# LOWER SOUTH SIDE STANDS

**at**

## LANSDOWNE PARK
## FRANK CLAIR STADIUM

## STRUCTURAL ADEQUACY REPORT 2007

**for**

## CITY OF OTTAWA
## REAL PROPERTY ASSET MANAGEMENT

**prepared by**


## ADJELEIAN ALLEN RUBELI LIMITED
## CONSULTING ENGINEERS

ADJELEIAN ALLEN RUBELI LIMITED CONSULTING ENGINEERS OTTAWA / TORONTO
75 Albert Street, Suite 1005, Ottawa, ON K1P 5E7 tel (613) 232-5786 fax (613) 230-8916 email aar@aar.on.ca


**September 2007**

**Structural Adequacy Report**
**AAR Reference No. 3061-26**

INDEX PAGE

## 1.0 Executive Summary:

Adjeleian Allen Rubeli Limited was engaged by the City of Ottawa to undertake a Structural Adequacy inspection of the Lower South Side Stands, a structure constructed in 1961. This inspection was one of a number of structural adequacy inspections of the entire Lansdowne Park-Frank Clair Stadium and Ottawa Civic Centre facility.

Our review was comprised of two parts:

- Visual examination of the structural elements of the Lower South Side Stands.
- Strength evaluation of critical portions of the structure with reference to the requirements of the 2006 Ontario Building Code.

A visual review of the structure indicated that, in our opinion, the structure is in poor condition with numerous indications of local distress and deterioration. A design check for structural adequacy, as determined by the 2006 Ontario Building Code, was undertaken for two reasons:

- Visual evidence of distress and deterioration of the Lower South Side Stands.
- The knowledge of the considerable reduction in shear capacity of beams without shear reinforcing compared to the code requirements prior to 1965.

A comparison of the 1960 NBC and the 2006 OBC (including the 2004 CSA A23.3, Standard on Concrete Design), revealed that the upper limit of stress in a beam, before shear reinforcing is required, is considerably lower than that permitted by the 1960 NBC. The design check for shear, particularly at sections of beams where no shear reinforcing steel was present, showed that the shear at certain sections exceeded, to a considerable extent, the shear capacity as determined by the 2006 OBC. In addition, there is evidence, from the visual inspection, of structural distress at some of the areas of high shear stress. Shear failure of beams without shear reinforcing is known to be sudden without any significant advanced warning.

It is our opinion that the Lower South Side Stands structure has reached the end of its service life unless extensive repairs are carried out to deal with both the deteriorated concrete and the over-stress in shear of critical portions of beams within the structure.

There are two options available to the City, in our opinion:

Option 1. Demolish the Lower South Side Stands structure.

Option 2. Repair defects and strengthen beams to meet current code requirements in order to prolong the life of the Lower South Side Stands.

It should be noted that for option 2, a seismic upgrade to current code levels would be required, in our opinion, in keeping with the "Arenas Structural Adequacy" Guidelines for Professional Engineers which advises that upgrades and repairs to structures designed prior to 1970 be upgraded to conform with the latest code in effect.

Furthermore, it is our opinion that the restoration costs for Option 2 could exceed the overall cost for a new structure, particularly when one considers that the extended life of the repaired structure will be shorter than that of a new facility.

Without remedial action indicated in this report, it is recommended that the Lower South Side Stands portion of the Frank Clair Stadium be closed for pubic occupancy.

## 2.0 Terms of Reference:

Adjeleian Allen Rubeli Limited was requested by the City of Ottawa to undertake a Structural Adequacy inspection of the Lower South Side Stands of Lansdowne Park. The assessment of the structural condition is based on the Guidelines for Assessment of Arenas for Structural Adequacy, as published by the Professional Engineers Ontario, January 1990.

The Ontario Ministry of Labour required that indoor arena structures be inspected at regular intervals. This guideline, which is no longer mandated for long span structures, was used as a guide during the assessment of this structure and includes the following key elements:

- Determine the general condition of the structural elements.
- Assess the ability of the structure to withstand loads specified by the Ontario Building Code (OBC).
- Seismic evaluation of the structure with reference to the current codes was not undertaken as part of this review and all references to loading, within this report, are related to gravity and wind loading on the building.

## 3.0 Building Description:

The South Side Stands of the stadium consist of two independent structures referred to as the Upper and Lower Sections. This report specifically references the Lower Section of the stands only and a separate report has been prepared for the Upper Section.

The South Side Stands are part of the Frank Clair Stadium located at Lansdowne Park in Ottawa. The Lower Stands were designed by Adjeleian Goodkey Weedmark and Associates Limited with drawings dated 1960. Construction was reported as completed in 1961.

The stands were constructed as a reinforced concrete structure using a combination of precast or cast-in-place elements supported on spread footings.

The concrete frames consist of cast-in-place sloped raker beams supported and cast over the top of three precast columns with cantilevered ends at both front and back. The top surface of the raker is stepped to facilitate the placing of the precast seating panels directly onto the frame. The frames have three lines of wind beams, either cast in place with the raker beams or welded to column cast-in-plates, connecting the frames together transversely.

The precast seating panels are detailed as doweled at each end to the frames except at frame line 10 where an expansion joint is located.

Access to the seating area consists of three ramp systems constructed of precast elements. Each ramp system connects to two separate vomitories. The precast panels are either supported by the concrete frames, or a support column. The precast elements are typically detailed as bearing supports and pinned at each support point. However ramp section R3 and ramp beam RB3 are hung. Ramp section R3 is hung off the side of the cantilever portion of ramp section R2 with ramp beam RB3 hung off the frame column corbels.

The back of the seating area had a steel canopy roof that was supported at the top end of each raker beam. The canopy was partially removed when the upper stands were constructed.

The ground level below the stands contains concession booths, a concourse, locker rooms, washrooms, mechanical rooms, electrical rooms and storage areas. The ground level service buildings were constructed of load bearing block walls with a structural steel roof. The rooms were built either with the stands in 1961 or later as an addition. The structural drawings for these areas, both original and additions, were not located in Adjeleian Allen Rubeli archives and were not required, in our opinion, for this review.

The Upper Stand Section was reported as constructed in 1975 and is located directly behind and extending over a portion of the Lower Stands. The Upper Stands has neither a structural connection nor an interconnected access for patrons either at or above grade.

## 4.0 Scope of Work

The adequacy review involved a visual examination of the critical structural elements including precast seating units, concrete frames, access ramps and visible structure within the washrooms, storage room and locker room below the stands.

Reference was made to the original drawings, previous reports and correspondence to identify potential problem areas in conjunction with visual observations and to review original design loads.

The building drawings are dated December 1960. The building code that was assumed to be used for design was the 1960 National Building Code (NBC) and ACI A318. The 2006 Ontario Building Code (OBC) and CSA A23.3-04 were used for reference as the current codes for loading and design evaluation.

The site review of the condition of the structure is based on direct visible observations at the time of the review. Any structural elements which were inaccessible or covered by finishes were not reviewed and are not included in this structural assessment.

This report is intended for the sole use of the named Client, the City of Ottawa.

## 5.0 Observations:

Site work was undertaken with the assistances of a restoration Contractor, ER Jonas Limited, during the week of August 7 to 10, 2007. The visual inspection included walking all areas of the building and using field glasses to inspect the upper areas. The areas noted during the field glass review were accessed with the use of a telescopic lift to allow for close examination of the upper areas. Areas that were not noted as a concern during the field glass review were also randomly reviewed from the lift.

## 5.1. Concrete Frames:

Concrete delamination, spalls and cracks were noted at various locations on the frames.

### 5.1.1. Columns:

The column base plates and anchor bolts were noted as not painted with surface corrosion observed. The corrosion was noted as not severe.

A delamination was noted on column C4 as noted in photo 03 & 04. This condition is located at the ramp landing and is associated with an as found interconnection between the ramp system and the column. The column was not originally connected to this ramp; however, it appears that an inadvertent connection was made resulting in this local distress.

