DISCUSSION of:

Comparison of Static and Dynamic Pile Load Tests at Thi Vai International Port in Viet Nam

Full Reference: Phan, T.L., Matsumoto, T., and Nguyen, H.H. (2013). *Comparison of Static and Dynamic Pile Load Tests at Thi Vai International port in Viet Nam*. International Journal of Geoengineering Case histories, http://casehistories.geoengineer.org, Vol.3, Issue 1, p.36-66. doi: 10.4417/IJGCH-03-01-04.

David Tara, *Principal, Thurber Engineering Ltd., Canada; email: dtara@thurber.ca*
Peter Middendorp, *Managing Director, Allnamics Pile Testing Experts BV, The Netherlands; email: middendorp@allnamics.eu*
Gerald Verbeek, *President, Allnamics USA LLC, USA; email: verbeek@allnamics-usa.com*

INTRODUCTION

The authors have presented an interesting case history that includes soil characterization, design, dynamic and static load testing results. The discussers are appreciative of the inclusion of the data shown on the figures and its value to the geotechnical community as it allows practitioners the opportunity to practice their Wave Matching Analysis (WMA) skills for soil conditions different from what they may typically encounter and to validate other software models. The discussers wish to share their experience with the authors and note that, in the process of reviewing the data in detail, some important issues came to light. This discussion focusses on the design and testing of the steel pipe piles, and hammer selection process, expanding on the authors' work and attempts to shed some light on the findings.

PILE DIMENSIONS, HAMMER SELECTION AND DRIVING STRESSES

The authors indicate (in Table 3 of Phan et al. 2013)) that TSP1 is a 1000 mm outside diameter (D) by 12 mm thick (t) pipe pile. The corresponding D/t ratio is 83 and the pile cross sectional area is 0.0372 m^2. The authors indicate that the yield strength of the steel is 360 MPa. Calculations presented in Table 2 of Phan et al. 2013 indicate the required vertical compression resistance is 8004 kN with a static load test (SLT) and 12006 kN without such a test and based on static analysis only. The corresponding Factor of Safety (FS) for the two cases is 2.0 and 3.0, respectively. For dynamic load testing, we infer that a FS of 2.0 to 2.5 would be acceptable such that the required mobilized resistance, without a SLT, would be about 8004 to 10005 kN.

Table 4 of Phan et al. 2013 indicates that the minimum required energy for pile installation (E_h) is 175.1 kJ and the maximum ram weight (W_r) 71 ton. The corresponding equivalent drop height (H_e) is 2.5 m. Table 5 indicates that a Delmag D100 13 with a 98 kN ram and an assumed H_e of 2.8 m was selected, resulting in an energy per blow between 214 and 334 kJ. Table 6 of Phan et al. 2013 indicates that the maximum design settlement (penetration) for this pile was 4.7 mm per blow but the authors report that the test pile was driven to an average settlement (penetration) of 0.6 mm per blow over the last 10 blows. A dynamic load test (DLT) was conducted at 34 days after installation using the same hammer with a recorded penetration per blow of 0.3 mm. A static load test was then performed another 14 days later.

Typically, hammer and pile selection must consider drivability, testing requirements (assuming the same hammer is used for the DLT) and the potential for pile damage, especially where piles are installed into dense or hard strata. Crapps (2004) provides guidance with respect to the minimum hammer size required to advance a pile. Using Crapps' criteria for H_e equal to 2.8 m, the minimum required hammer weight would be 35 kN. For a successful DLT, the hammer weight should typically be about 1% to 2% of the load required to be mobilized. Further, where good toe resistance is expected, our experience shows that the lower end of the range is generally adequate.

Submitted: 14 November 2014; Published: 5 February 2014
Reference: Tara, D., Middendorp, P., and Verbeek, G. (2014). *Discussion of Comparison of Static and Dynamic Pile Load Tests at Thi Vai International port in Viet Nam.* International Journal of Geoengineering Case histories, http://casehistories.geoengineer.org, Vol.2, Issue 4, p.291-297. doi: 10.4417/IJGCH-02-04-05

Using the 1% criteria, the required hammer weight to mobilize 8 to 10 MN would then be in the range of 80 to 100 kN. Although damage is often addressed in the industry by limiting the maximum driving stresses to 80% to 90% of the yield stress, this approach does not consider such issues as hammer alignment, denting and/or elastic buckling and pile material fatigue history.

