RE: Your request for access to information under Part II of the Access to Information and Protection of Privacy Act (Our File TW/014/2013)

On February 22, 2013, the Department of Transportation and Works received your request for access to the following records/information:

A copy of the engineering report related to the structural issues in the confederation building, in particular the issues associated with the 8th Floor of the East Block of the Confederation Building.

I am pleased to inform you that your request for access to this information has been granted in part and enclosed is a copy of the Preliminary Structural Report dated December 2009. For your information, I have further enclosed a copy of the Elevator Structure Assessment – Revision, dated October 2011.

Access to information contained within the records, has been refused in accordance with the following exception to disclosure, as specified in the Access to Information and Protection of Privacy Act (the Act):

Section 30(1) The head of a public body shall refuse to disclose personal information to an applicant where the disclosure would be an unreasonable invasion of a third party’s personal privacy.

As required by subsection 7(2) of the Act, we have severed information that is excepted from disclosure and have provided you with as much information as possible.

Your request specifically references an “engineering report related to structural issues”. Please note that the technical engineering interpretation of “structural issues” result in identifying the reports currently being provided. We are however requesting clarification as to if this addresses the intent of your request or if your request was intended to be broader, relating to other building elements, such as mechanical and electrical systems.
If you have any further questions, please feel free to contact the undersigned at 729-3676.

Sincerely,

JAMIE CHIPPETT
Deputy Minister

Enclosure
Department of Transportation and Works
Newfoundland & Labrador

St. John’s, NL

Elevator Structure Assessment - Revision

ER-203559

October, 2011
November 24, 2011

Department of Transportation and Works, NL
5th Floor West Block, Confederation Building
P.O Box 8700
St. John’s, NL
A1B 4J6

At the request of Mr. Derrick Moss, vibration measurements and a structural loading study were conducted on the NW Tower Elevator running from the 8th - 11th floor of the Confederation Building in St. John’s, NL. Acuren personnel [redacted] and [redacted] were onsite October 5th and 6th to conduct the field testing.

Vibration measurements were conducted using CSI model 2130-2 machinery analyzers, IBAscope software with 64-channel CM-8X Portable analyzers, Wilcoxon 799LF, 797L, 793 general purpose industrial accelerometers, and PCB model 086C41 medium modal sledge. All accelerometers and cables are checked using a PCB model 394C06 handheld calibration shaker. Strain Gage testing is conducted using a Binsfeld Engineering Inc. TorqueTrak 10K Torque Telemetry System.

Each test is briefly described below:

forced response – dual channel force input and vibration response measurements were acquired during machine shutdown at selected locations to determine system natural frequencies and damping characteristics.

continuous data testing – multi-channel vibration measurements were acquired simultaneously on selected measurement points over the machine frame during various testing conditions.

strain gage testing – strain gages are installed to measure torsion, and elongation stress and strain in the supporting structure of the elevator. A single data signal is transmitted wirelessly to a data acquisition unit for processing. Multiple units were used simultaneously.

Note that a standard directional axis is used throughout the report, where Z is vertical positive pointing up the shaft, X is positive running straight into the elevator shaft, and Y is positive running from right to left looking into the elevator shaft. In technical application apply the right hand rule facing the elevator.
Results and Observations – forced response

Resonance occurs when forcing frequencies are coincident with system natural frequencies. At resonance, system response (vibration magnitude) is amplified and a phase shift occurs. The amount of amplification depends on the system damping characteristics. A natural frequency may be described using Equation 2, shown below:

\[ f_n = \sqrt{\frac{k}{m}} \]  \[ \text{where } k = \text{stiffness, lb/in and } m = \text{mass, lb/in/sec}^2 \]  \[ \text{[Equation 1]} \]

Forced Response Testing (impact testing) is used to identify the natural frequencies of systems and components. The raw data acquired during impact testing is evaluated for amplitude, coherence and phase shift to verify the accuracy of the sample. This data is then crosschecked with system forcing functions to identify possible areas where resonance will occur.

Sample data plots for the impact testing are shown in the below figures. The data was acquired on the left side vertical I beam column support (looking into the elevator shaft). The indicated peak is 74.25 Hz. Note that other valid peaks are also present.

Figure 1 - spectrum of accelerometer response to impact by modal hammer, elevator column support
Results and Observations – forced response (continued)

The validity of an impact test is considered with reference to the dominance of the peak shown in the natural frequency transfer function compared with the coherence and the phase shift data. The coherence plot verifies that the vibration data acquired by the accelerometer is in fact a result of the impact by the modal hammer. In this test, a valid data point will have a coherence value of 0.75 or higher on a scale of 1.0, the below indicated 0.99.

![Coherence plot](image1)

Figure 2 - cross channel coherence plot between accelerometer and modal hammer, elevator column support

Additionally, a phase shift of approximately 180 degrees in the cross channel phase plot is consistent with resonance conditions.

![Phase plot](image2)

Figure 3 - cross channel phase plot between accelerometer and modal hammer, elevator column support

All of the data considered for this report was evaluated using the criteria presented in the previous figures prior to acceptance as a valid natural frequency of the tested component.
Results and Observations – forced response (continued)

Forced response testing indicates system natural frequencies, typically the 1st and 2nd bending modes. When forcing functions (such as the 1X set turning speed, pumps, fans, etc) are coincident with the natural frequency, a significant increase in vibration amplitude will occur (i.e. resonance).

Additional components of interest to natural frequency modes, other than peak frequency and amplitude, are damping and amplification. The amount of damping in a structure affects the resonant response, such that, with no damping, one would expect to get infinite motion with very small excitation. As the damping increases, the resonant response from a given force reduces.

In a lightly damped system, the frequency peak is narrow at its base in the spectral plot. This narrow peak implies large amplification over a narrow frequency range. Conversely, a highly damped system shows a peak at a natural frequency with a wide base. Associated with system damping is how much the system vibration amplitude will be amplified by a coincident forcing function. The degree of amplification may be quantified by Equation 2, where Q is the amplification factor, fc is the center or peak frequency, and fa and fb are the frequencies at which the amplitude peak is 0.707 of its maximum.

\[ Q = \frac{fc}{fb - fa} \]  

[Equation 2]

In figure 5 below, the amplification factor for the natural frequency peak of the data was acquired on the left side I beam column support. Peaks at 0.707 of the center frequency amplitude, and Hz, are used to calculate Q, the amplification factor. Therefore, the vibration amplitudes at a frequency of 74.25 Hz will be amplified by a factor of 22.

