foundation acceleration factor (F_a) and velocity factor (F_v) for a class C site and a class D site in Montréal.

4.4.2 Effects on the Retrofitted Single-storey Warehouse

The draft version of the upcoming edition of NBCC discussed in the previous section is applicable mainly to seismic design of new structures constructed in Canada. Upgrading an existing structure according to the full requirement of the new seismic design provisions can be demanding. As seismic screening and evaluation guidelines designed to accompany the new seismic design procedures are not yet published, it is thought that retrofitting the warehouse structure according to the 1995 NBCC seismic design code is appropriate. However, it is possible to estimate the impact of the new code on the retrofitted warehouse and other similar single-storey structures. Figure 4-23 illustrates the seismic design force level computed based on the current and the proposed seismic provisions for a ductile shear wall structure in Montréal with varying seismic importance. In recognizing that the soil type at the warehouse site will fall between the category of a class C site and a class D site, the design base shear coefficients for both soil types are generated for comparison.

In computing the structural period of the retrofitted warehouse using the equation for shear wall structures in Table 4-4, a value of 0.13 seconds is obtained and its corresponding design force according to the proposed 2005 NBCC is $0.185 \cdot W$ (class C site and I_E=1.5). As the proposed code allows a one-third reduction in the seismic design force for low-period structures with substantial ductility to account for the significant change in the design force level for low-rise structure with respect to the current seismic design code *(Heidebrecht, 2003)*, the design base shear may be reduced to $0.124 \cdot W$ (line 2 in Figure 4-23). If the soil type at the warehouse structure is analyzed to be a class D site, the design force will be significantly raised to $0.168 \cdot W$ (line 1 in Figure 4-23), which is nearly a 56% increase in the design force level with respect to the force level that the retrofitted structure is designed for (i.e., $0.108 \cdot W$). However, if the retrofitted warehouse is considered as a structure of normal importance, the retrofitted warehouse will have sufficient resistance to sustain the new design base shear force computed based on a class C soil condition (line 4 in Figure 4-23), and may be marginally unacceptable for a class D site (line 3 in Figure 4-23).

In order to access the torsion flexibility of the retrofitted structure, a torsion sensitivity ratio, $B_{x,}$ is calculated based on the deflection values given in Figure 4-21. Calculations show that the ratios for the y and x direction earthquakes are 1.15 and 1.06 respectively. These ratios indicate that the retrofitted warehouse is insensitive to torsional excitation and the use of the static procedure given by the proposed new code is acceptable. An ETABS computer model of the retrofitted structure is analyzed using the proposed static procedures of 2005 NBCC. The design base shear is taken as $0.124 \cdot W$ and a 10% accidental torsional moment on the structure is included in the analysis. The analysis has shown that the design forces in critical shear walls and the inelastic lateral drifts of the retrofitted structure are increased by approximately 15% relative to those calculated according to 1995 NBCC. Thus, the retrofitted structure is considered acceptable in terms of its lateral stiffness with respect to the proposed provisions; however, a minor increase of lateral strength will be needed for the warehouse to fully satisfy the proposed seismic design code. It should also be noted that with the penthouse seismic weight being less than 10% of the main structure, the proposed seismic design provisions permits the lateral load resisting system of the penthouse to be designed with a force modification factor different from that used for the main structure.

As the proposed seismic code has yet to be finalized, further amendments to the draft document may take place upon the review by relevant parties. In a brief discussion and application of the proposed 2005 NBCC seismic code, the retrofitted warehouse, which is considered generally safe under the provisions of the current 1995 NBCC seismic code, may require additional strengthening to fully satisfy the stringent requirements of the proposed 2005 NBCC seismic design provisions. However, such need of extra strengthening should be considered after the new building code and its associated seismic evaluation manual are finalized.

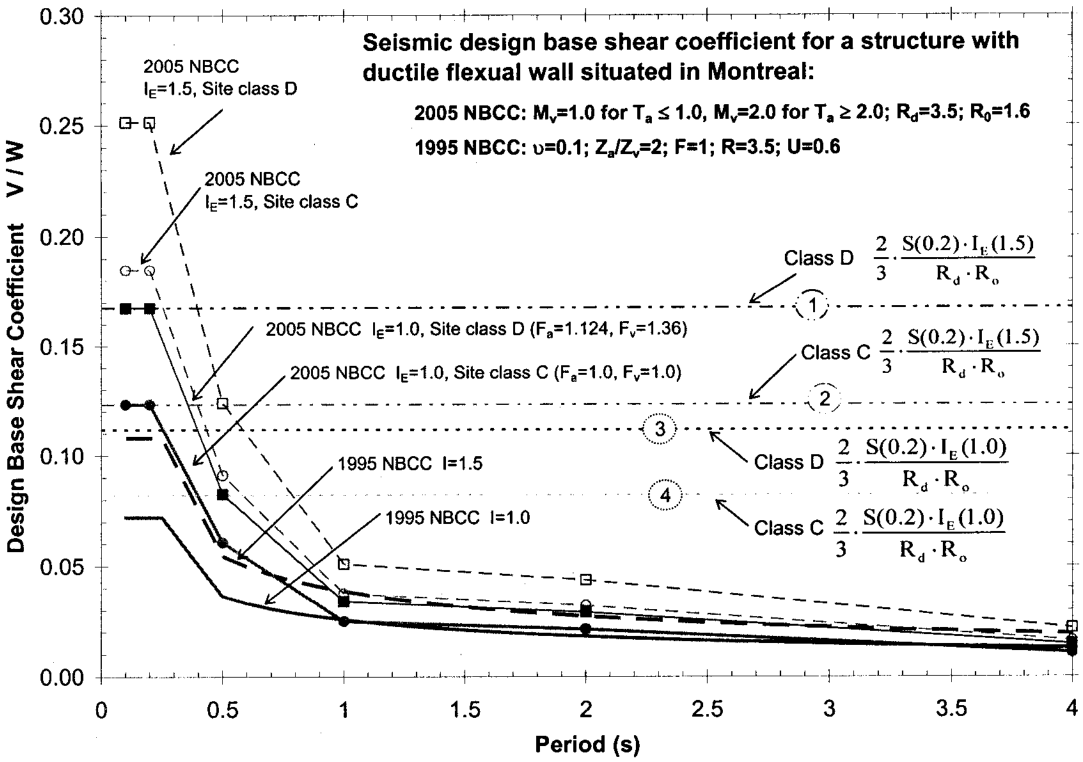


Figure 4-23: Comparisons of base shear coefficients computed according to the proposed 2005 NBCC and the current 1995 NBCC for a normal and a post-disaster structure located in Montréal.

Chapter 5: Conclusions

The present seismic evaluation project raises concerns for older precast concrete structures constructed in the 1960s. This issue is particularly crucial for post-disaster buildings and for other important structures such as historical structures. Building owners are encouraged to allow their building to be inspected and evaluated for potential seismic rehabilitation. It is expected that as the public and building owners gradually gain awareness of the importance of seismic mitigation and consequences of inaction, seismic screening, evaluation and retrofitting will become an essential activity to structural engineers. Therefore, it is imperative for engineers to avoid over-conservative analysis of existing structures and to come up with practical and less disruptive, as well as low cost retrofitting schemes to make seismic retrofitting a feasible option for structures with inadequate seismic resistance.

The choice of seismic retrofitting techniques varies from one building to another depending on the existing structural system and configuration. For a single-storey precast concrete frame structure, the present study has demonstrated a practical seismic evaluation and rehabilitation procedure. The old warehouse structure, which is identified to be a post-disaster building, is constructed without proper seismic resistant design and has a significant lack of structural redundancy. A visual inspection of the structure has reported obvious signs of distress in the building members and it was concluded that it is seismically vulnerable. An analytical assessment of the structure based on the 60% criteria for existing structure specified by 1995 NBCC and the seismic design guidelines of 1965 NBCC has further confirmed the seismic inadequacy of the structure with the column forces greatly exceeding the available resistances. The analysis has also shown that the partially infilled masonry walls are detrimental to the perimeter precast framing. In deciding to rehabilitate the seismic deficient warehouse structure for the full earthquake design load in 1995 NBCC, a conceptual seismic retrofit scheme for the structure was developed including: four reinforced concrete shear walls to be added just inside the main structure; tension-only steel braces to be installed at the penthouse; and the existing roof diaphragm to be upgraded to enable the formation of a rigid diaphragm. The rehabilitated structure has an improved structural redundancy and has gained substantial lateral strength and stiffness to resist a design earthquake. In addition, the impact of the proposed seismic design provisions of 2005 NBCC on the retrofitted structure is also discussed. It is concluded that since the proposed 2005 NBCC design load level is higher than the current standard, and a 15% increase in lateral strength of the retrofitted structure is required in order for it to fully satisfy the proposed standard.

