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A Case for Higher Resistance Factor - Piles Driven to Bedrock

Michael Snow, P.Eng., Golder Associates Ltd., Ottawa, Ontario, Canada
Tony Sangiuliano, P.Eng., Ministry of Transportation of Ontario

ABSTRACT

Static pile load tests (SPLTs) have been carried out in conjunction with high-strain dynamic pile testing (PDA) at three different sites to investigate the pile capacity in support of the structural design of two buildings and a bridge. Conducting the pile load testing was considered an investment to achieve more cost-effective foundation designs for current and future projects. The Canadian Highway Bridge Design Code and the National Building Code of Canada permit higher geotechnical resistance factors when static or dynamic pile load tests are conducted to reduce the geotechnical uncertainty. In each case the owners found value in undertaking the load tests in order to be able to use higher geotechnical resistance factors as outlined in the Canadian Highway Bridge Design Code and the National Building Code of Canada.

In eastern Ontario a very common deep foundation type is the use of driven steel piles, either H-piles or pipe piles, driven to refusal on bedrock. In some cases, the site conditions require pile lengths that can easily be over 25 m, representing a material cost of the overall structure. For three case histories presented, the authors proposed and were retained to complete pile load tests to support a 50 percent increase in the factored Ultimate Limit States (ULS) axial geotechnical pile capacity. In all three cases both dynamic (PDA) and static load tests were completed, and the results supported much higher pile capacities than originally anticipated otherwise.

In view of the results of the SPLT and PDA tests, the paper also discusses the appropriateness of the current geotechnical resistance factors for piles driven to refusal on sound bedrock where in all the cases undertaken the structural capacity of the pile was the limiting factor.

. RÉSUMÉ

Des essais de chargement statique sur pieux (ECSP) ont été effectués en conjonction avec des essais dynamiques de chargement dynamique sur pieux (ECDP) à trois sites différents afin d'étudier la capacité axiale de pieux enfoncer au refus pour appuyer la conception structurale de deux bâtiments et d'un pont. La réalisation des essais de chargement des pieux a été considérée comme un investissement pour rendre la conception des fondations plus rentables pour les projets actuels et futurs. Le Code canadien de conception des ponts routiers et le Code national du bâtiment du Canada permettent des facteurs de résistance géotechnique plus élevés pour la capacité à l'état limite ultime (ELUL) lorsque des essais statiques ou dynamiques de chargement sure des pieux sont effectués pour réduire l'incertitude géotechnique. Dans chaque cas, les propriétaires ont constaté qu'ils étaient utiles d'entreprendre les essais de chargement afin d'être en mesure d'utiliser des facteurs de résistance géotechnique plus élevés, comme le décrit le Code canadien de conception des ponts routiers et le Code national du bâtiment du Canada.

Dans l'est de l'Ontario, un type de fondation profond très commun est l'utilisation de piles d'acier enfoncées au refus, qu'il s'agit de pieux de type H ou de pieux tuyaux, sur le substratum rocheux. Dans certains cas, les conditions du site exigent des longueurs de pieux qui peuvent facilement dépasser 25 m, ce qui représente un coût important pour la construction de ces structures. Pour ces trois études de cas, on a proposé des essais de chargement de pieux pour soutenir une augmentation de 50 pourcent dans la capacité ELUL factorisé des pieux. Dans chacun des trois cas, les essais de charge dynamique et statiques ont été effectués, et les résultats ont soutenu des capacités de pieux beaucoup plus élevées que prévu à l'origine autrement. Compte tenu des résultats des tests ECDS et ECDP, cet article traite également de la pertinence des facteurs de résistance géotechnique actuels pour les piles enfoncées au refus sur le substratum rocheux où, dans tous les cas, la capacité structurelle du pieux était le facteur limitant.