Figure 4 - amplification factor calculation of natural frequency peak, elevator column support
Results and Observations – Continuous Data Acquisition Testing

Continuous acquisition of data was taken on the elevator shaft and surrounding structure. A total of 28 accelerometers were setup and connected to a computer and the data stored for post processing. Locations were strategically selected for areas of concern and those that are expected to produce the highest vibration readings. Data was also collected on the interior walls by means of epoxy gluing washers and attaching accelerometers.

During this testing numerous trials were conducted varying the load, location of load, method of going from floor to floor and direction of travel. As an example; one trial was completed by having 2 occupants on one side and a third centrally located in the cab, travelling from the 8th to the 11th level stopping on each floor and allowing the door to open and close. Similar trials were completed by using more occupants (up to a maximum of 7), or changing the sequence. Additionally trials were completed in reverse order by starting at the 11th and working to the 8th floor.

The following figures are waterfall plots of vibration amplitude and frequency over a period of time; in this case the data is split into 20 minute files. The amplitude; vertical axis, is measured in g’s of acceleration, frequency; horizontal axis, is shown in Hertz (Hz) and time; displayed as depth, is equally divided over the span of the plot in this case equal divisions for a total period of 20 minutes. Several locations are shown below and are representative of the analysis completed on all 28 channels of data acquired on the elevator and supporting structure.
Results and Observations – Continuous Data Acquisition Testing (continued)

Figure 5 – This data was taken on the right hand supporting column at the 11th floor, horizontal direction

In the above plot it is clear that vibration is present and that certain events trigger excitation of this location. What should be noted is the extremely low amplitude in which these recurring events happen. 0.001 inches per second @ 28Hz is an extremely low value, indicating very little transmission to the structure.
Results and Observations – Continuous Data Acquisition Testing (continued)

Figure 6 – This data was taken on the supports that tie in the columns to the horizontal beams located behind the interior walls, between the 9th and 10th floor.

What is important to note in the above figure is the location of the data. This sensor was placed on one of the column supports that connect into the horizontal steel beams that the masonry walls surround. The maximum vibration amplitude at this location was found to be 0.002g's at 25Hz or 0.005ips. The very low amplitude suggests that little vibration is being transmitted to the structure, thus not being transferred to the exterior masonry walls.
Results and Observations – Waveform Analysis

Consideration of instantaneous impacting which may occur from the elevator starting and stopping must be analyzed. Some of the energy transmitted into the structure may arise from one time impacts which will not be represented in the above waterfall plots. This is due to the process in which data is extrapolated from the raw time wave form and converted into the vibration plots shown above.

The following figures show the acceleration from single impacts caused by the elevator. This data is displayed as a plot of acceleration (g's) vs. time (hr:min:sec). These figures are representative of the analysis method used for all of the data points. Each plot is associated with a particular test conducted on the elevator.

![Waveform](image)

**Figure 7 – Raw waveform of acceleration on the column tie in located between the 9th and 10th floor**

The figure above is a detailed view of a raw time wave form for one of the accelerometers located on the column tie in. This figure shows data for a span of approximately 90 seconds. In this trial the elevator started on the 11th floor and moved to the 8th stopping on each floor. Each time it stopped the impacting is clearly visible. What should be noted; however, is the very low amplitude of these impacts, in the order of 0.15g's pk-pk.
Results and Observations – Waveform Analysis (continued)

Figure 8 – Raw waveform of acceleration on a horizontal support beam running between the columns, just above the 8th floor

The image above has been magnified to show; in milliseconds, the raw waveform data acquired on one of the horizontal supports where the hydraulic piston is attached to the supporting structure. This location shows the greatest amount of acceleration and impacting over the entire structure. Even at this location the level of excitation is very low, approximately 0.6g's pk-pk.
Results and Observations - Strain Gage Testing

Testing was conducted on both of the columns, in each case 2 strain gages were placed to measure torsion and longitudinal strain, which can then be related to stress by the use of Hooke's Law. The intentions for this testing were to reveal if the elevator was not properly moving along its guiding rails. If there were a misalignment of the rails or imbalance in the elevator car, the strain gages would determine the amount of strain being transmitted to the structure. The 4 strain gages were placed on the face of the vertical columns just above the 9th floor. See appendix for installation images. The following figures show some of results found from the gages.

Figure 9 – Strain gage output, left side column, measuring torsion

In figure 9 the strain gage output shown is for the left hand column, looking to the elevator shaft, measuring the torsion. The plot shows voltage in volts (vertical axis) and time in seconds, from this voltage the micro strain can be determined using a specific formula taking into consideration the system properties and particular setup for each gage. What is important to note is that the actual voltage figure is irrelevant in the analysis; the determining factor is the change (delta) in voltage between starting location and peaks in the data, as well as the overall change in voltage. Also please note that the length of the peaks (time span) does not represent the travel time from floor to floor. Instead the length represents the duration of instability of the strain, and the amplitude represents the
Results and Observations – Strain Gage Testing (continued)

magnitude of this instability. In the above example the output correlates to a trial where the elevator left
the 8th floor, stopping on each level and finishing on the 11th floor.

In this instance the change in voltage 0.104V comes from an initial 0.103V at rest, with the largest delta
resulting from moving from the 9th to 10th floor of 0.207V. This resulting voltage correlates to a stress of
approximately 80PSI which is negligible.

Elevator in
motion from 8th
to 9th floor

Elevator in
motion from 9th
to 10th floor

Elevator in
motion nearing
11th floor

Figure 10 – Strain gage output, right side column, measuring longitudinal strain

The largest outputs obtained came from both strain gages mounted to measure longitudinal strain on
each of the columns. Similar to the previous plot this trial was again conducted by starting on the 9th
level with stops at each floor. The largest swing in figure 10 occurs while the elevator is passing from
the 10th to the 11th level, near the end of its movement. In this case the calculated stress was
approximately 8500 PSI, again at this magnitude there is no reason to suspect abnormal deformation of
the structure. The stress must be attributed to friction between the car and the rail system, in that the
strain results from a “tugging” or pulling action, which in turn transfers the stress to columns. During
trials there was noticeable “bouncing” of the car at certain points during movement; however, there was
no correlation between the physical feeling of the bounce and the resulting output of the strain gage.
Results and Observations – Strain Gage Testing (continued)

14 different trials were performed varying the conditions and methods for running the elevator. In each case the data was collected and an analysis performed. The figures shown above are the worst case scenarios found during testing.

Results and Observations – Operating Deflection Shapes (ODS)

One particular test that can be completed using the vibration data obtained through accelerometers is an operating deflection shape. In this, a model can be built and the data applied to create an animation of exaggerated motions created by the vibration. In this report images will be included in the report and the animation movies will be attached as files in order to help demonstrate the motions described.

