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Design of River Piers for the Second Peace Bridge

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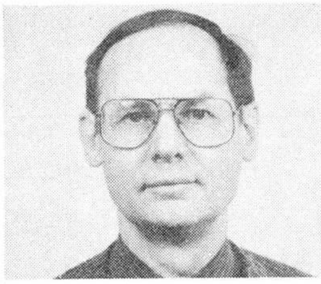

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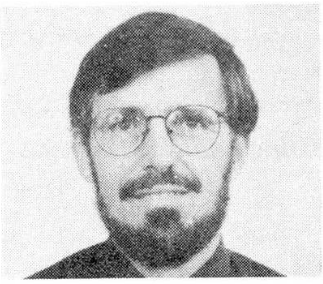

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SUMMARY

The Second Peace Bridge, to be completed in 2002, will double the capacity of the existing international Canada/U.S. crossing of the Niagara River, between Fort Erie, Ontario, and Buffalo, New York. The existing bridge, completed in 1926, consists of five steel arch spans crossing the river plus one truss span over the navigable Black Rock Canal on the U.S. side. The new bridge similarly consists of five steel arch spans of identical lengths, plus one large arch span over the canal. The total length of the bridge will be 1064 m. The five river piers were designed by Delcan Corporation, North York (Toronto), Canada. Design was carried out in accordance with the 1997 AASHTO LRFD Code [1] and required careful interpretation of the prescribed load combinations to determine the critical design loadings for the piers. Further, it was necessary to establish acceptable limits on the behaviour of the piers. Design was further constrained by strict hydrological regulations prohibiting increases in the upstream surface profile. With the assistance of detailed finite element flow models, pier geometries were refined and optimized, virtually eliminating backwater effects. The result was a design which meets all structural, environmental, economic and aesthetic criteria. Additional consideration was given to constructability issues as peak flow velocities in the Niagara River (best known for the spectacular Niagara Falls, downstream) can exceed 4 m/s and will present a major construction challenge to the contractor. In this paper, the authors wish to demonstrate that intelligent design cannot be achieved by mere reliance on design codes, but rather on an informed and judicious interpretation of design code clauses, the establishment of acceptable standards of performance and sound engineering judgement.

1. INTRODUCTION

The Second Peace Bridge, to be completed in 2002, will double the capacity of the existing international Canada/U.S. crossing of the Niagara River, between Fort Erie, Ontario, and Buffalo, New York. The existing bridge, completed in 1927, consists of five steel arch spans crossing the river plus one truss span over the navigable Black Rock Canal on the U.S. side. The new bridge similarly consists of five steel arch spans of matching lengths, plus one large arch span over the canal. A general arrangement is shown in Figure 1.

Design of the new Peace Bridge was awarded to an international design consortium consisting of DeLeuw, Cather & Company (Buffalo, NY), Delcan Corporation (Toronto, ON), McCormick Rankin Corporation (Mississauga, ON) and Bettigole Andrews Clark & Killam (Buffalo, NY). One of Delcan's major responsibilities on this project was the design of the five reinforced concrete river piers (Piers 5,6,7,8, and 9).

