Limestone rock sockets for Suncor's Athabasca River Bridge

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Abstract: Suncor Energy constructed a bridge over the Athabasca River to link its oilsand mining lease and processing facilities. The bridge is carried on four in-river piers each consisting of two piles founded on rock sockets each designed for 35 MN load (factored.) As the limestone in the area was known to be highly variable in strength and characteristics and is also subject to sinkhole activity, a detailed coring program was undertaken as part of the design. The paper will describe the above challenges, present details of the site investigation program, and the design of the rock sockets.

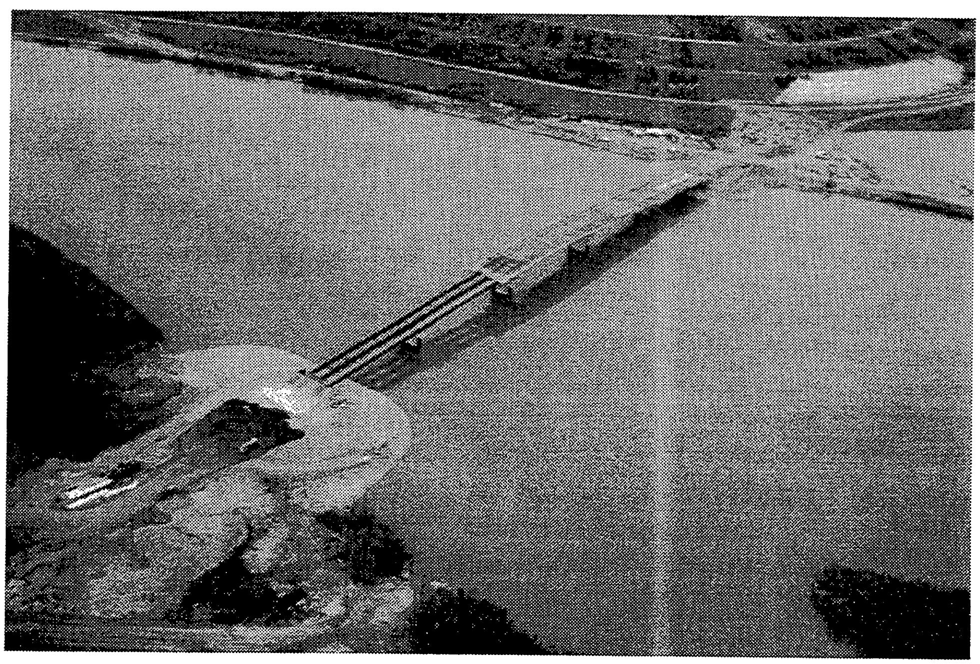
Overview of bridge

Suncor Energy required a major bridge crossing of the Athabasca River to connect their existing west bank operations and bitumen processing facilities with the new Steepbank and Millennium Mines on the east bank. In February 1996, Suncor released a request for competitive design / build tenders for this bridge. The successful tender was made by Peter Kiewit Son Co. Ltd, with Associated Engineering Services Company as Design Consultant, who sub-contacted geotechnical and related services to AMEC [formerly AGRA] Earth & Environmental Limited. AMEC were instructed to proceed with drilling and design services in June 1996. Site investigation was undertaken in July, with concurrent design activites, and installation of the main river piles was accomplished during September and October 1996. The bridge was opened to limited traffic on September 1997, about 2 months ahead of the final schedule. This however was about 12 months ahead of the initial schedule at the tender stage. The purpose of this paper is to discuss the issues and design approach for the limestone rock sockets required to support the main piers.

The bridge is a multi-span 384 m in horizontal length with four main piers and span lengths of 2@69 m and 3@82m. The piers extended to the bridge deck and were connected above water level by a 700 mm thick diaphragm. The vertical geometry of the bridge required a 6% grade for the river navigation channel.

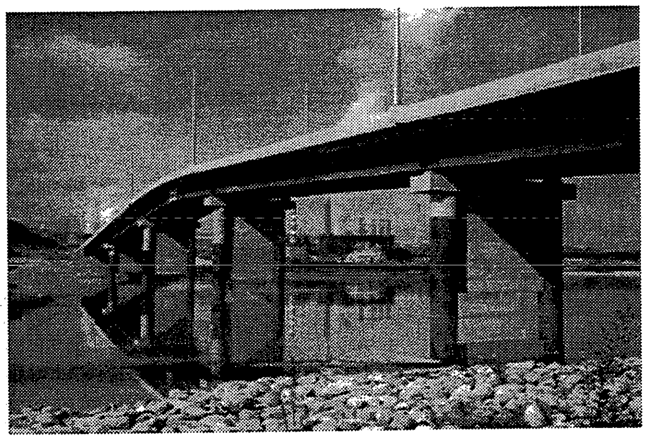
Fig. 1 provides an aerial view of the bridge under construction. This view was taken from the west bank looking to the south-west. The main piers, the abutment fills and the internal structures of the bridge are generally apparent. The wide west abutment fill includes a 7 m berm to permit migration of ungulates along the west bank of the Athabasca River. Fig. 2 views the structure from underneath, looking from the east abutment.