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Appendix A: Determination of NRC Seismic Screening Index

This section summarizes the method of determining the NRC seismic screening index, or known as Seismic Priority Index (SPI), described in Figure 1-9 of Chapter 1. The calculation of priority index and the accompanying screening system were developed by the Institute for Research in Construction of the National Research Council in 1993 *(NRC, 1993b)* based on the 1990 NBCC. The SPI index, which represents the seismic risk and the urgency of seismic evaluation of a building constructed in Canada, is computed as shown in Figure A1. The calculation of the index involves the addition of a structural index, which is the multiplication of selected parameters A, B, C, D, and E; and a non-structural index which is the multiplication of parameters B, E, and F. The values of each parameter are determined based on the building location, existing conditions, usage and the year of construction. The tabulation of the six parameters that are used in the determination of the SPI are reproduced from Foo *et al. (2001)* and are shown in Table A1 through Table A7.

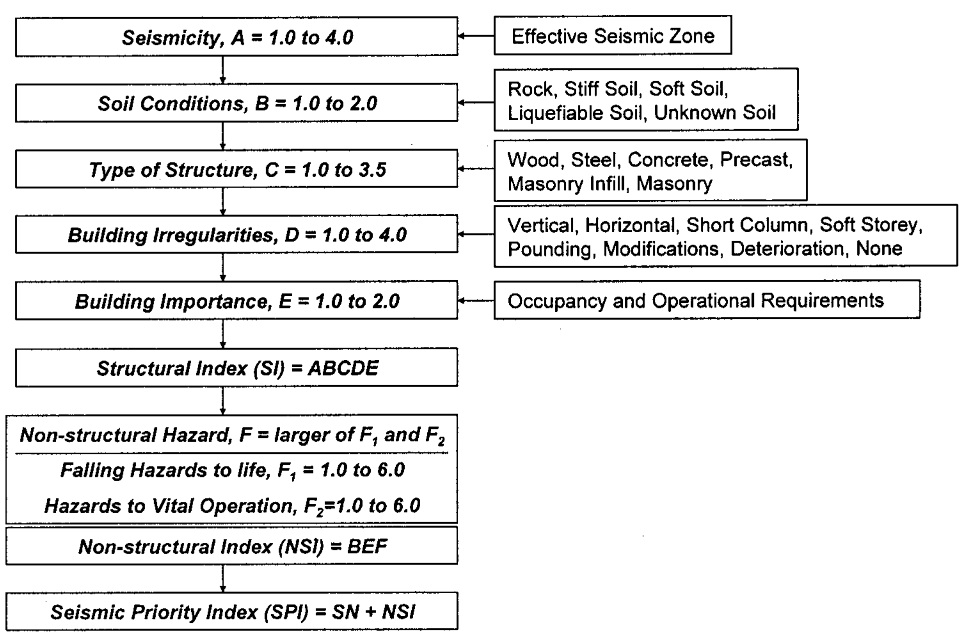

Figure A1: Determination of the NRC SPI index *(Foo et al., 2001).*

(I)

		Design NBC	Effective Seismic Zone (Z_a or Z_v+1 if Z_a>Z_v)				
			2	3	4	5	6
A	Seismicity	Pre - 65	1.0	1.5	2.0	3.0	4.0
		65 - 85	1.0	1.0	1.3	1.5	2.0
		Post 85	1.0	1.0	1.0	1.0	1.0

Table A1: Determination of seismicity parameter A based on the peak ground acceleration and velocity zone factors defined in the current 1995 NBCC *(Foo et al., 2001).*

(II)

		Design NBC	SoiL Category				
B	Soil Conditions		Rock or Stiff Soil	Stiff Soil >50m	Soft Soil >15m	Very Soft or Liquifiable Soil	Unknown Soil
		Pre - 65	1.0	1.3	1.5	2.0	1.5
		Post - 65	1.0	1.0	1.0	1.5	1.5

Table A2: Determination of the soil condition parameter B *(Foo et al., 2001).*

(III)

		Design NBC	Construction Type and Symbol												
			Wood		Steel				Concrete		Precast		Masonry Infill	Masonry	
C	Structure Type		WLF	WPB	SLF	SMF	SBF	SCW	CMF	CSW	PCF	PCW	SIW / CIW	RML / RMC	URM
		Pre - 70	1.2	2.0	1.0	1.2	1.5	2.0	2.5	2.0	2.5	2.0	3.0	2.5	3.5
		70 - 90	1.2	2.0	1.0	1.2	1.5	1.5	1.5	1.5	1.8	1.5	2.0	1.5	3.5
		Post - 90	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	-

WLF : Wood Light Frame
WPB : Wood, Post and Beam
SLF : Steel Light Frame
SMF : Steel Moment Frame
SBF : Steel Braced Frame
SCW : Steel frame with Concrete shear Walls
CMF : Concrete Moment Frame
CSW : Concrete Shear Walls
PCF : Precast Concrete Frame
PCW : Precast Concrete Walls
SIW : Steel Frame with Infilled masonry shear Walls
CIW : Concrete frame with Infill masonry shear Walls
RML : Reinforced Masonry bearing walls with wood or metal deck floors or roofs
RMC : Reinforced Masonry bearing walls with Concrete diaphragms
URM : UnReinforced Masonry bearing wall building

Table A3: Determination of the construction type parameter C *(Foo et al., 2001).*

(IV)

		Design NBC	Vertical	Horizontal	Short Concrete Columns	Soft Storey	Pounding	Modification	Deterioration	None
D	Building Irregularities	Pre - 70	1.3	1.5	1.5	2.0	1.3	1.3	1.3	1.0
		70 - 90	1.3	1.5	1.5	1.5	1.3	1.0	1.3	1.0

Table A4: Determination of the parameter D which reflects structural irregularities in the building; the parameter D is a product of all selected structural irregularity value and need not be greater than the maximum value of 4.0 *(Foo et al., 2001).*

(V)

		Design NBC	Low Occupancy	Normal Occupancy	School of High Occupancy	Post Disaster, Very High Occupancy	Special Operational Requirement
E	Building Importance		N < 10	N = 10 to 300	N = 301 to 3000	N > 3000	
		Pre - 70	0.7	1.0	1.5	2.0	3.0
		Post - 70	0.7	1.0	1.2	1.5	2.0

N = Occupied Area X Occupancy Density X Duration Factor*

Primary Use	Occupancy Density	Average Weekly Hours
Assembly	1	5 to 50
Mercantile, Personal service	0.2	50 to 80
Office, Institutional, Manufacturing	0.1	50 to 60
Residential	0.05	100
Storage	0.01 to 0.02	100

*Duration Factor = average weekly hours of human occupancy divided by 100, not greater than 1.0

Table A5: Determination of the parameter E which reflects the seismic importance of the structure *(Foo et al., 2001).*

(VI)

	Non-structural Hazards		Design NBC	None	yes	YES*	
F	F_1	Falling hazards to life	Pre - 70	1.0	3.0	6.0	F = max(F_1, F_2)
			Post - 70	1.0	2.0	3.0	
	F_2	Hazards to vital operation	Any Year	1.0	3.0	6.0	

Table A6: Determination of the parameter F which reflects the seismic hazards of the non-structural components; the column YES* applies when one or more of the following descriptors is selected: structural type of SMF, structural type of CMF, soft storey, and horizontal irregularities *(Foo et al., 2001).*

The selected values of each SPI parameter are combined as shown in Figure A1. Structures that are subjected to the seismic screening are ranked based on their SPI, such that structures with a high risk score (i.e., a high SPI value) is prioritized for a detailed seismic evaluation. As an example, the SPI for the single-storey precast concrete warehouse structure investigated in the report may be calculated as shown in Table A8.

SPI Value	Evaluation Priority
SPI ≥ 10	Low
10 < SPI ≤ 20	Medium
SPI > 20	High

Table A7: Prioritizing the seismic evaluation of structures based on the computed SPI.

Subject: Single-storey precast concrete structure with masonry-infilled walls constructed in 1963		
Parameter	Description	Value
A	Pre 65, Montreal Z_a=4, Z_v=2, Z_v+1 =3	1.50
B	Pre 65, Stiff Soil	1.00
C	Pre 70, Concrete building with masonry-infilled walls	3.00
D	Pre 70, Short columns, and deterioration	1.95
E	Pre 70, Post-disaster	2.00
F	Hazard to vital operation	6.00
SPI = ABCDE + BEF = 29.55 (> 20, High)		

Table A8: Calculation of SPI index for the single-storey precast concrete structure investigated in the report.