Selection of an appropriate hammer must also consider the pile size and relative dimensions to mitigate the risk of damaging the pile. CFEM (2004) provides guidance to limit pile damage based on hammer energy and recommends that "the rated energy of the hammer should be limited to a value of 6 x 10^6 J (Newton metre) times the cross sectional area of the pile" in square metres. This approach does not consider the relative dimensions of the pipe. API (2000) provides more comprehensive recommendations for hard driving conditions that are diameter dependent in that they consider the D/t ratio of the pipe. API also has a more conservative limit of about 4200 kJ times the cross sectional area of the pile subject to a minimum pile wall thickness (in millimeters) as follows:

$$t = D/100 + 6.35 \quad (1)$$

Tara (2012) used the Crapps, API and CFEM criteria to successfully select hammers for the installation of 610, 914 and 1824 mm diameter steel pipe piles for the Pitt River Bridge project completed in the late 2000s. Using the graphical representation of the API and CFEM criteria, Figure 1 shows a number of relevant case studies presented in Mostafa (2011), Tara (2012) and from internal files. The case studies shown include piles with and without reported damage. For reference, TSP1 is included on the figure (using an energy level of 280 kJ and the cross sectional area of 0.0372 m2). By inspection, none of the piles included in the data set situated below the API boundary exhibit damage. However, given the location of TSP1 on Figure 1 relative to the API and CFEM thresholds, especially relative to the D/t limit (61 = 1000/(1000/100 + 6.35)) for 1000 mm pipe and the energy thresholds of both API and CFEM, we should at least consider that TSP1 was at risk of damage during driving.

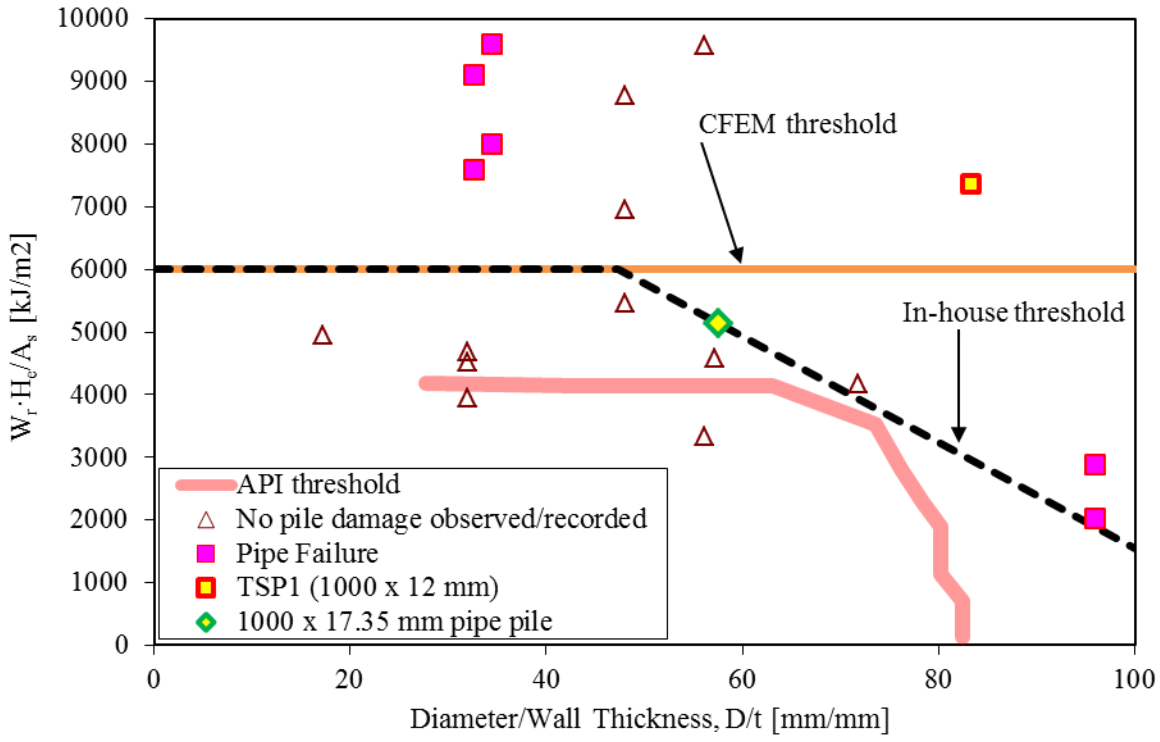


Figure 1. API, CFEM and in-house threshold criteria with relevant case histories including TSP1.