Fig. 1 The Suncor Bridge under construction viewed from the north-east with Tar Island Dyke visible beyond the west abutment



The main piers consisted of two 2362 mm steel piles supported by 2438mm (96") rock sockets advanced by a rock auger. Pier foundations were installed through the vibration of a 2400 mm diameter steel casings, down to the bottom of the river alluvium, and into limestone, in some installations this casing had to be re-driven into the limestone to get a seal. The casings were then cleaned out with an air lift and the piles were then socketed into the underlying limestone. The total pile length was then typically about 20 m in river alluvium and 12 m into the limestone. Most of the sockets were drilled in from a barge, but the western most Pier 1 was installed from a temporary causeway fill. Much of the superstructure was rigged working off an ice bridge.

Fig. 2 Looking under the bridge from the south-east



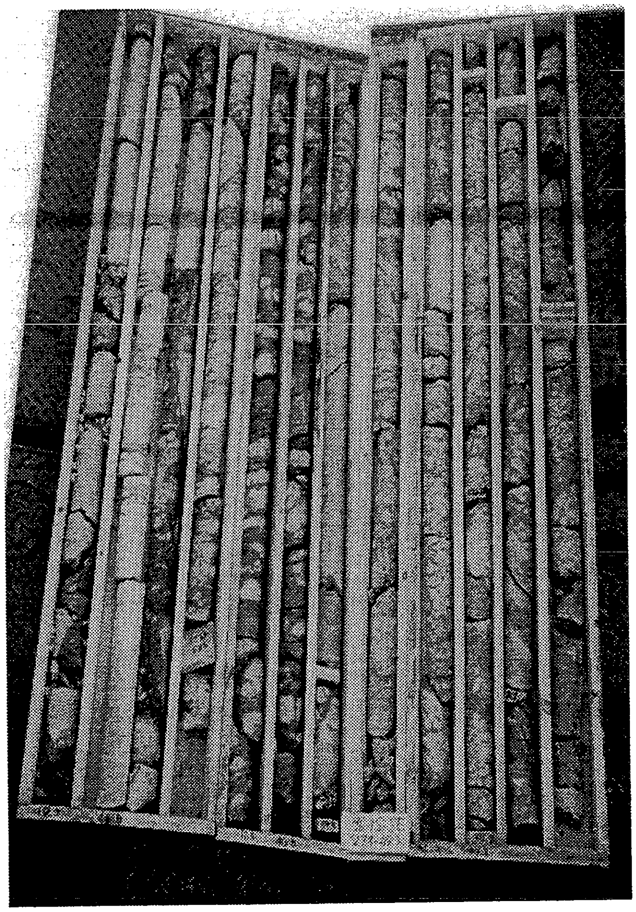
The bridge specifications included the requirement for a design [empty] heavy haul truck one-way, light 2-way traffic, and covered pipelines in an above-deck configuration. Design loading including the effects of ice jamming as well as direct and glancing impacts of possible barge traffic along the Athabasca River. The vertical design loads provided to AMEC were a factored load of 35,000 kN, and an unfactored load + Ice of 30,000 kN. These loads were provided at a specified elevation of 224.5 m. As pile tips were installed as low as elevation 196.6 m, loads were increased below this depth by adding the unfactored weight of additional concrete to both factored and unfactored loads. This additional loading was up to about 10% of the design load. Laterally loading including allowance for scour was also a major design issue, but this is not considered in the paper.

Devonian limestone foundation

Geology and structure

Preliminary in-river site investigation, and general site knowledge established site conditions consisting of a variable depth of medium dense fluvial deposits overlying limestone. For geotechnical purposes the limestone was characterized by a classification system due to Matthews et al (1980). This system subdivided the Waterways Formation limestones into 5 major categories of Class A, B, C, C/D and D. These five units containing rocks from weak calcareous shales that can be crumbled by hand, to bioclastic zones that ring when hit by a geologists hammer. In the region, the Devonian succession rests unconformably on the Precambrian surface. Fig. 3 illustrates the variation in limestone with depth in a set of cores. A variety of structural features observed on both the Precambrian surfaces and within the Devonian succession indicate that regional tilting, faulting, salt collapse, folding and faulting have modified the deposit. The Devonian surface itself has been described as an egg-crate due to the abundance of collapse and karst features that are present. Some of these features may not be karstic, but rather result from the upwards penetration of collapse features initiated by salt collapse at depth. These collapse or karst features are infilled with a range of weak limestone rubble, calcareous shales, and in some instances coal and oilsands.

Fig. 3 Typical limestone core



The various members of the Devonian limestone are highly bedded. However the complex history of both the original depositional environment, coupled with post depositional changes has resulted in highly complex bedding. Exposures of limestone in pit floor quarry pits, and drilling investigations on both Syncrude and Suncor leases make it clear that extrapolation of conditions between corehole locations even as close as 10 to 20 m is problematic.

The geology of the limestone near to and within the valley of the Athabacsa River is deceptive. Reconnaissance along the river north of Fort McMurray will reveal the frequent exposures of folded and faulted limestone, and the impression could be gained that bedrock will be close to river level. This is not the case, and there is evidence at several locations of a deep channel incised into bedrock and infilled with fluvial deposits. Several sites can be considered summarized on Table 1. Prior to the detailed design of the bridge this information as well as the results of preliminary drilling on the Suncor bridge alignment were known. Subsequently drilling for both abutments confirmed the west channel and in addition as listed on Table 1, a deep channel on the other side of the active channel underlying the east abutment.