In the analysis completed two predominant frequencies of vibration were present and subsequent animations were created at these values, 22Hz and 28Hz respectively. One difficulty in this modeling is knowing how to represent the connection of the structure at the top of the elevator shaft. Without being able to visually confirm this condition it could be classified as a “fixed” or “free” connection, thus changing the characteristics of the model. For that reason both models have been created and attached to this report.

![Figure 11 – Model of Elevator Structure](image)

The model shown above is representative of the elevator supporting structure. This is not a scale recreation of the structure; it is simply a visual aid in determining how the structure motion is being affected by vibration. Please note the white floor level indicators on the left had side of the model are for reference only.
Results and Observations – Operational Deflection Shapes (ODS) (continued)

22Hz Fixed Columns: In this model there is noticeable movement in the vertical direction of the horizontal supports on the rail. In addition some torsion of these supports is present near the top of the structure. In this animation the top of the columns have been represented as fixed elements.

28Hz Fixed Columns: In this animation there is much more horizontal deflection in the Y axis as well as more vertical displacement of the horizontal supports in the lower floors. Displacement of the joining wall in the X direction is also evident.

22Hz Free Columns: Using this model much greater reactions can be seen throughout the system. Large amounts of torsion and vertical displacement are present near the top of the structure. This would be expected in the case of a “free” connection of the vertical columns.

28Hz Free Columns: Again more displacement of the system is present; however, at this level the majority of the movement is a “swaying” motion in the Y direction.

In all the above cases the relative motion of the structure could be contributed to the elevator and the supporting structure. The issue with the motion shown in the animations is that the magnitude of amplification required in seeing it. The levels of vibration obtained suggest that it would not be feasible to expect such reactions during operation of the elevator.
Results and Observations – Architectural and Masonry Concerns

Exterior masonry walls that are constructed of solid masonry are subject to the damaging effects of freezing of any entering rain water which will surely penetrate the exterior wythe. The rain water that penetrates the exterior wythe will freeze in place because it cannot escape. The frozen water occupies a larger volume than the entering water and will push the exterior wythe away from the rest of the wall. The freeze/thaw cycle is inexorable and will necessitate repairs on a regular and recurring basis. The proper solution is to provide a cavity wall so that there is a lane through which the entering rain water can escape before it freezes. Alternatively, the exterior surface can be coated with an impermeable coating so that the rain water cannot reach the brick facade.

Brick walls are not water proof although they are water resistant and so rain water will enter and freeze in place if it cannot run away.

There are three alternative to regulate the problem with the exterior walls of the building as follows:

- Remove the exterior wythe and rebuild the wall providing a cavity so that the entering rain water can escape before it freezes. It will run down the inside face of the exterior façade and needs to be forced out at the bottom of the wall and above window and door lintels using weep holes and a flashing that slopes downward between the backing masonry and the façade.

- Coat the exterior face of the facade with a water proof membrane to exclude the entrance of the rain water.

- Repair the cracks in the façade from time to time resulting from the freeze/thaw cycle.
Conclusions and Recommendations

At the request of Mr. Derrick Moss, vibration measurements and a structural loading study was conducted on the NW Tower Elevator running from the 6th - 11th floor of the Confederation Building in St. John's, NL. Acuren personnel [REDACTED] and [REDACTED] were onsite October 5th and 6th to conduct the field testing.

The testing was requested due to cracking in the motor and masonry on the exterior of the NW Tower. The intentions were to show if the vibration, impacting or stresses of the elevator motion were being transmitted to the building structure thus causing damage to the exterior walls. The data collected, and subsequent analysis reveal the following conclusions:

- Vibration amplitudes being transmitted to the elevator and surrounding structure are extremely low
- Very low level impacting present caused by the piston starting and stopping
- Stress levels found in the supporting vertical columns were very low, in both torsion and in longitudinal direction
- Recommend a more defined routine maintenance plan on elevator. Conditions found at time of testing suggest better operation of the elevator could be expected by regular maintenance. Initially very little lubrication of the guide rails was present, correction of this issue and subsequent runs lead to smoother operation of the elevator. Small oil leak present in the mechanical room below elevator pit should be addressed as soon as possible. All items and other could be addressed and corrected given regular maintenance
- Further investigation of the architectural flaws present on the NW towers façades should be completed to confirm the lack of proper drainage. It is recommended this be completed prior to any further action to the NW tower.

If you have any questions or comments regarding this report or any other subject, please contact us at 800-252-1774.

Report prepared by: [REDACTED]  
Report Review by: [REDACTED]
Appendix A

Figure 12 – Setup of strain gages on one of the supporting columns
Figure 13 – Accelerometer setup on one of the column supports to measure vibration transmitted to this location
Figure 14 – Accelerometers set up on the top of the elevator driving piston to measure vibrations levels at this location
Figure 15 – 3D View of animation model built for visual aid in analysis, including sensor locations
PRELIMINARY STRUCTURAL REPORT

Confederation Building
Window Replacement
St. John's, NL

Submitted to:
Ron Fougere Associates
P.O. Box 21118
St. John's, NL A1A 5B2

Submitted by:
fga Consulting Engineers Limited
2 Hunt’s Lane
St. John’s, Newfoundland and Labrador
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FGA File No: 4040-36
Date: December 18, 2009

Section 30(1)
INTRODUCTION

Fga Consulting Engineers has been engaged by Ron Fougere and Associates Ltd. to provide structural engineering input into the Phase 1 tender package for replacement of windows on the north face/west wing of the Confederation Building East Block facility. The scope of the project has included the determination of design wind pressures for the new glazing system, and a review of the existing structural framework so as to define restraints in the tender documents for how the window mullion system would attach to the structure.

During the course of this work two additional issues were raised, one with regard to the attachment of limestone window surrounds and, the second being a vertical crack in masonry at the North West corner of the central tower. Our firm was asked to include the latter two items into the scope of our work. In the following paragraphs, we will review progress to date and identify action items for future investigation.

DISCUSSION

During a recent building inspection of the façade completed by [redacted] of Jokinen Engineering Services, concern was raised with respect to the attachment of limestone surrounds which outline the perimeter of the existing window system. In one location, it was revealed that there was no obvious mechanical anchorage of the limestone to the building. In subsequent inspections that were completed by IMV, a section of masonry wall was removed, revealing the presence of a masonry anchor, but there was still concern as to how the unit worked, and if it was typical throughout the building. In the Phase 1 package, which is currently being completed by Eastern Contracting, the contractor is required to mechanically anchor the surrounds, in addition to removing a number of the units to determine the overall method of installation.