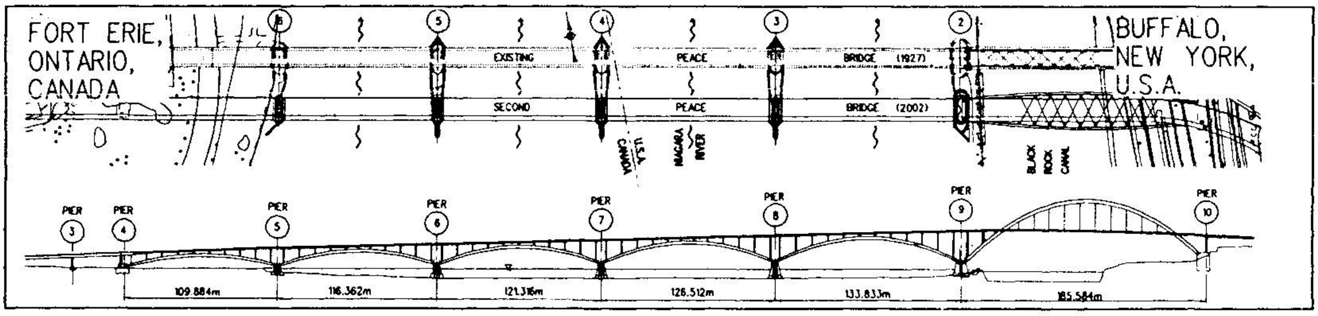

Figure 1. General Arrangement Showing Existing and New (Second) Peace Bridge.

2. INNOVATIVE PIER GEOMETRY

The piers are to be of mass concrete and, like the existing piers, will rest directly on the bedrock at the bottom of the river, with no anchorage into bedrock except for some form of shear keys.

The general layout of the piers was essentially constrained by two factors: (1) the general shape of the piers should be architecturally similar to those of the existing structure; (2) requirements by permitting authorities that the new structure not cause any increase in water elevation levels immediately upstream of the bridge, or hence, in Lake Erie.

Preliminary drawings of the piers called for them to be of equal width (in the longitudinal direction of the bridge) to the piers of the existing bridge. It was recognized, however, that the piers could be made thinner given that the pin separation (between adjacent arches) of 4.0 m was substantially less than the 7.01 to 9.45 m separation on the existing bridge. At the same time, detailed hydraulic computer models prepared by Delcan's hydrotechnical engineers indicated that this preliminary configuration would lead to an unacceptable increase in the backwater effect. Thus, it was decided to reduce the width of the piers to mitigate the backwater effect. This was found to have some beneficial effect on the hydraulic profile, but not enough to satisfy the strict regulations.

The hydraulic modeling indicated that the backwater effects were largely the result of eddy effects between the proposed and existing piers. Delcan proposed an innovative scheme in which the new and existing piers would be linked to provide a single, continuous object around which the river would flow. As a further refinement, a teardrop shaped tail was added to the rear of the existing piers, much like the trailing edge of an aircraft wing [4]. Revised hydraulic models proved the validity of the proposed link and tail as backwater effects were reduced to near-zero. A typical river pier, including link and tail, is shown in Figure 2.

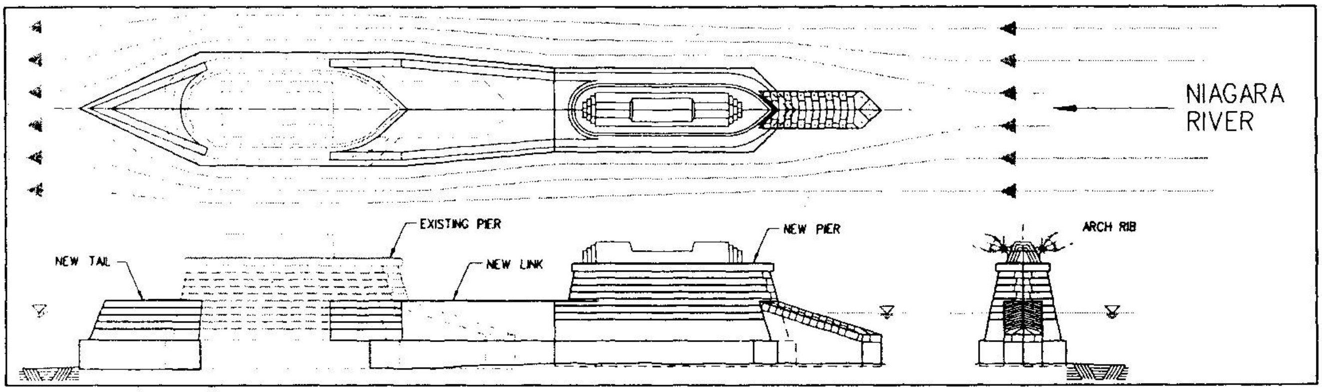

Figure 2. Typical River Pier, Including Link to Existing Pier and New Tail on Existing Pier

