Rehabilitation of Railway Bridge Piers Heavily Damaged by Alkali-Aggregate Reaction

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ABSTRACT

This paper deals with the evaluation of the damages caused by the alkali-aggregate reaction. As the concrete above the water line had to be replaced, the bridge loads were transferred through steel columns to deeper layers of concrete.

INTRODUCTION

Numerous old structures showing signs of deterioration are found in the Beauharnois-Valleyfield area located south west of Montreal. Deterioration due to alkali-silica reactivity is related to sandstone aggregates of the Postdam groupe (Bérard and Lapierre, 1977). This type of rock, although common (Clark, 1972), is rarely used as aggregate due to its hardness and abrasivity. Nevertheless, when available from important excavation sites it has been used as a concrete aggregate with ordinary alkali rich cements.

Such was the case in the 1930's when a 30 km long water canal was excavated to derive most of the St-Lawrence flow and bring it to the Beauharnois power house which can produce 1570 MW.

One of these structures is a railway bridge, almost a kilometer long, having 44 simple 22 meter spans built with two steel plate girders. Each girder is simply supported on 100 mm thick bearing plates resting on piers measuring 6 feet (1.8 m) by 12 feet (3.6m). The height of the piers (from top to bedrock), varies between 10.5 and 17.4 meters and an average height of 14 meters is under water. The bridge piers were built "in the dry" as normal foundations, before the excavation of the canal. A complete reconstruction, if necessary, of all 45 piers in such depth of water in one working season, would require an excessive number of heavy duty cofferdams and preliminary studies indicated that in that case a new type of bridge with fewer piers and longer spans would be required.

DAMAGE TO CONCRETE PIERS

All visible parts of the piers showed considerable damage due to alkali-silica reaction causing polygonal surface fracturing aggravated by subsequent freezing and thawing. The part of concrete piers protruding above water (2 meters) was heavily cracked. In many instances the sliding bearings were not functionning and thermal movements had cracked the piers (fig.1). In some cases, the central crack seen on the photograph was opened up to 35 mm, in winter. A detailed investigation revealed that some bearings had settled into the concrete pier tops some 20 mm.

SOLUTIONS INVESTIGATED

The following repair/replacement modes were initially investigated.

1. Building a complete new bridge having only ten piers with a new steel superstructure; preliminary estimates reached 60 x 10^6 can $.
2. Demolishing and repair of part of the concrete piers. This entailed the removal of all girders and their storing on shore.
3. Lifting the spans through specially designed supports bearing on the sound concrete of the pier, while damaged concrete was removed and replaced. The load was transferred though steel columns embedded 5-10 m deep in the coring holes (fig. 2).

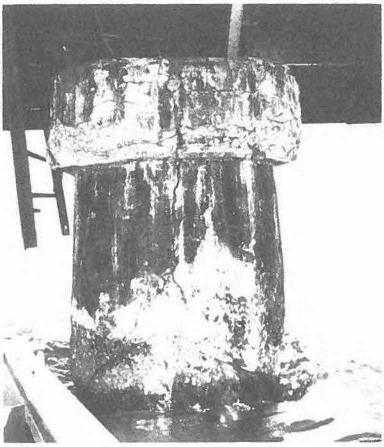

Figure 1: Typical pier damage

CONCRETE QUALITY INVESTIGATION

Field and laboratory studies were undertaken to ascertain the concrete strength and to determine through push-out tests the anchorage length of the steel inserts required to carry the bridge loadings. Laboratory investigations included detailed petrographic analysis of coarse and fine aggregate and compression and splitting tensile tests on fifty two cores. Moduli of elasticity were measured on 16 cores.

Quality of concrete was determined by large diameter cores (150 mm) obtained by diamond drilling of a 200 mm hole. The size of the bore hole was chosen so that it could accommodate a big enough insert (H-beam, pipe or solid steel shaft). Drilling operations were done from a lorry pulled along the existing railway tracks.

Two holes, 6-7.5 meter long were drilled in eight of the 45 piers. Three of the sixteen holes were further drilled down to 15 meters and two were continued into the bedrock to check the concrete-rock interface.

PETROGRAPHIC ANALYSIS

Petrographic analysis revealed that both coarse and fine aggregates are almost exclusively orthoquartzite sandstone. This orthoquartzite is a sedimentary rock composed of quartz grains having an average diameter of 0.3 mm, cemented by a siliceous cement which is reactive to alkalis and thus responsible for the expansion of the aggregates and the observed microcraking.

At the concrete rock interface, the rock showed glacial striae and it was evident that the piers had not been anchored into the bedrock. Loss of drilling water occured at two piers indicating an opening between concrete and bedrock. Drilling into the bedrock often revealed a superficial rock crust followed by friable sandstone or seams filled with gravel. Drilling was continued such that one anchor per pier could be grouted in a rock layer having a minimum thickness of 1.5 meters.

CORE EXAMINATION

All cores were visually inspected and signs of deterioration observed, with extension of opened microcracks noted. Concrete above water level was seen to be badly fractured but was substantially better at lower levels. It was then decided to test specimens from three nominal zones: the heavily fractured top zone (0-2 m), the proposed anchorage zone (3 to 7.5 m) and the lower part (7.5 to 16 m). The following effects of alkali-silica reaction were observed all the way down the pier:

a) Polygonal cracking; b) reaction rims around coarse and fine aggregates; c) many large size aggregates were fractured with fractures continuing in the cement paste; d) presence of silica gel. In the voids, silica gel regularly showed desiccation cracks resulting from dehydration. At the exposed part of the pier freezing and thawing had increased the crack sizes.