1 INTRODUCTION

Piles Driven to refusal on sound bedrock are a common type of deep foundation for buildings and bridges in Eastern Ontario and Western Quebec. The presence of thick deposits of Champlain Sea clay, glacial till which is often nominally consolidated, underlying sound sedimentary or pre-Cambrian bedrock, and market conditions has led to this often being the deep foundation of choice. The Ministry of Transportation of Ontario (MTO) had undertaken an extensive series of static pile load tests back in the 1980s across Ontario reflecting different pile types and ground conditions. Notwithstanding, the undertaking of static and/or dynamic pile load tests is not often undertaken to support the design of bridges and buildings due to the cost and time required.

The 2019 edition of the Canadian Highway Design Bridge Code (CHBDC), Table 6.2, and the 2006 edition of the Canadian Foundation Engineering Manual (CFEM), Table 8.1, contain geotechnical resistance factors to be applied to the axial ultimate limit states (ULS) capacity of deep foundations – factors which reflect higher uncertainty originating with the method and nature of the assumptions used to obtain the ultimate geotechnical capacity. Generally, three methods are used to establish the ultimate axial capacity: some computational method and an idealized stratigraphic profile and material properties, dynamic (PDA) load tests and static load tests, each with its own distinct geotechnical resistance factor. Table 1 below summarizes these factors.

Table 1. Geotechnical Resistance Factors

Pile Axial Capacity Method	CFEM 2009	CHBDC 2019 (typical)
Analysis/Soil Model	0.4	0.4
Dynamic Load Test	0.5	0.5
Static Load Test	0.6	0.6

The guidance in Table 10.5.5.2.3-1 of AASHTO LRFD (2012) outlines a distinct series of over six different approaches for establishing the ultimate axial capacity of driven piles, not necessarily to refusal on sound bedrock, each with a distinct geotechnical resistance factor that generally range from 0.4 to 0.8.

2 SITE DESCRIPTIONS AND HISTORY

The three sites are located in eastern Ontario and relate to the design of an industrial complex (Site A), a Highway 417 bridge over Ramsayville Road for the Ministry of transportation of Ontario (MTO) (Site B), and an emergency response facility (Site C). The depth of overburden at each site is about 36 m, 53 m and 31 m at Sites A, B, and C, respectively.

The stratigraphy at Site A consist of about 1 m of surficial sands, 32 m of very soft to stiff Champlain Sea clay and about 3 m of glacial till over a shale bedrock with unconfined compression strengths of 97 to 230 MPa.

The stratigraphy at Site B consist of about 2 m of embankment fill, 39 m of firm to very stiff Champlain Sea clay and about 12 m of glacial till over a shale bedrock with unconfined compression strengths of 40 to 85 MPa.

The stratigraphy at Site C consist of about 6 m of very stiff Champlain Sea clay and about 25 m of glacial till over a dolostone bedrock with unconfined compression strengths of 190 to 230 MPa.

3 TEST PILE PROGRAM

Site A included 6 test piles, 2 closed toe steel pipe piles 245 mm x 12 mm, 2 closed toe steel pipe piles 324 mm x 12.7 mm and 2 steel HP 310 x 110 piles. The steel pipe piles were of ASTM A252 Grade 3 steel had a yield strength of 310 MPa. The HP piles (all sites) were of ASTM A572 Grade 50 steel had a yield strength of 345 MPa. All 6 piles were driven to refusal on the shale bedrock. All 6 of the piles were subjected to dynamic load tests at the end of initial drive (EOID) and following a beginning of restrike (BOR), while one of each pile type was subjected to static load tests. The work was completed between February and May 2019.

Site B included one test pile consisting of a steel HP 310 x 110 instrumented pile. The pile was driven to refusal on the shale bedrock. The pile was subjected to a dynamic load test at the EOID and BOR, and was subsequently subjected to a static load test. The instrumented test pile was also part of a downdrag research project being undertaken by the MTO, which included strain gauges, pile extensometers, with the addition of protective steel angles and pipes welded over certain lengths of the pile. The work was completed between August and September 2019.