Appendix B: Design of Seismic Upgrading of the Precast Structure

The design of seismic upgrading follows Clause 21 of CSA A23.3-94 *(1994)* and Clause 27 of CSA Standard S16.1-01 *(2001)*. To facilitate the calculations under the Canadian standards, S.I. units are used as the primary unit system and the equivalent values in Imperial units will be provided in parenthesis.

B.1 Design of Shear Walls

Shear walls are designed according to Section 21.5 of the A23.3-94 Standard with a force reduction factor of R=3.5. The design shear forces and moments obtained from ETABS are shown in Table B1. The shear walls aligned in the N-S and E-W directions are designed for the critical loadings occurring in SHW1 and SHW3, respectively.

Shear Walls	E-W EQ				N-S EQ.			
	Shear Force		Base Moment		Shear Force		Base Moment	
	(kips)	(kN)	(kips ft)	(kNm)	(kips)	(kN)	(kips ft)	(kNm)
SHW1	67	298	848	1150	885	3937	11149	15116
SHW2	67	298	848	1150	652	2900	8213	11135
SHW3	807	3590	10172	13791	68	302	848	1150
SHW4	730	3247	9193	12464	68	302	848	1150

Table B1: Shear and moments reactions on the shear walls (R=3.5).

B.1.1 Dimensions and Stability Requirements

The dimensions of each shear wall is 6500 mm long by 300 mm thick (26.2 ft by 11.8 in.) and 4 m (13 ft) high. The minimum wall thickness, b_w, is computed according to Clause 21.5.3.2:

$$b_w > \frac{l_u}{10} = \frac{4000}{10} = 400 \text{ mm } (15.7 \text{ in}) \qquad \text{(A23.3 – Cl. 21.5.3)}$$

Since the selected wall thickness is less than the dimension limitation, the flexural compression depth of the wall, c_c, will be checked with Clause 21.5.3.2 later in the design to ensure stability of the wall.

B.1.2 Concentrated Reinforcement

The reinforcing details in the concentrated reinforcement region are shown in Figure B1. The concentrated reinforcement at each wall end consists of eight No. 25 longitudinal bars and the bar size is within one-tenth of the wall thickness as specified by Clause 21.5.4.4. The longitudinal bars are clearly spaced at 100 mm (4 in), that is within the 150 mm (6 in.) limitation stated in Clause 7.6.5.5. The clear spacing between the two bars at the wall end is 170 mm (6.7in.) and it is also within the 500 mm (20 in.) limit of Clause 7.4.1.3. The longitudinal bars are confined by two No. 10 column ties with seismic hooks at their ends (i.e., a minimum 135° bend angle and a bar extension of 6 d_b). The tie spacing within the concentrated region is determined according to Clause 21.5.6.5:

1. $6 \cdot d_b$ (150 mm)
2. 24 tie diameter (240mm)
3. One-half of wall thickness (175mm) (A23.3 – Cl. 21.5.6.5)

$\therefore$ Minimum tie spacing = 150 mm (6 in.)

The minimum concentrated reinforcement ratio at each end of the wall is checked with Clause 21.5.6.4:

$$A_s \geq 0.002 \cdot l_u \cdot b_w = 0.002 \times 300 \times 6500$$
$$\geq 3900 \text{ mm}^2 \ (6.0 \text{ in}^2) \qquad \text{(A23.3 – Cl. 21.5.6.4)}$$

A_s provided $= 8 \times 500 = 4000$ mm^2 (6.2 in.^{2}) $\therefore$ ok.

The maximum concentrated reinforcement ratio at each end of the wall is checked with Clause 21.5.4.3:

$$\rho = \frac{A_s}{A_g} = \frac{8 \times 500}{480 \times 300} = 0.028 \ (< 0.06) \ \therefore \text{ok.} \qquad \text{(A23.3 – Cl. 21.5.4.3)}$$

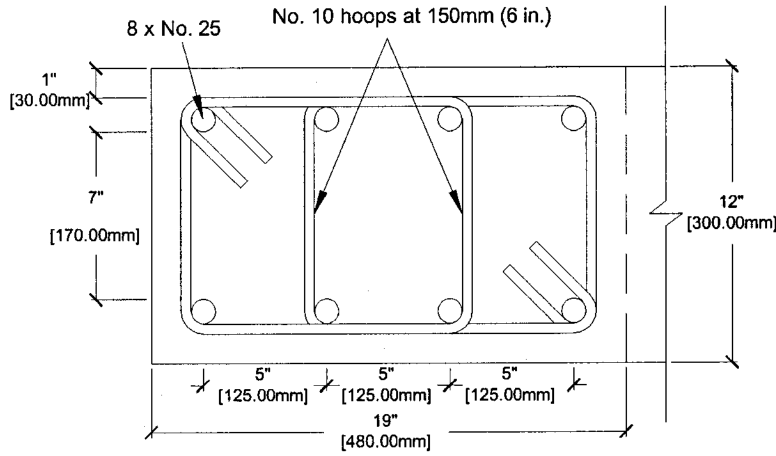

Figure B1: Details of concentrated reinforcement in the shear walls.

B.1.3 Moment Resistance and Distributed Vertical Reinforcement

Two curtains of No. 10 vertical reinforcement are provided in each wall and the amount of vertical wall reinforcement in both shear walls SHW1 and SHW3 are shown in Table B2. The factored moment resistance, M_r, of each reinforced wall section is computed using RESPONSE-2000 and they are compared with the design moments computed using ETABS in Table B3. The moment resistances of both walls are found to be adequate.

Shear Walls	Spacing of Long. Reinforcemnt (2 curtains of No.10 bars)			Clause 21.5.5.1 ρ	
	layer	(in.)	(mm)	Provided	Min Req.
SHW1	34	6	165	0.0040	0.0025
SHW3	27	8	210	0.0032	

Table B2: Longitudinal reinforcement required in the shear walls.

Shear Walls	RESPONSE-2000 M_r		ETABS M_f		$\gamma_w = M_n/M_f$
	(kips ft)	(kNm)	(kips ft)	(kNm)	
SHW1	11185	15165	11149	15116	1.55
SHW3	10223	13860	10172	13791	1.58

Table B3: Design moments and factored moment resistances of the walls.

The distributed vertical reinforcement ratio of each wall, ρ, is checked according to Clause 21.5.5.1. For shear wall SHW1:

$$\rho = \frac{A_s}{s \cdot b_w} = \frac{2\times100}{165\times300} = 0.0040 \; > 0.0025 \; \therefore \text{ok.} \qquad \text{(A23.3 – Cl. 21.5.5.1)}$$

Similarly, for shear wall SHW3:

$$\rho = \frac{A_s}{s \cdot b_w} = \frac{2\times100}{210\times300} = 0.0032 \; > 0.0025 \; \therefore \text{ok.} \qquad \text{(A23.3 – Cl. 21.5.5.1)}$$

To ensure ductility in the plastic hinge region, Clause 21.5.7 requires the compression depth, c_c, of each wall shall be less than $0.55l_w$:

$$c_c \le 0.55 \cdot l_w = 0.55\times6500 = 3575 \text{ mm (141 in.)} \qquad \text{(A23.3 – Cl. 21.5.7)}$$

Clause 21.5.7 also demands that if compression depth c_c is over $0.14{\cdot}l_w{\cdot}\gamma_w$, extra confinement criteria shall be applied over the length of c_c. In conservatively using a low overstrength value, γ_w, of 1.2 for this check:

$$0.14 \cdot \gamma_w l_w = 0.14\times1.2\times6500 = 1092 \text{ mm (45 in.)} \qquad \text{(A23.3 – Cl. 21.5.7)}$$

Furthermore, the lesser conditions imposed by Clause 21.5.3.2 should also be checked for a wall thinner than $l_u/10$:

$$\text{(a)} \quad 4 \cdot b_w = 4\times300 = 1200 \text{ mm (47 in.)}$$
$$\text{(b)} \quad 0.3 \cdot l_w = 0.3\times6500 = 1950\text{mm (77 in.)} \qquad \text{(A23.3 – Cl. 21.5.3.4)}$$
$$\therefore c_c \le 4 \cdot b_w = 1200 \text{ mm (47 in.)}$$

The compression depths, c_c, obtained from RESPONSE-2000 are checked with the above conditions in Table B4 and they are well within all the limitations imposed by the clauses.

Shear Walls	Compression Depth c_c		Clause 21.5.7 $0.55l_w$		Clause 21.5.7 $0.14l_w\gamma_w$		Clause 21.5.3.4 $4b_w$	
	(in.)	(mm)	(in.)	(mm)	(in.)	(mm)	(in.)	(mm)
SHW1	21	539	141	3575	43	1092	47	1200
SHW3	23	575						

Table B4: Various checks on compression depths of the two shear walls.

