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REHABILITATION OF A MAJOR 38 YEAR OLD STRUCTURE CHARACTERIZED BY SEVERE ALKALI-AGGREGATE REACTIVITY

John Ryell and Rick Mlynarczyk
Trow Consulting Engineers Ltd.
1595 Clark Boulevard Brampton, Ontario, L6T 4V1

Robert Falcioni
The Regional Municipality of Sudbury
200 Brady Street, P.O. Box 3700, Station A, Sudbury, Ontario, P3A 5W5

ABSTRACT

The concrete in an important large bridge structure carrying traffic under the main CPR rail lines in Sudbury, Ontario was severely deteriorated due to the expansive effects of an alkali-silica reaction compounded by the effects of freeze-thaw disintegration and rebar corrosion spalling. The local gravel coarse aggregate in the concrete, consisting of metamorphosed sedimentary rocks of the Huronian Supergroup (argillites, siltstones, greywackes and sandstones/arkoses), is generally considered to be slowly alkali-silica reactive. In this structure a major rehabilitation effort was required to maintain the structure in a satisfactory condition many years before the anticipated service life was reached.

This paper discusses the condition of the structure, the reconstruction alternatives examined and the rehabilitation procedures. The use of the hydrodemolition procedures for the partial depth removal of the deteriorated concrete from the abutments, columns and retaining walls was one of the keys to the success of the project both technically and financially.

The problem in controlling cracking in the wall refacement is discussed and data related to the frequency of cracking in relation to concrete temperatures and curing conditions is presented.

The rehabilitation effort was completed in 1998 for a total cost of approximately $3.9 million.

Keywords: Alkali-silica reactions, reconstruction alternatives, rehabilitation, hydrodemolition, refacement cracking.

INTRODUCTION

The Brady Street Underpass, located in the heart of the City of Sudbury, was constructed in 1960/61 to alleviate traffic congestion caused by the busy level crossing of the main line tracks of the Canadian Pacific Railway (CPR). The structure is owned and maintained by The Regional Municipality of Sudbury and services approximately 18,000 vehicles per day.

The structure consists of a roadway crossing under the two span CPR and adjacent Elgin Street bridge decks with two abutments, four retaining walls and a median with a centre line of pier columns. The structure is supported by steel H piles founded on bedrock 18 to 25 m below the concrete footings. An unusual feature of the structure is the asphalt surfaced 230 mm thick reinforced concrete road slab supported on concrete strut beams that provide a horizontal tie to the bridge footings and abutments. The overall length of the two bridge abutments is 55.4 m with a minimum height of 6 m. Retaining walls at each end of the structure extend more than 40 m beyond the abutments.

Compared to most highway structures many of the concrete elements on the Brady Street Underpass are quite massive in size. The abutments to the CPR structures, for example, vary in thickness from 2.6 m at the footing to 1.2 m at the deck level and some of the retaining walls are 2 m wide at the base. The pier columns are 0.9 m by 1.2 m in cross section.

CONDITION OF THE STRUCTURE

In 1997 when a decision was made by the owner to rehabilitate the structure some concrete components were affected by severe deterioration; the worst examples were the exposed retaining walls (Fig. 1). It is clear that the main contributing factor in the deterioration process is alkali-silicate reaction (ASR). Freeze-thaw damage of the cracked concrete and surface spalling caused by corrosion of the steel reinforcement has served to accelerate the rate of deterioration..

The last 30 years has seen a number of reports related to alkali-aggregate reaction (AAR) in concrete structures in northern Ontario mainly by researchers from Ontario Hydro and the Ministry of Transportation, Ontario. For highway structures in the vicinity of Sudbury Magni and Rogers (1987) reported that twenty-six bridges were affected by AAR damage and that three older bridge structures had been replaced as a result of deterioration promoted by AAR. Petrographic examination carried out by Magni and Rogers confirmed the presence of cracking, aggregate particle reaction rims and alkali silica gel in the damaged concrete. Magni and Rogers further reported that the reactive rock types in the Sudbury area gravels were predominantly argillites and greywackes and that the reactive particle content in the gravels varied from 65 to 90 per cent. The 1987 report contained a photograph of a Brady Street retaining wall; the deterioration was described as a moderate to severe reaction and it was noted that considerable maintenance and rehabilitation work had been carried out.