Site C included 4 test piles consisting of 2 steel HP 310 x 110 piles and 2 steel HP 360 x 152 piles. All 4 piles were driven to refusal on the dolostone bedrock. Three out of 4 piles were subjected to dynamic load tests at the EOID, with one pile showing signs of damage. One of each pile type was tested during BOR and were subsequently subjected to static load tests. The work was completed between November 2019 and February 2020.

The HP piles for Sites A and C were reinforced with 12 mm thick steel plates to the lower sections of the pile flanges. For Site B the pile was provided with a standard bearing point driving shoe. For the close-ended pipe piles at Site A, the base of the piles were reinforced with a 25 mm thick plate welded to the pile base.

For the purposes of this paper, sound bedrock is considered as having an RQD greater than 50 and a UCS greater than 20 MPA.

3 Dynamic Load Testing

3.1 Equipment and Methodology

The dynamic load tests performed at each site were completed in general accordance with ASTM D4945-12 using the following hammers:

1. Site A – During EOID a hydraulic hammer with a rated energy of 70 kJ, while during BOR a drop hammer with a rated energy of between 143 and 238 kJ (depending on drop height);
2. Bermingham B-32 with a rated energy of 110 kJ for Site B; and
3. Site C – During EOID a drop hammer with a rated energy of 65 to 125 kJ (depending on drop height) and during BOR a hydraulic hammer with a rated energy of about 80 kJ.

At Site B the nature of the instrumented pile required that only limited energy be used to seat the pile and check for damage, and thus the fully mobilized capacity of the pile could not be verified in this dynamic manner.

3.2 Dynamic Load Test Results

The energy transferred during the EOID and BOR tests, as well as the ultimate capacity obtained, and percentage of skin friction are presented in Table 2 and 3. Figure 1 shows an example of the dynamic load test setup for Site B.

Table 2. Dynamic Load Test Results EOID

Pile/Site	Energy transferred (kJ)	Ultimate capacity (kN)	Skin Friction (%)
Site A 245x12 #1	59	2,804	7.5
Site A 245x12 #2	60	3.098	7.1
Site A 325x12.7 #3	68	3,250	12
Site A 325x12.7 #4	65	3,030	14
Site A HP310x110 #5	64	4,470	22
Site A HP310x110 #6	63	4,556	21
Site B HP310x110	36	2,125(1)	44
Site C HP360x152 #2	78	2,300	17
Site C HP310x110 #3	66	2,240	26
Site C HP360x152 #4	118	3,600	17

(1) The pile was not fully mobilized in order to protect the instrumentation.

Table 3. Dynamic Load Test Results BOR

Pile/Site	Energy transferred (kJ)	Ultimate capacity (kN)	Skin Friction (%)
Site A 245x12 #1	77	2,662	34
Site A 245x12 #2	103	3,380	26
Site A 325x12.7 #3	110	3,426	28
Site A 325x12.7 #4	106	2,960	30
Site A HP310x110 #5	102	5,490	20
Site A HP310x110 #6	120	5,290	22
Site B HP310x110 (1)	n/a	n/a	n/a
Site C HP310x110 #3	69	3,750	9
Site C HP360x152 #4	64	5,100	17

(1) The pile was not restruck to avoid damaging instrumentation.

3.3 Discussion

The dynamic testing results presented indicate that the pile capacity obtained from such testing is materially dependent on the ability of the pile driving hammer to mobilize the tip of the pile resting on competent bedrock and the structural capacity of the pile to withstand the impact energy of the hammer, assuming the pile has not been materially damaged during driving through the glacial till or seating on the bedrock. Furthermore, the results also indicate that of the overall pile capacity upon BOR, about 15 to 35 to percent is derived by side friction along the pile. A stronger pile material (e.g., higher strength steel) and a bigger hammer would likely lead to higher capacities for the same conditions. Higher strength piles would also likely be more resistant to damage during pile driving.