Table 1 Evidence of Deep Channel in Athabasca River

Location	Method	Surface Elevation MSL [m]	Limestone Contact MSL [m]	Depth of Channel [m]
Suncor Bridge and vicinity	Drilling	232	< 182 west 195 east	> 50
Lougheed Bridge near Fort Mackay	Drilling	232	182	50
Isadore's Lake	Drilling	233	147	86
Daphne to Sutherland Island	Airgun/ Uniboom On Athabasca River	230 +/-	135	95 to deep reflector

Infilled sinkholes are often encountered in ore resource corehole programs and when the mine pit floor is stripped to limestone. Sinkholes are entirely random in location with no known predictive tie to geology, although there is a tendency to find more closer to the Athabasca River. A summary of sinkhole observations is given Table 2. The area ratio (number of sinkholes per unit area) is higher at the sizer location an inpit facility where the limestone surface was carefully exposed. These data on sinkhole geometry are for sites at some distance from the river. There was no reason to believe that the sinkholes would not be found beneath the river and in fact conditions may be worse, not better due to more intense karsting activity associated with a possible ancient incised valley of the Athabasca River.

Table 2 Summary of Sinkhole Observations

Location	Number of sinkholes /area	Diameter of sinkholes	Probability Density [p]: Area of sinkholes / Area investigated
Suncor Lease 86/17	6 to 12/370,000 m^2	24 to 46 m say 30 m average	0.012 to 0.024 or possibly lower for a lower average size, but not all sinkholes encountered.
Pit6 Sizer	3/14,000 m^2	9 to 21 m	0.035

Sinkhole implications

Sinkholes are infilled with very weak material relative to the limestone, and if encountered at a river pile location would result in significant design and foundation schedule implications within an already tight design, fabrication, and construction schedule. Going deeper with much longer sockets was not a practical option based on the type of equipment available to the contractor. While no sinkholes had been found in the feasibility phase, sinkholes could not be ruled out. In order to support the significant cost of the site investigations required to confirm foundation conditions, it was to decided to undertake a statistical analysis. A statistical analysis was considered relevant given the general observation that sinkholes could occur anywhere and with no geological clues to their locations.

Approach

The problem is how many boreholes must be drilled to give a good level of confidence that a sinkhole will not be encountered within the bridge alignment, and more specifically at the pier locations. Baecher (1972) has presented the probability of locating a given sized target as a function of the target diameter/ grid spacing ratio. The target in this case is a more-or-less round sinkhole. It is first instructive to apply this theory to the sinkhole database. Using statistical theory it can be shown that for the lease drilling spacing of about 60 m there is a 99 percent chance of hitting a 85 m diameter sinkhole, 80 percent of hitting a 61 m sinkhole and 25 percent of hitting a 43 m size. Even if the actual drilling was at a spacing of 30 m there still would be a 50/50 chance of missing a 24 m sinkhole. Therefore many smaller sinkholes may be missing within the mining database. Given the prospects for higher karsting activity in the river bed itself it was argued that the sinkhole area to spacing area probability of 0.05 was possible, with a 0.025 ratio being quite reasonable in light of the data. The next issue is what is the realistic minimum size of sinkholes.

They have been seen with a surface expression of as little as 3 m, but this is unusual and the typical minimum size is 6 to 9 m was adopted.

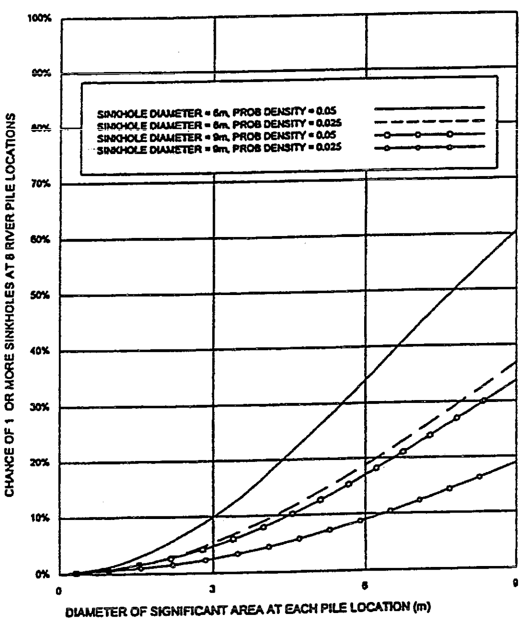
Chances of a sinkhole being present

Let us now consider the design area and suggest that the area of impact or design concern for one pile is called a "significant area", or $A_{significant}$. This is considered to be an area based on somewhere from two to three times the pile diameter, or 5 to 7.5 m. There are 8 pile locations. The probability of finding one or more sinkholes of area $A_{sinkhole}$ at 8 consecutive pile locations is represented by a series of independent Bernoulli trials, following a procedure suggested by Vick and Bromwell (1989) to be:

[1] $P\{\text{finding}\} = 1 - [(1 - (p)^n]$

and where $n = 8 \times A_{significant} / A_{sinkhole}$. With a probability density p of 0.025 to 0.05, P{finding} can be calculated. Consider that the design would wish to ensure that there is no sinkhole within an area $A_{significant}$ represented by at least 2 pile diameters (i.e., 5 m). The chances of actually having one or more sinkholes at 8 consecutive pile locations is therefore from about 10 to 25% based on this method of analysis. A summary of this analysis is given in Fig. 4.