Figure 1: Severe AAR deterioration in structure retaining wall. Generally the outer face steel reinforcement is corroded.

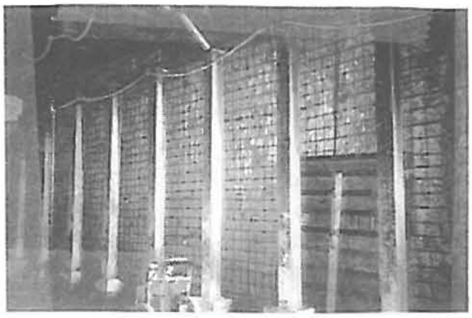

Figure 2: Commencement of formwork erection to the CPR structure abutment. At this stage the outer 200 mm of deteriorated concrete has been removed and the epoxy coated steel reinforcement and mechanical anchors are in place. Note the temporary steel structure supporting the bridge deck beams under the CPR rail lines.

Figure 3: Removal of concrete by hydrodemolition

By 1997, as noted above, some components of the structure were affected by severe deterioration and since pieces of concrete could be removed by hand at some locations there was clearly a danger of deteriorated concrete falling onto the Brady Street Roadway. The retaining walls, abutments, centre pier and the outside face of the Elgin Street deck slab were vulnerable in this respect.

The Elgin Street deck slab was known to be in a relatively poor condition from an earlier report (Velji and Iamonaco 1988). The top surface of the concrete deck slab was disintegrated to a depth of 110 mm, the concrete was non air entrained and a corrosion potential survey indicated that 73% of the top layer of steel reinforcement was actively corroding.

Core samples removed from the structure in 1993 and 1995 confirmed that ASR had developed in the concrete. Silica gel was detected in voids and fractures in the cement paste; dark peripheral rims and internal fracturing characterized many of the coarse aggregate particles and the paste was extensively fractured at some locations.

Petrographic examination of core samples indicated that between 54% and 81% of the coarse aggregate particles consisted of rock types susceptible to ASR including greywacke, argellite, quartzite, quartzwacke and arkose. The fine aggregate consisted primarily of silicate minerals and rock fragments with a lithology similar to the coarse aggregate. The interior concrete behind or below the deteriorated surfaces was affected by hairline and microscopic cracks in the cement paste. Such cracking was extensive in the exposed components such as the abutments and retaining walls and in one core sample approximately 40 transverse fractures per meter of core length were detected. In more sheltered bridge components such as the paved road slab, the lower portion of the Elgin Street deck slab and the footings, the concrete contained only a moderate or few narrow cracks and microfractures.

Examination of core samples indicated that most of the concrete was not air entrained.

RECONSTRUCTION ALTERNATIVES

The deteriorated condition of some components of the Brady Street Underpass, a number of traffic issues and Regional plans for the extension and development of the Brady Street corridor, required a comprehensive engineering and feasibility study to evaluate the various options for the reconstruction of this large, important structure. The engineering feasibility study (Mlynarczyk et al 1995) addressed three broad options for reconstruction of the underpass viz: rehabilitation of the structures so as to achieve a further 20 years safe service life with minimum ongoing maintenance costs; demolition and replacement of the structures to the present geometry and demolition and replacement of the structures to accommodate six lanes of traffic on Brady Street. Safe movement of pedestrians was addressed in each of the three options viz: a pedestrian tunnel behind the North abutment for the structure rehabilitation option, and raised sidewalks inside the underpass structure for the two replacement options.

The feasibility study indicated that the estimated construction costs including contingency allowances and engineering were $4.5 million for the rehabilitation option, $16.5 million for the replacement four lane structure and $19.8 million for a replacement six lane structure. The construction of a pedestrian tunnel in conjunction with the rehabilitation option was estimated at an additional $1.6 million.