Deterioration was noted in the columns such as at grid location C2. This could have initiated as cracks as a result of local bending distress. With time, water would have entered the cracks causing rusting of the reinforcing steel and additional cracking and delamination. See Photo 02.

Large concrete delaminations were found at the top of columns at grid locations C7 and C9. The concrete was sounded and determined that the concrete could spall within the next two years; as such the Contractor was instructed to remove the potential defective concrete. The reinforcing steel at grid location C7 was exposed with corrosion evident on the vertical corner bars and the ties. See Photo 01. The column ties were found to have corrosion with loss of area while the vertical bars were noted to have surface corrosion only.

Concrete delaminations found at other columns, such as along grid line A, were not removed as the risk, as a result of falling and causing damage, was considered minor.

### 5.1.2. Sloped Raker Beam

The cast-in-place concrete beams on frame lines 2, 3 and 4 were noted to have structural steel added to provide both flexural and shear reinforcing for the beam at grid line A. See Photo 05. The year in which the beams were reinforced was unknown and was reportedly required to reinforce the beams after the area was overloaded as a result of a temporary loading condition not related to the intended use of the seating.

Some of the raker beams were noted at to have shear and / or flexural cracks in the either the front cantilever section and / or the interior span at column line A. See Photo 07 & 08. The cracks in raker beam 5 were measured with the largest recorded in the range of 2 mm at a location between the column on Line A and RB1. This width is substantially larger than a 0.3 mm hair line crack which could be expected for acceptable performance of reinforced concrete structures.

Concrete delaminations were noted in these raker beams in correspondence written in 1994 and 1998 when portions of the stands were reviewed.

Concrete repair areas previously undertaken to both the top and bottom of the beams were randomly hammered and noted as being in good condition.

Concrete delaminations were found at various locations of these beams not previously repaired. The areas were sounded to determine the extent of delamination and were typically found to indicate that corrosion of the reinforcing was ongoing. The delamination along frame line 19 was noted as major and the sounding indicated that, in our opinion, a spall could occur within two years; as such the Contractor was instructed to remove the concrete as evident in photo 06. Upon removal it was noted that corrosion of the reinforcing was substantial and loss of reinforcing area was evident.

The connection of the canopy roof structure, now partially removed, to the raker beams were reviewed randomly with no concerns identified.

### 5.1.3. Wind Beams:

No distress or deterioration was noted.

### 5.2. Precast seating:

The precast seating panels were reviewed from both the top and bottom. The underside was reviewed with field glasses and then random areas were then reviewed from the lift.

Concrete delaminations were noted in these precast panels in correspondence written in 1988 and 1998 when portions of the stands were reviewed.

The precast seating panels have been repaired at various times over the life of this building with the repair patches typically as evident in Photo 09. The latest known date of concrete repair was reported as 2004. The repair patches were randomly hammered with patches noted as sound and in good condition.

Concrete delaminations were found at various locations on the soffit of the panels not previously repaired.

The upper surface of the precast seating sections was found to have local areas of spalled concrete with the reinforcing steel exposed and deteriorated as evident in Photo 10.

The seating connections to the precast panels were found to have no noted distress.

### 5.3. Ramps:

### 5.3.1. Precast ramps:

The ramps consist of four different sloped precast panels with one precast landing section. These panels, as indicated on the original drawings, are pinned at each support. See appendix B for reference ramp locations. Each section of ramp consists of two precast sections with a left and right panel, as indicated on the original drawings.

The ramps were noted to have local areas of new delaminations or areas where the delamination now extends beyond the original repair area.

The areas that were previously repaired were randomly sounded with various patches found to be again delaminated as evident in photo 11. The areas where the patches have again delaminated were typically where ramp section R3 is hung off ramp section R2. Ramp section R2 has a crack, as evident in photo 12, down the centre of the cantilever portion that supports the hangers. This condition was noted to varying degrees at a majority of these ramp locations.

Ramp section R4 had a section replaced with cast-in-place concrete in 2001. The reason for the replacement was not available.

Ramp section R4 was noted to have cracks in the beam near the support at the column tee as evident in photo 13.

The landing slabs were noted to have cracks in the top of slab that have existed overtime as evident in photo 14. Diagonal cracks are evident at the side of the panels at the supports that align with the top of slab crack as evident in photo 21.

### 5.3.2. Ramp support beam:

The three ramp support beams were reviewed with ramp beams RB3 along grid line C, at the bottom of the ramp, noted to have the majority of deterioration. These beams are the only ramp beams with previous repairs evident.

Ramp beams RB1 and RB2 were noted to have local distress / deterioration at one location only. Ramp beam RB2 is cracked at the support at frame line 6 as evident in photo 15.

Ramp beams RB3 were noted at this time to typically have visible deflection (photo 16) with varying forms of longitudinal and transverse cracks (photo 17). In addition, these beams have areas of delamination which are either new or at and beyond previous repairs (photo 18). The brackets supporting these beams at the columns were noted to have two different repair details with the most recent repair detail including a new corbel support to the column. The beam connections to the columns were all typically noted as cracked.

### 5.3.3. Ramp Support Columns:

The ramp support columns are precast and T-shaped as evident in photo 19 with the stem having a steel base plate bolted to the foundation. The ramp slabs are pinned to the top of the tee portion of these columns.

The sloped ramp sections R3 and R4 are supported by both the top of the tee and supplementary structural steel brackets that were previously installed as a remedial detail for additional bearing support as evident in photos 19 & 21. The angles were bolted through the column tee portion.

The tee portion of the column has longitudinal and transverse cracks with diagonal longitudinal cracks evident at the ends. The angle and through bolts appear to have been added to reinforce the tee, providing additional support for the sloped ramp sections. The landing ramp side of the tees are now distressed as evident, in photo 21, by the diagonal crack.

## 5.4. Guardrails:

### 5.4.1. Ramps:

The access ramp guardrails consist of reinforced 150mm concrete blocks with a concrete cap as shown in Photo 20. These rails were reconstructed in 2001 and are considered in good condition.

The anchorage of the block railing to the ramp landings was found to have the concrete extension below the slab, as shown in photo 21, to suit the required development of the reinforcing steel into the precast slab. At section BB, many of the railing dowel concrete sections below the slab were noted as missing with deterioration of the slab edge. See photo 04 and 22. The design of the railings required the development of the reinforcing through the precast slab and as such these rails are considered locally as no longer anchored.

### 5.4.2. Seating Areas:

The railing post connections to be precast panels on the sides of the seating area that have been reinforced were found to be in good condition.

The original posts are currently deteriorated. See Photo 24. The railing replacement undertaken in 1990 was to reinforce the railing by adding new verticals to supplement the posts that have deteriorated.

The side sections of railings for seating include a kick stop that is cast on top of the precast seating panels. These sections of concrete were noted as missing at various locations. See photo 06.

The front railing posts and horizontal rails were reconstructed in 1990 and currently have local deteriorated concrete at an anchor location and a section of upper rail is missing. The concrete supporting a post has delaminated as evident in Photo 23. The top railing at the front of the aisle is required to be higher than at the front of seating. It was noted that at one of the aisle locations, the verticals are in place without the top extension rail section, above the seating rail height, as evident in Photo 24.

## 5.5. Ground Level Service building:

The ground level service buildings are located along the full length of the stand between the front and middle column lines. The buildings are typically load bearing block with steel roof framing.

No structural distress or deterioration was evident. It was noted, however, that at frame line 5, a structural steel bracket has been modified with a bolt head cut off as evident in photo 08. This modification was only evident at this location; however, similar modifications may be present at all hidden connections where the freezer was installed.

## 5.6. Expansion Joints

The main building expansion joints were generally noted to be performing well with no signs of deterioration visible from this random review.

## 6.0 Design Loading:

The design loading for both dead and live loads have the same values in both 1960 National Building Code and the 2006 Ontario Building Code.

The superimposed live loading for occupancy of this facility for this type of structure in accordance with both the 1960 NBC and the 2006 OBC is 100 psf.