Fig. 4 The results of Bernoulli trials predicting the chances for finding one or more sinkholes



CHANCE OF 1 OR MORE SINKHOLES
AT 8 RIVER PILE LOCATIONS

Tender strategy and drilling grid spacing

At the time of the tender, the possible presence of sinkholes had not been addressed in the feasibility study (undertaken by others) and may not have therefore been a subject of concern in other tenders. While sinkholes might also be present at the abutments they could be more readily handled during final design. A real issue concerning the tender was the overall schedule, and the necessity to commit to an in-river pier spacing in order to pre-fabricate structural elements before the eight river piles were installed. This was a significant risk to both the contract in terms of bonus / malus provisions, and to Suncor on the overall east bank mine development. The tender identified the issue, developed a plan to confirm the suitability of each pile location, and most importantly the bridge designer developed a plan to allow movement of a pier location up to 20 m in the unlikely event that a sinkhole was not located during the drilling phase, but was located during the actual pile installation. This turned out to be a successful risk management strategy at the tender stage.

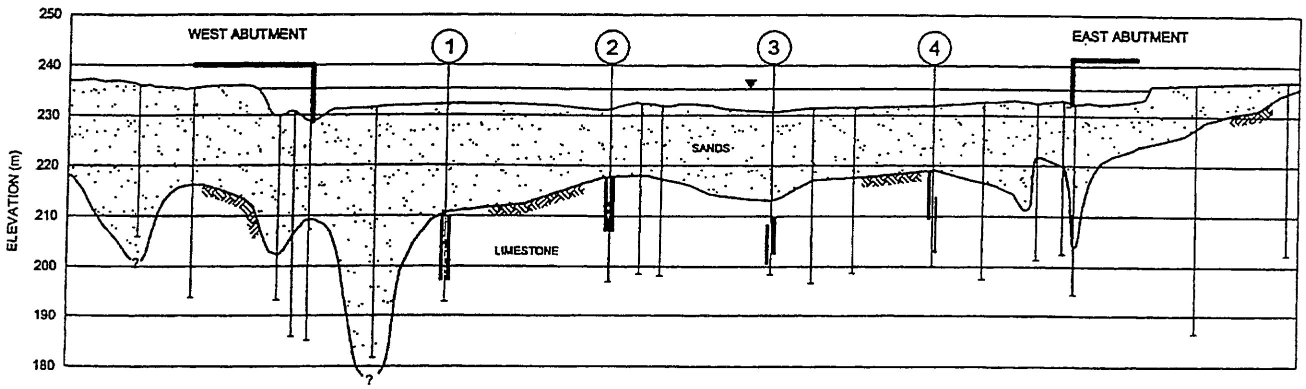
Based on the significant probability of actually encountering one or more sinkholes the next question is what grid spacing is required to give a high level of confidence that if a sinkhole is present at a pile location it in fact can be located. For a high confidence level, Baecher's prediction require a spacing of 5 m to 7.5 m for 6 to 9 m targets (i.e., the minimum sinkhole diameter) to give a high confidence (ie converging on 100%) that if present that a sinkhole is detected.

For each pile location, the recommended coring configuration was a total of 5 coreholes. One was located at the centre of the pile, and four at the corners of a 6m square centred on the pile centreline. In order to accomplish this Kiewit mobilized in a large barge which allowed all 10 holes for a pier location to be drilled from the barge surface with no moves at a pier location. The barge deck provided a template of 10 cut-outs exactly located at the required spacing. Two drilling rigs were used to speed up the site investigation program.