B.1.4 Shear Resistance and Distributed Horizontal Reinforcement

The shear strength of each wall is designed to resist the probable shear force, V_p', associated with the development of the probable moment resistance (Clause 21.7.2.3). The probable moment resistances of both walls are taken as 1.47 times the factored moment resistances given in Table B3. The probable design shear forces acting on the shear walls are computed as shown in Table B5.

Shear Walls	M_p=1.47 M_r		M_p/M_f	ETABS V_f		V_p'	
	(kips ft)	(kNm)		(kips)	(kN)	(kips)	(kN)
SHW1	16442	22293	1.47	885	3937	1305	5806
SHW3	15027	20374	1.48	807	3590	1192	5303

Table B5: Design shear forces associate with the development of probable moment resistances

The shear reinforcement of the walls is provided in the form of two curtains of horizontal No.15 bars and they are designed according to the shear design guidelines in Clause 11.4.3. Clause 21.7.3.2 states that the inclination angle of diagonal compressive stress, θ, shall be taken as 45°. The strength contribution from concrete, V_c, is conservatively ignored. The shear depth of each wall is assumed to be 80 % of the wall length. With a horizontal bar spacing of 120 mm (5.5 in.) the shear resistance, V_{rg}, of shear wall SHW1 is:

$$\begin{aligned} V_{rg} &= V_c + \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v \cdot \cot\theta}{s} \\ &= \frac{0.85\times(2\times 200)\times 400\times(0.8\times 6500)\times\cot 45^{\circ}}{120}\times 10^{-3} \qquad \text{(A23.3 – Cl. 11.4.3)} \\ &= 5893\ \text{kN (1325 kips.)} > V_p' \ \therefore \text{ok.} \end{aligned}$$

In accordance with clause 11.4.3, the shear resistance of the wall cannot exceed:

$$\begin{aligned} 0.25\cdot\phi_c\cdot f_c'\cdot b_w\cdot d_v &= \frac{0.25\times 0.6\times 35\times 300\times(0.8\times 6500)}{1000} \\ &= 8190\,\text{kN (1841 kips)} \qquad \text{(A23.3 – Cl. 11.4.3)} \end{aligned}$$

$$V_{rg}\text{=5893 kN (1325 kips)} < 0.25\cdot\phi_c\cdot f_c'\cdot b_w\cdot d_v \ \therefore \text{ok.}$$

The distributed horizontal reinforcing ratio of SHW1 is checked according to Clause 21.5.5.1:

$$\rho = \frac{A_s}{s \cdot b_w} = \frac{2 \times 200}{120 \times 300} = 0.0111 > 0.0025 \;\therefore \text{ok.} \qquad \text{(A23.3 – Cl. 21.5.5.1)}$$

Similarly, shear wall SHW3 is reinforced with 2 curtains of No.15 horizontal bars spaced at 130 mm (5 in.). The factored shear resistance of SHW3 is checked:

$$\begin{aligned} V_{rg} &= V_c + \frac{\phi_s \cdot A_v \cdot f_y \cdot d_v \cdot \cot\theta}{s} \\ &= \frac{0.85 \times (2 \times 200) \times 400 \times (0.8 \times 6500) \times \cot 45^\circ}{130} \times 10^{-3} \qquad \text{(A23.3 – Cl. 11.4.3)} \\ &= 5440 \text{ kN (1223 kips.)} > V_p' \;\therefore \text{ok.} \end{aligned}$$

$$V_{rg} = 5440 \text{ kN (1223 kips)} < 0.25 \cdot \phi_c \cdot f_c' \cdot b_w \cdot d_v \;\therefore \text{ok.}$$

The distributed horizontal reinforcing ratio of SHW3 is also checked according to Clause 21.5.5.1:

$$\rho = \frac{A_s}{s \cdot b_w} = \frac{2 \times 200}{130 \times 300} = 0.0102 > 0.0025 \;\therefore \text{ok.} \qquad \text{(A23.3 – Cl. 21.5.5.1)}$$

The results of shear resistance calculations of both walls are summarized in Table B6.

Shear Walls	Spacing of Hori. Reinforcemnt (2 curtains of No.15 bars)			Clause 21.5.5.1 ρ		V_{rg}	
	layer	(in.)	(mm)	Provided	Min Req.	(kips)	(kN)
SHW1	33	4.7	120	0.0111	0.0025	1325	5893
SHW3	31	5.1	130	0.0103		1223	5440

Table B6: Horizontal reinforcement and shear resistance of shear walls.

B.1.5 Design of Dowel Connections

The dowel connections between the shear wall and the roof diaphragm are designed to resist the probable shear force V_p' developed within each shear wall. The dowel connections between diaphragm topping and the shear walls are designed according to shear-friction design method in Clause 11.6.2. For shear wall SHW1, a total of 73 No. 20 dowels (a total area of 21900 mm^2 or 34 in^2) are distributed over the 6.5 m long (21.3 ft) wall. Appropriate parameter values are substituted into the shear-friction design equations given in Clause 11.6.1 where: permanent load N acting perpendicular to the shear plane is taken as zero, $\lambda = 1$ for normal density concrete; $\mu = 1$, $c = 0.5$ MPa for concrete placed against hardened concrete with the surface clean and intentionally roughened to a full amplitude of at least 5 mm; $\alpha_f = 90°$ for dowels aligned perpendicular to shear plane; A_{cv} is the cross sectional area of the wall; $\phi_c = 0.6$ and $\phi_s = 0.85$. The corresponding shear strength of the dowel connections, V_r, is computed to be:

$$\begin{aligned} V_r &= [\lambda \cdot \phi_c \cdot (c + \mu \cdot \sigma)] \cdot A_{cv} \\ &= (0.3 \times 300 \times 6500 + 0.6 \times 73 \times 300 \times 400) \times 10^{-3} \qquad \text{(A23.3 – Cl. 11.6.1)} \\ &= 5841 \text{ kN (1313 kips)} > V_p' \end{aligned}$$

However, the shear-friction strength, v_r, need not exceed the limits imposed by Clause 11.6.1:

$$\begin{aligned} &0.25 \cdot \phi_c \cdot f_c' = 5.25 \text{ MPa} \\ &\qquad \text{and} \qquad\qquad\qquad \text{(A23.3 – Cl. 11.6.1)} \\ &7 \cdot \phi_c = 4.2 \text{ MPa} \end{aligned}$$

$$\begin{aligned} v_r &= [\lambda \cdot \phi_c \cdot (c + \mu \cdot \sigma)] \\ &= 0.3 + 0.6 \times \frac{21900}{300 \times 6500} \times 400 \\ &= 3.00 \text{ MPa} < 0.25 \cdot \phi_c \cdot f_c' \text{ and } 7 \cdot \phi_c \therefore \text{ok.} \end{aligned}$$

For shear wall SHW3, a sufficient amount of dowels are allocated to resist the design shear force and it is assumed that the shear keys formed by the interlocking double-tee stems will provide the remaining resistance required to resist the probable shear force. A total of 48 dowels (a total area of 14400 mm^2 or 22.3 in^2) are selected. The calculated shear resistance of the dowel connections is:

$$\begin{aligned} V_r &= [\lambda \cdot \phi_c \cdot (c + \mu \cdot \sigma)] \cdot A_{cv} \\ &= (0.3 \times 300 \times 6500 + 0.6 \times 48 \times 300 \times 400) \times 10^{-3} \qquad \text{(A23.3 – Cl. 11.6.1)} \\ &= 4041 \text{ kN (908 kips)} > V_f \end{aligned}$$

The shear-friction strength is checked according to Clause 11.6.1:

$$
\begin{aligned}
v_r &= \left[\lambda \cdot \phi_c \cdot (c + \mu \cdot \sigma)\right] \\
&= 0.3 + 0.6 \times \frac{14400}{300 \times 6500} \times 400 \qquad \text{(A23.3 – Cl.11.6.1)} \\
&= 2.07 \text{ MPa } < 0.25 \cdot \phi_c \cdot f_c^{'} \text{ and } 7 \cdot \phi_c \;\; \therefore \text{ok.}
\end{aligned}
$$

Based on the above calculations, the dowel connections provided in SHW3 will resist 76 % of the probable shear force arises from flexural hinging of the wall. A summary of the dowel connection design of both walls is provided in Table B7.

Shear Walls	No. of Dowels in Each Wall	V_r		V_p'	
		(kips)	(kN)	(kips)	(kN)
SHW1	73 No. 20	1313	5841	1305	5806
SHW3	48 No. 20	908 *	4041*	1192	5303

* The deficit shall be carried by the "shear keys"

Table B7: Resistances of the dowel connections between the diaphragm and the shear walls.

B.2 Design of Rigid Diaphragm

A 50 mm thick (2 in.) cast-in-place concrete topping with welded wire fabric (WWF) reinforcement is constructed on the existing roof. The WWF are sized based on their strengths available to: (1) resist the probable shear forces from the shear walls; (2) assist the drag bars in transferring inertial forces to the walls; and (3) provide the additional flexural strength needed by the diaphragm.