The building frames are spaced at 18 feet centres with the columns of the frames located 32 feet apart. Depending on the tributary area supported by columns and beams, there are provisions for live load reductions which have changed since the issue of the 1960 Code. For beams which support a tributary area greater than 200 sq. ft., a live load reduction factor of 0.85 was permitted under the original 1960 Design Code. Also, a load reduction factor of .85 for the columns would have applied under the 1960 Code, regardless of the size of the tributary area. The current 2006 Code does not allow a live load reduction factor when the tributary area for beams or columns, for assembly occupancy, is less than 860 sq. ft., a tributary area value which is greater than at any of the columns at this structure.

.

## 7.0 Member Capacity Subject to Shear and Bending Moments:

The determination of member capacities is different in the current codes compared to the codes in 1960. Currently, design is based on “factored loads” and “factored resistances” compared to “service load design” used prior to 1965. Despite this, reinforcing requirements for flexural or bending moments are not significantly different between the code and standards in 1960 compared to today.

For shear resistance, however, there have been dramatic changes in the determination of shear capacities. This is a result of the evaluation of numerous laboratory testing of beams, both reinforced (vertical stirrups) and un-reinforced. The current design standards have responded to the evidence of lower shear capacities of beams compared to previous knowledge with more stringent requirements, particularly beams with no shear reinforcement. The impact of these code changes is the requirement of stirrups at much lower shear stress levels than was required in the codes prior to 1965. These new shear requirements in current codes are particularly important as shear failures in un-reinforced beams are typical non ductile in nature, with failures occurring typically without significant advanced warning.

The visual site review indicated a number of cracks which we believed were shear cracks combined with flexural cracks. An initial review of the structural drawings indicated significant sections of beam elements without shear reinforcement.

As we are aware of the current lowering of shear capacities of un-reinforced beams in shear, and evidence of failures of beams in shear on older structures, we thought it advisable to review portions of the structure at critical shear areas, even if no shear cracks were visible, in addition to beams showing evidence of shear cracks.

Critical areas of the structure reviewed for shear stress would indicate that the shear stress due to design loads did not exceed allowable shear stress permitted by the code in effect at the time the Lower South Side Stands was constructed.

In our opinion, the Lower South Side Stands service life should not be extended without a complete review of the structure and suitable repairs to:

a) Correct the deterioration noted.

b) Upgrade the member capacities to suit current code requirements particularly with regard to beam shear.

## 8.0 Review of Selected Members for Member Capacity

A structural design review was undertaken to assess the structural members that were noted during the visual assessment as distressed and later expanded to critical areas of other members not noted as having visible distress.

The design review was undertaken to assess both the flexural and shear adequacy of the raker beams and ramps at critical areas. The design review consisted of reviewing the concrete section and reinforcing steel at areas determined as critical. The reinforcing steel used in the design analyses check was determined from the original structural drawings. These design reviews were not all inclusive and did not include a full design assessment of all structural elements.

These assessments of the structural adequacy of the noted primary structural elements are as follows:

### 8.1.1. Concrete Frames:

A review of the concrete sections and reinforcing steel at critical areas on the raker beam, would indicate, in our opinion, that the flexural capacity is satisfactory.

At a number of locations on the raker beam, particularly areas without shear reinforcing, the shear capacity in our opinion exceeds that permitted by the current OBC. In many cases, the exceedance of stress levels was very high resulting in our assessment that occupation of this structure by the public was risky.

As an example, the calculated shear value and shear resistance at an un-reinforced critical area, near the support of ramp beam RB1 on the raker beam, was determined using the required design dead and live loads in accordance with the current code. The following results were obtained:

- Calculated factored shear: 290 kN
- Beam shear resistance under the current design standard: 186 kN

It should be noted that, in this area, cracks which, in our opinion, are cause by shear, were noted.

### 8.1.2. Ramps:

#### 8.1.2.1. Precast Ramps:

A review of the concrete sections and reinforcing steel at critical areas would indicate, in our opinion, that the flexural capacity is satisfactory

All precast concrete ramps, excluding section R2, were determined, in our opinion, to have satisfactory shear capacities.

Precast ramp R2, at the cantilever that supports ramp section R3, at the area without shear reinforcing, was investigated. The shear load was found to exceed the shear capacity permitted by the current design standards.

#### 8.1.2.2. Ramp Support Beam:

##### 8.1.2.2.1. Ramp Support Beam RB1:

A review of the concrete sections and reinforcing steel at critical areas would indicate, in our opinion, that both the flexural and shear capacity of beam RB1 is satisfactory.

##### 8.1.2.2.2. Ramp Support Beam RB2 and RB3:

A review of the concrete sections and reinforcing steel at critical areas on RB2 and RB3 would indicate, in our opinion, that the flexural capacity is satisfactory.

At these beams RB2 and RB3, particularly areas without shear reinforcing, the shear capacity, in our opinion, exceeds the capacity of that permitted by the current OBC.

##### 8.1.2.2.3. Ramp Support Column:

A review of the concrete sections and reinforcing steel at critical areas on the tee portion of columns would indicate, in our opinion, that the flexural capacity is satisfactory.

At the cantilever end of the tee supporting the ramp, section R4, particularly at areas without shear reinforcing, the shear capacity, in our opinion, exceeds the shear capacity of that permitted by the current OBC.

## 9.0 Summary of Remedial Action:

The items noted in "Observations", Section 5 of this report, will require attention after resolution of items noted in "Member Capacity", Section 8 of this report. These items are summarized as follows:

### 9.1. Remediation of Distress and Deterioration

**9.1.1.** Column C4 has local distress as a result of thermal movement and concrete repair to the column and ramp landing slab will be required.

**9.1.2.** The deteriorated concrete at areas on the frames, precast seating panels, columns and ramp system will require repairs, in our opinion, within two years.

**9.1.3.** Concrete repairs to suit corrosion of the reinforcing steel will be on going and can increase exponentially overtime. Repairs undertaken to date are performing satisfactory accept in some of the members listed as per the current code as subjected to high stresses. It is anticipated that concrete repairs will be required regularly for the life of the structure.

**9.1.4.** All guard rails: front of aisle, side of seating and at access ramps will require remedial action.

**9.1.5.** Concrete removed during this inspection will require additional concrete removal and repairs within the next year to protect the reinforcing steel.

**9.1.6.** Exposed structural steel that has evidence of rusting is currently not a concern. However, cleaning and priming will extend the life of the exposed steel members.

**9.1.7.** The service building roof beam support bracket at concrete frame 5 will require additional assessment to determine if the as found condition can support the service building roof design loading.

### 9.2. Remediation of Elements Overstressed in Shear:

The elements of the Lower South Side Stands structure that have been found to be overstressed with reference to supporting the code level design loads, in accordance with the current building code, are as follows:

**9.2.1.** Raker beams at five separate locations on all 19 frames.

**9.2.2.** Ramp support columns at all 6 locations.

**9.2.3.** Ramp support beams RB2 and RB3 at all 12 locations.

**9.2.4.** Ramp panels R2 at 6 of the 12 locations.

**9.2.5.** It appears that in 1960, seismic design was not a requirement under the building code. The current building code has significant requirements for the determination of design seismic forces on building structures. In light of the recommendations of the "Arenas Structural Adequacy Guidelines for Professional Engineers", as this structure was designed previously to the 1970 code, if a retrofit or repairs were to be carried out, the completed repaired structure should meet all of the requirements of the current code, including seismic.

## 10.0 Conclusions and Recommendations:

After a visual review of the structure, it is our opinion that the Lower South Side Stands is not in good condition with numerous indications of local distress and deterioration which will require remedial attention and further assessment if it is the intent to continue occupancy.

A design assessment of key structural elements was undertaken for two reasons:

- Visual evidence of stress and deterioration of the Lower South Side Stands.
- The knowledge of the considerable reduction in shear capacity of beams without shear reinforcing compared to the code requirements prior to 1965.

The design review determined that at numerous locations, beam shears exceeded shear capacities as determined from current code standards and at some of these overstressed areas there was evidence of distress and deterioration, particularly at areas with no shear reinforcing steel.