Design

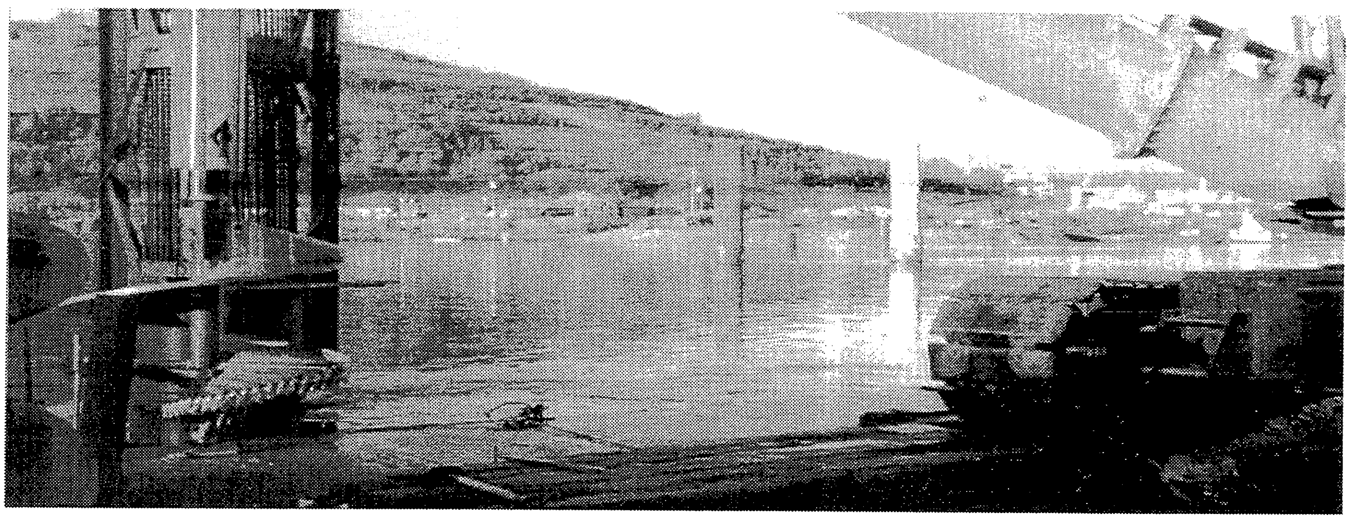
A representative cross section along the approach fills and bridge alignment, see Fig. 5, shows the variation of the fluvial sands overlying the limestone. The deep channel along the west bank, extending to deeper than elevation 182, is apparent. There is an indication of an additional deep channel west of the west abutment. No sinkholes were found at the 8 socket locations. However a deep channel was also found immediately beneath the east abutment, see Fig. 7. It is possible, discounting the seismic survey, that the interpreted deep channels of Table 1 and Fig. 5 are nothing more than deep sand infilled sinkholes.

Fig. 5 A representative cross section along the bridge alignment showing the abutments, elevation of the limestone, deep sand channels, and the 8 rock sockets



However the mechanism as to how such deep 'holes' could be infilled with relatively loose sands and gravels with occasional clay layers is not clear. The presence of a deep channel along the thalweg of the Athabasca River is certainly indicated. Fig. 6 provides a view of the west bank as well as the 2438 mm auger used to install the rock sockets.

Fig. 6 On the river at Pier 3 looking west, and showing the 2438 mm rock auger used for drilling out the rock sockets



Limestone characterization

The method used to characterize the limestone is presented in Table 3. This system was used to describe all core. Example of the core log set for Sockets 3N and 4S are given in Fig. 7 and Fig. 8. These figures shows all 5 holes drilled at a socket location, and were prepared for each socket. Hole 96-3N-C was drilled at the socket centreline and the other 4 holes at the corners of a 6 m square centred on the middle hole. This set of holes indicated an extreme example of the variation of the top of the limestone. At this location the channel deposit / limestone contact dropped about 8 m over a horizontal distance of 4.2 m. Whether this was an intact limestone pinnacle or a block of disturbed material was never resolved, but the socket was placed below it. The variability of the rock with depth and laterally is also indicated on Fig. 6.

For the socket design , the other major source of pile design data is the unconfined compressive strength UCS (i.e., also referred to as q_u or σ_c) which forms the primary basis of the methodology used for the design of rock sockets. As shown on Table 3 several methods of determining the design UCS values are shown and are listed as follows. In Table 3 these values can be compared to the literature as reported earlier by Matthews et al (1980) and reported in Table 3. It can be seen that reasonable agreement exists for Class A and B, but site data requires a much lower design selection for the other classes. Table 3 also reports the conservative design strengths adopted for pile design.

