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Laments Over Ignorant and Costly Foundation Design

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Recently in a tranquil and pleasant Ontario town, the local municipality designed a viaduct and let it out for bid. The viaduct is a 200 m long 5-span bridge consisting of two parallel 15 m wide roadways supported on four piers and two abutments. Each pier and abutment rests on a common footing of breadth of 4 m and length about 32 m.

The soil profile consists of an about 1.5 m thick layer of silty sand on an about 20 m thick layer of compressible silty clay followed by silty sand which changes into glacial till becoming very dense with depth. The till is deposited on a horizontal layer of shale limestone at a depth of about 30 m.

The general area is being developed with multi-storey office buildings and much local foundation construction experience is available. The buildings are placed on about 30 m to 35 m long, closed-toe, pipe piles driven into the glacial till and concrete-filled after the driving. Typically, the pile sizes are 178 mm (7.0 inch) and 244 mm (9.625 inch) with wall thickness ranging from 8 mm (0.315 inch) through 16 mm (0.625 inch). The actual pile used and its size (diameter and wall thickness) are proportioned to the magnitude of the load acting on each pile group. Normally, the pile groups are small, typically four to six piles, and the piles are not loaded to full structural capacity. (Reducing the number of piles in a small pile group by one pile would require the remaining piles to carry too high a load. Adjusting the wall thickness to optimize the use of the pile strength, would require an uneconomical and impractical supply of different pile sizes).

For example, the foundations of a building located a few hundred yards from the viaduct included a total of 120 piles. One pile was rejected due to excessive bending, a failure rate of 0.8 percent, which is very satisfactory and indicative of, of course, a good performance by the contractor, but also of relatively easy site conditions.

The piles supporting the building are 244 mm (9.625 inch) pipe piles with a 12-mm (0.472 inch) wall assigned a total design load of 1,156 KN (130 tons) and 178 mm (7.0 inch) pipe piles with varying wall thickness and assigned a design load ranging from 805 KN (90 tons) through 960 KN (108 tons) depending on the wall thickness of the pipe. The pile embedment lengths are 30 m through 35 m (100 ft through 115 ft). **Structural analysis** of piles strength indicates that the total load taken as dead load plus drag load due to fully developed negative skin friction is smaller than half the structural strength - a more than adequate margin. Of course, factored analysis indicates a similarly acceptable margin. **Static analysis** of bearing capacity indicates that piles must be driven to a resistance in the till of about 1,000 KN (110 ton) to ensure both adequate safety against failure and to eliminate the possibility of unacceptable settlement.

The piles were driven with a hydraulic hammer having a ram weight of 33 KN (3.6 ton). Wave Equation Analysis (GRLWEAP) was used to determine the termination criteria at End-of-Initial-Driving (EOID) and in Restriking (RSTR) one week after EOID to confirm capacity.

Dynamic testing and analyses by means of the Pile Driving Analyzer and the CAPWAP program were used to confirm hammer performance and verify bearing capacities and termination criteria at EOID and RSTR. To determine this by means of static testing would have been prohibitively expensive and not very conclusive.

Another building located even closer to the viaduct site is founded on 219 mm (8.625 inch) and 244 mm (9.625 inch) piles with 8 mm (0.315 inch) and 9 mm (0.350 inch) wall thickness and design loads of 815 KN (92 ton) and 1,030 KN (116 ton), respectively. The piles were driven with a 40 KN (4.5 ton) drop hammer. Again, wave equation analysis and dynamic monitoring were used to determine termination blow count criterion and to verify pile capacity.

The mentioned pile foundations are representative for many areas with similar soil conditions. The methods used in finalizing the design of combining static and dynamic analyses as well as the verification in the field using the Pile Driving Analyzer have been in common practice in Ontario since the mid-seventies. One would, therefore, expect that the designers of the pile foundation for the viaduct mentioned in the first paragraph would take advantage of the available local site experience as well as apply the knowledge about pile design acquired by Ontario engineers during the past about 15 years, but NO! The pile foundations of the viaduct were designed as follows.