The extent of the shear overstress in critical beams together with the degree of structural distress and deterioration of the structure represents a high risk, in our opinion, for the occupancy of this building by the public.

The Lower South Side Stands is nearing the 50 year mark. The Lower South Side Stands is a structural frame without finishes to protect it from the elements constructed, with 1960 durability requirements for concrete which is not equivalent to today's standard and, as such, deterioration was not unexpected during this review.

The Lower South Side Stands, in our opinion, has reached its service life. Major remedial action will be required to extend the life of this structure. The remedial action includes the repair of the deteriorated concrete and reinforcing steel and the strengthening of the shear capacity of beams overstressed in shear. It should be noted that should the remedial action, as outlined in this report, be carried out, we would recommend that the structure be upgraded, simultaneously with the structure repairs, to meet the current code requirements for structures which would include a seismic retrofit in keeping with the recommendations of the "Guidelines for Assessment of Arenas for Structural Adequacy".

The restoration costs to repair, reinforce and upgrade the Lower South Side Stands to suit current structural code requirements to permit continued usage will, in our opinion and based on currently known conditions, approach, if not exceed, the cost of a new structure.

These repair costs would only extend the current building program usage for a limited time as concrete deterioration will continue over time resulting in ongoing additional repair costs.

Without the remedial action indicated in this report, it is recommended that the Lower South Side Stands of Frank Clair Stadium, including the field area directly under the cantilever seating, be closed for public occupancy.

**ADJELEIAN ALLEN RUBELI LIMITED**

CERTIFIED ENGINEERING TECHNOLOGIST
OACETT
ONTARIO
GARRY T. VOPNI
No 016094

Garry Vopni, C.E.T.

REGISTERED PROFESSIONAL ENGINEER
C. M. ALLEN

C. Michael Allen, P. Eng.

# APPENDIX A:

## PHOTOS

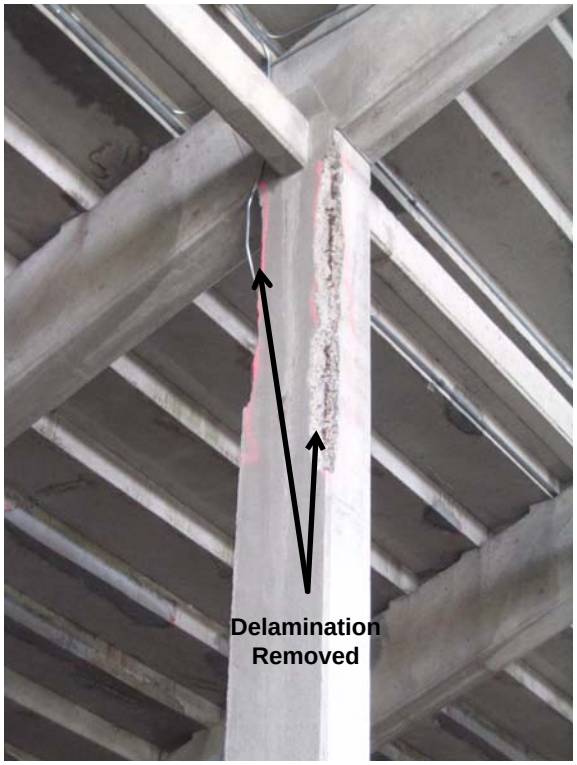



Photo 01 - Column @ Raker Beam: Concrete Spall


Photo 02 – Column C2:  Local Deterioration

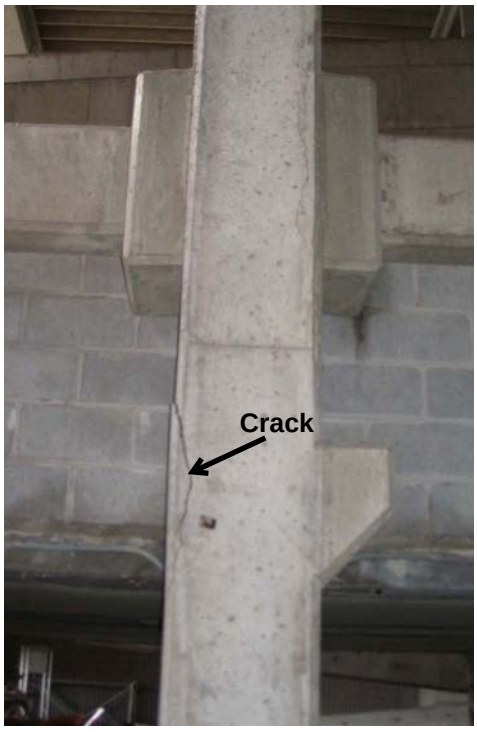



Photo 03 – Column at Landing Ramp: Concrete Delamination

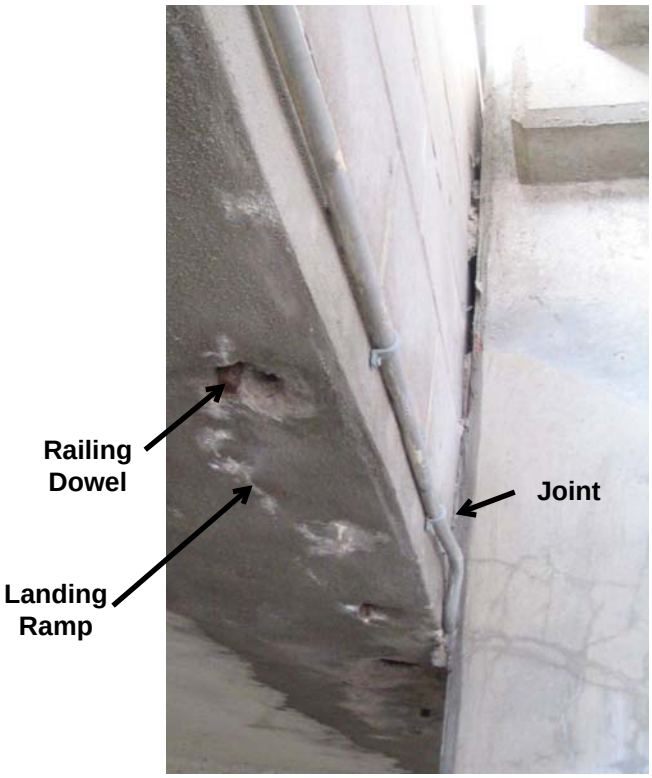



Photo 04 – Landing ramp @ column C4:
Joint built solid; guard rebar anchors

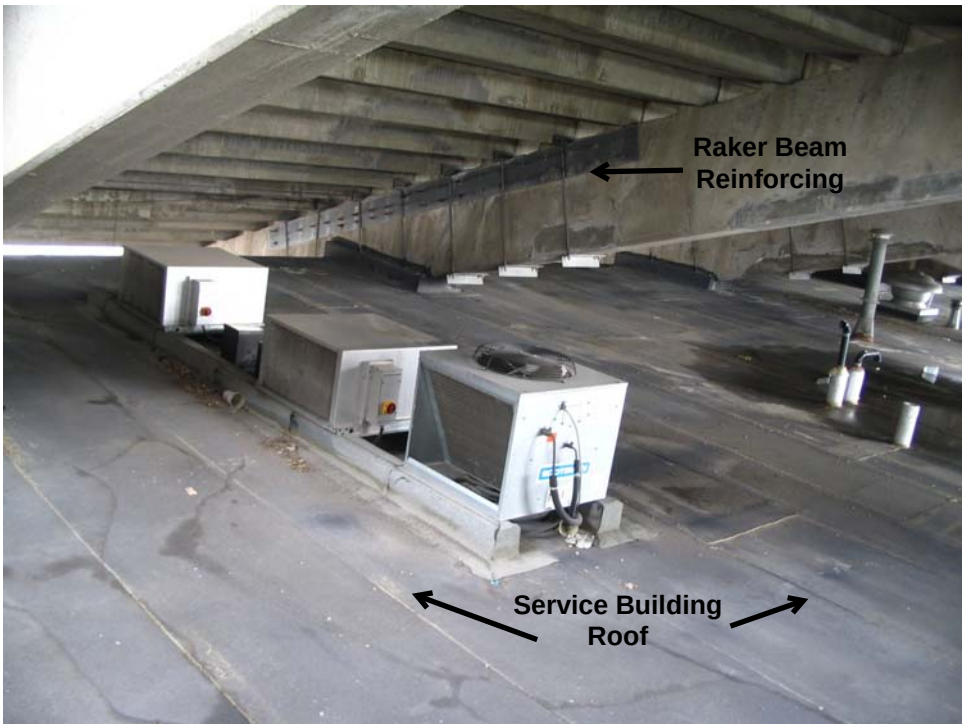



Photo 05 – Raker Beam over service building:
Structural steel reinforcing of Raker Beam