B.2.1 Shear Resistance of Diaphragm

The WWF in the E-W direction is designed to resist the probable shear force V_p' (5806 kN or 1305 kips) of SHW1 (Figure B2) using a shear-friction model given in Hawkins *et al. (2000)*. The amount of steel area, A_{vf}, needed across the diaphragm width (73.51 m or 241 ft) is:

$$
\begin{aligned}
A_{vf} / \text{unit length} &\geq \frac{V_p^{'}}{\phi_s \cdot f_y \cdot \mu \cdot D_d} = \frac{5806}{(0.85 \times 400 \times 1.0 \times 10^{-3}) \times 73.51} \\
&\geq 232 \text{ mm}^2/\text{m } (0.11 \text{ in}^2/\text{ft})
\end{aligned}
$$

The WWF in N-S direction is designed to resist the design shear force V_p' of SHW3 (5303 kN or 1192 kips). The amount of steel area needed across the diaphragm length (122.28 m or 401 ft) is:

$$A_{vf} \text{ / unit length} \geq \frac{V_p'}{\phi_s \cdot f_y \cdot \mu \cdot D_d} = \frac{5303}{(0.85 \times 400 \times 1.0 \times 10^{-3}) \times 122.28}$$
$$\geq 128 \text{ mm}^2 \text{ / m } (0.06 \text{ in}^2 \text{ / ft})$$

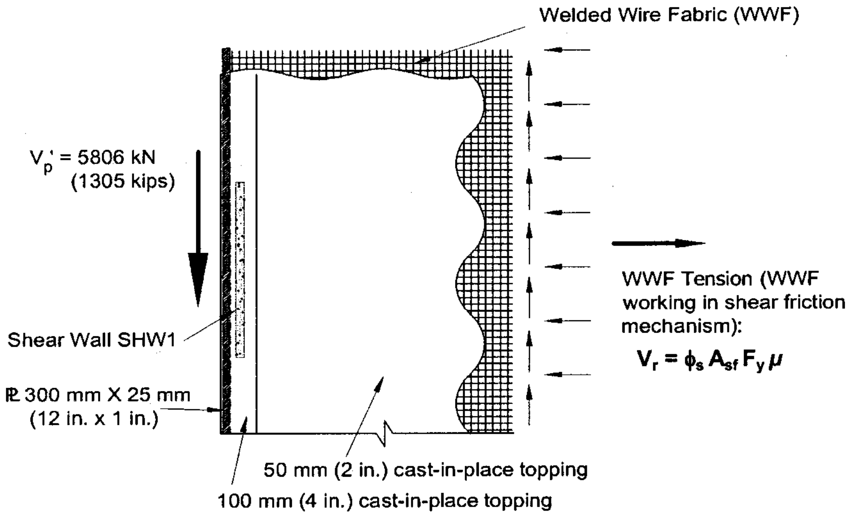

Figure B2: Shear-friction model for design of topping reinforcement.

B.2.2 Inertial load transfer

Drag-struts at the diaphragm perimeter are provided in the form of 300 mm wide by 25 mm thick (12 in. by 1 in.) Grade W300 steel plates. The drag bar has an area of 7500 mm^2 (12 in.^{2}) and its factored tensile resistance, T_r, is:

$$T_r = \phi_s \cdot A_s \cdot f_y = 0.9 \times 7500 \times 300 \times 10^{-3}$$
$$= 2025 \text{ kN (455 kips)}$$

Since the tensile resistance of the drag bar is insufficient, it is decided to engage the WWF to assist the transfer of inertial load to the shear walls (Figure B3). In the N-S direction, residual force, T_p', to be carried by WWF (Figure B4):

$$T_p' = 0.52 \cdot V_p' - T_r = 0.52 \times 5806 - 2050$$
$$= 969 \text{ kN (218 kips)}$$

The minimum amount of steel area, A_s, required for inertial load collection in the N-S direction is:

$$T_p' \le T_r = \phi_s \cdot A_s \cdot f_y$$

$$A_s \,/\, \text{unit length} \ge \frac{T_p'}{\phi_s \cdot f_y \cdot \frac{D_n}{2}} = \frac{969}{(0.85 \times 400 \times 10^{-3}) \times \frac{122.28}{2}}$$

$$\ge 47\ \text{mm}^2/\text{m}\ (0.02\ \text{in}^2/\text{ft})$$

In the E-W direction, the residual force resisted by WWF (Figure B5) is:

$$T_p' = 0.6 \cdot V_p' - T_r = 0.6 \times 5303 - 2050$$

$$= 1132\ \text{kN}\ (254\ \text{kips})$$

The minimum amount of WWF needed in the E-W direction for inertial load collection is:

$$A_s \,/\, \text{unit length} \ge \frac{T_p'}{\phi_s \cdot f_y \cdot \frac{D_n}{2}} = \frac{1132}{(0.85 \times 400 \times 10^{-3}) \times \frac{73.51}{2}}$$

$$\ge 91\ \text{mm}^2/\text{m}\ (0.043\ \text{in}^2/\text{ft})$$

The amount of WWF per unit length needed for shear-friction reinforcement and inertial load transfer are compared in Table B8. Comparisons show that shear-friction reinforcement governs the design of the WWF in the roof diaphragm.

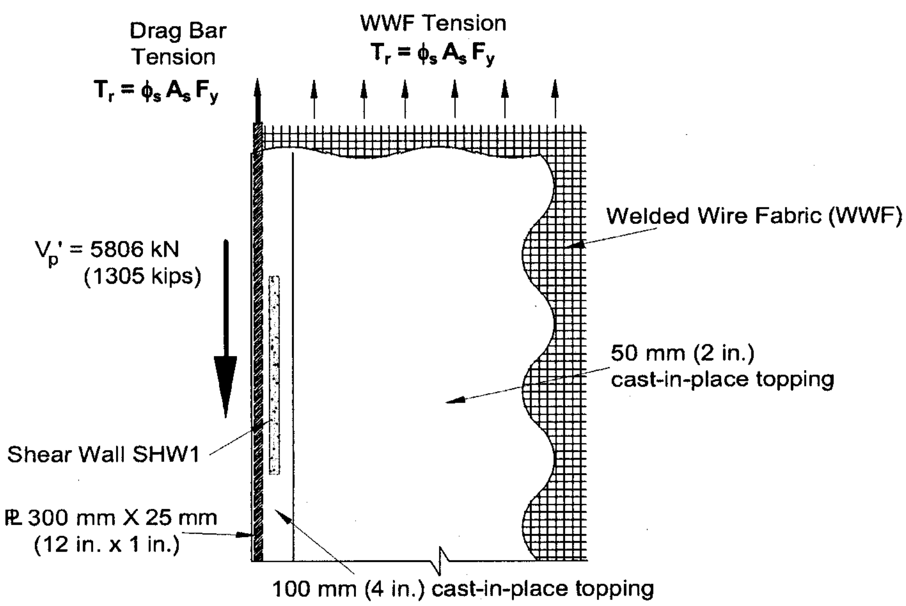

Figure B3: Welded wire fabric assisting the drag bar to transfer diaphragm inertial load.

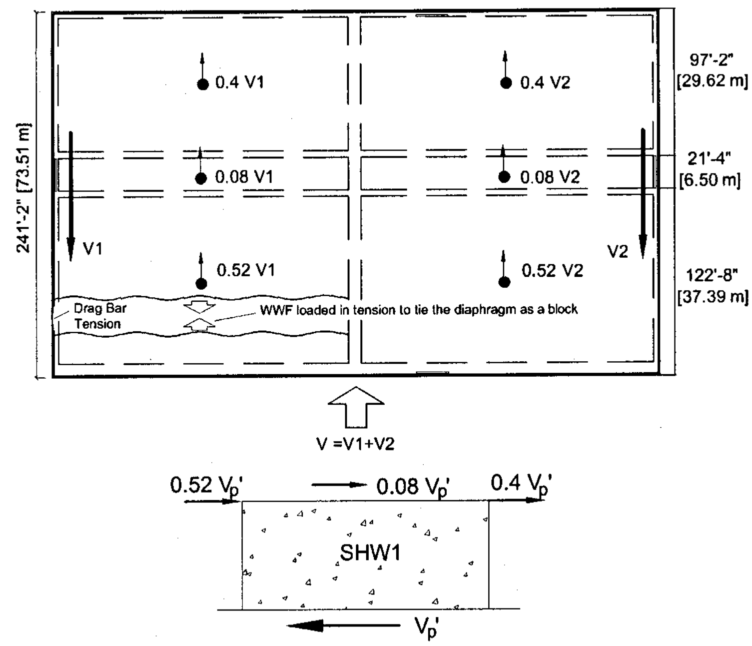

Figure B4: Distribution of earthquake inertial force in the N-S direction.