Between the EOID and BOR results, we can see an increase in capacity in part gained by an increase in skin friction. However, a more detailed comparison between EOID and BOR results is nuanced by using different hammers between the two PDA test for all the sites, except Site B, where driving energy was managed to protect instrumentation.

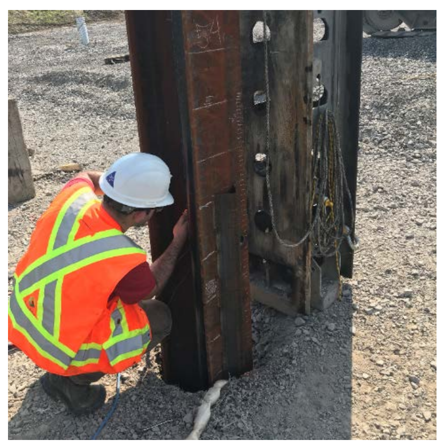

Figure 1. Typical Dynamic Load Test Setup (Site B)

4 Static Load Testing

4.1 Equipment and Methodology

The static load tests performed at each site were completed in general accordance with ASTM D1143, Procedure C. Figures 2 through 4 show the static load test setup for Sites A through C, respectively. For Site's

B and C, the load frame was specified to be able to handle loads up to 120 percent of the specified target load, to provide greater stability of the load frame. For each pile the structural capacity was set as corresponding to about 90% of the yield strength of the pile steel, with the load capacities given in Table 4.

The specified target load was selected to represent the structural capacity of the pile at about 90 percent of the steel's yield strength. Table 4 shows the maximum axial load achieved for each test.

The configurations of the different load test setups vary based on the number of tests being planned and the loads being reached. With the exception of Site A, the load tests were conducted with no material instability of the test setup (non-uniform displacement, tilting or misalignment of the frame).

Table 4. Maximum Static Test Loads

Pile/Site	Target Load (kN)	Maximum Load Achieved (kN)
Site A 245x12	2,716	2,767
Site A 325x12.7	3,848	3,912
All Sites HP310x110	4,400	4,400
Site C HP360x152	6,600	6,600

At Site A, the load tests for the 245mm x 12 mm pipe pile and the HP 310 x 110 pile had to be unloaded and reloaded to adjust the loading frame and improve frame stability. This adjustment may have affected the permanent plastic deformations recorded. Furthermore, the 24 hour final reading on the 245mm x 12 mm pipe pile at Site A could not be completed as weather had affected the electronic dial gauges.

Figure 2. Static Test Setup (Site A)

4.2 Static Load Test Results

The results of all the static load tests are presented on Figures 5 through 8. All the tests were able to achieve or slightly exceed the targeted loads as indicated in Table 4.. The permanent deformations recorded for all the tests achieved criteria set out in ASTM D1143 and were considered acceptable except for the tests on the 324 mm x 12.7 mm pipe pile at Site A and the HP 360 x 152 pile at Site C. For these piles the ultimate pile capacity was reduced slightly to reflect these higher permanent deformations, which in the case of the HP 360 x 152 pile is likely the result of pile damage near the toe, as observed from the PDA results.

Figure 3. Static Test Setup (Site B)

Figure 4. Static Test Setup (Site C)

5 DISCUSSION

The focus of this paper is to suggest that the current Limit States approach to assessing the factored ultimate limit states (ULS) axial capacity of piles driven to refusal on sound bedrock provides overly conservative outcomes. This pile capacity problem is reflected in the three case studies presented herein reflects the conditions at all three sites and all six of the static load tests presented herein.