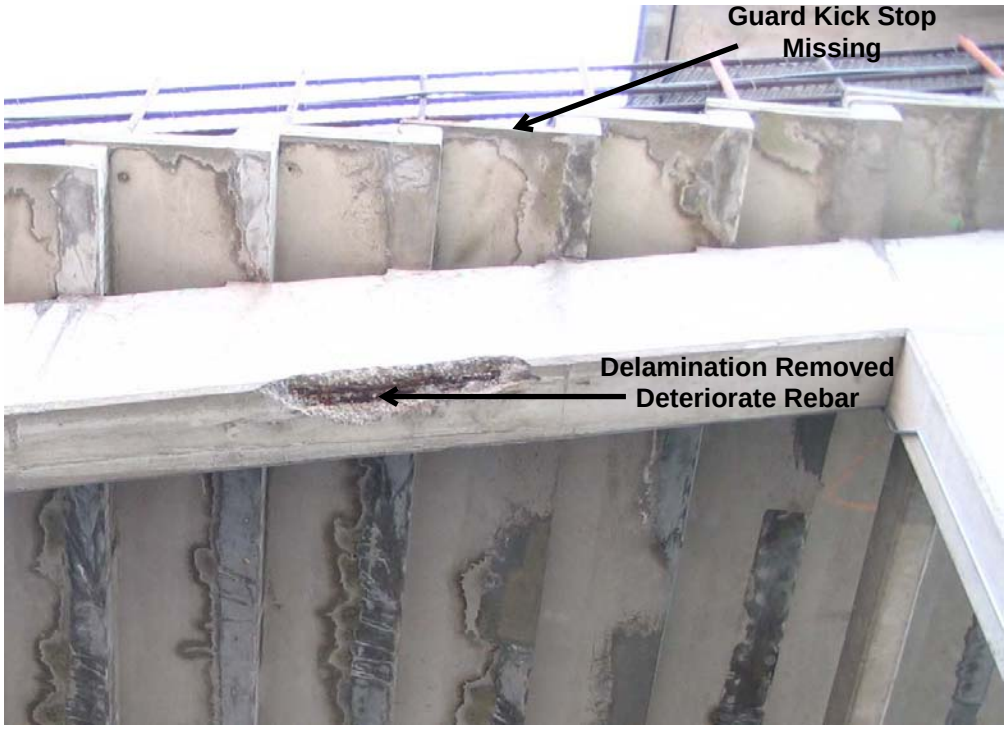



Photo 06 – Raker Beam 19: Deteriorated concrete/rebar;
railing kick stop missing

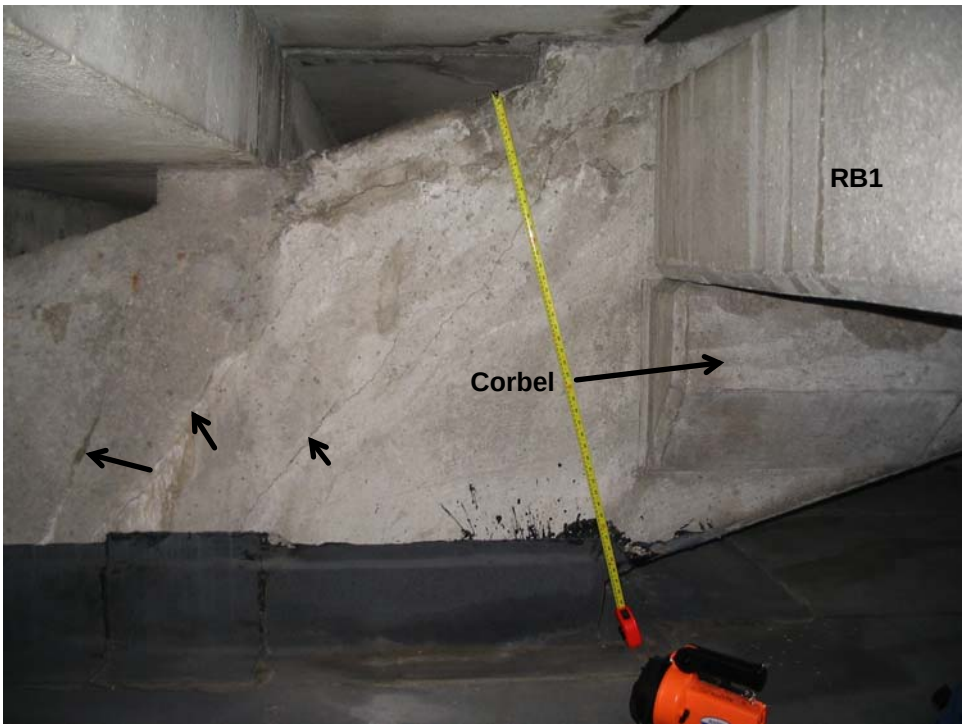


Photo 07 – Raker 5: Cracks

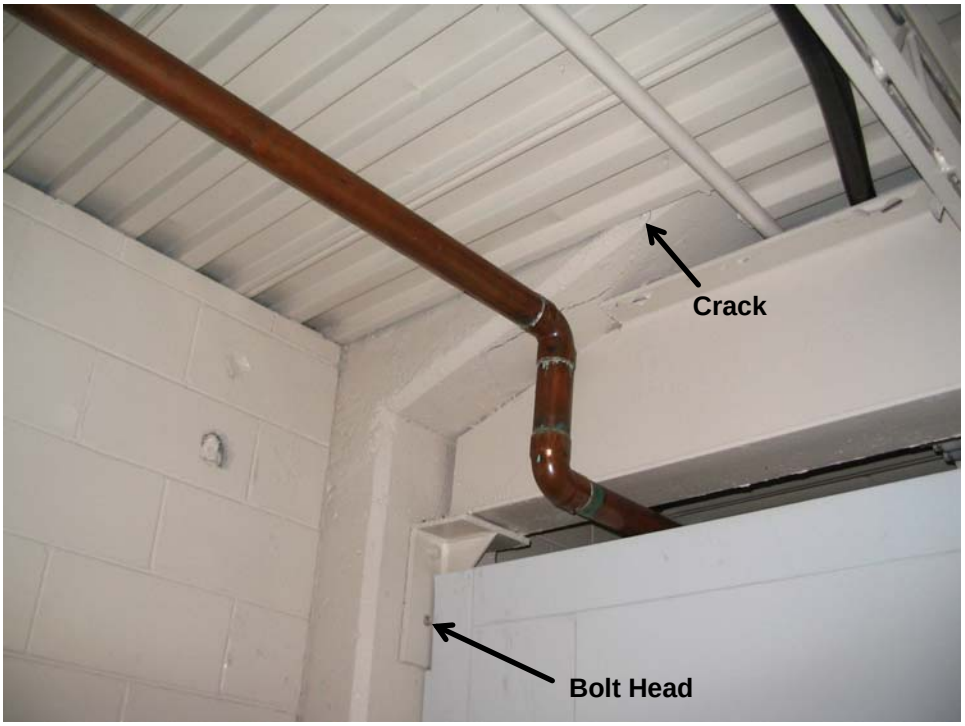


Photo 08 – Service building roof support at Frame 5:
Bolt head removed; Raker Beam Cracks