Fig. 7 The 5 coreholes drilled for Socket 3N and showing the as-built socket

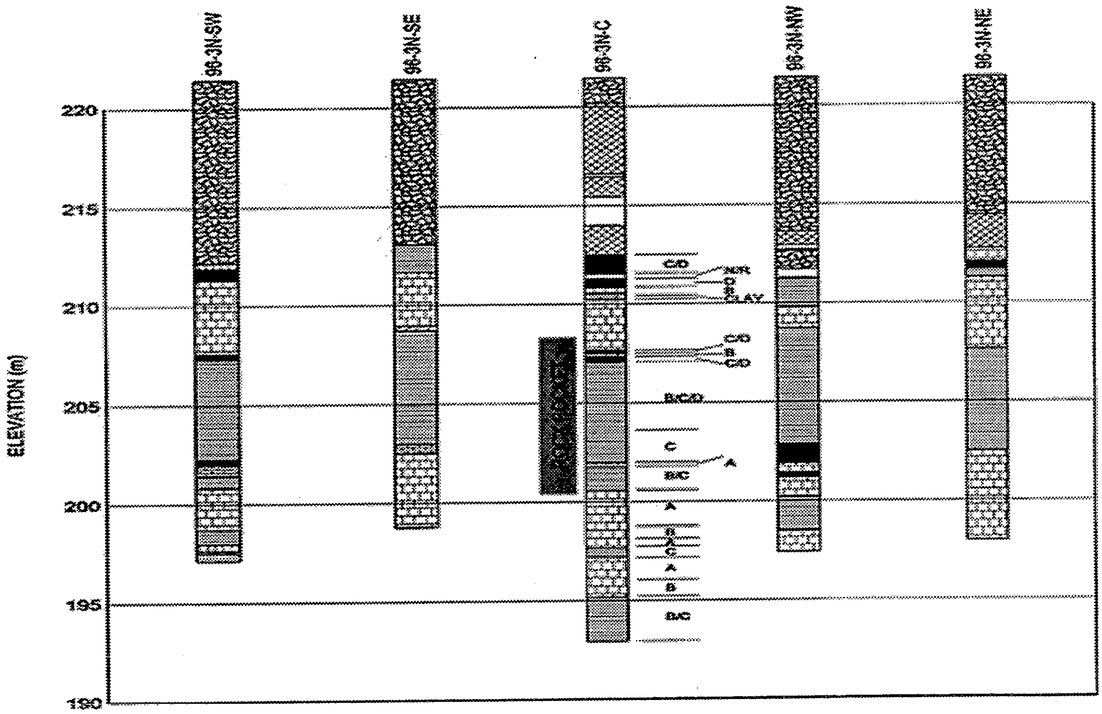
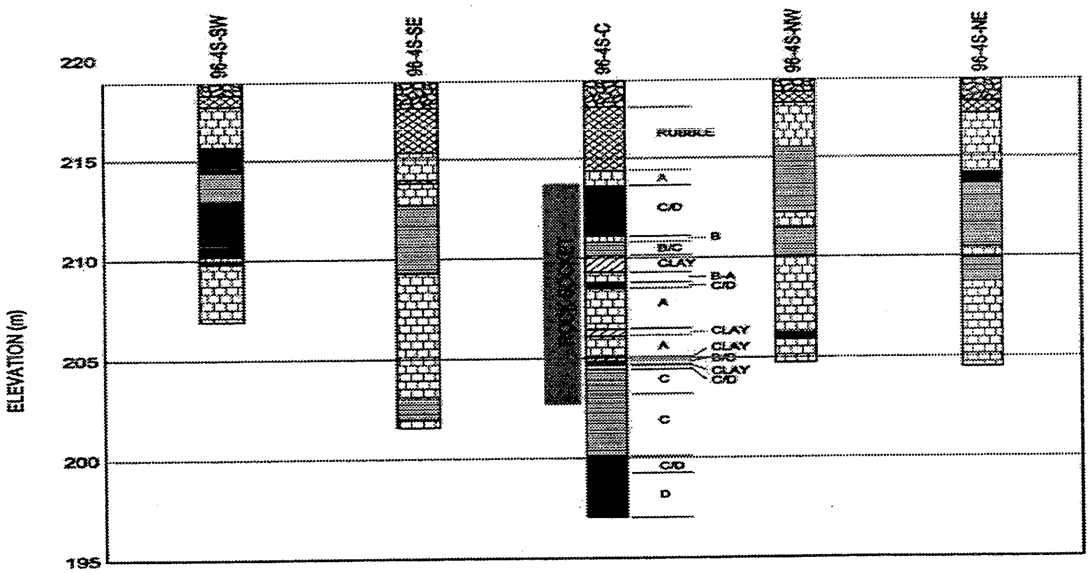


Fig. 8 The 5 coreholes drilled for Socket 4S and showing the as-built socket



Pile load test

A pile load test was undertaken in Class C/D limestones. The test was undertaken jacking upwards, and testing a 3.0 m long shaft in compression. The testing was undertaken using a bi-directional Osterberg Cell and with the testing being done by Loadtest Inc. This test indicates the ultimate capacity of the jack was reached at 7.5 mm deflection of the base of the shaft. A possible slip of the bond occurred at 2 mm with subsequent increase in load to full deflection. Assuming the pile slipped at 2 mm the inferred bond stress was 0.72 MPa. The q_u of one test in the C/D shaft wall was 7.4 MPa, but which was considered to be unrepresentatively high. The bond factor is therefore 0.26 expressed as $\tau = 0.72 = 0.26(q_u = 7.4)^{1/2}$ The inferred rock modulus is about 1,550 MPa, which is considerably higher than the value adopted for design for Class C/D rocks (which was based on a sensible lower bound compressive strength), but is more in accordance with the high strength measured.

Table 3 Major Groups in Limestone

Type	Description	Unconfined Compressive strength / q_u [MPa]				ISRM System
		Literature	Mean	Lower Bound	Design	
A	Strong massive with occasional stringers of calcareous shale or argillite	34	56	35	35	Med Strong to Strong Rock
B	Moderately strong, generally massive or thickly medium bedded argillaceous limestones (with numerous random shale or argillite stringers).	28	25.3	17.4	16	Weak to Medium Strong Rock
C	Moderately weak to weak, thinly interbedded limestones and calcareous shales. This material may be susceptible to minor slaking on exposure.	14	8.4	3.4	2	Very Weak to Weak Rock
D/C	Interbedded moderate weak to weak shale or limey shale. This material is susceptible to slaking on exposure.	10	2.2	1.1	1.1	Extremely Weak to Very Weak Rock
D	Weak, very thinly bedded shale or limey shale, grading to massive beds 1 to 3 feet thick. Material strongly susceptible to slaking on exposure	0.3 to 3	0.63	0.37	0.37	Extremely Weak Rock to Very Stiff Clay

Design parameters

In order to satisfy all parties involved, the design considered both working stress and limit states designs using both the unfactored and factored loads presented earlier. Thus both factored shaft and tip resistances were obtained by using a factored resistance $f_c = 0.5$ for cohesion. The working stress designs considered an overall factor of safety of 2.5.