Early in 1997, a decision was made by the Owner to rehabilitate the structure to obtain a further 20 year safe service life. Construction of the pedestrian tunnel was not included in the 1998 rehabilitation contract.

REHABILITATION

The construction contract for the rehabilitation work was tendered in October 1997. The scope of the tendered contract was to carry out the rehabilitation of the retaining walls, abutments, Brady Street road slab and pier during a two year period. Other related work in the contract involved modifications to the existing storm sewer systems, replacement of the pedestrian and bridge railings, waterproofing, earthworks, bridge lighting replacement, and landscaping. The contract was awarded to Peter Kiewit Sons Co. Ltd. in 1997 who proposed to carry out all of the specified work in one construction season. The primary difference between Kiewit's concrete removal method and that of the other contractors who tendered for the work was the use of hydrodemolition for the partial depth removal of the concrete.

During progress of the tendered rehabilitation work the removal and replacement of the Elgin Street bridge deck and the painting of the CPR structural steel deck and girders was added to the project.

The primary objective of the rehabilitation option was to extend the service life of the structures for at least a further 20 years. For the vertical bridge components the removal of sufficient amounts of deteriorated concrete and the placement of a new refacing layer of dense, protective concrete well bonded and mechanically anchored to the underlying sound concrete, was the basis of the rehabilitation work. Based on the core samples removed from the structure (Mlynarczyk and Iamonaco 1993), rehabilitation included the removal of an approximate 300 mm depth of the surface concrete and all reinforcing steel from the retaining walls, removal of 200 mm of concrete from the abutments and 75 to 100 mm of concrete (25 mm beyond the reinforcing steel) from the pier. The top one third of the pier was in better condition probably due to the absence of road splash and local patching was considered satisfactory. The entire road slab, excluding the strut beams, was removed and replaced. The work was carried out while maintaining at least one lane of traffic flow through the work in both directions at all times except near the end of the work when Brady Street was closed for approximately 7 weeks to facilitate completion of the additional work comprising the Elgin Street deck replacement and the painting of the CPR structural steel.

The specification for concrete quality was intended to provide a number of specific characteristics, as follows:

- Minimum availability of alkalis in the new concrete which could fuel further AAR in the existing concrete.
- Low concrete placement temperatures, low concrete peak temperatures and low concrete temperature differentials to control as far as practical thermal stresses in the concrete.
- Vertical formwork left in place for a period of not less than 7 days to minimize thermal and drying shrinkage stresses in the concrete.
- High workability, 30 MPa, superplasticized concrete to support proper compaction of the concrete in relatively congested sections.
- Use of non-reactive concrete aggregate from an MTO approved source.

The total cementitious content of the air-entrained, 30 MPa, refacement concrete was 355 kg/m^3 consisting of 85% Type 20, Moderate Portland cement and 15% granulated blast furnace slag. The total alkali content of the concrete was very low (1.73 kg/m^3) mainly due to the properties of the portland cement (0.49% Na_2O equivalent).

The factors in the rehabilitation option that support the owner's objective of achieving at least a 20 year additional service life include:

- The likelihood that any ongoing ASR in the original 38 year old concrete will continue at a much reduced rate and that any resulting expansion will not cause significant deterioration to the relatively thick concrete refacement.

- The refacement concrete is air entrained to make it resistant to freezing and thawing, the low permeability characteristics will minimize the ingress of moisture and salt solution to the ASR concrete and the low alkali content will not fuel continued ASR.
- The very substantial thickness of the deteriorated structural components, e.g. abutments and retaining walls is adequate from a strength viewpoint to support dead and live loads and the quality of the interior concrete is adequate as a base for the mechanically anchored refacement concrete.

The rehabilitation work began in April 1998 with the removal of the asphalt pavement on the Brady Street road slab and the installation of the temporary shoring to the CPR steel deck. Prior to the partial depth removal of 200 mm of concrete from the abutments up to and around the bearings, the entire CPR deck was jacked up approximately 6 mm off the abutment bearings and supported on steel columns to transfer the load to the footings (Fig. 2). This jacking of the CPR steel deck was necessary for the proper placement of the concrete in the top part of the abutments. The outer 100 mm of concrete on the retaining walls, abutments and piers including the reinforcing steel (excluding piers) was removed using a Hoe Ram.