In the mid 2000s based on a limited data set, the primary discusser developed a D/t threshold for in house pile and hammer selection criteria as shown on Figure 1. These criteria can be transformed (Tara, 2012) for a 1000 x 12 mm pipe pile similar to TSP1 as shown in Figure 2. Ignoring the D/t threshold, Figure 2 suggests that an appropriately sized hammer for TSP1 using API and CFEM guidelines would be in the ram weight range of 55 to 80 kN assuming a 2.8 m drop height. If the D/t criterion is adopted, the ram weight would need to be limited to about 37 kN to mitigate the risk of pile damage.

Obviously this is quite different from the selected ram weight of 98 kN and, in the discussers' opinion, requires additional consideration. Further, given the range of ram weights suggested by Figure 2, drivability and testing requirements must also be reviewed. Using a smaller hammer ram weight, decreasing H_e and/or further increasing the pile wall thickness would be required to shift the data point to the in house threshold shown. Examination of Figures 1 and 2 suggests that reducing H_e alone would likely not be sufficient to mitigate the risk of pile damage as it is unlikely that an APE D100 would function with an effect stroke of only about 1.1 m. To reduce the risk of pile damage during pile driving or testing for the same hammer, drop height and driving conditions as used for the Thi Vai Port project, application of the primary discusser's in house criteria (shown on Figure 1) would require increasing the pile wall thickness to at least 17.35 mm. For reference, the corresponding data point for a 1000 x 17.35 mm pipe pile is shown on Figure 1. As shown, the corresponding D/t ratio would decrease from 83 to about 58.

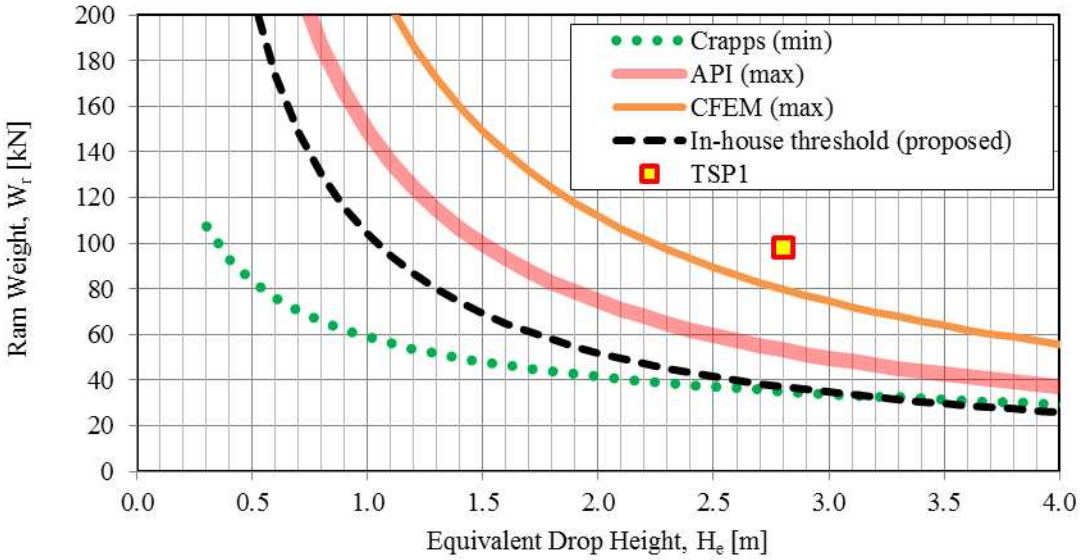


Figure 2. Hammer ram weight versus effective drop height for a 1000 x 12 mm pipe pile.