CORE TEST RESULTS

All fifty two compression test results are plotted on figure 3. Results vary from a low of 22.8 MPa up to a high of 43.6 MPa. An horizontal line representing the water level shows qualitatively that concretes above and below that line belong to two different populations.

Almost all compression results on the concrete from the upper 3 meters of the pier are below 30 MPa, varying from 22.8 MPa to 31.6 MPa. Results of compression tests for concrete situated below the water level is significantly above the 30 MPa value. Mean value for this group, which follows a normal distribution is 36.4 MPa, with a coefficient of variation of 9.8 %.

Splitting tensile test results has the same type of distribution. Variation of strength with depth follows the trend seen in the compression test results, although in a less definite manner. Considering the test results for specimens taken below the water level, an average value of 2.5 MPa and a coefficient of variation of 16% are obtained. Ratio of this average splitting tensile strength to corresponding average compressive strength is 0.07. Moduli of elasticity were observed to vary between 13 000 MPa and 22 800 MPa with an average value of 17 700 MPa. (Houde and Lacroix, 1985).

BOND TESTS

A small experimental program of push-out tests was done to measure the bond properties of the actual materials: proposed grout, cores from the piers and steel rods having the same finish as the proposed 150 mm steel columns. The ultimate bond stress between grout and concrete and between grout and steel was found to be approximately 15% of the grout compressive strength. With the length of embedment and the actual field grout compressive strength,

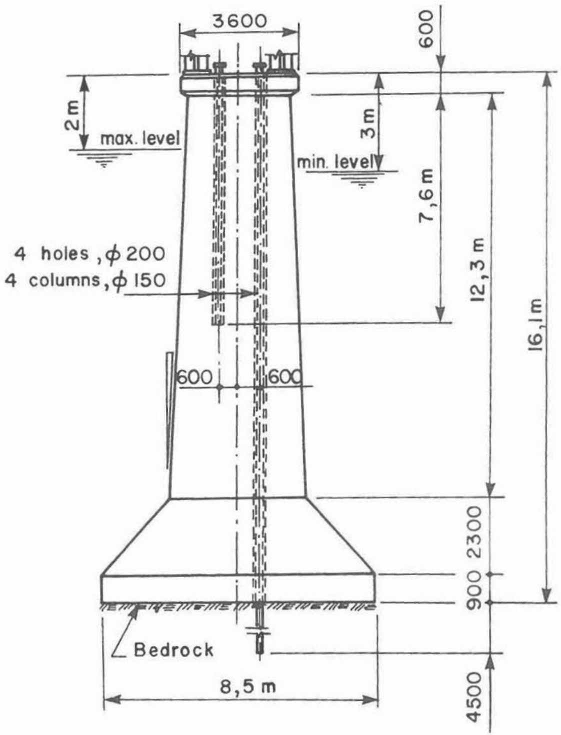

Figure 2: Proposed load transfer columns

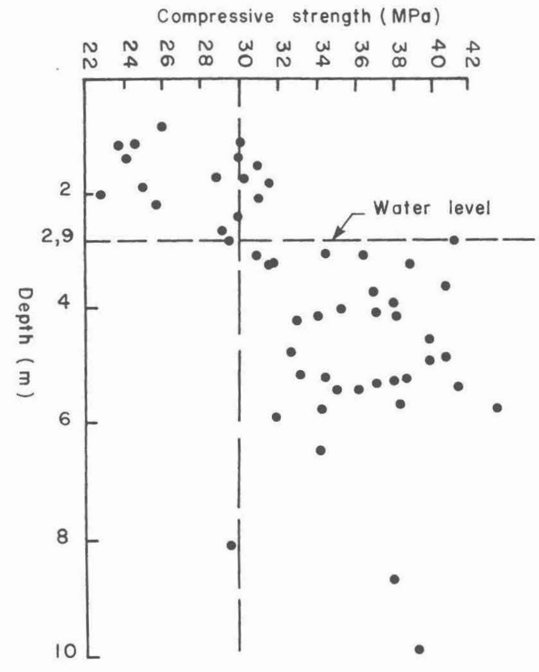

Figure 3: Variation of compressive strength with depth

the bond stress, under service loads, is limited to two percent of the grout compressive strength.

REPAIR WORK

The construction program was organized in such a way that drilling could be done in fall and winter while fabrication of the 172 steel columns and 43 permanent transfer load beams was underway; everything would then be in place so that repairs could be done the following construction season.

Steel columns were positionned within 0,40 mm, in elevation, and within 9.5 mm in the two horizontal directions. To facilitate the handling and support operations the ends of the columns were threaded on a 330 mm length. When the grout had hardened sufficiently, the bridge girders were jacked up and lowered on temporary support beams resting on sleeves threaded to the column ends.

At that stage,concrete was removed to a depth of at least 1.5-1.8 meter, even if sometimes deterioration did not reach that level. This was done because sufficient space was needed to install the lifting mechanism which was fixed to the steel column through 4 split collars. The lifting mechanism was thus independent of the height of concrete removed and above all no lifting operation relied on the bearing capacity of the alterated layers of concrete. After installation of the load transfer beam, the lifting mechanism was removed and reinforcing bars set up. Installation of the formwork reached below the water level to allow replacement of the chipped concrete around the sides of the pier.

CONCLUSION

The chosen method of repairs was successful and rehabilitation of the fourty five piers could be done in one working season, using only five lifting mechanisms. All field operations had been designed so that they could be completed in one working day. After the unavoidable initial adjustments, cruising speed was attained and the last repair operation was completed on one pier, every working day.

The rehabilitation project was realized at a cost of 6×10^6 can \$, considerably less that the price (60×10^6 can \$) of a new bridge.

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