3. ESTABLISHMENT OF PERFORMANCE CRITERIA

The AASHTO LRFD Code provides the engineer with a degree of latitude in the design of piers. The only explicit instruction, given by Clause 11.7 is that "[p]iers shall be designed to transmit the loads on the superstructure, and the loads acting on the pier itself, onto the foundation." For the mass concrete piers under consideration, this essentially implies that the piers shall be able to resist sliding and overturning under all relevant combinations of vertical and horizontal loads.

3.1 Sliding

For the pier buoyant self-weight and applied arch forces shown in Figure 3(a), the available sliding friction resistance, $F_R = \mu\ (F_{W,V} + F_{E,V} + W_b)$ must exceed the unbalanced lateral force, $F_L = F_{W,H} - F_{E,H}$, shown in Figure 3(b) Performance under sliding may be expressed as a demand / capacity (D/C) ratio, $D/C = F_L / F_R$, with values less than 1.0 indicating that sliding does not occur.

3.2 Overturning

Similarly, overturning about some arbitrary point, O, will not occur if the available resisting moment exceeds the applied overturning moments, as shown in Figure 3(c). The resisting moment, $M_R = R_V \times d$, is the product of the vertical reaction, $R_V = F_{W,V} + F_{E,V} + W_b$, multiplied by the maximum deviation, $d = w/2 - b/2$, of the reaction from the centreline of the pier; b is the minimum width of the compression block which will just sustain the reaction force without exceeding the compressive strength of the concrete or bedrock. Again, performance may be expressed as $D/C = M_O / M_R$, with values less than 1.0 indicating that there is sufficient overturning resistance.

3.3 Other Criteria

While the sliding and overturning criteria ensure stability of the piers at the ultimate limit state (ULS), it is also desirable to ensure satisfactory day-to-day performance at the service limit state (SLS). After some discussion, it was decided that in order to preserve the integrity of the pier foundation / bedrock interface, the entire interface should remain in compression for all SLS loads. Assuming elastic behaviour at SLS, the net compression stress, σ_{net}, at the extreme edge of the foundation is equal to $\sigma_{axial} - \sigma_{moment}$, where $\sigma_{axial} = P / A$, and $\sigma_{moment} = M / S$. Negative values of σ_{net} indicate unacceptable uplift. As an additional measure of confidence, a minimum acceptable value of $\sigma_{net} = 0.25\ \sigma_{axial}$ was established.

While the foundation was to be in compression, it was recognized that at other sections (the foundation / body interface, particularly) there may be a light tension field under certain loading conditions. This was deemed to be acceptable, provided that such tensions were well below the concrete cracking strength and would not require special reinforcing treatment.