Shaft Capacity

The shaft capacity adopted a relationship between τ_u the ultimate shaft capacity and the USC as:

[2] τ_u =factor (q_u)$^{1/2}$(MPa)

A major study by Rowe and Armitage (1987) as well as work on weak clay shales by Hooley and Brooks (1993) was used to establish the design parameters provided in Table 4.

Table 4 Shaft Design Parameters

CLASS	q_u MPa	τ_u kPa	E_r MPa
A	35	1500	800
B	18	1000	500
C	3	500	200
C/D	1.1	200	150
D	0.37	100	60

A deformation modulus E_r was required in order to provide an estimate of pile deflections, as well as undertake designs for combined side shear and end bearing sockets. Based on work presented by Rowe and Armitage (1987) and experience in the area, the E_r as presented in Table 3 were selected.

Tip Capacity

A summary of tip capacities is given in Table 5. For conventional end bearing, Rowe and Armitage (1987) and Wyllie (1992) recommend allowable tip capacities, with factors of safety of 2.5 to 3.0 as $Q_a = q_u$ (tip area) is given in Table 5. There are several conditions to be met to rely on this approach. The pile tip rock contact must be horizontal.

The base of the socket must be at least one pile diameter below the rock surface, and the rock to a depth of at least one pile diameter below the base of the socket is either intact or tightly jointed with no compressible seams or gouge-filled seams.

For heavily jointed rocks, the CFEM (1992) consider that the allowable bearing pressure is:

[3] $q_a = K_{sp}\, d\, q_{u,average}$

where K_{sp} is based on discontinuity spacing and is considered to be in the order of 0.1 for moderately close discontinuity spacing of 0.3 to 1 m. Given the observed vertical joint spacing of 0.4 to 0.6 m, a K_{sp} of 0.1 is appropriate for Class A and B rocks. The depth factor d depends on L/D and cannot exceed 3 for L/D > 5 which is typical for the river piles. However given the variable nature of the rock above the nominal tip elevators this "d factor" was discounted to 2.0. A summary of the capacities calculated is given in Table 5. Ultimate Limit States or Factored capacities of 8.8, 4.5, and 0.75 MPa are selected for Class A, B and C rocks. Other class rocks were not considered as the capacities are considered to likely be too low to provide any tip resistance.

Table 5 Summary of End Bearing Capacity (MPa)

Rock Type	$q_A = q_u$	q_a CFEM $K_{sp} = 0.1$ D =2.0	q_{uls} =1.25 x $q_{a,CFEM}$
Class A	35	7	8.8
Class B	18	3.6	4.4
Class C	3	0.6	0.75

The potential application of a tip formed by the shape created by the 96" rock auger proposed by Kiewit was also considered in the design stage. This shape is formed by a semi-cone-shaped concave down bit with about 0.9 m protrusion forming an approximate cone angle of $2\theta = 106^0$ from the tip to the outside diameter. While Kiewit indicated there would be some potential of cleaning out off the socket base, some disturbance and broken rock had to be considered. A model was developed assuming that the competent rock is fractured into the equivalent of a dense, interlocked, but rubbly "gravel". Within this model a key assumption was that the weight of the fluid concrete placed to river level will exert an effective stress of about 350 to 500 kPa from the highest to lowest tip. The use of retarders, and low insitu temperatures would result in the full weight of concrete to the barge surface being exerted on the rocks potentially disturbed by the tip. This load will act to firm up the looser beds or rubble, and this assumption formed the basis of the method considered. This will tend to consolidate any loose beds within the disturbed zone of influenced by the bit. Methods in Reese and Wright (1977) were used to estimate an allowable load at a tip deflection consistent with shaft deflection.

Pile capacity

Geological interpretations made it clear that the obvious design choice was to found the socket on the competent Class A and B limestones. All piles could be founded on such units assuming that measures were taken to ensure a clean smooth contact. However, because of schedule and timing considerations the contractor did not want to spend any more time than necessary on each hole, and did not wish to rely on a complicated cleanout requirement. For this reason the design evolved to a consideration of both shaft and end bearing, including consideration of the possible fractured tip as discussed above. Designs were done using a spreadsheet model in which limestone stratigraphy contained the class units with depth and incrementally calculated the factored capacity for each class / depth increment. As part of this spread sheet a weighted rock modulus was also calculated. This modulus was used to predict the pile deflection under the unfactored load (with ice and increased pile weight below Elevation 224.5 m.) using the procedures given by Pells and Turner (1979).

