



SEISMIC UPGRADE OF CHIMNEY STACK AT LIONS GATE HOSPITAL IN NORTH VANCOUVER, CANADA

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SUMMARY

The 122 feet high chimney stack of Lions Gate Hospital is a tapered hollow reinforced concrete tubular structure built in 1958 surrounded by three critical buildings housing patients, administration and emergency functions. The seismic evaluation of the original structure determined that its flexural capacity was only 30% - 40% of that required by current codes, had little ductility, and it constituted a high risk to the adjacent buildings in the event of the code level earthquake.

This paper outlines the evaluation and cost comparison of six different retrofit options, and the details of the design, construction, and cost aspects of the selected option.

The analysis of the options evaluated the impact of the selected scheme to the occupants of the nearby buildings, and encompassed alternatives such as wrapping the stack with Fibre Reinforced Polymers (FRP), reduction of the height, application of shotcrete or addition of a reinforced concrete skin, structural steel liner bonded to the stack, independent structural steel enclosure and a new stack.

The option of wrapping the stack with FRP was ultimately implemented because of its cost-effectiveness and the minimum disturbance of the construction works, minimizing the dust and noise and eliminating disruption of the existing operating equipment integral with the stack. The existing foundation was also enlarged to provide the required overall stability and anchorage for the FRP.

INTRODUCTION

I

n 2001, the North Shore Health Region retained Sandwell to carry out the seismic evaluation and detailed upgrade design for a 122 feet high chimney stack structure which is a part of the Lions Gate Hospital complex located in North Vancouver, B.C.

This paper outlines structural and particularly seismic deficiencies, and compares the implications and order of magnitude of the costs for various upgrade schemes.

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The scope of this seismic upgrade project included:

- Assessing seismic safety of existing chimney structure with respect to the provisions of most recent building codes.
- Performing material testing to determine concrete and steel material properties, determining the concrete shell thickness, and the amount and distribution of steel reinforcement.
- Recommending feasible seismic upgrade and structural repair solutions including corresponding construction costs.

Reference Material

A limited amount of background information on the existing stack structure was available, mainly based on one original construction drawing showing an elevation of the stack structure produced in 1958. Information related to the concrete shell thickness, material properties, and the reinforcement was not available. A materials testing firm, Levelton Engineering Ltd., was retained to perform the material testing in order to provide some of the missing information. Geotechnical parameters were determined based on the soil investigation performed by the geotechnical consultant for other locations in the hospital complex.

SEISMIC ANALYSIS AND EVALUATION

Structure Description

The 122 feet high chimney stack structure, which is the part of the Lions Gate Hospital complex, was designed in 1958. At the present time, the stack is in function as a part of the in-house natural gas fuelled heating system and as the exhaust for incineration of hospital waste.

The chimney is a tapered hollow reinforced concrete tubular structure. Overall chimney height from the ground level (elevation 338'-6") to the top is around 122 feet. The overall height from the top of the foundation to the top of the structure is on the order of 136 feet. The outer diameter of the hollow concrete section is 9'-7" at the base level and it tapers down to 6'-0" at the top of the stack. The shell thickness varies from 9 inches at the base to 6 inches at the elevation of 368'-6", and is constant from that elevation upwards. The concrete shell is reinforced with #4 vertical steel rebar spaced at 12 inches on centre throughout the height. Horizontal reinforcement consists of #3 steel rings at variable spacing, ranging from 4 inches on centre at the ground level, to 6 inches on centre at elevation 15 feet above ground level, and at 8 inches on centre at the elevation of 368'-6" upwards. Clear cover to longitudinal reinforcement is on the order of 2 inches. The material testing revealed that the reinforcement consists of normal deformed bars with yield strength of 276 MPa. Concrete compressive strength varies from 28 MPa to 40 MPa. It should be noted that the description of the concrete shell thickness, amount and distribution of reinforcement, and concrete and steel material properties is based exclusively on the findings of material testing.

The interior of the stack is lined with refractory blocks of variable thickness. Site inspection by Levelton determined that the thickness of the block layer is 16 inches at the base and it is assumed that it tapers down to 8 inches from elevation 368'-6" upwards.