Currently, a sill and a vertical surround have been removed. It was determined that the sill was set in a masonry bed. A slot was set into the top surface of the sill and a metal bar was inserted and tied back to the window system. Upon removal of the vertical piece, it was confirmed that a mechanical anchor had been used. A sketch of this arrangement has been enclosed. Refer to sketch Sk1. Calculations were completed by our office in accordance with the National Building Code 2005 requirements for wind and seismic design. It revealed that there was sufficient capacity to resist the governing wind condition. Since the sampling completed to date has been small, and there was a unit without mechanical anchorage, it was the recommendation of all parties that a method of
site verification be developed as window units are removed, to verify the existence of these anchors. Our office has been in discussion with Stantec to determine the best non-destructive testing approach to complete this inspection. In the near future, a methodology will be forwarded, along with costing for the work. Since the location of the intended anchors is accurately known, we are optimistic that a method can be used that will avoid removal of concrete block. There is no remediation planned for the sills, and in view of the fact that the framing for the new window system will be set further out on the face of the wall, the likelihood of sill movement is extremely unlikely.

The header section bears on a galvanized shelf angle which was bolted, and field welded to a structural steel box shape. The reader is referred to sketch Sk 10. The original drawings indicate that a 0.5 inch rod was to be welded on the surface of the galvanized angle, which would fit into a recess of the limestone, preventing bottom kick out. In has observed movement of the sills at other locations, and it was considered appropriate to remove a header to confirm the presence of the rods, and determine if corrosion has occurred. Leaking of the roof in the area of the parapets has been reported in the past, which may have enabled water to travel behind the stonework, creating a possibility for corrosion, and stone deterioration from freeze/thaw action.

An additional concern was raised by our firm with respect to the presence of horizontal cracking on many of the limestone pieces. This presents another risk, that being delamination. The sill was removed from site and was examined using digital radiography, completed at our facility. A copy of the report prepared by our office has been attached to this report in Appendix “A”. The use of digital radiography was experimental, since we are not aware if it has been used on limestone, and generally, the technique is used to accurately locate reinforcing steel. Using calibrating blocks of steel and concrete (with an intentional defect installed) within the image, the process was unable to identify internal cracks.

The piece was subsequently transported to Stantec for additional testing. From their preliminary review, they confirmed the piece to be limestone. Inspection using magnifying equipment confirmed the presence of longitudinal through cracks in the piece. Given the number of cracks and their length, epoxy injection is not considered a viable option for repair. However, a procedure for pinning the section together, either mechanically, or by grouting a number of holes along the length under low pressure, may ensure continued long term performance of the vertical sections. A diagram of the proposed repair is contained in sketch Sk9. We would recommend that a repair methodology be developed based on this concept, and a unit exhibiting cracking be selected for repair and evaluation. The piece should be located in an area accessible by man lift. In developing the procedure, we might suggest the casting of several precast units having intentional internal debonding. Some of these could be repaired. Flexural, destructive testing could then be completed to evaluate the ability of the repair procedure to improve the capacity of the units. Refer to sketch Sk8 for a depiction of this concept.

Cracking of the face brick at the northwest corner of the tower has been a concern for considerable time. An investigation was initiated in 2001 by Newfoundland Design
Associates Limited leading to a report entitled “Brick Repair - Confederation Building East Block, Tower Section”, a copy of which has been attached for reference purposes in Appendix “B”. The document also identifies work completed as early as 1998 on the subject. The NDAL investigation was unable to conclusively state the cause of the cracking but did speculate that the absence of control joints and the rigidity of the wall system was the most likely mechanism. Following the report a repair was completed, which included the construction of a vertical control joint, additional steel brackets, brick ties and replacement brick.

Cracking was once again observed in this area by [redacted] during his inspection of the façade in 2009. Once again the cause was not obvious, but because of visible signs of interior structural remediation, [redacted] speculated that the center of resistance of the structure may have changed, resulting in torsional affects that might cause brick failure. Therefore, it was his recommendation that a structural engineer be engaged to complete an analysis of the building to determine how the structure behaves under wind loading. Our firm was requested to provide this service, and the first course of business was to obtain drawings of the building so as to prepare a fee proposal. In addition to the NDAL report, we received a disk containing structural drawings for the East block, dating from 1958 to the present.

As we read the NDAL report we noticed what appeared to be a downward displacement of the brick on the north face. We refer the reader to photographs 2 and 3 of the NDAL report. Separation of the mortar joint near the top of the tower in photographs 18 and 19, and snapping of the header bricks in photograph 9 were also noted. Significant deterioration of mortar beds just above the through wall flashing above the north wing roof was also obvious from photographs 21 and 22. These deficiencies suggested that vertical support of the wall, perhaps at the north wing roof, had been compromised.

As reported in the report of [redacted], the Larson system was used for the construction of the building envelope. The interaction between the steel frame and this masonry system is quite complex. Unlike today’s construction techniques, where there is an intentional separation of the steel and masonry to allow vertical deflection under gravity load, the vertical face of masonry is continuous while the backup block is interrupted at the floor elevations. The interior concrete block may be installed tight to the underside of the spandrel beam, in which case steel and masonry act as a composite wall, with no ability for the beam to deflect before load is transferred to the level below. This is depicted on sketch Sk11. There may also be a situation where a gap exists below the beam, allowing deflection of the beam and load transfer through the face brick. Refer to sketches Sk12 and 14. This created deflection compatibility issues between the rigid masonry and the more flexible steel beam. This latter condition can produce some interesting load transfers within the wall. The face brick is bonded to the interior masonry with mortar, and is also supported to the interior block by header courses at every sixth course, as shown on sketch Sk7. From an examination of the structural drawings, it revealed that a wide plate was welded to the top flange of the spandrel beams to support the interior portion of the wall, but only the cantilever performance of the headers prevents the entire weight of the brick from being resisted by the vertical mortar.
bond or support at the bottom of the wall, in the plane of the north wing roof. Complicating this picture is the flexibility of the support that might exist at the north wing roof. At the corner column location, the interior block terminates at the steel column and only the face brick wraps the column. The structural drawings show a horizontal angle welded to the column at the north and west face, at the same elevation as the top beam plate of the beam. Refer to sketch Sk3 to Sk6. This does provide support for some filling in with concrete block, and the ability to embed the header brick at every sixth course. However, from our experience, masonry that is installed within column flanges tends to be chopped up, loose and debonded from the column flanges, decreasing the performance of the headers.

During a review of the list of over four hundred structural drawings, we highlighted a project completed in 1992. This involved the installation of a new elevator, which travels from the eighth to the eleventh floors. A stairway from the eighth floor to the eleventh floor had existed from the original construction. It was necessary to remove the stair, and the floor construction at the eighth level. A new floor was constructed at an elevation 1.5 meters below the eighth level, creating an elevator pit, and the reaction point for the elevator. In addition, vertical guide posts were erected on the floor, which were tied back to each spandrel beam, providing horizontal guide rail support at each floor level.