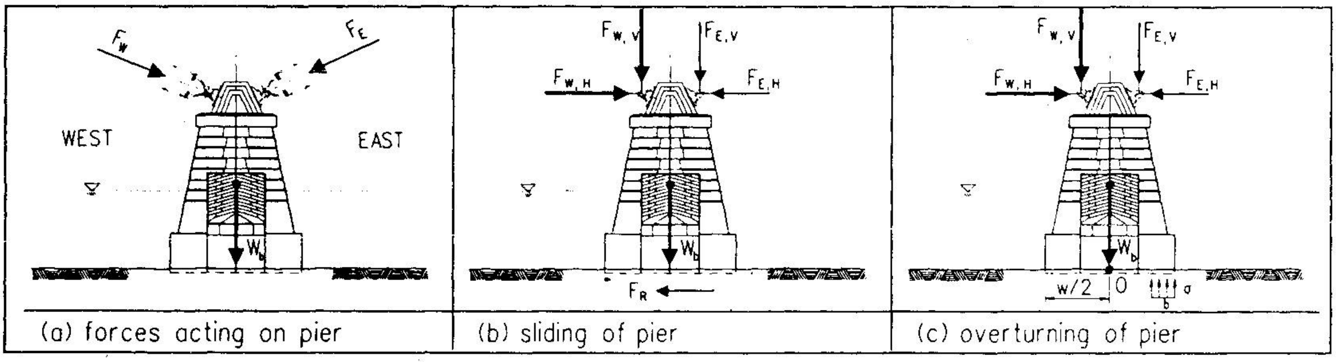

Figure 3. Sliding and Overturning Forces Acting on a Typical Pier

4. RELEVANT LRFD LOAD CASES AND COMBINATIONS

Load cases, factors and combinations were determined, in general, as set out in AASHTO LRFD, Clause 3.4. For each design element under consideration, it was necessary to determine the load combination(s) controlling the design of that element. Of particular interest for the pier design, was the fact that loads are imposed on the pier by arch ribs from adjacent arch spans which are structurally independent of one another. While this presented the possibility that different load factors and combinations could be used for adjacent spans, to produce the most severe loads on the piers, Delcan recognized the need to use sound engineering judgement and to consider the *intent* of the LRFD provisions, to determine conservative, but realistic loads.

First, it was recognized that LRFD ULS combinations typically represent a particular loading event, so its not realistic to use different ULS combinations on adjacent spans (e.g. ULS-III, which represents the instance of very high winds, but no vehicles: this is not compatible with load combinations where live loads are present). Second, whereas load factors for permanent loads, defined in LRFD Table 3.4.1.2, may have high or low values (e.g. 1.25 or 0.90 for dead load), it is not likely that actual permanent loads would be less than their nominal values in one span and greater than nominal in the adjacent span. Indeed, this is recognized explicitly in the LRFD Commentary C3.4.1. Permanent load factors were thus kept constant within a given combination.

Distribution of live loads was of particular importance given the structural independence of the arch spans. That is, the presence of traffic on one span, but not on the adjacent span produces an extreme case of unbalanced thrust on the pier between these spans. While the probability of such an event was believed to be very small, specific situations where this might occur are conceivable (e.g. re-opening the bridge to traffic after a closure) and the decision was made to include this distribution in the applicable SLS and ULS combinations.

6. DETAILED DESIGN

Detailed design of pier reinforcement proceeded in accordance with LRFD Section 5. As with general pier design considerations, LRFD is rather vague regarding reinforcing details for large concrete piers. It is of some interest to note that the piers of the existing bridge are unreinforced below the pier caps, and only lightly reinforced above.

6.1 Pier Foundations

Foundations were found to be primarily in compression and were thus treated as structural mass concrete elements. Nominal shrinkage and temperature steel consisting of 20M bars at 300 x 300 mm spacing was provided in accordance with LRFD Clause 5.10.8.3.

6.2 Pier Bodies and Pier Caps

Pier bodies were initially regarded as compression elements. Strict adherence to LRFD Clause 5.7.3.3.2 requirements for minimum compression reinforcing, however, would have resulted in extreme and unnecessary quantities of vertical steel. Careful consideration of the problem indicated that the typical compression stresses were very low (in the range of 0.4 MPa), and thus, the pier body could best be regarded as a bending element, where light precompression is beneficial [3]. Further analysis showed the maximum tensile stress in the concrete (caused by bending moments) to be in the range of 0.5 MPa, well below the concrete modulus of rupture, about 3.5 MPa. Thus, it was not necessary to provide any more steel than the nominal face steel required for mass concrete.