Each of the four pier and two abutment footings are to be placed on 44 pipe piles of 324 mm (12.75 inch) diameter with a 13 mm (0.500 inch) wall thickness. The design load is a mere 698 KN (78 ton) [as judged from that the required pile bearing capacity (ultimate resistance) is given as 1,396 KN (157 ton)]. Because of an anticipated dragload of 580 KN (actually an underestimated value), for the abutment piles, the design requires that the abutment piles be bitumen coated of to reduce negative skin friction. (That also the pier piles will be subjected to the same magnitude of drag load was obviously not understood by the designer).

As a mistake one can hope will be corrected before it is ɔo late, the specifications call for the bitumen coated piles to be coated over the full length! The coating is intended to reduce undesirable negative skin friction due to the settling surficial soil layers. Coating the lower portion of the piles will reduce the desirable - and necessary - positive shaft resistance. The piles have significant lateral capacity, yet no horizontal loads are indicated in the design. To resist horizontal loads, the piles are inclined 3 (vertical) to 1 (horizontal) and 5 (vertical) to 1 (horizontal) in about equal number of piles for each inclination.

Furthermore, no fewer than six static loading tests are included in the contract specifications, but no dynamic testing, and the termination criterion is to be governed by a dynamic formula (Hiley) with the same criterion applied to both initial driving and restriking conditions!

The dynamic formula includes a factor for pile rebound for the impact. The tender drawings include a nomogram over the use of the formula with a numerical example given implying that a rebound value of 10 mm is a representative rebound value. Inserting the 10-mm value into the dynamic formula, the termination criterion for the specified pile size becomes 4 blows/25 mm. However, for long piles driven in soils typical of the site, the rebound is more often in the range of about 18 mm through 25 mm. For rebound values of 18, 20, 22, and 23 mm, the formula results in blow counts at termination of 8, 13, 26, 54 blows/25 mm, respectively. This variation should be considered in the light of that, due to the natural variations and limited accuracy of measurements, it is futile to expect to obtain the rebound value with a better accuracy that ±3 mm. Furthermore, a rebound value of 25 mm is also possible and inserting this value in the formula returns a negative value for the blow/count: -46 blows/25 mm! Clearly, the dynamic formula has no place in modern foundation engineering. Applying it to the pile installation could jeopardize the foundation safety and including it in contract specifications could open up the project to a potential dispute with the contractor about the termination criteria and one that the Owner will not win should the dispute go to litigation. (The Ontario Ministry of Transportation 1983 Bridge Design Code cautions against dynamic formulae and the 1985 Canadian Foundation Engineering Manual discourages

TABLE I STRUCTURAL STRENGTH

PILE DIA-METER	WALL THICK-NESS	STEEL AREA	CONCR AREA	STEEL YIELD	CONCR Cyl	STRUCTURAL STRENGTH Unfact	0.8EA/1000	Factored	Mod. E
mm	mm	cm^2	cm^2	MPa	MPa	KN	KN	KN	GPa
324	12.7	124	700	241	30	5090	3710	2630	56
244	12.0	88	382	303	40	4190	2510	2210	67
244	13.8	100	369	303	40	4520	2680	2420	71
244	15.1	109	361	303	40	4750	2800	2560	74
244	12.0	88	382	359	40	4670	2510	2500	67
244	13.8	100	369	359	40	5080	2680	2750	71
244	15.1	109	361	359	40	5350	2800	2920	74
244	12.0	88	382	483	40	5760	2510	3150	67
244	13.8	100	369	483	40	6320	2680	3500	71
244	15.1	109	361	483	40	6700	2800	3740	74

0.8EA/1000 is resistance at 0.8 millistrain

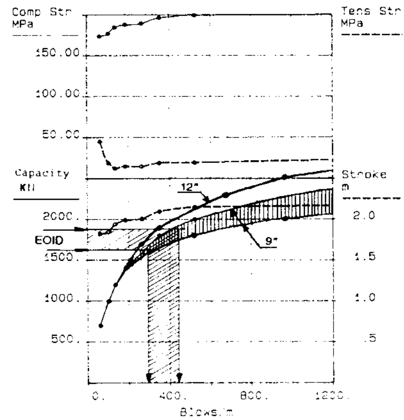

Figure 1 Bearing Graph for the specified twelve-inch pile and for nine-inch piles.