The discovery of this structural modification in the approximate location of the bottom of the crack was considered extremely important and was identified as a possible contributing factor for the creation of the cracks. Given the complexity of the interaction between the building frame and the rigid masonry, introduction of a repetitive, dynamic load, suggested that joint fatigue failure of the mortar may have commenced some time after the elevator was placed into service. Repetitive, dynamic loading could potentially lead to crack growth, followed by water penetration and subsequent freeze/thaw action. The repairs undertaken in 2001 would have repaired the brick, but continued operation of the elevator might explain why the crack reopened.

Given the two scenarios presented above, we believe that some investigative work is required to either verify or reject these potential causes, prior to proceeding with a much greater task of modeling the entire structure. Such a modeling is a labor intensive effort. The purpose for such modeling is to determine if stress “hot spots” exist because of building shape. In the mid 1980’s, major renovations were undertaken to the interior of the tower, specifically relating to interior stair design. We know of the existence of the project, but as of yet, have not determined the extent of structural renovations that occurred at that time. Future modeling would create the structure as originally intended, and a second model modified, incorporating the significant structural changes that occurred in the mid 1980’s. A comparison of the two models under identical load conditions might highlight a change in performance that could explain a build up in stress in the North West corner. The intricacy of the model is dependent on the overall purpose. Not every structural component needs to be included, and therefore, some short cuts can be taken.
Unfortunately, there are no guarantees as to what may come out of such an analysis. There may not be a "hot spot". When structural engineers design buildings, they make simplifying assumptions. One of those is that only key structural components, such as bracing, shear walls, or rigid frames participate in the lateral resistance of the building. Many of the exterior walls and all of the interior walls are ignored, since it would be impossible to correctly model them in the first instance, and removal of such walls in the future, could compromise the structure from a lateral load resisting perspective. The simplifying assumptions make it easier to design a building, but make it difficult to compare predicted and actual performance.

For this reason, we would propose to hold off on the development of a computer model, until the more localized theories have been properly assessed. We have already extracted the drawings and details of the localized area of the floor plan. Blow ups of these areas are contained in Appendix "C". We have initiated the procurement of load information from the elevator supplier, and shortly, a visual inspection of the structure will be completed in localized areas, and are attached for reference purposes. Those areas include a close up inspection of the framing in the north wing roof immediately below the north tower wall. This will provide information with respect to what actually supports the face brick of the tower, and if there has been any evidence of modification, deterioration, or vertical deflection of the steel. Refer to sketch Sk13.

The second area of focus will be in the area of the elevator. A visual inspection at the elevator floor will be completed to determine if the exterior cracking has propagated through to the interior. It will enable us to determine if the 1992 design was constructed as intended. It will also provide us with an opportunity to investigate the possibility of monitoring structural movement over a period of time. By use of accelerometers and strain gauges, it may be possible to correlate the forces created by the deceleration of the elevator with floor movement, and potential vibration transfer to the wall assembly. It may also be able determine wind load affects during that time. The monitoring of the structure will either prove or disprove the theory that elevator construction is playing a key role in wall cracking. The monitoring will also provide a bench mark to compare structural performance, if after subsequent structural reinforcing is determined necessary.

**SUMMARY**

Replacement of the window system is currently underway in the Phase 1 area, and demolition to date has confirmed that the assumptions made during the drawing preparation were accurate. It is important that shop drawings from the window manufacturer provide a set of design calculations that will enable our office to verify that the design load requirements have been met and that the transfer of horizontal and vertical loads has been properly distributed to the existing structure.
Our office is currently assembling information with respect to a methodology that will ensure that anchors for the vertical limestone sections can be verified not only for the ongoing project, but for the remainder of the window replacement program. We have also recommended further investigation into a process whereby the affects of cracking can be quantified, and where required, a rehabilitation process developed to minimize the potential of limestone delamination in the future. In our recent meeting, it was also recommended that a limestone header section and supported stone be removed to verify the existence and condition of the kick out bars, and as well, determine the type condition of other anchorages.

With respect to the ongoing concern regarding brick cracking at the northwest corner of the central tower, plans are in place to inspect the elevator support floor below the eighth level, which may also lead to a testing program that will determine the magnitude of load transferred to the wall system as a result of elevator movement. An examination of the support system at the north wing intersection will also be completed to determine if there are any weak points of construction in this area. Depending on the findings of these two investigations, a decision would then be made on proceeding with a more encompassing computer model of the building.
APPENDIX "A"

DIGITAL RADIOGRAPHY
LIMESTONE SILL INSPECTION
DIGITAL RADIOGRAPHY
LIMESTONE SILL INSPECTION

Prepared for:

Works, Service & Transportation
Confederation Building
P.O. Box 8700
Prince Phillip Drive
St. John's, NL
A1B 4J6

File Number: 4040-36
Date: December 15, 2009

Prepared by: [Redacted]
TABLE OF CONTENTS

1.0 SCOPE OF WORK ................................................................. 1
2.0 SUMMARY OF RESULTS .................................................. 1
3.0 EXAMINATION DETAILS .................................................... 2
   3.1 SYSTEM ................................................................. 2
   3.2 PERSONNEL ............................................................ 2
   3.3 PROCEDURE ........................................................... 2

APPENDICES

APPENDIX A: DIGITAL RADIOPHIC FINDINGS .......................... 4
APPENDIX B: LIMESTONE SILL DIAGRAM ................................ 11
APPENDIX C: PHOTOS .......................................................... 13

LIST OF TABLES

TABLE 1: THICKNESS READINGS .......................................... 1
TABLE 2: EQUIPMENT LIST .................................................... 2
1.0 SCOPE OF WORK

To acquire digital radiograph of one (1) limestone sill approximately 48" long by 8" x 6" square to evaluate for gross internal discontinuities.

2.0 SUMMARY OF RESULTS

Radiographs were taken in six (6) positions. Each radiograph was taken for a total of thirty-two (32) minutes at 30". We achieved a sensitivity of approximately 10-30%. Three (3) radiographs were taken parallel to the 8" plane and three (3) radiographs were taken parallel to the 6" plane. Defects orientated parallel to the radiation plane have the greatest potential for detection. Normally radiography performs best in locating volumetric defects. Ultrasonics is often used in consort with radiography to increase the detectability of planar defects. Ultrasonic inspection was attempted using our standard fault finding equipment but we could not ascertain any reference reflections indicate the ultrasonic beam was highly dispersed and attenuated. A reference standard is included with each radiograph to demonstrate sensitivity. Two holes, approximately 20% and 40% deep were drilled in an 8" thick brick. The holes are readily visible on all radiographs demonstrating the RT technique has the ability to detect aligned planar defects of 20% the sill thickness.