DYNAMIC LOAD TEST RESULTS

DLTs were performed on Pile TSP1 as shown on Figures 27 and 28 of Phan et al. 2013, respectively. The end of drive (EOD) and beginning of restrike (BOR) data for force (F), impedance times velocity (Z·v) and upwave (U↑) are reproduced on Figures 3 (F and Z·v) and 4 (U↑). It should be noted that some minor corrections were made to the F and Z·v data to address minor proportionality and arrival time discrepancies. Further, the BOR data appears to have been cut off rather early as neither the F nor Z·v traces are stabilizing. For discussion purposes, the return travelling wave time in the pile (2L/c) is also shown on the figures. What becomes immediately apparent in the BOR traces is the decrease in F and increase Z·v at about 4 ms before the pile toe. This change can only be due to a decrease in pile impedance that the discussers attribute to possible pile damage.

WAVE MATCHING ANALYSIS

The authors present the results of WMAs performed for EOD and BOR conditions using a 3D FE program developed by the authors. The discussers conducted WMA of the EOD data using the software program AllWave-DLT by Allnamics-USA, a recent implementation of TNOWAVE developed by TNO Building and Construction Research (Middendorp and Verbeek, 2006). The EOD WMA results for the U↑ match are summarized on Figure 5 and it is clear that the match quality is good. The total mobilized resistance determined by the WMA is about 6838 kN with the shaft and toe resistances accounting for 5720 and 1118 kN, respectively.

The WMA results were then used to generate a simulated SLT using the same parameters and resistance distribution as for the DLT. The discussers also used Allwave-DLT to estimate the SLT response assuming a plugged condition and an equivalent toe quake of 6 mm. The results are shown on Figure 6 along with the authors' simulated and real SLT results

from Figure 31 of Phan et al. 2013. By inspection, the discussers' estimated load movement responses of the pile match the SLT reasonably well.

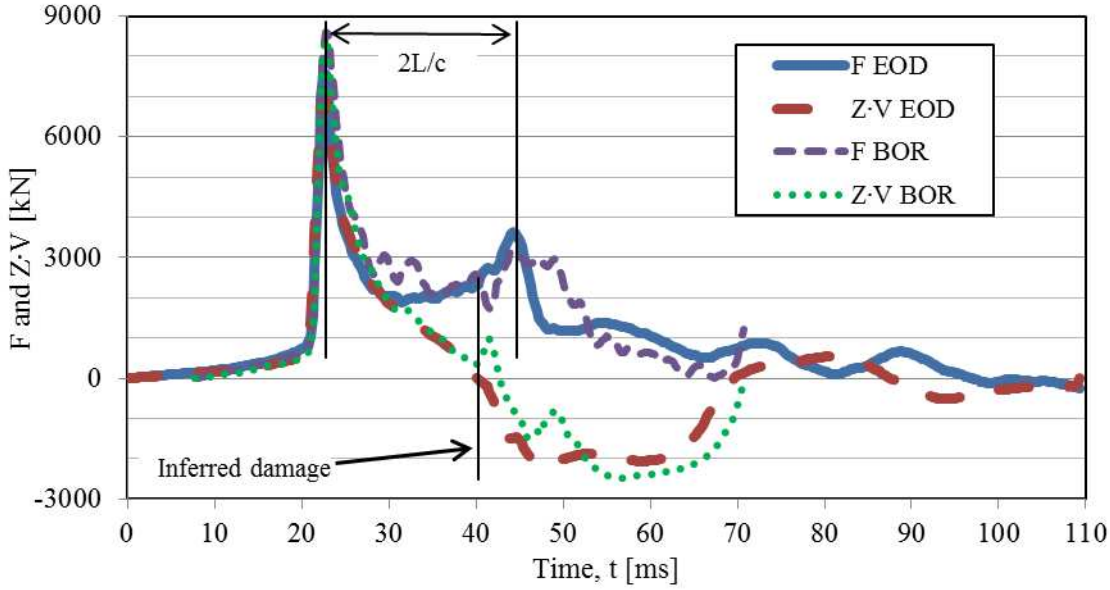


Figure 3. Force and Z·v for EOID and BOR tests of TSP1.