The foundation structure for the stack consists of a 20'-0" octagon mat. The top of the foundation is located at 10 feet below grade.

A photograph of the chimney before the seismic upgrade during the early stage of the upgrade work is shown in Figure 1.



Figure 1 Lions Gate Hospital Chimney prior to Upgrade

Regional Seismicity

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

- A magnitude 7.2 earthquake located some 100 km southeast of Vancouver in 1872
- A magnitude 7.0 earthquake located off the West Coast of Vancouver Island in 1918
- A magnitude 7.3 earthquake located on Vancouver Island in 1946.
- Numerous smaller earthquakes.
- Evidence that larger earthquakes, perhaps greater than magnitude 8 have occurred.

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

Reference Seismic Code and Seismic Design Criteria

According to information provided by the original structural drawing, the chimney structure was designed in 1958 and it is very likely that it was designed with limited (if any) earthquake resistant features. In order to assess the compliance of the existing structure with the seismic requirements of current building

codes, the latest edition of the British Columbia Building Code (1998) and the National Building Code of Canada 1995 (NBCC1995) was used in this study. However, NBCC1995 does not offer specific guidance on special structures like chimneys. As such a more relevant document was also used: Standard Practice for the Design and Construction of Cast-in-Place Reinforced Concrete Chimneys (ACI 307-98) issued by the American Concrete Institute. Concrete design in Canada follows the guidelines presented in the CSA Standard CAN3-A23.3-M94 Design of Concrete Structures for Buildings.

Following the 1995 NBC seismic provisions, the base shear force for a building structure can be expressed as:

$$V = V_e \cdot U / R$$

Where:

$$V_e = v \cdot S \cdot I \cdot F \cdot W$$

R = Force modification factor that reflects the capability of a structure to dissipate earthquake energy through inelastic behaviour, commonly defined as ductility. This factor also recognises the existence of alternate load paths or redundancy in the critical structural elements, thus increasing a number of locations where energy can be dissipated and also reducing the risk of structural collapse as a result of failure in the individual elements. Values of R factor vary from 1.0 for non-ductile structures to 4.0 for well-detailed ductile steel or concrete structures. Chimney structures do not fall into any of the structural categories listed in Table 4.1.9.B of NBC 1995, and therefore a value of R = 1.5 may be considered as appropriate in this case, keeping in mind that there is a very limited redundancy and ductility capacity associated with older concrete structures of this type.

U = This is a calibration factor equal to 0.6, and it has a constant value for all types of structures.

v = Factor based on ground motion associated with a 10% probability of exceedance in 50 years (475 years return period). For North Vancouver v = 0.2.

S = Seismic response factor, related to fundamental period of the structure. NBC1995 recommends several formulas for the calculation of fundamental period of building structures, however none of them is appropriate for chimneys. Based on the modal dynamic analysis performed using SAP2000 software, a fundamental period value of 0.90 seconds was obtained, corresponding to S = 1.6.

I = Importance factor. For ordinary buildings, the factor used is 1.0. For post-disaster facilities like hospitals, the factor is increased by 50% i.e. I = 1.5. The latter value was used in this study.

F = Foundation factor related to soil conditions, varying in the range from 1.0 to 2.0. In this case, a factor of 1.0 was used based on the geotechnical report recommendations. It should be noted that the maximum value of the product of F x S required for the design of this type of structure in this seismic zone is limited to 3.0.

W = Dead load (concrete plus refractory). In this case there is no need to add 25% of snow load as for majority of building structures.

Note that the above outlined seismic force expression has been established for use in ultimate strength design procedures.

Equivalent static analysis is a commonly used and widely accepted method for determining effects of earthquake actions to majority of typical building structures. However, in the framework of such a simplified procedure it is very difficult to account for the effects of higher vibration modes that may considerably affect the performance of some types of structures during an earthquake. Chimney stacks, as slender cantilever structures, fall into a category of structural systems that require alternative procedures of seismic analyses. Rational treatment of this type of structures calls for a response spectrum analysis. NBC 1995 also recommends response spectrum method as an additional tool in seismic analysis of special

structures. The Code prescribes a standard response spectrum curve calibrated to an earthquake event with 10% probability of exceedance in 50 years. Damping ratio of 5% was used in this project; this value is found to be appropriate for majority of reinforced concrete structures.