6.3 Pier Thrust Blocks

Pier thrust blocks were idealized as compact compression elements between the arch bearings. Reinforcing was provided in accordance with LRFD Clauses 5.7.3.3.2, and 5.7.4.6 pertaining to minimum requirements for longitudinal and transverse confining steel. Additional end-hooked vertical bars were supplied to provide complete confinement of the block in three orthogonal directions. These vertical bars also serve to enhance shear transfer from the thrust block to the pier cap, as required under conditions of unbalanced arch thrust loads. Reinforcement for a typical pier is shown in Figure 4.

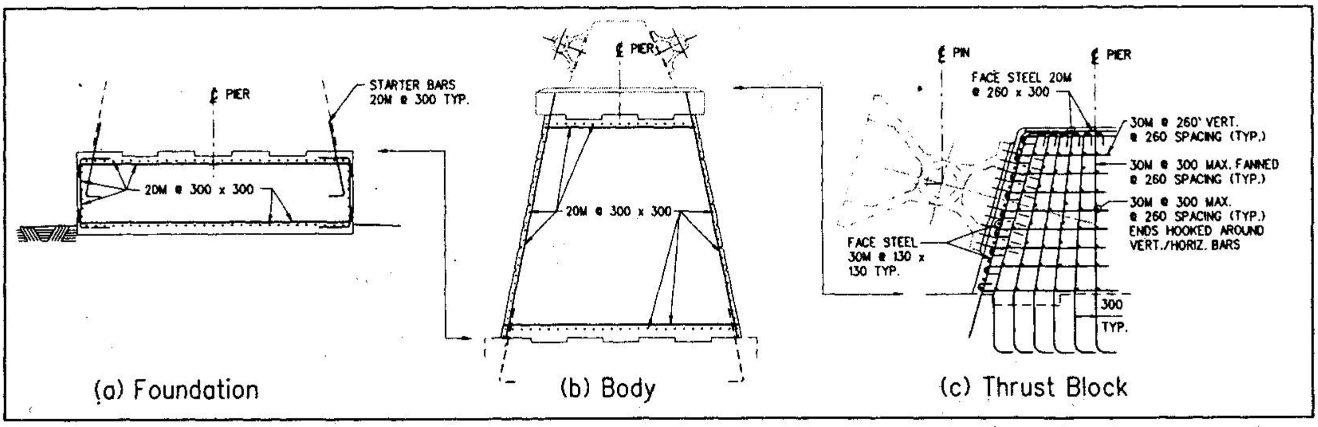

Figure 4. Typical Pier Reinforcement

7. CONSTRUCTABILITY ISSUES

Construction of the piers for the existing structure was performed using pre-fabricated braced timber cofferdams, sheathed in sheet piling and sealed using bag concrete placed by divers. By all accounts [2], placement of the cofferdams was very difficult in the fast-flowing Niagara River. It is anticipated that a refined cofferdam scheme will be used for the construction of the new piers. Delcan's design for constructability addressed a number of issues including the following:

Compatibility With Cofferdam or Alternative Construction Methods - the simple layout of reinforcement, particularly in the foundation and pier body, facilitates the use of braced cofferdams as the design is not particularly sensitive to gaps in the reinforcement required to allow bracing members to pass. Further, should proponent contractors wish to explore alternatives such as "dry-dock" fabrication, necessary modifications to the general design concept may be easily achieved.

Simplicity of Reinforcement Placement - whereas the geometry of the thrustblocks is irregular and changes from pier to pier, it is not necessary to employ complex reinforcement layouts to achieve the desired performance. Reinforcement has been laid out in simple grid patterns to facilitate its placement and to make allowance for the placement of anchor bolts to fasten the pin bearing plates.

8. CONCLUSIONS

The Design of such unique pier structures requires careful consideration, and it is necessary that the engineer have a fundamental understanding of the intent of the applicable design code clauses. With this in mind, Delcan's engineers have produced an efficient design which meets all structural, aesthetic, environmental, and constructability criteria.

The authors look forward to presenting a companion paper on the construction of the piers and bridge after the project goes to construction in the Spring of 1999.

9. ACKNOWLEDGEMENTS

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