The remaining deteriorated concrete, approximately 100 to 200 mm in thickness, was removed by hydrodemolition using a one man hydraulically operated robotic device with a water pressure of 86 MPa (Fig. 3). The hydrodemolition equipment, using water at approximately 190 liters per minute, removed up to approximately 0.5 cubic meters of concrete per hour. A total of 393 cubic meters of concrete was removed by hydrodemolition using approximately 17 million liters of water. Following the major removal operation by mechanical means and hydrodemolition, only minor amounts of fractured or loose concrete remained; these were removed by a hand held water lance operated at pressures of approximately 50 to 60 MPa (Fig. 4).

Mechanical dowel anchors were installed to tie in the refacement concrete to the existing concrete in the abutments and retaining walls. The anchors were spaced at 1 m centres vertically and horizontally and grouted to a depth of 300 mm into the existing concrete. A mat of epoxy coated reinforcing steel was installed near the outer face of the abutments and retaining walls and tied to the dowel anchors (Fig. 5).

For the refacements of the abutments and retaining walls vertical construction joints in the concrete were placed at the location of existing joints but at a spacing not exceeding 12 m. The mat of epoxy coated reinforcing steel was continuous through all construction joints except that every second bar was cut at vertical joints. Vertical control joints were spaced at 6 m spacings using a 30 mm deep crack-inducer strip. Horizontal construction joints were placed at the mid height of the abutments and on retaining walls over 4.5 m high.

It was specified that the Brady Street road slab could not be removed prior to the removal of the braced CPR deck load from the footings. Once the refacement of the abutment walls was completed and the shores removed, the full thickness of the road slab over the transverse strut beams was removed by hydrodemolition. Following this operation, the slab reinforcement was cut by torch and the slab sections were lifted out by loader. Removal of debris and minor areas of deteriorated concrete from the strut beams and footings, placement of new granular materials, followed by conventional bridge deck concrete placement techniques were used to complete the new concrete road slab (Fig. 6). A total of 780 cubic meters of concrete was placed in the road slab and medians.

The 200 m^3 of reinforced concrete in the Elgin Street deck slab were demolished in 10 hours using a Hoe Ram (Fig. 7). The deck replacement consisted of eighteen precast 0.54 m thick concrete beams cast at a local precast plant and installed by crane (Fig. 8) plus waterproofing and a bituminous pavement.

The Brady Street structure, following completion of the rehabilitation work is shown in Fig. 9.

CRACKING IN REFACEMENT CONCRETE

Considerable efforts were made at the design stage and during construction to minimize cracking in the refacement concrete due to the effects of thermal gradients and drying shrinkage. A maximum concrete placement temperature of 20°C and a maximum in-place temperature of 60°C were specified and achieved. Formwork remained in place for seven days plus such additional time necessary to reach a maximum average difference in temperature of 10°C between the in-place concrete and the outside ambient conditions.

Fig. 4: Water blasting cle in up and surface preparation of the roadway crash wall and piers prior to concrete refacement. Typically the outer 75 mm to 100 mm of deteriorated concrete has been removed and some steel reinforcement has been replaced.

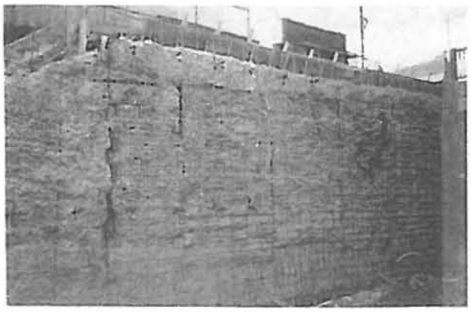

Fig. 5: Placement of epoxy coated reinforcing steel in retaining wall. At this stage the outer 300 mm of deteriorated concrete has been removed and the mechanical anchors and form ties have been installed.