It should be noted that the effect of the vertical component of an earthquake on the seismic response of a chimney structure has been determined to be insignificant. Based on the weight of knowledge resulting from the studies carried out by the ACI Committee 307 on Chimney Structures, it appears that the effect of the vertical component of earthquake motion adds only a few percent of vertical stresses to those resulting from the dead load and horizontal component of an earthquake. In addition to this, it is well known that peak responses to vertical and horizontal components of earthquake motion do not generally occur simultaneously. Moreover, seismic evaluation of the structure based on the combined effect of the vertical and horizontal components of an earthquake would be unduly conservative.

Based on the NBCC1995 seismic design procedure used in the evaluation, the value of design base shear force expressed as a fraction of W is:

$$V = 0.19 \cdot W$$

SEISMIC EVALUATION

Evaluation of the existing stack structure is done according to the seismic requirements of the National Building Code of Canada 1995 that typically applies to new structures. The assessment criteria used in this project are therefore more stringent as compared to the recommendations offered by the National Research Council of Canada Guidelines for Seismic Evaluation of Existing Buildings, 1992. It should be noted that the higher importance factor value $I = 1.5$ was also used in the evaluation as discussed in the previous section.

Seismic evaluation of the chimney structure is largely based on the findings of numerical seismic analysis performed using two different analytical methods: equivalent static and response spectrum analysis. In the analysis, the chimney was modelled as a free-standing stick structure fixed at the base level (top of foundation).

The following two options were considered in evaluation of the chimney structure: i) Full height of the chimney structure, and ii) Reduced height chimney structure; the height reduced by 80 ft. It should be noted that, in the case of the reduced height chimney, seismic force value has been increased to $V = 0.36 \cdot W$ as compared to the force for full height of the chimney, $V = 0.19 \cdot W$. This difference is due to the change in dynamic properties of the shorter and stiffer chimney resulting in a shorter period with a corresponding higher S value than in the former case. Furthermore, for every seismic retrofit option the base shear was re-calculated using the related dynamic properties of the modified chimney accordingly. The results of seismic analysis carried out in relation to the above two options are summarised in Tables 1 and 2 shown below.

It should be noted that the values given in the last column of each table indicate a range of ratios from the bottom to near the top of the stack between flexural capacity (C) and flexural demand (D) of the structure analysed using the above listed seismic design parameters. According to earthquake-resistant design philosophy, capacity of a structural element or a structure as a whole should be at least equal to the structural demand, which is expressed here as the effect of factored loads acting on a structure. In order to provide a level of seismic performance approximately equivalent to a new structure designed per the

provisions of the current building codes, capacity/demand (C/D) ratio for each structural element should be at least equal to 1.0.

Table 1 - Full height chimney structure, $V = 0.19 \cdot W$

Case No.	Type of Analysis	I	R	Flexural C/D range
1	Equivalent static	1.5	1.5	0.3 to 0.95
2	Response spectrum NBC 1995 1/475 eq.	1.5	1.5	0.4 to 0.98

Table 2 - Reduced height chimney structure - height reduced by 80 ft. $V = 0.36 \cdot W$

Case No.	Type of Analysis	I	R	Flexural C/D range
3	Equivalent static	1.5	1.5	0.6 to 3.1
4	Response spectrum NBC 1995 1/475 eq.	1.5	1.5	0.63 to 3.2

The flexural capacity (C in the table above) of various chimney sections was determined from interaction diagrams based on the recommendations of the CSA Standard CAN3-A23.3-M94 Design of Concrete Structures for Buildings.

Relevant findings of the seismic assessment, as well as the general recommendations for upgrading the observed structural deficiencies, are summarised below.

Flexural capacity of the structure.