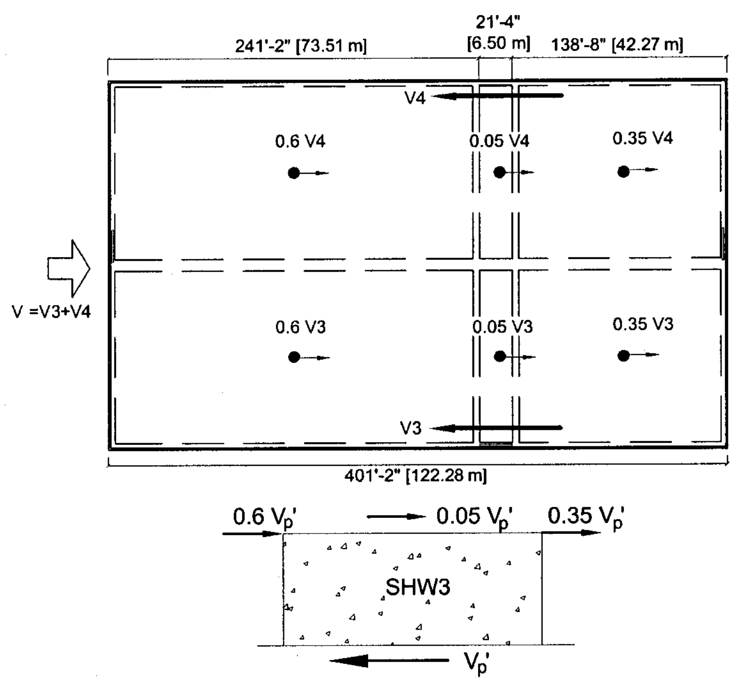

Figure B5: Distribution of earthquake inertial force in the E-W direction.

Welded Wire Fabric (WWF)	Shear-friction		Inertial Load Transfer	
	(in.^{2}/ft)	(mm^2/m)	(in.^{2}/ft)	(mm^2/m)
NS Direction	0.06	128	0.02	47
EW Direction	0.11	232	0.04	91

Table B8: Comparisons of welded wire fabric needed in the two orthogonal directions.

B.2.3 Moment Resistance

Using the deep beam analogy, the probable forces internal to the roof diaphragm in the two orthogonal directions are shown in Figure B6. The coupling tensile forces acting within the chord reinforcement are resolved from the midspan moment and with the moment arm being the diaphragm length or width.

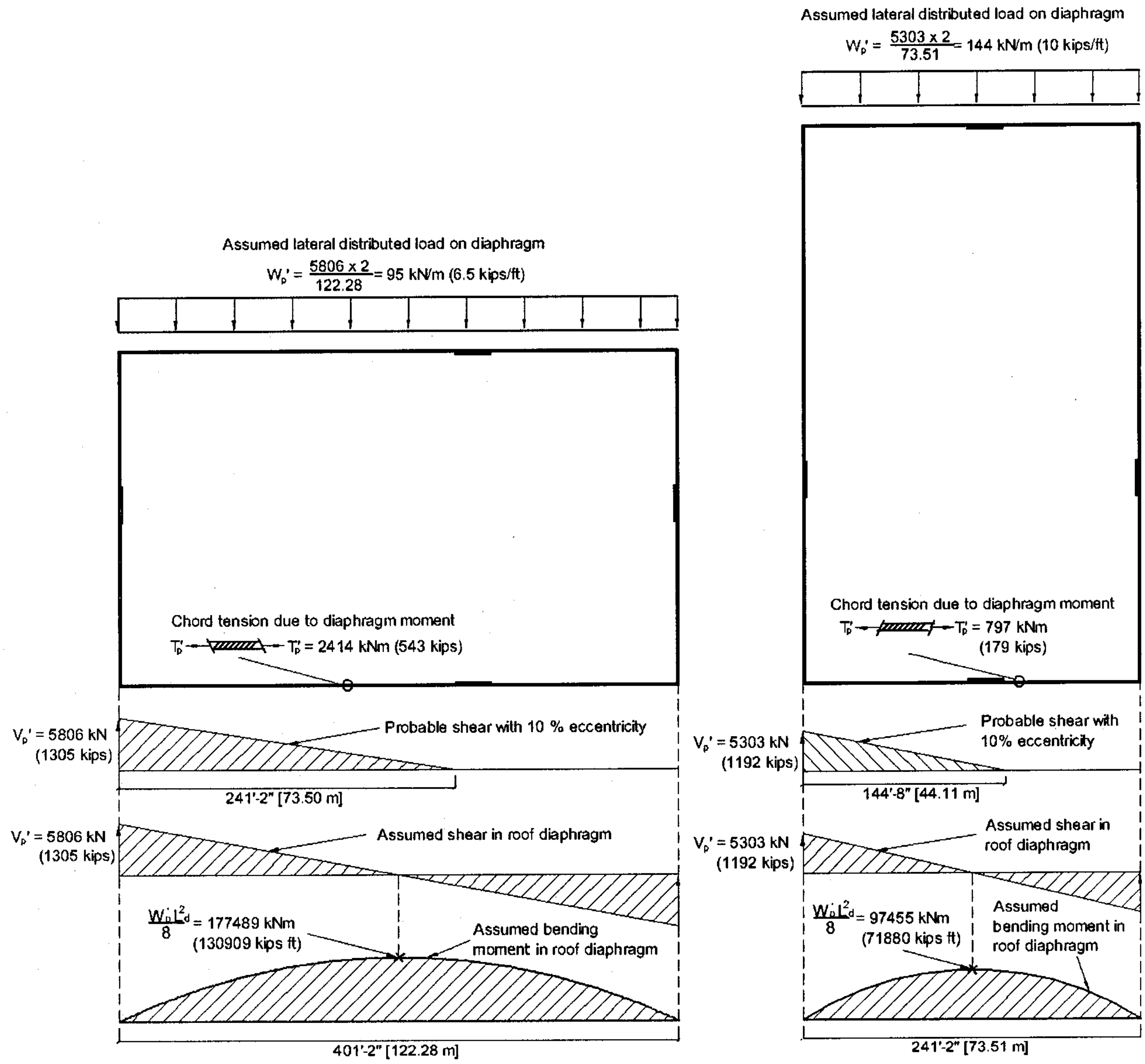

Figure B6: Assumed values of design moments to ensure the elastic behavior of the diaphragm.

Using the previously chosen drag bars as the primary chord reinforcement, the factored moment resistance, M_r, of the diaphragm in the E-W direction is:

$$M_r = \phi_s \cdot A_{chord} \cdot f_y \cdot D_d$$
$$= 0.9 \times 7500 \times 300 \times 122.28 \times 10^{-3}$$
$$= 247612 \text{ kNm (182635 kips ft)} > M_p^{'} \text{ in diaphragm } \therefore \text{ok}$$

The diaphragm chord flexural reinforcement located on the N-S edges is adequate. In the E-W direction:

$$M_r = \phi_s \cdot A_{chord} \cdot f_y \cdot D_d$$
$$= 0.9 \times 7500 \times 300 \times 73.51 \times 10^{-3}$$
$$= 148858 \text{ kNm (109793 kips ft)} < M_p^{'} \text{ in diaphragm } \therefore \text{not ok.}$$

Since the steel plates on the E-W edges are insufficient, it is decided to engage some of the E-W welded wire fabric as additional flexural reinforcement. However, as the maximum diaphragm moment occurs at midspan, where the shear force is minimal, the WWF designed based on the previous shear-friction requirements (Table B8) is adequate. The selected transverse and longitudinal sizes of welded wire fabric are summarized in Table B9.

Welded Wire Fabric	Designation (Wire Size / Spacing)		Area / Unit Length		Wire diameter	
	Metric	Imperial	($in.^2/ft$)	(mm^2/m)	in.	mm
Transverse (N-S)	MW13.3 / 102 mm	M2.1 / 4 in.	0.06	130	0.16	4.12
Longitudinal (E-W)	MW25.8 / 102 mm	M4.0 / 4 in.	0.12	253	0.23	5.74

Table B9: Welded wire fabric (WWF) needed in the diaphragm.