TABLE II BEARING CAPACITY

Pile Size	STATIC RESISTANCE INITIAL DRIVING Shaft	Toe	Total	RESTRIKING Shaft	Toe	Total	Factor of Safety	Load at N. P.
	KN	KN	KN	KN	KN	KN	--	KN
Installed for 700 KN Design Load								
9-inch	700	600	1300	1400	800	2200	3.14	1400
12-inch	900	800	1700	1800	1000	2800	4.00	1800
Installed for 1,200 KN Design Load								
9-inch	700	1000	1700	1400	1200	2600	2.17	1800
12-inch	900	1000	1900	1800	1300	3100	2.58	2200

their use and recommends using dynamic analysis and measurements instead).

The static tests will be carried out to a nominal maximum load of 1,396 KN (157 ton). (Actually, as the specifications do not prescribe the use of a separate load cell, the applied test load will include a significant error and the maximum load will probably be at least about 10 to 15 percent smaller than the specified 1,396 KN, probably be no more than about 1,200 KN). For this project, unless the static test is carried beyond the 1,396 KN load, the static testing will not provide much useful information - it will be a waste of money.

With the combination of contractor's charge and the engineers time, each static loading test at this site will cost at least about $6,000 to $8,000. As perhaps all six tests may not be executed, the total costs for static testing amount to, say, about $30,000. In contrast, a dynamic test costs about $500 to $1,000 per pile tested. If dynamic testing would be used in lieu of the static testing, in addition to verifying pile capacity, important information would be obtained on hammer performance, termination criteria, and required pile lengths. Variations between piles can be considered because several piles are normally tested on each occasion. Even if piles were tested in each pier and both abutments, the total amount would be a fraction of the cost of six static loading tests.

It does not take a specialist to know that the twelve-inch pipe pile can easily be installed to a final bearing capacity much in excess of the required value, and be allowed for a total load considerable greater than 700 KN. An allowable load of 1,200 KN is commonly used. As to the dragload due to negative skin friction developing at the site, the structural strength of the pile is such that there is no need for any bitumen coating even when using the relatively low strength concrete (30 MPa - 300 psi) called for in the specifications.

Perhaps the designers wanted to have a safe and conservative design. Perhaps they had little recent experience of pile foundation design. Perhaps they were short of time. Be this as it may-finding out will not change the situation. Before we shrug our shoulders, however, we should look at the cost consequence of the design. Then, perhaps, we will recognize that the profession has a problem that is costing more than just embarrassment.

The cost as bid per pile is $3,200 plus an extra $370 for bitumen coating the full pile length. The project includes a total of 260 piles. Total costs are therefore $832,000 + $96,000 = $928,000. Replacement costs for rejected piles are not included and the specifications release the contractor from any responsibility for rejected piles.

Increasing the load per pile to 1,200 KN, a value more in tune with the pile, would eliminate 110 piles (the new total is 150 piles) at a savings of

$393,000 and a new total cost of about $535,000. If at the same time the bitumen coating is eliminated, the total savings would be about $489,000. Finally, replacing the static testing with dynamic testing would bring the savings to at least about $500,000! One can wonder what potential savings might be available on the superstructure (total bid price $8,000,000) if the design approach for the superstructure is similar to that of the foundations.

The designers must have felt that perhaps there was a margin for improvement, because the specifications solicited bids for "alternative piles sizes producing the same or higher capacity". Such bids were received. A low bidder offered a 245 mm (9.625 inch) pile (with a minimum 12 mm (0.472 inch) wall and no extra charge for a thicker wall if this would be required) at a price of $2,375 per pile making a total of $617,500, a more than $200,000 savings on the pile foundations as designed. Actually more, because the contractor realized that the bitumen coating was excessive and indicated that a reduced amount of bitumen coating at a charge of $23,000, a savings of $70,000. He also offered to take on the responsibility of any rejected piles, which would reduce the total costs additionally.