The existing chimney structure only has a flexural capacity of 30%-40% of that required by current codes. Even if 80' of the stack was removed, the chimney would still only have a flexural capacity of some 60% of that required by current codes. The deficiency has been identified almost throughout the chimney height, starting at the base level and continuing up to an elevation 90 feet above ground.

Shear capacity of the structure.

Shear strength of the chimney structure was found to be adequate for all seismic actions considered, except locally at openings as discussed later. The analysis was carried out considering contribution of horizontal reinforcement to the shear strength Concrete contribution to shear strength was neglected in the analysis.

Lateral confinement.

Results of numerous studies, as well as the experience gained in previous earthquakes, have confirmed that closely spaced ties in a concrete element ensure adequate confinement to the concrete core and also provide restraint to the main longitudinal reinforcement. According to the minimum provisions of the Canadian Concrete Code (CAN3-A23.3-M94) related to elastic response of structures, lateral confinement provided by the existing horizontal rings is deficient by over 50 percent. Moreover, lateral confinement was also evaluated using the recommendations of ACI 307-98, which are considered more appropriate for evaluation of chimney structures. The results of the evaluation indicate that two layers of vertical and circumferential reinforcement are required, however only one layer is present in the existing structure.

Splice lengths of reinforcement bars.

Information on the splice length and anchorage for vertical and horizontal reinforcement was not available. Typically, seismic evaluations of older concrete structures show that the splice lengths are not adequate per the current code requirements. As a result, it may be expected that the full capacity of stack sections in bending and shear cannot be reached. The assumption made in this evaluation was that the splice lengths are adequate.

Vertical reinforcement ratio.

Ratio of existing vertical reinforcement varies from 0.28% in the top 90 feet of the stack to 0.20% at the base level. According to ACI 307-98, vertical reinforcement ratio of minimum 0.25% should be provided in a chimney structure. Except for the base of the structure, this clause has been complied with throughout the chimney height.

According to the minimum provisions of the Canadian Concrete Code (CAN3-A23.3-M94) related to the minimum flexural reinforcement requirements, all stack sections are considered to be deficient. The cracking moment value for all cross-sections of the stack structure is significantly larger than the moment of resistance, whereas the code requires that the moment of resistance value should be at least 20 % larger than the cracking moment. This is characteristic for older concrete structures, which are considered to be under-reinforced per current code requirements. As a consequence of this deficiency, there is an increased chance for a brittle mode of failure of concrete elements in an earthquake.

Reinforcement around the openings.

According to the ACI 307-98 provisions, a substantial amount of reinforcement is required in areas around openings. The existing structural drawing shows one breaching opening of 3'- 4" inner diameter at approximately 6 feet above the ground level. Details of reinforcing bars provided around the opening are not known, and it may be expected that the existing provisions are inadequate. Therefore, at the time of structural upgrade particular attention should be given to strengthening the areas around the existing opening.

Lateral displacements.

Lateral displacements at the top of chimney structure induced by design earthquake have also been evaluated. The analysis indicated that the top lateral displacements induced by the NBC1995 design earthquake (return period of 475 years) are within the limits prescribed by ACI 307-98.

RETROFIT CONCEPTS

The following retrofit concepts were considered in this project:

Retrofit Concept 1

Retrofit concept 1 consisted of wrapping Fibre Reinforced Polymers (FRP) fabric around the chimney structure by using one of the commercially available products. The system was designed to add flexural capacity and ductility to the existing structure by the use of high strength hybrid glass fibre/epoxy composites. A major advantage of this retrofit concept lies in the ease of installation and the lightweight of FRP material. It also allows for the preservation of exterior appearance of the existing structure. In order to ensure the effective force transfer from the stack to the foundation, special anchorage arrangements were developed for this concept.

About FRP

Fibre-reinforced polymers (FRP's) are a combination of a polymer matrix (epoxy, in general terms) and a fibre reinforced material such as glass or carbon. They also may contain fillers, or core materials to provide the FRP with additional reinforcing functions.