B.3 Design of Penthouse Retrofit

B.3.1 Design of Tension-only Concentric Diagonal Braces

Each penthouse bay is concentrically braced by a pair of double-angles built-up braces. These tension-only braces are designed according to Clause 27.6 of the S16.1-2001 standard with a force reduction factor of R=2. Each built-up brace is composed of two Grade W300 L76×64×6.4 angles and the built-up section has a gross area of 1690 mm^2 (2.6 in.^{2}). The factored tensile resistance, T_r, of the selected built-up brace member is:

$$\begin{aligned} T_r &= \phi_s \cdot A_s \cdot f_y = 0.9 \times 1690 \times 300 \times 10^{-3} \\ &= 456 \text{ kN (102 kips)} \quad > T_f \ \therefore \text{ok.} \end{aligned}$$

The factored tensile resistance of the brace member is adequate in comparison with the design forces obtained from the ETABS analyses (Table B10). The slenderness ratio of the tension-only braces should be less than 300 as stipulated by Clause 27.6.3. The weak axis slenderness ratio of each built-up brace member is calculated by taking the effective brace length, k·L, as 45% of the original length, L, to account for the lateral supports available at midpoint (0.5L) and at the gusset plate connections (0.05L) at the brace ends. The slenderness ratio of BR1 is:

$$\begin{aligned} \frac{k \cdot L}{r_x} &= \frac{1 \times (0.45 \times 12.6) \times 10^3}{24} \\ &= 236 \ < 300 \ \therefore \text{ok.} \end{aligned}$$

Similarly, the slenderness ratio of BR2 is:

$$\begin{aligned} \frac{k \cdot L}{r_x} &= \frac{1 \times (0.45 \times 15) \times 10^3}{24} \\ &= 282 \ < 300 \ \therefore \text{ok.} \end{aligned}$$

Since the braces are slender, there is no flat-width ratio requirement for the braces (Clause 27.6.3). Bridgings between the angles are provided in the form of 70 mm long by 40 mm wide by 8 mm thick (2.8 in. by 0.9 in. by 0.3 in.) steel plates welded to the top and bottom of the angles at a 500 mm (20 in.) spacing.

Braces	E-W Earthquake Axial Force		N-S Earthquake Axial Force	
	(kips)	(kN)	(kips)	(kN)
BR1	66	294	6	27
BR2	5	22	62	276
BR3	5	22	53	236
BR4	50	222	6	27

Table B10: Axial tension forces in the penthouse braces (R=2.0).

To ensure the braces are designed with sufficient lateral stiffness, each bracing bent is checked in accordance with the interstorey drift limit of $0.01 \cdot h_s$. In conservatively disregarding the stiffness available from the columns, the elastic force-displacement relationship of a laterally loaded tension-only braced frame is:

$$F = \frac{E \cdot A}{L} \cdot \cos^2 \theta \cdot \Delta_{elastic}$$

where E, A and L are the, elastic modulus, cross-sectional area and length of the brace. Using the geometry provided in Figure B7, the inelastic displacement of the BR1 bracing bent is estimated as:

$$\begin{aligned} R \cdot \Delta_{elastic} &= \frac{R \cdot T_f \cdot L}{E \cdot A \cdot \cos \theta} \\ &= \frac{2 \cdot 294 \cdot 12600}{200000 \cdot 1690 \cdot (\cos 14.6^\circ) \times 10^{-3}} \\ &= 22.6 \text{ mm } (0.46 \text{ in.}) < \text{Drift limit } 0.01 \times 3180 = 32 \text{ mm } \therefore \text{ok} \end{aligned}$$

A linear amplification factor, U_2, for the braces BR1 is calculated to account for P-Δ effects on the braced frame. The combined gravity load acting on each bracing bent is conservatively estimated to be 1600 kN (360 kips) The lateral force acting on a braced frame is resolved from the maximum factored tensile loads in Table B10. Thus, the amplification factor of BR1 is:

$$\begin{aligned} U_2 &= 1 + \left(\frac{\sum C_{1 \cdot D + 0.5 \cdot L} \cdot \Delta_{Inelastic}}{\sum V_f \cdot h_s} \right) \\ &= 1 + \left(\frac{(\frac{1600}{2}) \times 22.6}{(294 \times \cos 14.6^\circ) \times 3180} \right) \\ &= 1.02 < 1.4 \text{ Lateral load resisting system is adequate. } \therefore \text{ok.} \end{aligned}$$

The amplified tensile load is checked against the factored tensile resistance of BR1:

$$T_f \cdot U_2 = 294 \times 1.02$$
$$= 300 \text{ kN (12 kips)} < T_r \;\; \therefore \text{ok.}$$

Similarly, the inelastic displacement and the corresponding P-Δ amplification factors of the BR2 braced frame is checked as follows:

$$R \cdot \Delta_{elastic} = \frac{R \cdot T_f \cdot L}{E \cdot A \cdot \cos\theta}$$
$$= \frac{2 \cdot 276 \cdot 15000}{200000 \cdot 1690 \cdot (\cos 12^\circ) \times 10^{-3}}$$
$$= 25 \text{ mm (1 in.)} < \text{Drift limit} = 32 \text{ mm} \;\; \therefore \text{ok}$$

$$U_2 = 1 + \left(\frac{\sum C_{1 \cdot D + 0.5 \cdot L} \cdot \Delta_{Inelastic}}{\sum V_f \cdot h_s} \right)$$
$$= 1 + \left(\frac{(\frac{1600}{2}) \times 25}{(276 \times \cos 12) \times 3180} \right)$$
$$= 1.02 < 1.4 \text{ Lateral load resisting system is adequate. } \therefore \text{ok.}$$

$$T_f \cdot U_2 = 276 \times 1.02$$
$$= 282 \text{ kN (63 kips)} < T_r \;\; \therefore \text{ok.}$$

The connections of the braces shall be governed by the tensile forces arising from yielding of the braces. The expected yield strength of the material is taken as $R_y \cdot f_y$, where $R_y=1.1$ and $R_y \cdot f_y$ need not be less than 385 MPa. Thus, the maximum design force for all brace connections, T_f', is:

$$T_f' = R_y \cdot A_s \cdot f_y \qquad (R_y \cdot f_y \geq 385 \text{ MPa})$$
$$= 1690 \times 385 \times 10^{-3}$$
$$= 651 \text{ kN (146 kips)}$$

The double angle braces are connected to the gusset plates at both ends by four (n = 4) M325 No.22 bolts (f_u = 830MPa). There are two shear planes (m = 2) for each bolt, and to account for possible intercepted bolt threads, the shear strength of the bolts is multiplied by 0.7. The shear resistance of the bolted connection is:

$$V_{r\,bolt} = 0.7\times(0.6\cdot\phi_b\cdot n\cdot m\cdot A_{bolt}\cdot f_u)$$
$$= 0.7\times(0.6\times0.67\times4\times2\times380\times830\times10^{-3})$$
$$= 710\text{ kN (160 kips)} \quad > T_f^{'} \;\therefore \text{ok.}$$

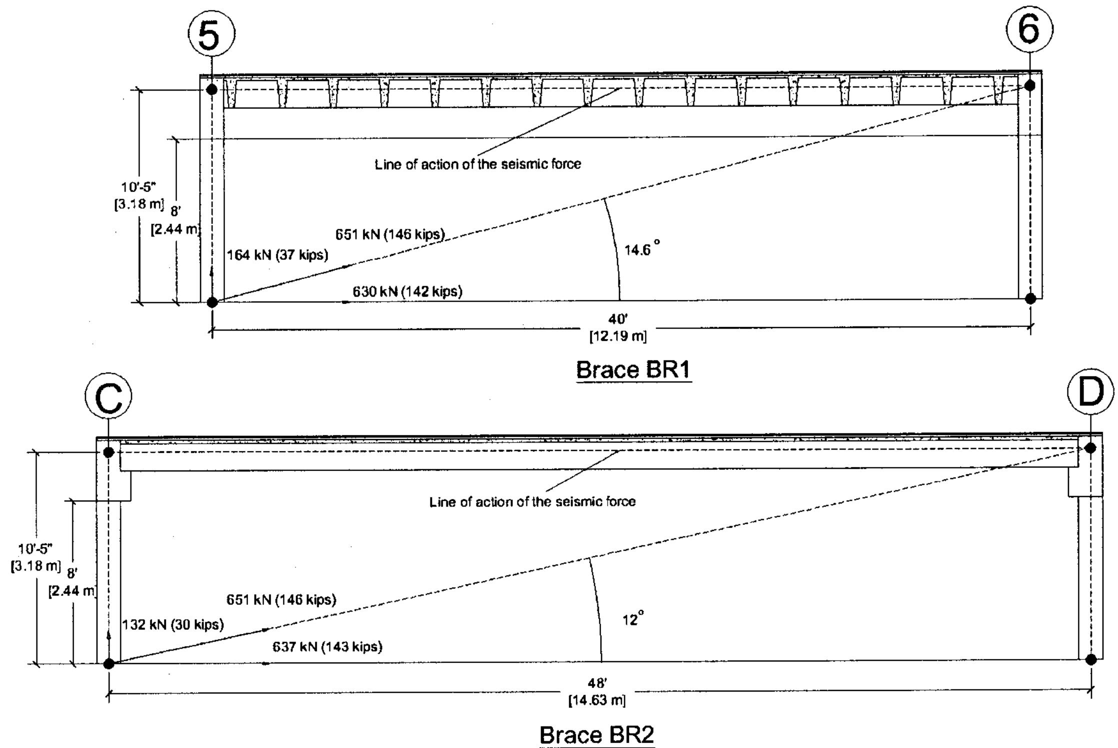

Figure B7: Geometry of the brace bents and probable tensile forces to be resisted by the connections.