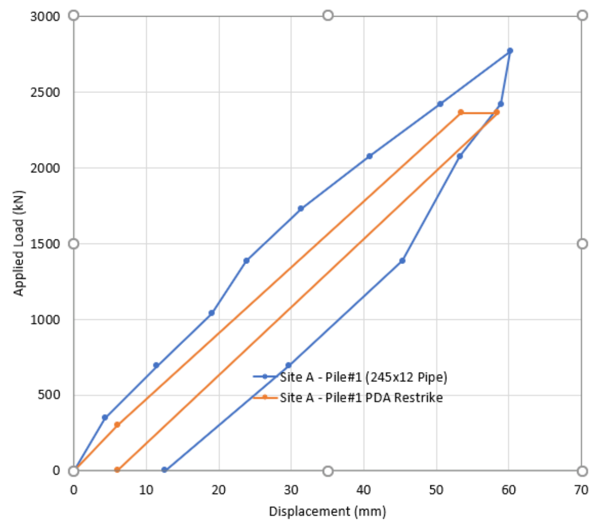

Figure 5. Site A 245x12 Pipe Results

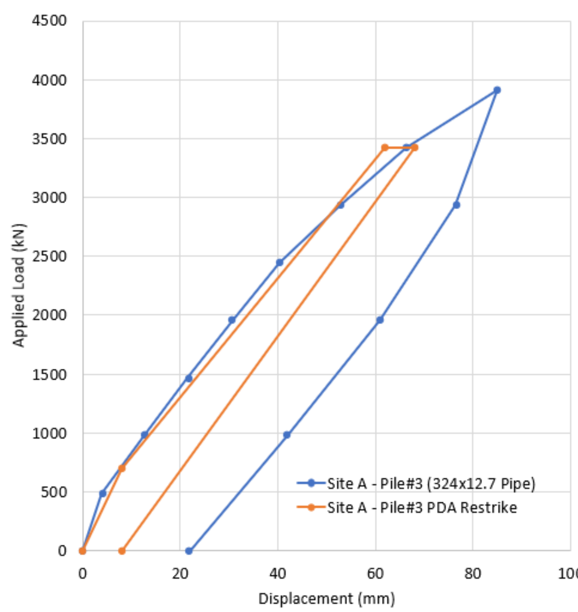

Figure 6. Site A 324x12.7 Pipe Results

The authors suggest that the limited work on the geotechnical axial capacity of a pile driven to refusal on sound bedrock do not provide a rigorous approach and have limitations. For the purposes of this discussion, sound bedrock is considered to have a Rock Quality Designation (RQD) greater than 50 percent and an unconfined compressive strength (UCS) in excess of 10 MPa. In fact, historically in eastern Ontario the approach was to state that the axial capacity of such piles was only limited by their structural capacity and not by the geotechnical capacity of the sound bedrock.