The nine-inch pile is made not with the mild steel used for the twelve-inch pile, but with high strength steel. Considering axial structural strength, the nine-inch pile is about equal or stronger than the twelve-inch pile depending on wall thickness and concrete strength. As to bearing capacity, the nine-inch pile can be installed to a capacity of an equally safe margin. As a side benefit, the smaller diameter will result in that a smaller drag load will affect the pile. And last, but not least, the mentioned experience from the nine-inch pile in the same soil conditions a few hundred yards away indicates the assured performance of the nine-inch pile with regard to both installation and capacity.

Table I compares the structural strength of the specified twelve-inch pile to that of a nine-inch pile of wall thickness from 12 mm through 15 mm. The strengths and stiffness values vary, of course, but the strengths are in all cases more than adequate for the specified design loads and the maximum loads occurring at the neutral plane (dragload condition).

Table II compares calculated bearing capacity and static resistance for the nine and twelve-inch piles considering both the specified design load of 700 KN and the more economical design load of 1,200 KN. Values indicated as "Initial Driving) relate to what termination criterion to apply to end-of-initial-driving conditions. Similarly, values indicated "Restriking" apply to restriking conditions after soil set-up has occurred (about one week after the initial driving).

To determine termination criteria, one should not fool, or worse, oneself or one's client with a dynamic formula. Computations using a wave equation analysis are made about as quickly and they produce results which correspond to reality. It has been argued that the advice to employ wave equation analysis in foundation engineering design is bad, because the engineer is then required to have access to a computer, he or she must also know what parameters to use as input to the computer. Such criticism is tantamount to rejection on the grounds that engineers should not need to have proper tools nor know what they are doing!

Figure 1 presents a Bearing Graph resulting from a wave equation analysis by means of the GRLWEAP program applied to the viaduct piling. The analysis assumes the pile driving hammer to be the B300 Berminghammer. The bearing graph shows the static resistance (capacity) for the specified twelve-inch pile and the range for the nine-inch pile sizes as function of the blow count (penetration resistance). Guided by the static analysis presented in Table II, the graph aids the selection of termination criteria at end-of-initial-driving and at restriking conditions. The WEAP analyses include the previous experience from the area and, therefore, the graph is expected to be representative for the conditions at the viaduct site. However, it is given here only as illustration and should not be applied without field confirmation by means of dynamic measurements.

Despite that the technical facts were presented to the designers, they advised the municipality to reject the contractor's alternative bid saying that the nine-inch pile was not of sufficient capacity! Although the technical merits of the rejection advice are lacking, the politicians had no recourse other than to follow the advice of their engineering staff and consultants.

Of course, the viaduct case is not unique nor restricted to this particular community. The situation is not that much different in other areas of the province and, for the matter, across the nation and the continent. Foundation design and construction is often wasteful and made without much consideration of recent advances. How to improve the situation? Would we want to go in for value engineering in the grand scale? Would we want the society to require an engineer to obtain continuing education credits as a stipulation for maintaining a professional engineering license? Would we want to require that all special areas of design and construction be carried out only by engineers with specially designated licence?

Value engineering as a review of the technical and economical merit of a design before it goes out for tender might be useful and particularly for government projects. I am not sure that making continuing education compulsory is the way to go, though. Encouraging voluntary upgrading is probably more efficient. Besides, creating specialist designations would be a step toward guild thinking and exclusivism. The first to be excluded would be the design quality.

We are living in a society where systems change rapidly. To maintain professional ability, one must keep abreast with the development of the art. The profession must increase its effort to provide its members with means to upgrade skills. Members should be encouraged to attend seminars and short courses to maintain and improve professional ability as well as standard of knowledge in design. Employers should be made to realize that engineers need to spend time on continuing education. Owners and clients should learn to demand that engineers or engineering firms hired to do a job can show that they have indeed kept abreast.

Specialists such as us have an obligation to assist the profession in making presentations to seminars and short courses of our experience. We must also make an effort to disseminate the evolving state-of-the-art by publishing more case history papers. We must get more involved in code standards committees.

If we try to do this to the full extent possible, our society will be better served by us and our politicians will be able to spend our tax dollars and limited resources more wisely.