### Table 1: Line Thickness Readings

<table>
<thead>
<tr>
<th>JOINT/LOCATION</th>
<th>SENSITIVITY</th>
<th>FINDINGS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1</td>
<td>20%</td>
<td>NO DEFECT EVIDENT</td>
</tr>
<tr>
<td>1-2</td>
<td>20%</td>
<td>NO DEFECT EVIDENT</td>
</tr>
<tr>
<td>2-3</td>
<td>20%</td>
<td>NO DEFECT EVIDENT</td>
</tr>
<tr>
<td>0-1R</td>
<td>20%</td>
<td>NO DEFECT EVIDENT</td>
</tr>
<tr>
<td>1-2R</td>
<td>20%</td>
<td>NO DEFECT EVIDENT</td>
</tr>
<tr>
<td>2-3R</td>
<td>20%</td>
<td>NO DEFECT EVIDENT</td>
</tr>
</tbody>
</table>
3.0 EXAMINATION DETAILS

3.1 System

The equipment used is listed in Table 2 below:

<table>
<thead>
<tr>
<th>TABLE 2: EQUIPMENT LIST</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TECHOPS GAMMA CAMERA</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>COMPUTER RADIOGRAPHY DIGITAL SCANNER</strong></td>
</tr>
</tbody>
</table>

3.2 Personnel

Section 30(1)

personnel carried out the inspection.

3.3 Procedure

The examination was carried out according to Acuren inspection procedure W1-ACUREN-024, Radiography Examination - Vessels.

Note: Unless otherwise instructed, we shall dispose of all parts and test samples sixty days from the date of this report.
APPENDICES
APPENDIX A - DIGITAL RADIOGRAPHIC FINDINGS
Photo 1: 0-1 32 minutes @ 30 inches
Photo 2: 0-1R 32 minutes @ 30 inches
Photo 3: 1-2 32 minutes @ 30 inches
Photo 4: 1-2R 32 minutes @ 30 inches
Photo 5: 2-3 32 minutes @ 30 inches
Photo 6: 2-3R 32 minutes @ 30 inches
APPENDIX B -
LIMESTONE SILL DIAGRAM
APPENDIX C - PHOTOS
Photo -1:

Photo -2:
Photo - 3:

Photo - 4:
Photo -5:
Brick Repair - Confederation Building
East Block, Tower Section

Project No. 119905035

December 2001

Prepared for: Department of Works, Services & Transportation

Prepared by: Newfoundland Design Associates Limited
ATTENTION: Mr. Derek Button, P. Eng.

Dear Sir:

RE:   BRICK REPAIR - CONFEDERATION BUILDING
       EAST BLOCK, TOWER SECTION
       PROJECT #:  119905035

We have carried out an inspection of the fractured brick work on the exterior northwest corner of the Confederation Building East Block Tower to assess the extent of damage and to recommend immediate repair options. Please find attached our report detailing findings, conclusions and recommendations.

I trust you find the above satisfactory. If we can be of further assistance with this or any other matter, please contact us at your convenience.

Yours truly,

NEWFOUNDLAND DESIGN ASSOCIATES LIMITED

[Redacted]
Project Engineer
TABLE OF CONTENTS

1.0 INTRODUCTION .................................................. 1

2.0 BACKGROUND .................................................... 2
   2.1 Reports ....................................................... 2
   2.2 Drawings .................................................... 2

3.0 SITE INVESTIGATION ............................................. 3
   3.1 Exterior Observation ....................................... 3
   3.2 Elevator Shaft ............................................... 4
   3.3 Restoration .................................................. 4

4.0 CONCLUSIONS .................................................... 6

5.0 RECOMMENDATIONS ............................................... 7

List of Appendices

Appendix A - Photographs
Appendix B - Sketches
Appendix C - Technical Notes
1.0 INTRODUCTION

The Firm of Newfoundland Design Associates Limited was engaged by the Government of Newfoundland and Labrador, Department of Works, Services and Transportation, to investigate a fracture in the brick veneer at the Northwest corner of the Confederation Building East Block Tower Section and to prepare details for the immediate repair in accordance with the proposal as submitted by Newfoundland Design Associates Limited on September 11, 2001.

Generally, the scope of work included:

- Close-up visual inspection of the damaged brick area.
- Gather and review pertinent details to evaluate the problem.
- Obtain advise and opinion from professional architect.
- Determine the cause or causes of the problem where possible.
- Recommend details for immediate repair through approval with the Department of Works, Services and Transportation.
- Submit a report summarizing findings.
2.0 BACKGROUND

2.1 Reports

The following reports and correspondence were obtained from the Department of Works, Services and Transportation:

- East Block - Confederation Building correspondence, September 2000, Design Services Ltd.
- Confederation Building - East Block correspondence, August 2001, Design Services Ltd.
- Exterior brick at Northwest Corner of the East Block Tower - Confederation Building, August 2001, AMEC E&C Services Ltd.

Reference was also made to the Brick Industry Association Technical notes and various other texts relating to the subject.

2.2 Drawings

The following applicable drawings were obtained from the Department of Works, Services and Transportation:

- Major Renovations to Confederation Building, East Block by BFL Consultants Limited:
  Sheets W-S-021.1 to W-S-024.4;
  Sheets W-A-3.1 to W-A-6.2;
  Sheets W-S-3.1 to W-S-6.1; and
  Sheets W-M-3.1 to W-M-6.1
3.0 SITE INVESTIGATION

3.1 Exterior Observations

In addition to a review of existing reports and drawings, a close visual inspection of the exterior building envelope was carried out. This necessitated the partial demolition of the brick masonry facade in the vicinity of the affected area. A photograph log is included in Appendix A.

The building structure is steel frame with concrete block masonry infill and brick veneer. The concrete block and brick are interconnected to form a solid wall system, but with a felt membrane sandwiched between. Wall section drawings obtained through DWST show a 1 inch air space between the brick veneer and concrete block, however inspection of the actual construction revealed that the void is filled with grout. The brick and header block are tied together by continuous header bricks at every 6th brick course. At this point, the waterproof membrane appears to be discontinuous. The sketches contained in Appendix B show the wall construction.

We were unable to state conclusively, the cause of the cracks in the exterior brick, however the following observations and comments are offered:

- Our initial inspection was on September 11, 2001. The vertical crack had been present long enough to permit the growth of moss, and it appeared that freeze/thaw cycles most likely resulted in the deteriorated condition.