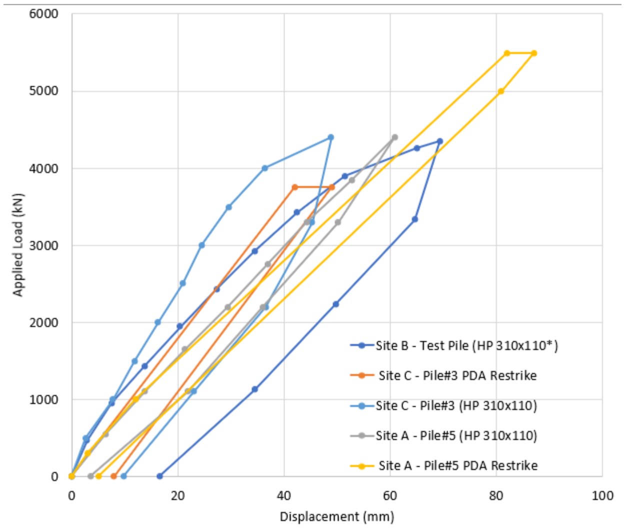

Figure 7. HP 310x110 Results All Sites

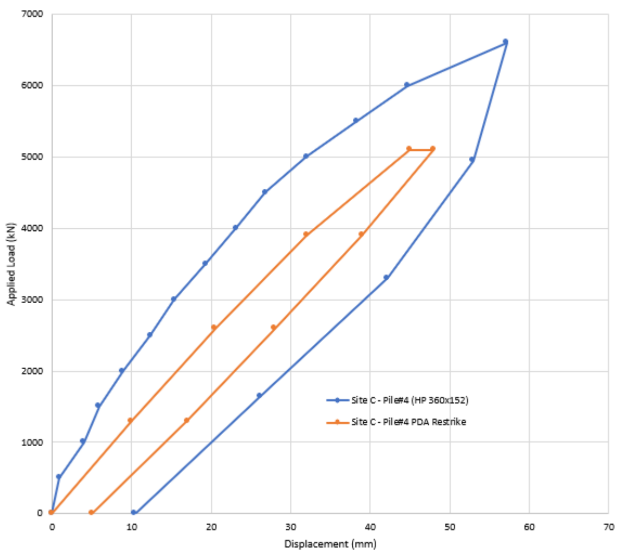

Figure 8. Site C HP 360x152 Results

Work by Rehnman and Broms (1971) on the capacity of steel pins jacked into bedrock suggests that the ultimate capacity is about 4 to 6 times the UCS of the bedrock. Goodman (1980) presented similar factors which were dependent on the type of bedrock and ranged from 2.4 to 8.5. As recommended in the commentary (section C6.11.2) of the CHBDC the "neat" cross-sectional area of the HP piles was used and similarly for the pipe piles only the pipe wall end area was used and not the base area. Using a "plugged" or partially "plugged" end area for the HP pile or the full end surface of the pipe pile would provide much larger capacities than presented in Table 5. When compared to the maximum static test loads achieved, the computed capacities range from 10 percent lower to 320 percent higher. The range of computed capacity relative to the load test capacity may reflect the complexity of the geometry of the analytical model and the structural limitations of the load test to achieve higher loads.

Furthermore, it seems clear that the ultimate capacities presented below, with the exception of the instrumented pile at Site B, are well in excess of the capacities that could be proven using a static load test.

Table 5. Estimated ULS Pile Capacities

Pile/Site	Computed ULS capacity (kN)[1]	Fraction of Max. Achieved Load (%)
Site A 245x12	4,090	~148
Site A 325x12.7	5,740	~147
Site A HP310x110	6,245	~142
Site B HP310x110	2,400	~55
Site C HP310x110	14,210	~320
Site C HP360x152	19,555	~300

(1) Using the median factor proposed by Goodman (1980) for the specific rock type and the average UCS.

The values in Table 6 present the factored ULS capacities using the geotechnical resistance factors presented in Table 1 and assuming that the consequence factor is 1.0 for each case.

Table 6. Factored ULS Pile Capacities

Pile/Site	Computed capacity (kN)	PDA capacity (kN)	Load Test Capacity (kN)
Site A 245x12	1,636	1,331	1,660
Site A 325x12.7	2,296	1,713	2,050[1]
Site A HP310x110	2,498	2,745	2,640
Site B HP310x110	960	n/a	2,640
Site C HP310x110	5,684	1,875	2,640
Site C HP360x152	7,822	2,550	3,960

(1) The ultimate capacities were slightly reduced to reflect higher permanent deformations.

Of the data presented in Table 6 using the three different approaches, all the Site A case values were within 15 percent of the median values, the Site B values were within 47 percent of the median value, and all Site C case values were within 50 percent of the median values.

The variability of factored ULS axial capacities presented in Table 6 would have significant material implications on design costs. In fact, it is surprising that more case studies of piles driven to refusal on bedrock are not encountered in the published literature. We postulate that the likely reason is that for cases where the computed pile capacities are higher than the factored structural capacity of the pile, then the latter governs the design of the element. In fact steel strengths would need to increase materially, with a corresponding increase in pile driving hammer capacities, before the bedrock itself underwent failure to the point of affecting foundation performance.

The authors are suggesting that in such cases where piles on driven to refusal on sound bedrock, the pile capacity will always be practically limited to the structural capacity of the pile and not the capacity of the bedrock. As such the current framework of geotechnical resistance factors should be replaced with appropriate structural resistance factor that reflect the potential variability of the steel pile material parameters as well as possible damage induced into the pile during driving, the nature of the loading and potential long-term damage from environmental factors.