Photo 09 – Precast Seating Panels: typical soffit repair patches

Photo 10 – Top of Precast Seating Panels:
Deteriorated concrete and reinforcing


Photo 11 – Ramp R3 Hangers at R2:
Delamination repair cracked

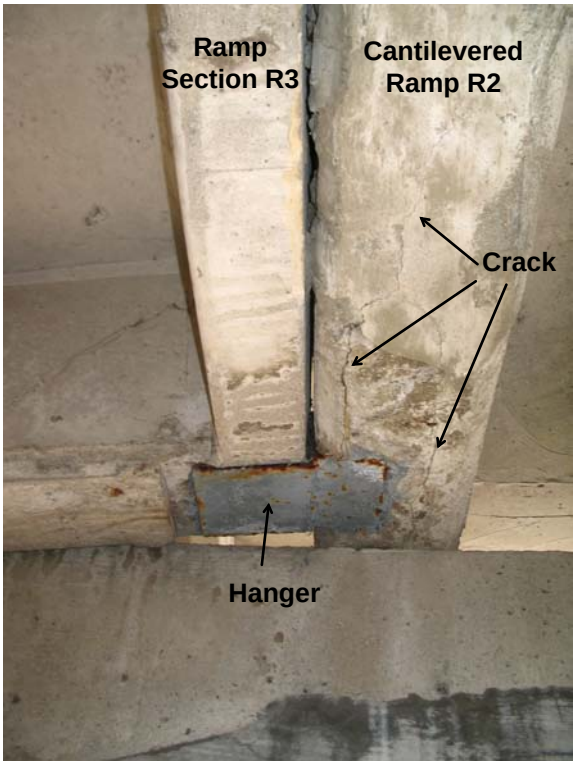



Photo 12 – Ramp R3 Hanger @ R2: cracks

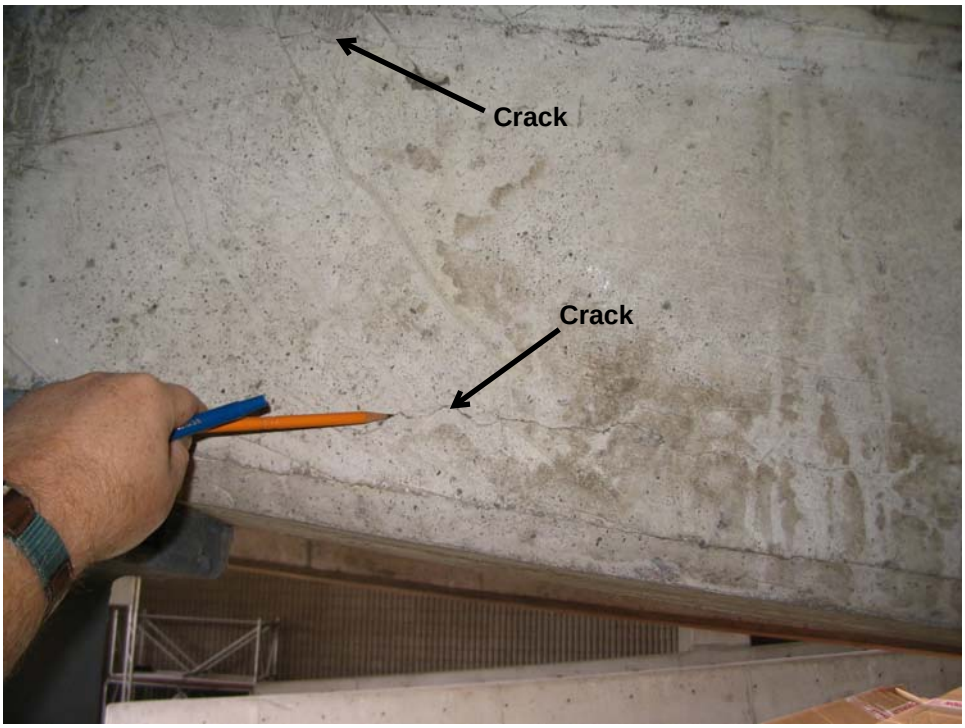


Photo 13 – Ramp R4 @ Column Support: cracks

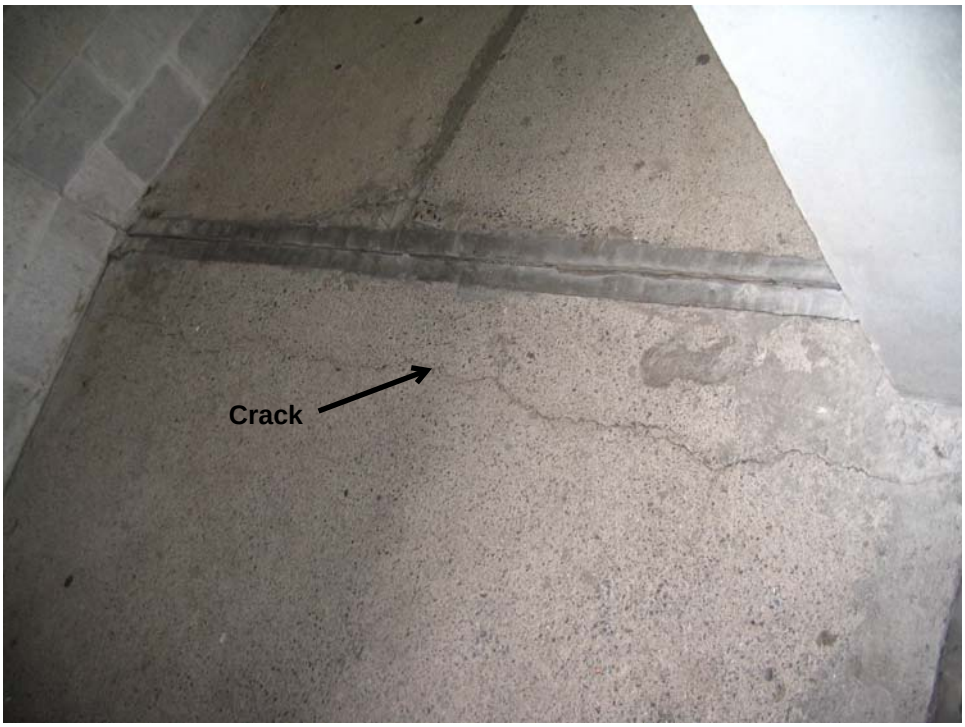


Photo 14 – Ramp Landing @ Support Column:
Top of slab crack

Photo 15 – Ramp Support Beam RB2: crack at support


Photo 16 – Ramp Beam RB3: Deflection

Photo 17 – Ramp Support Beam Cracks


Photo 18 – Ramp Support Beam Delamination

Photo 19 – Ramp Column Support @ Landing Slab:
Guard rebar anchors; landing slab cracks;
ramp column tee crack