- Mugford’s Contracting Ltd. (retained by DWST) were then instructed to remove the damaged brick work on each side of the vertical crack to a “firm hold”. Approximate limits of 0.6m x 20.0 m were established for removal.

- During our second inspection on September 28, 2001, we were able to view the masonry backup wall and interlocking brick system. Discussion with the work crew and close-up inspection verified that header bricks were split in two.

A small section of masonry block back-up was also removed at approximately the 8th Floor level to expose the structural frame and interior masonry wall details. It was observed that the steel column is encased by the masonry construction and is rusting.

At the time of the observation, the inner brick work was wet, thus confirming the penetration of moisture and the likelihood that freeze/thaw cycles are contributing to the condition of the brick.
On October 4, 2001, a visual inspection was performed on the north face at the west corner to further define the extent of damage. Overall, the brick veneer on the north face appears sound with the exception of a horizontal crack in the mortar bed approximately 1.0 m down from the top of the brick face. We also observed several areas in joint caulking that could permit the entry of water into the masonry construction.

3.2 Elevator Shaft

On October 12, 2001, the exterior wall of the elevator shaft was inspected in an attempt to further define the wall section detail and to investigate the possibility of the fracture propagating through the wall assembly. A vertical crack in the comer of the elevator shaft opposite the exterior fracture occurrence was observed.

3.3 Restoration

M.R.A.I.C. of Gibbons Hampton Architects Ltd. was retained as a consultant to assist in evaluation of the problem and preparation of repair details. Photographs and measurements of the wall construction were evaluated and drawn to scale using CAD.

Given the time of year and location of the work involved, the urgency of completing the repairs necessitated immediate engagement of the Contractor to do the work. Thus, a repair detail was required that would maintain the existing wall assembly appearance and construction type, prevent the occurrence of further cracking or deterioration, and to the degree possible seal the envelope from environmental loadings.

A vertical control joint at the plane of intersection with the steel column was decided upon as the most favorable option to prevent future unsightly cracking of the brick veneer. Steel angle brackets were incorporated into the repair detail to support the vertical weight of the new brick and tie the brick work to the back-up masonry horizontally, and to accommodate the existing brick pattern. Horizontal brick ties were also used between the steel angle brackets.

On November 9, 2001, we performed a follow-up inspection of the restoration activities and found the repair work to be proceeding as detailed with the exception that several fasteners were relocated slightly in order to achieve secure anchorage.
We also verified that several existing header brick courses were broken past the limit of initial demolition. The brick and mortar remained sound however. “Dur-O-Wall” restoration anchors were installed at 600 mm on centre at every second header course to the horizontal limits of the swingstage assembly to secure the brick veneer to the back-up masonry block. Technical information for the “Dur-O-Wall” anchors are included in Appendix C.
4.0 CONCLUSIONS

In conclusion, we are unable to conclusively state the cause(s) of the initial fracture in the brick veneer at the northwest corner of the East Block Tower. Considering the observed wall construction, and with reference to the observations prepared by the Design Services Board of the Ontario Realty Corporation, the lack of control joints and the rigidity of the wall construction is the most likely mechanism that created the necessary forces to cause the initial crack which then acted as a port for entry of water to further the deterioration through freeze/thaw cycles.
5.0 RECOMMENDATIONS

Due to the time of year, the repair of the affected area had to be carried out immediately. Thus with the Department of Works, Services and Transportation's concurrence, the remedial work as detailed in Appendix B was completed. This remedial work will secure the brick veneer and protect against further deterioration in the short term. We recommend that an in-depth study of the complete building envelope be undertaken to evaluate the condition of the overall masonry wall construction. We further recommend to repair broken caulking and joints throughout the Tower section and remaining building to prevent water infiltration and to install control joints as required throughout the building to prevent future cracking and deterioration of the exterior wall construction.
Appendix A
Photographs
Confederation Building East Block
Tower Section
Brick Repair
October 2001

2  Initial Inspection Window on Fracture

3  Fracture in Brickwork
Confederation Building East Block
Tower Section
Brick Repair
October 2001

4  Affected Area Extends Beyond Corner

5  Moss growing in Crack
Confederation Building East Block
Tower Section
Brick Repair

October 2001

Evidence of Previous Repair

Note Back-up Block Construction and Membrane
Confederation Building East Block
Tower Section
Brick Repair

October 2001

8 Note Fractured Header Bricks

Fractured Header Bricks - Photograph 8 Continued
Confederation Building East Block
Tower Section
Brick Repair
October 2001

Extent of Brickwork Removed

Typical Construction - Note air cavity filled with grout at corner
Masonry back-up block removed to expose steel frame

Note rust accumulation on splice bolts
Typical wall construction

North Face at Top of Brick Course
Confederation Building East Block
Tower Section
Brick Repair
October 2001

16. Cracks in mortar in granite panels above brick face

17. Crack in caulking around flashing at top of brick course
Horizontal crack in mortar bed on North Face at 11th Floor

Continued from Photo #18. Note opening in caulkin above crack
Mortar condition on North Face at 6th Level

Inspection window inside Elevator shaft
Confederation Building East Block
Tower Section
Brick Repair
October 2001

22. Cracking in plaster inside elevator shaft

23. Crack in plaster inside elevator shaft at 11th Floor Level
Impression of Former Stair Landing in Elevator Shaft
Bricks removed for inspection of Header Bricks
Confederation Building East Block
Tower Section
Brick Repair

November 26, 2001

New Brick Support Angle

Veneer Anchor and Setting Tool
Confederation Building East Block
Tower Section
Brick Repair

November 26, 2001

Completed Repairs
APPENDIX “B”

BRICK REPAIR – CONFEDERATION BUILDING
EAST BLOCK TOWER SECTION
Appendix B
Sketches
EXISTING CONDITION
PLAN DETAIL - EXTERIOR CORNER

8" +/- BRICK OR
8" +/- BRICK/BLOCK

8" +/- BRICK/BLOCK OR
8" +/- BLOCK (ASSUMED)

VERTICAL CRACK IN BRICK
HORIZ. LOCATION VARIES
HORIZONTAL CRACK IN HEADER BRICK
(HORIZONTAL WIDTH VARIES)