Benefits of fibre-reinforced polymers that make them ideal for structural strengthening applications are that they can be used for high tensile and flexural strengths and have very little creep. Another benefit of FRP's is how lightweight they are; in general, they weigh one-fifteenth the weight of steel and as such usually make application easier. In addition to these properties, through the use of fillers and additives, composites can be made to be resistant to chemical reactions, inhibit ultraviolet rays, and have low water absorption.

The most common use of FRP for wall upgrades consists of a fabric that makes use of primary carbon fibres or glass fibres with optional polyaramid fibres stitch bonded. The fabric is applied with epoxy against the wall surface constituting a composite system formed by embedding continuous fibres in a resin matrix which binds the fibres together to provide additional strength to the existing walls. In bonding FRP's to the structural element, proper surface preparation is very important, and it is obtained by standard mechanical procedures such as grinding or sanding.

Design and construction with FRP's are regulated by the Canadian Standard Associations document CSA S806-2 Design and Construction of Building Components with Fibre-Reinforced Polymers.

Retrofit Concept 2

Retrofit concept 2 consisted of demolishing the upper 80 feet of the stack structure. Alternate option (2a) was to only reduce the height of chimney and (2b) to replace the demolished upper part with a new steel shell. This scheme has a benefit of lowering the centre of gravity of the structure. In addition, option 2b has the benefit of replacing a portion of the existing 40-year-old concrete stack with a new steel shell.

This option cannot achieve compliance for 100% of the code specified base shear, but only some 60% for the option of removal or removal/replacement respectively. This option was presented for consideration only if funding was unavailable to achieve the full upgrade, and if reducing the height of the stack emissions was environmentally acceptable.

Retrofit Concept 3

Retrofit concept 3 consisted of adding a new, 4 inch thick, reinforced concrete overlay at the outer face of the existing concrete shell. In order to upgrade the flexural capacity of the existing structure to current code requirements, the addition of reinforced concrete skin is required for the lower 90 feet of the stack height. In order to provide an adequate connection between the existing and the new concrete, adhesive anchors (dowels) need to be installed.

Retrofit Concept 4

Retrofit concept 4 consisted of installing a new structural steel plate "liner" outside of the stack for the lower 90 feet. The liner would be doweled to the existing concrete to upgrade the flexural capacity of the existing structure to current code requirements. This concept is structurally similar to retrofit concepts 1 and 3.

Retrofit Concept 5

Retrofit concept 5 consisted of totalling demolishing the existing stack and constructing a new stack made of a structural carbon steel plate protected by an insulated system and an inner shell of corten steel or stainless steel mechanically attached to the structural steel. This was costed for comparison purposes.

Retrofit Concept 6

Retrofit concept 6 consists of installing a new structural steel “enclosure” outside of the existing stack, made of vertical steel plates perpendicular to the surface of the stack with horizontal stiffeners. This option would not involve any modifications to the existing stack surface as the enclosure is not bonded to the existing stack. One advantage of this option is the minimum amount of dust and noise compared to the other retrofit concepts because the majority of steel work could be done offsite. If this option is selected for the detailed design, analysis will be required to determine the deformation compatibility of the ductile steel enclosure relative to the "brittle" stack structure. An isometric sketch of this concept is illustrated in Figure 2.