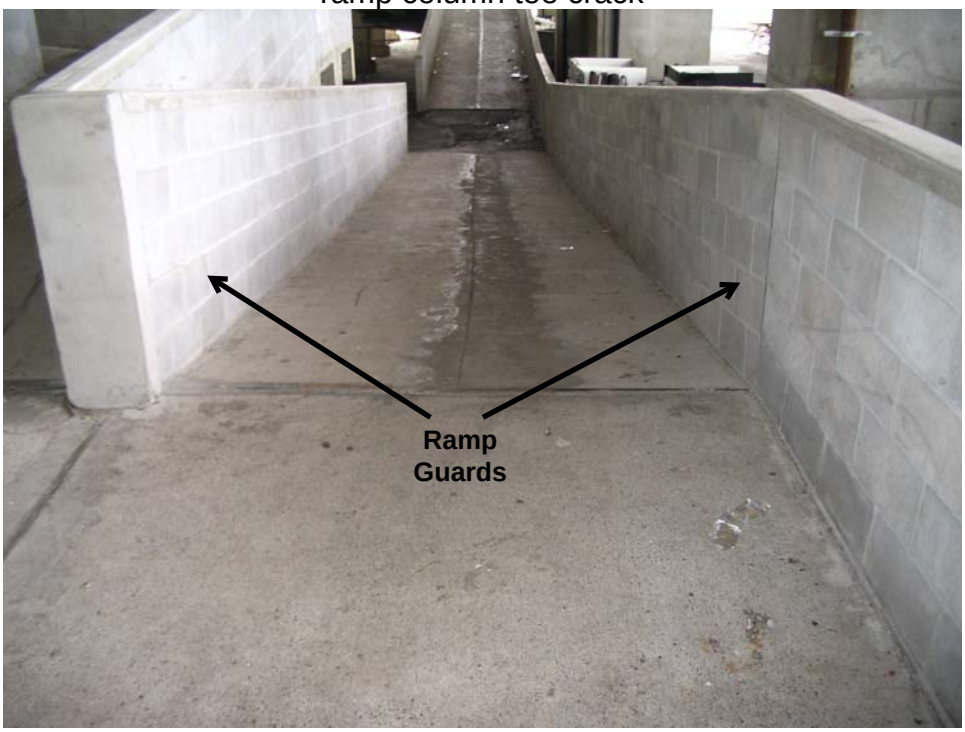



Photo 20 – Ramp R4: Block Guards

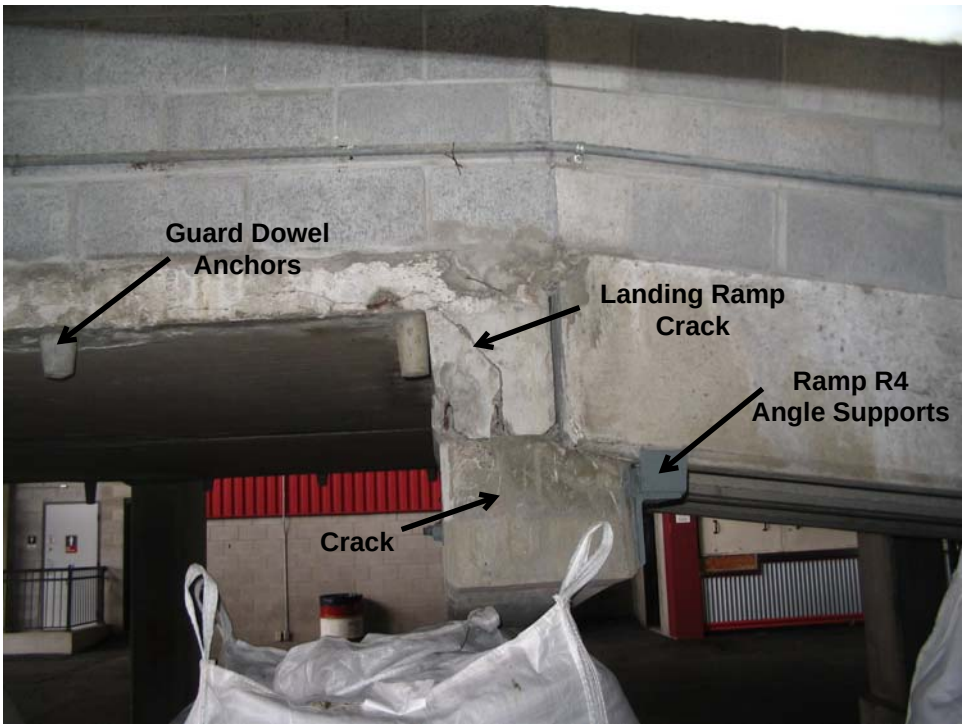


Photo 21 – Ramp Column End View: Guard rebar anchors; Landing slab cracks; Ramp column cracks; Ramp R4 bearing support previous repairs

Photo 22 – Landing Ramp: Delamination slab edge; Guard rail anchors

Photo 23 – Front Guard Rail: Deterioration base


Photo 24 – Front aisle guard rail:  Top horizontal rail missing

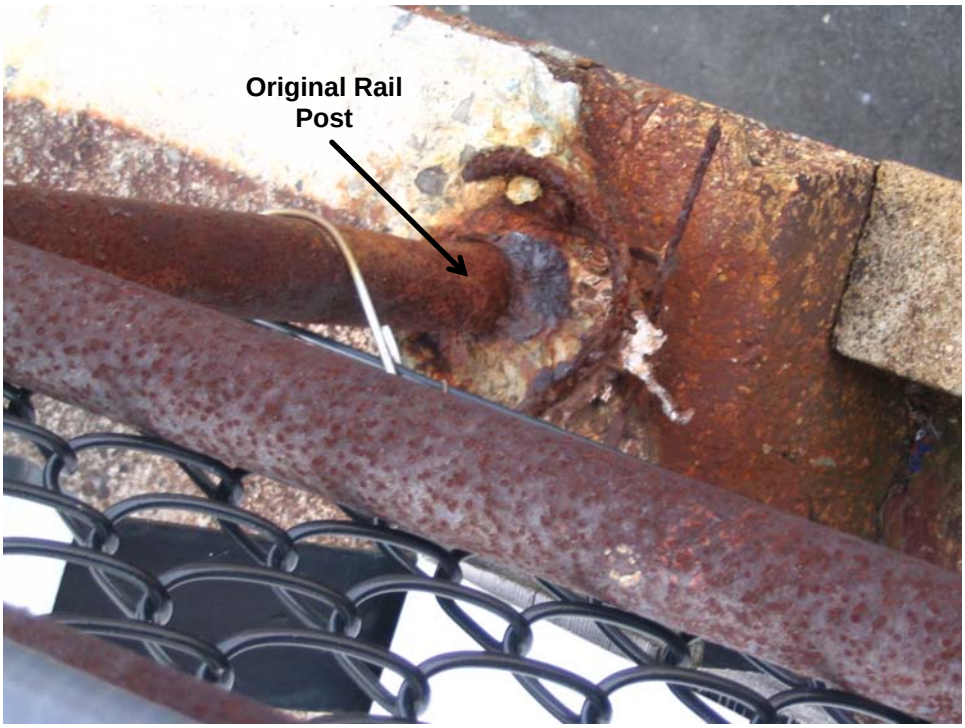
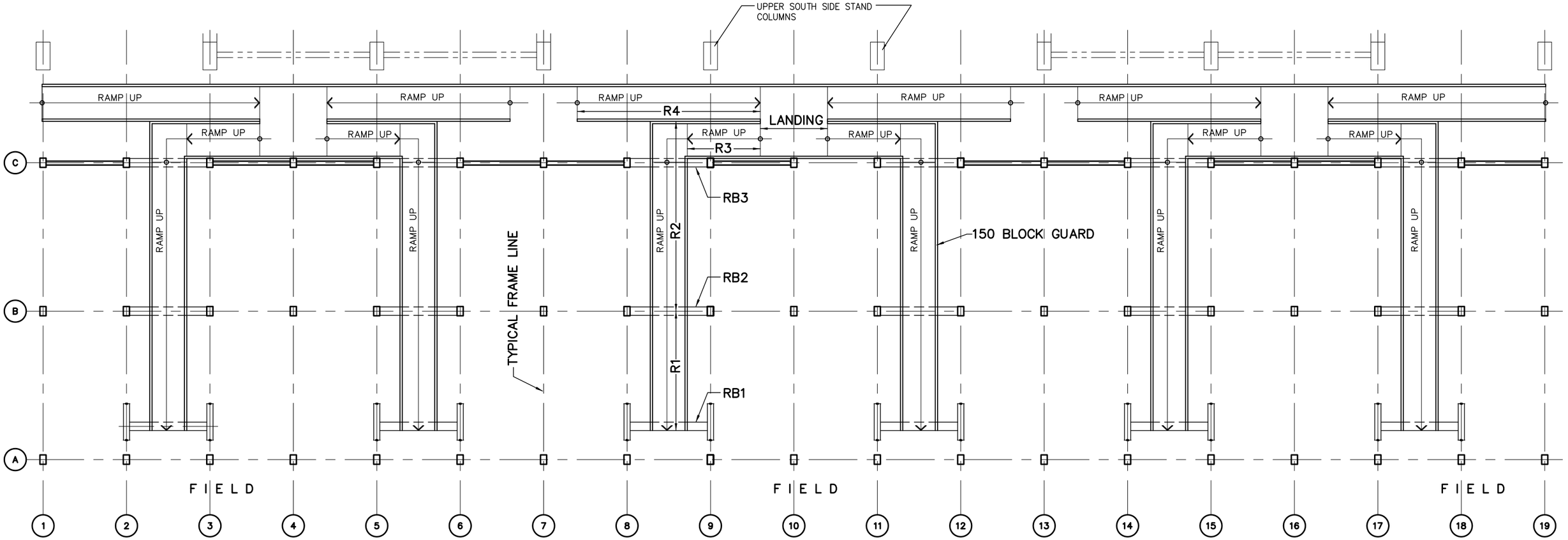