GROUT FILLED VOID
FELT MEMBRANE

FACE OF FLANGE
CAVITY IN SOME LOCATIONS

PLASTER

Section 30(1)
NEW PLAN DETAIL @ EXTERIOR CORNER

**NEW BRICK AT CORNER**

**CAULKING**

**BACKER ROD AND CAULKING**

**NEW 16mm CONTROL JOINT** *(TO EXTEND FULL HEIGHT OF BRICK FACE)*

**NEW L3"x3"x1/4" BRICK** *SUPPORT BOLTED TO EXISTING HEADER BRICK AS DETAILED*

**NEW BRICK**

**HILTI EPOXY ANCHOR**

**HIT-HAS + HY 150**

**3/8"012" C/C MAX.** *(INTO SOLID EXISTING HEADER BRICK COURSE)*

**EXISTING CONCRETE BLOCK**

**CAVITY**

**INSTALL NEW "BLUESKIN" OR EQUAL PEEL AND STICK AIR VAPOUR BARRIER**

**FILL JOINT WITH MORTAR**

**MASONRY THIS LOCATION UNKNOWN**

**EXISTING WALL CONSTRUCTION**

---

**NEW PLAN DETAIL @ EXTERIOR CORNER**

---

**ARCHITECTURAL**

**BRICK REPAIR - CONFEDERATION BLDG. EAST BLOCK TOWER SECTION**

---

**DETAIL**

**SK-2**

---

**Section 30(1)**
CAULKING

NEW BRICK SUPPORT ANGLE BOLTED TO EXISTING HEADER BRICK. LOCATE AT EVERY 4th HEADER BRICK

NEW BRICK

FILL JOINT WITH MORTAR

NEW BRICK TIES (GALV) AT ALL HEADER BRICK EXCEPT AT ANGLE LOCATIONS

EXISTING FELT MEMBRANE

NEW BRICK SUPPORT ANGLE BOLTED TO EXISTING HEADER BRICK. LOCATE AT EVERY 4th HEADER BRICK

CAULKING L3" x 3" x 1/4" HOT DIPPED GALV.

EXISTING BRICK

EXISTING TIE (HEADER) BRICK

EXISTING BRICK

EXISTING BLOCK

INSTALL NEW "BLUESKIN" OR EQUAL PEEL AND STICK AIR VAPOUR BARRIER AS PER MANUFACTURERS RECOMMENDATIONS

EXISTING WALL CONSTRUCTION

HILTI EPOXY ANCHOR HIT-HAS + HY 150 3/8" O12" c/c

SECTION A-A
Appendix C
Technical Notes
SERIES ANCHOR: 5000

<table>
<thead>
<tr>
<th>APPLICATION</th>
<th>ANCHOR SELECTION</th>
</tr>
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<tbody>
<tr>
<td>VENEER</td>
<td>BACK-UP</td>
</tr>
<tr>
<td>Brick</td>
<td>Brick</td>
</tr>
<tr>
<td>Solid Brick</td>
<td>Solid Brick</td>
</tr>
<tr>
<td>Precast</td>
<td>Precast</td>
</tr>
<tr>
<td>Stone</td>
<td>Stone</td>
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<tr>
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<td>5005074</td>
</tr>
<tr>
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<td>5005084</td>
</tr>
</tbody>
</table>

The 5000 series anchor provides an excellent method of reanchoring a solid facade >3" to various solid backups. The anchor is installed by drilling a standard 1/2" masonry hole through the veneer into the back-up at a "T" joint location. Anchor placement edge distance = 6", 1 anchor per 2-4 ft" of masonry. Anchors are installed with the 5550001 setting tool, via 50-100 lb torque. Custom lengths available upon request.

<table>
<thead>
<tr>
<th>SERIES 5000</th>
<th>ULTIMATE CAPACITY</th>
</tr>
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<tbody>
<tr>
<td>VENEER MATERIAL</td>
<td>COMPRESSION (lb)</td>
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<tr>
<td>BACK-UP</td>
<td>avg</td>
</tr>
<tr>
<td>Brick + Precast/CC + Brick</td>
<td>300</td>
</tr>
<tr>
<td>Brick + Precast/CC + Precast/CC + Brick</td>
<td>929</td>
</tr>
<tr>
<td>Brick + Precast/CC + Precast/CC + Brick</td>
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<td>Precast/CC + Precast/CC + Brick</td>
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<tr>
<td>Precast/CC + Precast/CC + Precast/CC + Brick</td>
<td>2169</td>
</tr>
</tbody>
</table>

SPECIFICATION REFERENCE FOR DUR-O-WAL
FACADE REPAIR ANCHOR SERIES: 5000 - Recognized ICBO 4575

GENERAL

PRODUCTS

Components

Execution

Hole Size

Anchor Length

Drilling Technique

---

*Substrate or substrate should not exceed ultimate design performance. Refer to anchor performance characteristics before performance specifications.
APPENDIX "C"

LOCALIZED TOWER FLOOR PLANS
APPENDIX “D”

STRUCTURAL SKETCHES
NOTE:
HEADER ANGLE ASSUMED TO PERFORM AS SHELF SUPPORT
PRE-CAST CONCRETE TEST SPECIMEN

POLYETHYLENE SHEET FOR CRACK INITIATION

SECTION
N.T.S

TEST LOAD TO RUPTURE

SAMPLES
- SOLID PIECE
- CRACKED PIECE WITHOUT REPAIR
- CRACKED PIECE WITH REPAIR

RON FOGUERE ASSOCIATES
CONFEDERATION BUILDING
WINDOW REPLACEMENT

DATE  SCALE  REV. No.  DWG. No.  JOB No.
09.12.16  1:10  A  SK8  4040-36
IN-PLACE CRACK STABILIZATION

MASONRY

ZONE OF EPOXY PENETRATION

NATURAL BED CRACK PLAINS

SECTION
N.T.S

RON FOUGERE ASSOCIATES
CONFEDERATION BUILDING WINDOW REPLACEMENT

DATE
09.12.16

SCALE
N.T.S

REV. No.
A

DWC. No.
SK9

JOB No.
4040-36
2x3/6" TIE PLATES @ 1'-6" CENTERS

C12x23

ALIGNMENT BOLTS TO BE REMOVED AFTER WELDING

12mm BAR
150 LONG
150 SPACING

6x4x3/8" Z
(GALVANIZED)

4"

LIMESTONE

TYPICAL WINDOW HEADER
DEFLECTION

LOAD TRANSFER
DIRECT TO BEAM WITH POTENTIAL FOR BEAM DEFLECTION

GAP ASSUMED
INFORMATION REQUIRED ON BRICK SUPPORT

NORTH WING ROOF

THIRD FLOOR
FLOOR LOAD

STRESS REGION DUE TO DEFLECTION INCOMPATIBILITY

DEFLECTION DUE TO WEIGHT OF WALL ABOVE AND BEAM REACTION

POTENTIAL TENSILE STRESS BUILD UP

DEFLECTION

EXAGGERATED BEAM DEFLECTION/ ROTATION