The AASHTO LRFD (2012) guidance in Section 6.5.4.2 outlines a series of driven steel pile structural resistance factors with values of 0.6 and 0.7 for normal pile driving conditions and 0.5 and 0.6 for severe pile driving conditions, for steel H piles and pipe piles, respectively.

Performing PDA analyses during driving of the piles assists with establishing appropriate driving criteria to seat the piles on sound bedrock but also in assessing potential damage to piles during driving. In cases where the piles cannot be driven sufficiently to be supported on sound bedrock, perhaps due to obstructions, then the PDA results are helpful in establishing a reduced pile capacity.

6 CONCLUSIONS

The case studies presented herein offer an interesting comparison of different methods for estimating the ULS axial capacity of piles driven to refusal on sound bedrock. Whether a computational, PDA or static load test approach is used, each approach comes with limitations with respect to establishing the geotechnical ULS capacity of such piles.

For all the cases, the static load test was able to validate that the factored ULS capacity was only limited by the structural capacity of the pile. For cases where the computed or PDA factored ULS capacity was higher than the static load test (structural) capacity of the pile, the design was still advanced using the limiting structural capacity of the pile.

Computational approaches are challenging to model for such cases as indicted by the varied range of geotechnical capacities computed in Table 5 relative to static load test results.

PDA results typically are limited by the ability to drive the pile harder than its structural tolerance, and as such provide a good indication of structural capacity with some accounting for damage from pile driving where noted.

The cost of load tests during the design phase of projects is lengthy and expensive, and as such is rarely undertaken. The cost of static load tests can easily reach several hundred thousand dollars (CAD) and take several months to organize and complete.

The authors suggest that for the case of steel piles driven to refusal on sound bedrock (i.e., RQD > 50 and UCS > 20 MPa), developing a limiting geotechnical capacity does not provide any protection against geotechnical failure and in fact penalizes a foundation system that is largely limited by the structural capacity of the pile itself. Rather the resistance side of the Limit States equation for such piles should solely be based on the factored structural resistance of the piles as defined in codes, accounting for damage during pile driving and for environmental impacts on these structural elements.

Based on the results presented herein, it would seem appropriate that for the case of steel piles driven to refusal on sound bedrock, that the ultimate axial capacity of the pile be established solely on the factored structural capacity of the pile. In such cases the resistance factors outlined by AASHTO LRFD (2012) would appear to be a reasonable approach to defining a factored ULS capacity for steel piles driven to refusal on sound bedrock. These geotechnical resistance factors recognize the structural limitation od driven piles, and even provide a consideration for the greater potential for pile damage in certain conditions.

Using the recommended approach provided herein, available factored capacities for piles driven to refusal on sound bedrock could be increased by 10 % to 30 %. For a typical HP 310 x 110 driven pile under normal driving conditions and using a structural resistance factor of 0.6, this would lead to a factored axial ULS capacity of 2,640 kN which corresponds exactly to the results of the three static load test geotechnical factored capacities for said piles. Clients should be able to realize the full benefit of the driven pile structural capacity using structural resistance factors and not geotechnical resistance factors, for the case of piles driven to refusal on sound bedrock.

7 REFERENCES

Canadian Foundation Engineering Manual, Fourth Edition, 2006. BiTech Publishers, 488 p.

Canadian Highway Bridge Design Code – CSA S6:19. Canadian Standards Association, 1197 p.

LRFD Bridge Design Specifications, 2012, AASHTO.

Rehnman, S.E., Broms, B., Bearing Capacity of Piles Driven into Rock, *Canadian Geotechnical Journal*, Vol. 8, no. 2, pp. 151-162, May 1971.

Goodman, R.E., 1980. Introduction to Rock Mechanics, Wiley Publishers.