Brittle net section fracture at the brace connection should be avoided. To ensure the brace members will behave in a ductile manner, each angle is reinforced with a 90 mm wide by 8 mm thick (3.5 in. by 0.3 in.) steel plate at the bolted connection region (Figure B8). The possible fracture failure paths of the reinforced angle are illustrated in Figure B9. Assuming that the bolt holes are drilled (bolt diameter + 2 mm) and with the shear lag factor for the angle being taken as 0.8, the net section fracture strength of a reinforced brace connection is:

$$A_{net11} = 2\times\left[(0.8\times(847-(22+2)\times6.4))+(90-24)\times8\right]$$
$$= 2165\text{ mm}^2\ (3.4\text{ in.}^2)\quad\text{(Controls)}$$
$$A_{net12} = 2\times\left[0.6\times(252-(24\times3.5))+(28+35)\right]\times(6.4+8)$$
$$= 4717\text{mm}^2\ (7.3\text{ in}^2)$$

$$
\begin{aligned}
T_{net11} &= 0.85 \cdot \phi_s \cdot A_{net11} \cdot f_u \\
&= 0.85 \times 0.9 \times 2165 \times 450 \times 10^{-3} \\
&= 745 \text{ kN (168 kips)} \quad > T_f^{'} \ \therefore \text{ok.}
\end{aligned}
$$

The bearing strength, B_r, of the reinforced connection is also checked:

$$
\begin{aligned}
B_r &= 3 \cdot \phi_b \cdot t \cdot d_{bolt} \cdot n \cdot f_u \\
&= 3 \times 0.67 \times (6.4 + 8) \times 22 \times 4 \times 450 \times 10^{-3} \\
&= 1146 \text{ kN (257 kips)} \quad > T_f^{'} \ \therefore \text{ok.}
\end{aligned}
$$

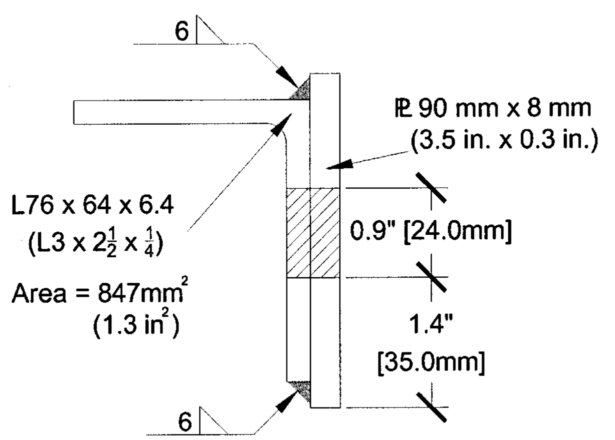

Figure B8: Steel angle reinforced with steel plate at the bolted connection.

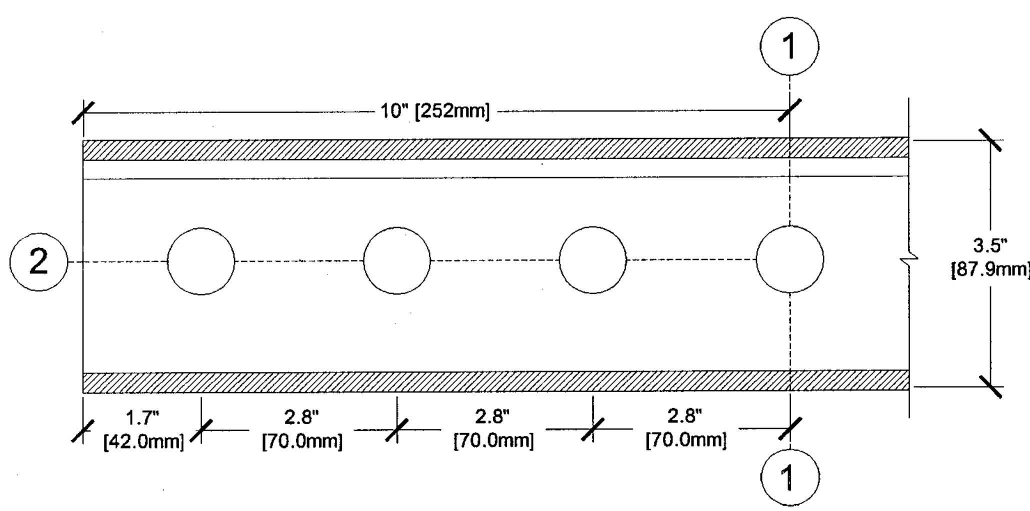

Figure B9: Net section fracture failure paths in the reinforced angle.

B.3.2 Design of the Roof Cross-braces

The roof cross-braces are designed as tension-only members and they should remain elastic while the vertical braces undergo inelastic behavior. Figure B10 shows that forces due to yielding of the E-W vertical braces dominate the design of the roof ties.

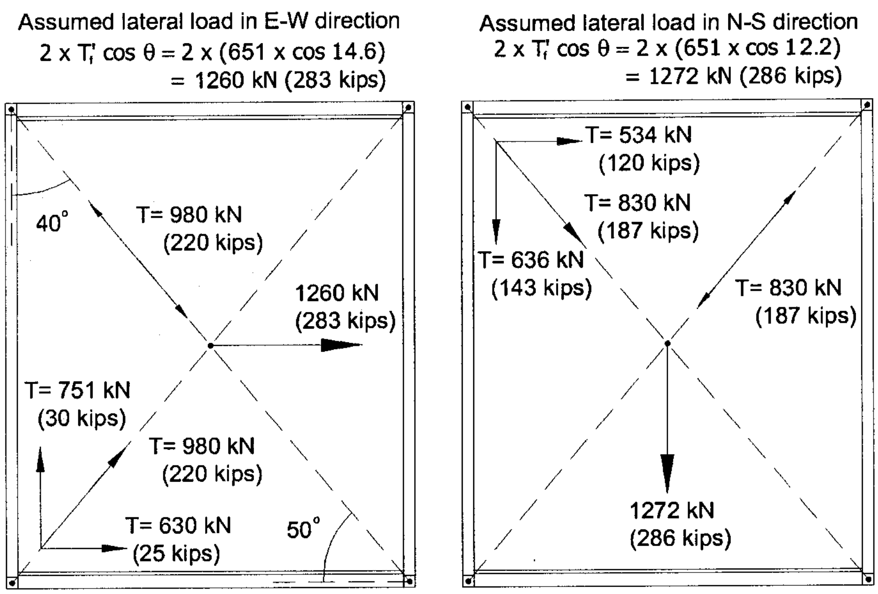

Figure B10: Assumed design forces acting in the tension-only roof braces.

The roof braces are provided in the form of WT100×26 Grade W350 structural tee members. The gross sectional yielding strength, T_r, of the tee section is checked:

$$
\begin{aligned}
T_r &= \phi_s \cdot A_s \cdot f_y \\
&= 0.9 \times 3310 \times 345 \times 10^{-3} \\
&= 1027\ \text{kN (231 kips)} \quad > T = 980\ \text{kN (220 kips)} \therefore \text{ok.}
\end{aligned}
$$

Assuming 420 mm (16.5 in.) long welds are provided along both legs of the structural tee member, the reduced net cross-sectional area due to shear lag is:

$$
\begin{aligned}
A_{net} &= A_{gross} \cdot (1 - \frac{\overline{x}}{L_w}) \\
&= 3310 \times (1 - \frac{17.5}{420}) \\
&= 3172\ \text{mm}^2\ (5\,\text{in}^2)
\end{aligned}
$$

The corresponding net section fracture strength, T_{net}, of the roof cross-braces is:

$$
\begin{aligned}
T_{net} &= 0.85 \cdot \phi_s \cdot A_{net} \cdot f_u \\
&= 0.85 \times 0.9 \times 3172 \times 450 \times 10^{-3} \\
&= 1091\ \text{kN (246 kips)} \ > T = 980\ \text{kN (220 kips)} \ \therefore \text{ok.}
\end{aligned}
$$

In order to keep the slenderness ratio of the cross ties within the 300 limit, lateral supports are provided at quarter point of each brace so that the weak axis slenderness ratio is reduced to an acceptable value of 167.