The spreadsheet applied the factored shaft capacities to each increment of rock according to the unit capacity and irrespective of the thickness of a given unit. For really thin say 0.3 m or less units of strong rock sandwiched between less competent weaker rocks it is possible that local bond failure of the thinner segment might occur. But, in view of the conservative position adopted in arriving at unit shaft capacities, these thinner layers were not further reduced. The factored capacity of a combined socket depends on the assumption made between strain compatibility of the shaft bond and the mobilization of tip resistance. If the shaft bond was brittle, side-slip would occur with load being transferred to the base and only a lesser or residual value available in the shaft. It is generally held that field tests indicate that shear failure in the concrete rock bond is plastic and that no drop off in resistance occurs. Two methods were considered. Firstly it was assumed that a plastic slip occurred and the full factored shaft could be added to the factored end bearing. This is the upper bound to behaviour. A second model based on Carter and Kulhawy (1988) assumes zero dilatancy along the shaft and reduces combined behaviour to a frictional model, which severally down rates shaft capacity. This model is a lower bound of expected factored capacity.

Deflections were calculated using both Pells and Turner (1979) as well as a procedure given by Rowe and Armitage (1987). Predicted deflections were in the order of 10 to 30 mm, depending on shaft length and the strength and stiffness of the limestone below the tip.

The field installation proceed by first vibrating in a steel casing caisson through the channel deposits to and into the limestone contact. River sands and gravels were extracted by air lifting and disposed of in the channel [in accordance with environmental permits]. A 1245 mm (48") diameter auger was then used to create a pilot hole to within 0.6 m of the design tip elevation. The hole was then reamed out using the 2438 mm auger. The shaft was cleaned with a wire brush, with some attempt at grooving. The hole was then cleaned out by air lifting and a 'one-eyed' bucket. Most installations were conduced off a barge, which introduced the schedule critical element of getting off the river before freeze-up. Work was completed by 31October 1996. Rates of drilling advance from both augers were found to correlate well with the corehole data. In 6 of 8 sockets the as-built tips were either at or up to 0.3 m above design elevation. In one case the tip was 0.7 m and in the other 2.9 m above design. In these cases deeper penetration by the auger was not possible. In both cases the design change was accepted by the field evidence of reasonably hard layers for bearing, and the relatively low percentage of end bearing that was required in these two cases.

The as-built sockets are located on Fig. 5. and summarized in Table 6. Most sockets were in the range of 7.4 to 13.4 m effective length. Effective length was defined as the length from the as-built base of steel casing to the tip. The total length of pile from the riverbed varied from 23.3 m to 33.7 m. For Piers 1 and 2 the sockets were both near the contact and had quite similar lengths, see Fig. 5. For Pier 3 poor limestone and seating issues resulted in a top of socket well below the contact. Pier 4 had the biggest difference between Socket 4N which was much higher relative to 4S. This was due to very poor rock encountered in the first 5m of 4S.

Table 6 Summary of As-Built Piles

Pile	Sand and Gravels, and poor limestone (m)	Effective Limestone Socket (m)
1N	20.3	13.4
1S	20.5	12.9
2N	12.7	10.6
2S	13.2	10.7
3N	23.8	7.9
3S	22.6	7.4
4N	13.9	9.4
4S	19.0	11.0

Conclusions

The investigation, design, and construction of the rock sockets, and the bridge in general were accomplished within a tight schedule, using a design build approach. In fact a whole year was taken out of the initial schedule given in the Owner's tender request, and the bridge was open to light traffic 2 months ahead of the final schedule that was established.

The geotechnical design recognized the possible impact of sinkholes and the uncertainty introduced to both the design and the construction schedule. The possible presence of a sinkhole was expressed statistically and this was found to be a useful tool in defending the site investigation program required. The recognition of possible deep sinkholes within the river bottom and ways to deal with them was a significant component of the winning bid. The field exploration methods taken to prove-out each rock socket location proved successful, and were of substantial assistance in optimising socket design. In the event, no sinkholes were encountered at the rock socket locations, however an additional depression in the river bottom was encountered at the east abutment.

Acknowledgements

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References

Baecher G.B. (1972) Site Exploration: a probabilistic approach Ph.D. Thesis Massachusetts Institute of Technology.

CFEM (1992) Canadian Foundation Engineering Manual 3rd Edition.

Carter J.P. and Kulhawy F. (1988) Analysis and design of drilled shaft foundations socketed into rock, Electric Power Research Institute EPRI Project 1493-4.

Hooley P. and Brooks S.R. (1993) The ultimate shaft frictional resistance mobilized by bored piles in over-consolidated clay and socketed into weak and weathered rock In: The engineering geology of weak rocks, Balkema, Cripps et al (eds). pp 447-455.

Mathews K., Rawlings G., and Bharti S. (1980) Investigation of the geotechnical and mining characteristics of the Devonian Limestones. In:Applied Oilsands Geoscience Conference. Edmonton, June 11-13, 1980.

Pells P.J and Turner R.M. (1979) Elastic solutions for the design and analysis of rock socketed piles Canadian Geotechnical Journal 16(3) pp 481-487.

Reese L.C. and Wright S.J. (1977) Drilled shaft design and construction manual. Vol 1. US DOT Federal Highways Administration Office Washington D.C.

Rowe R.K. and Armitage H.H. (1987) A design method for drilled piers in soft rock Canadian Geotechnical Journal 24(1) pp 126-142.

Vick S.G. and Bromwell L. (1989) Risk Analysis for Dams in Karst ASCE JGE 115(6) pp 815-835.

Wyllie D.C. (1992) Foundations in Rock. E&FN Spon.