Photo 25 – Side Guards: Corroded post; kick stop missing

# APPENDIX B:

## BUILDING PLAN AND SECTIONS

ADJELEIAN ALLEN RUBELI LIMITED CONSULTING ENGINEERS OTTAWA / TORONTO
75 Albert Street, Suite 1005, Ottawa, ON K1P 5E7 tel (613) 232-5786 fax (613) 230-8916 email aar@aar.on.ca



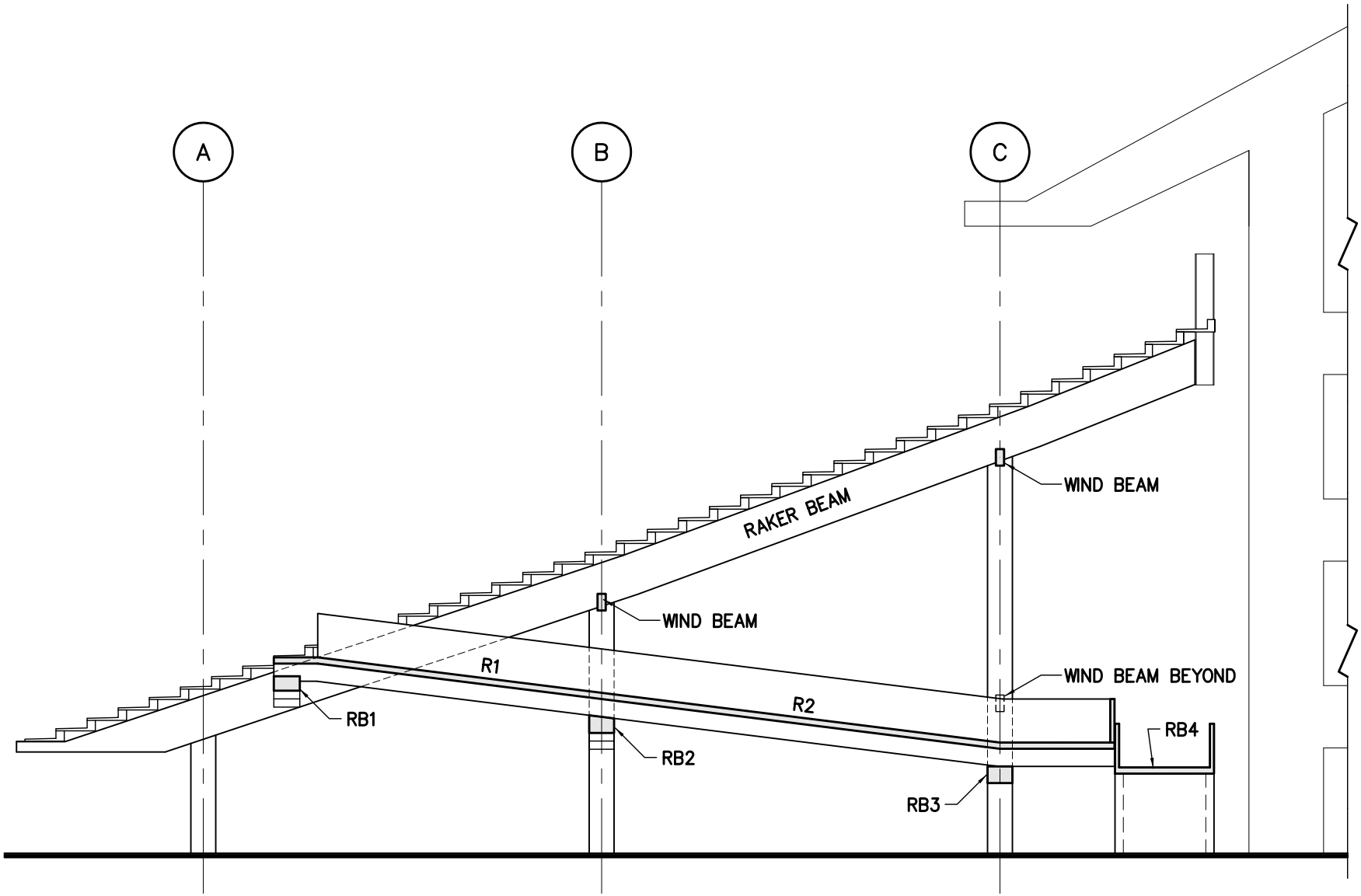

# Frank Clair Stadium

## Lower South Side Stands

AAR No.: 3061–26

Plan / Ramps Columns and Lower Beams

**Adjeleian Allen Rubeli Limited**
Consulting Engineers
75 Albert Street, Suite 1005,
Ottawa, Ontario




# Frank Clair Stadium

## Lower South Side Stands

AAR No.: 3061–26

Section Ramp / Frame

**Adjeleian Allen Rubeli Limited**
Consulting Engineers
75 Albert Street, Suite 1005,
Ottawa, Ontario

# APPENDIX C:

**August 31, 2007**
**Loading Restriction**

ADJELEIAN ALLEN RUBELI LIMITED CONSULTING ENGINEERS OTTAWA / TORONTO
75 Albert Street, Suite 1005, Ottawa, ON K1P 5E7 tel (613) 232-5786 fax (613) 230-8916 email aar@aar.on.ca

**August 31, 2007**

**City of Ottawa**
Department of Corporate Services
Real Property Asset Management
100 Constellation Crescent
Ottawa, Ontario

**Attention: Mr. Richard Davis**

Dear Mr. Davis,

**Re: Lansdowne Park - South Side Stands of Frank Clair Stadium Inspection**
**Lower Stands - Structural Distress**
**Our Reference No. 3061-26**

Adjeleian Allen Rubeli Ltd has undertaken an inspection and structural assessment of the Upper and Lower South Stands of Frank Clair Stadium.

The inspection of the lower portion of the south side stands commenced on August 8th 2007 with local distress and deterioration identified. The distressed areas were analysed with reference to the current 2006 building code and the 2004 concrete design code.

Our findings will be summarized in an Engineering Report that is currently being prepared. The as found conditions per the design review just completed are such that it is our opinion that sections of the lower stand frames and ramp supports are structurally at risk. This risk, in our opinion, based on the current concrete design code, is high and we are, therefore, recommending that the City of Ottawa immediately take the appropriate steps to ensure that the south side lower stands not be occupied by the public either on or below the ramps and seating areas including the locker rooms and access routes at the ground floor level. The field area directly under the cantilever seating of the stadium can, in our opinion, be occupied by the public at this time.

The ground level and locker room area under the stands will require portions of the structure to be shored to allow public occupancy for the remaining of the fall season.

It should be noted that the upper south side stands were reviewed and no restrictions are required and this upper stand portion of the complex is not effected by the above conditions or occupancy restrictions.

Yours truly,

**ADJELEIAN ALLEN RUBELI LIMITED**

REGISTERED PROFESSIONAL ENGINEER
C. M. ALLEN
PROVINCE OF ONTARIO

Garry Vopni, C.E.T.

C. Michael Allen, P.Eng

cc Lansdowne Park (Mark Hennigar)