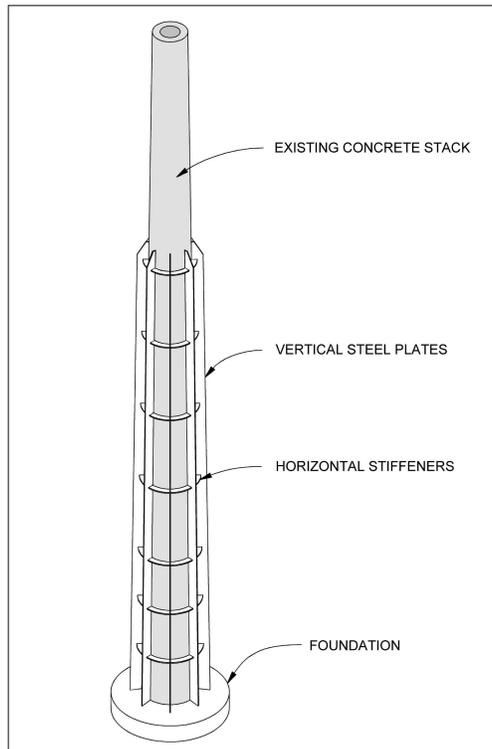


Figure 2 Isometric Sketch of Retrofit Concept 6

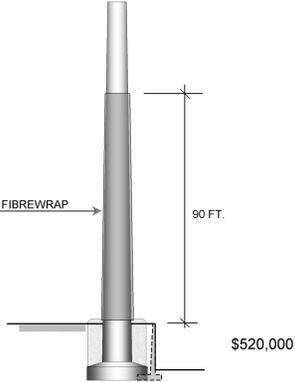
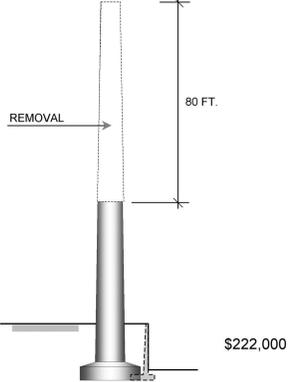
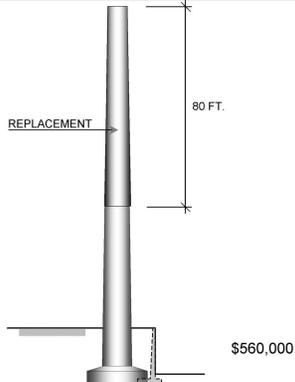
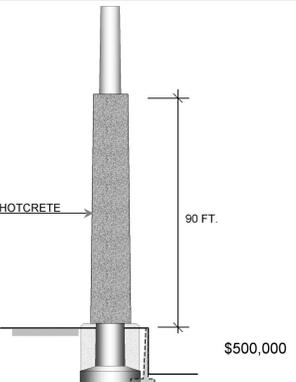
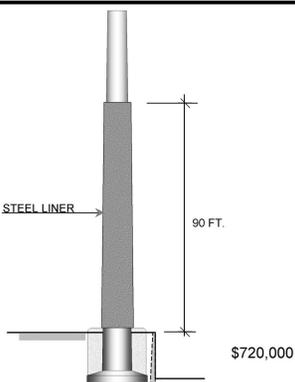
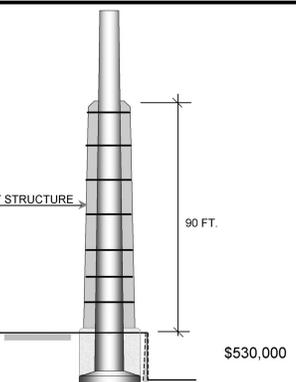
Table 3 summarises all the retrofit options including their advantages and disadvantages, and capital cost estimates, and Table 4 provides a graphical summary.

Table 3 Summary of Retrofit Options

 Table 3 - Summary of Retrofit Options and Capital Cost Estimates								
Retrofit Concept	DESCRIPTION	1	2a	2b	3	4	5	6
1	Upgrade by installing FRC fibrewrap at the outside (90 ft. high portion)	☞						
2a	Removal of the upper 80 Ft. of the stack structure		☞					
2b	Demolition of upper 80 Ft. of the stack structure and replacement with a new steel liner.			☞				
3	Casting of new reinforced concrete skin at the outside of the stack in the lower 60 ft. of the chimney height, from the pedestal.				☞			
4	Upgrade by installing new steel liner at the outside of the stack doweled to the pedestal (lower 60 ft. of the chimney height)					☞		
5	New stack with steel liner						☞	
6	Upgrade by providing independent steel structure at the outside of the stack							☞
a	Code level achieved	100%	66%	60%	100%	100%	100%	100%
b	Reinforce around the breeching opening	✓	✓	✓	✓	✓		
c	Excavation below grade and anchorage into the existing foundation through a pedestal	✓			✓	✓		✓
d	New foundation	✓					✓	
e	Disruption to operation of the existing chimney structure		✓	✓			✓	
f	Connection works to the existing facilities						✓	
g	Environmental disruption such as strong noises or dust on chimney works		✓	✓	✓	✓		
h	Demolition works and surface preparation	✓	✓	✓	✓	✓		
i	Cost estimate	\$520,000	\$222,000	\$560,000	\$500,000	\$720,000	\$990,000	\$530,000