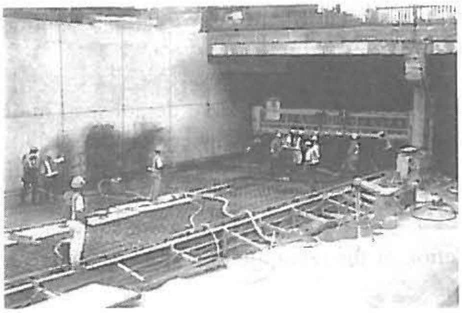

Fig. 6: Replacement of concrete road slab using epoxy coated reinforcing steel. At this stage the refacement of the structure abutments and retaining walls is complete but the Elgin Street deck slab has not been replaced (note the significant AAR deterioration on the exposed face).

Fig. 7: Demolition of Elgin Street cast-in-place deck slab. The CPR structure is in the background.

Fig. 8: Installation of precast concrete deck units, Elgin Street Overpass. The CPR structure is in the background.

Fig. 9: Rehabilitated structure, November 1998. The replaced Elgin Street deck slab is in the foreground.

However some cracking did occur in the refacement concrete placed on the abutments and retaining walls. In the early stages of the work hairline cracks (less than 0.2 mm wide at the surface), and generally vertical, developed at intervals of between 1.8 and 2.1 m on the face of the concrete. Such cracks extended for the full height of concrete placement and were visible on removal of the formwork. An extensive program of temperature monitoring during the curing period indicated average peak concrete temperatures of approximately 35°C at 33 hours following concrete placement and approximately 24°C at the end of the "forms-on" seven day curing period.

The 7 day and 28 day compressive strengths of the concrete average 32.8 MPa and 38.0 MPa respectively.

In an effort to reduce the frequency of vertical cracking and following a review of the concrete placement temperatures, ambient temperatures, compressive strengths, slump and curing procedures, the spacing of the vertical construction joints and/or control joints was reduced from 6 m to 3 m and the depth of the crack inducer strip was increased from 30 mm to 60 mm. This change generally had little effect on the pattern of cracking in the refacement concrete. Additional concrete cylinders were taken on one wall section and tested for compressive strength and splitting tensile strength at 1 day, 2 days, 3 days, 7 days and 28 days.

Comparison between the broken faces of the splitting tensile test specimens and the appearance of the fractured faces of core samples taken through vertical cracks, with particular attention paid to fractures in the coarse aggregate particles, indicated that some of the vertical cracks were propagated between 18 and 24 hours after placement of the concrete.

The experience gained on the Brady Street bridge rehabilitation, in particular the refacement of the abutments and retaining walls, indicates that some cracking in refacement concrete, placed over significant areas of existing concrete, will occur. Specifications that limit concrete placing temperature, control concrete temperature gradients and provide for a long period of curing with the vertical formwork in place, will not completely eliminate this vertical cracking. Factors on the Brady Street structure that contributed to the occurrence of the cracking in the wall refacement concrete may have included movement in the existing abutment and retaining wall concrete due to ambient conditions, vibrations from traffic on the rail structure and road structure and the restraint provided by the regularly spaced dowels and anchors that tied the refacement concrete to the original concrete.

It is interesting to note that cracking did not occur in the smaller area of refacement concrete applied to the pier columns, pier cap and pier crash wall.

During the 1999 construction season, i.e. the year following completion of the rehabilitation contract, 935 meters of cracks, mainly vertical, in the abutments and retaining walls were sealed with a two component flexible polyurethane resin system at a cost of approximately \$100,000. Some very narrow hairline cracks would not accept the resin material and were left unsealed; such cracks are not expected to effect the overall performance of the structure.

CONCLUSIONS

Rehabilitation of the Brady Street underpass was completed in 1998 and periodic monitoring of the structure will provide valuable data on the long term effectiveness of the rehabilitation methods used.

The rehabilitation of this large, important concrete structure contained its own special challenges, including the maintenance of the existing road and rail traffic, the relocation of services, the removal and replacement of large amounts of concrete, and the completion of significant additional work. Projects of this type require a partnership of experienced engineers and contractors working with the owner so that the work is successfully completed on time, within budget, and to the highest standards of quality.

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