A number of other upgrading provisions are needed as part of the seismic upgrade. Concepts 1, 3, 4, and 6 include enlarging the foundation mass to resist the seismic overturning loads. In addition, for these alternatives anchorage of the external reinforcing is required, thus a cast-in-place reinforced concrete pedestal has been proposed to achieve both requirements. As previously mentioned, the breeching opening located in the lower portion of the chimney structure will need to be strengthened by providing additional reinforcement locally for retrofit concepts 1 to 4. In addition, for all the options related to upgrading the existing stack, it was necessary to provide restraint for the existing internal refractory bricks as collapse of the brick liner during or after an earthquake would obstruct the chimney operation; the restraint would consist of a stainless steel mesh placed over the surface of the brick liner and fixed to the concrete wall with appropriate adhesive anchorage. Repair of minor cracking was recommended as a part of the upgrade project. Allowance for the cost of all these items were considered in the cost estimates.

Table 4 Graphical Summary of Retrofit Options

 Table 4 - Graphic Summary of Retrofit Concepts	
 <p>\$520,000</p>	 <p>\$222,000</p>
<p>RETROFIT CONCEPT 1 - FIBREWRAP</p>	<p>RETROFIT CONCEPT 2a - REMOVAL OF THE UPPER PART OF THE STACK</p>
 <p>\$560,000</p>	 <p>\$500,000</p>
<p>RETROFIT CONCEPT 2b - REPLACEMENT OF THE UPPER PART OF THE STACK</p>	<p>RETROFIT CONCEPT 3 - SHOTCRETE</p>
 <p>\$720,000</p>	 <p>\$530,000</p>
<p>RETROFIT CONCEPT 4 - STEEL LINER</p>	<p>RETROFIT CONCEPT 6 - INDEPENDENT STRUCTURE</p>

COST COMPARISONS

Cost estimates presented in Table 3 summarize the costs of various upgrade concepts as discussed in previous sections. The comparison of construction impact is also described in the table.

Developing a cost estimate for such specialized and unique work is a challenge. As such, a conservative estimate has been developed using the following parameters:

- Union labour work force; because the work is highly dependent on essentially low-skilled labour, non-union labour rates could significantly reduce costs.
- Assuming conventional internal jump forming for concrete; certain contractors may have specialised equipment that could reduce such costs.
- Adequate costs to ensure all Workers Compensation Board requirements could be met while working inside the chimney.
- Current market conditions; future market conditions may result in aggressive bidding that could further reduce costs, or a heated construction market that could increase costs.

Including a 20% contingency the costs are summarized below:

• Option 1, 90' FRP reinforcing	\$ 520,000
• Option 2a, reduce stack height by 80' [partial upgrade only]	\$ 222,000
• Option 2b, as per 2a plus new steel upper 80' [partial upgrade only]	\$ 560,000
• Option 3, reinforced concrete shell 90'	\$ 500,000
• Option 4, bonded steel liner 90'	\$ 720,000
• Option 5, New stack	\$ 990,000
• Option 6, independent steel frame 90'	\$ 530,000

SELECTED OPTION

Among the retrofit concepts that provide 100% of the current seismic code requirements, the Option 1 reinforcing the stack with FRP was selected. Although potentially slightly higher in cost than option 3, it was selected due to lesser construction impact with regards to noise and duration. Detailed design followed and the upgrade of the stack was built accordingly.

Figure No. 3 shows a photograph of the stack after the upgrade.

From an aesthetic point of view, the FRP option resulted in a final appearance nearly identical to the existing stack (only millimetres thicker than the original structure).



Figure 3 Overview of the Chimney after the Upgrade Work