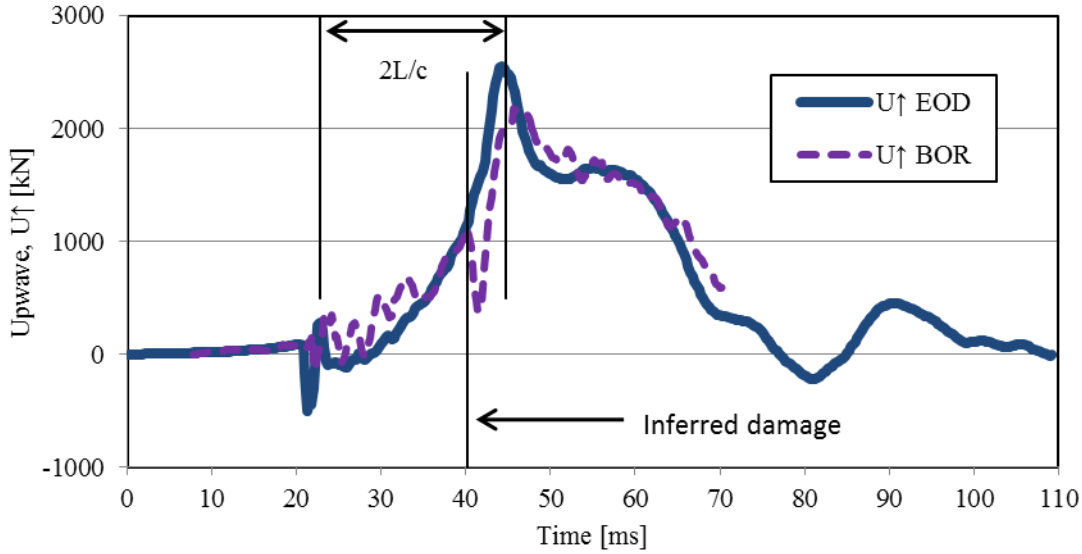


Figure 4. Upwave for EOID and BOR tests of TSP1.

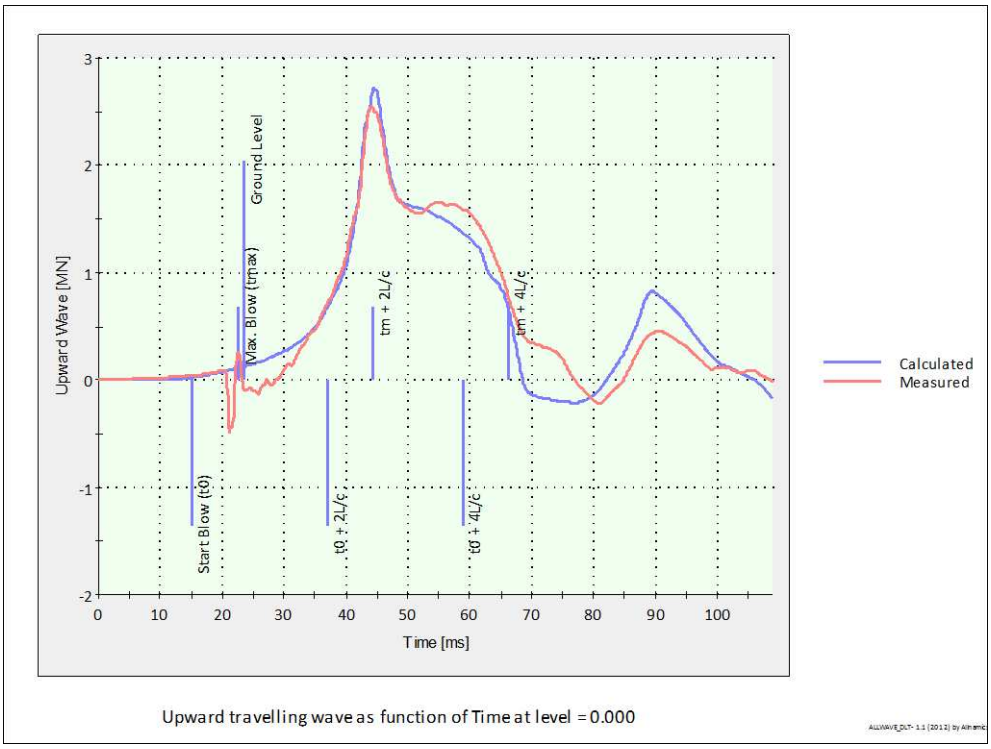


Figure 5. Results of WMA for TSP1 at EOD using AllWave-DLT.

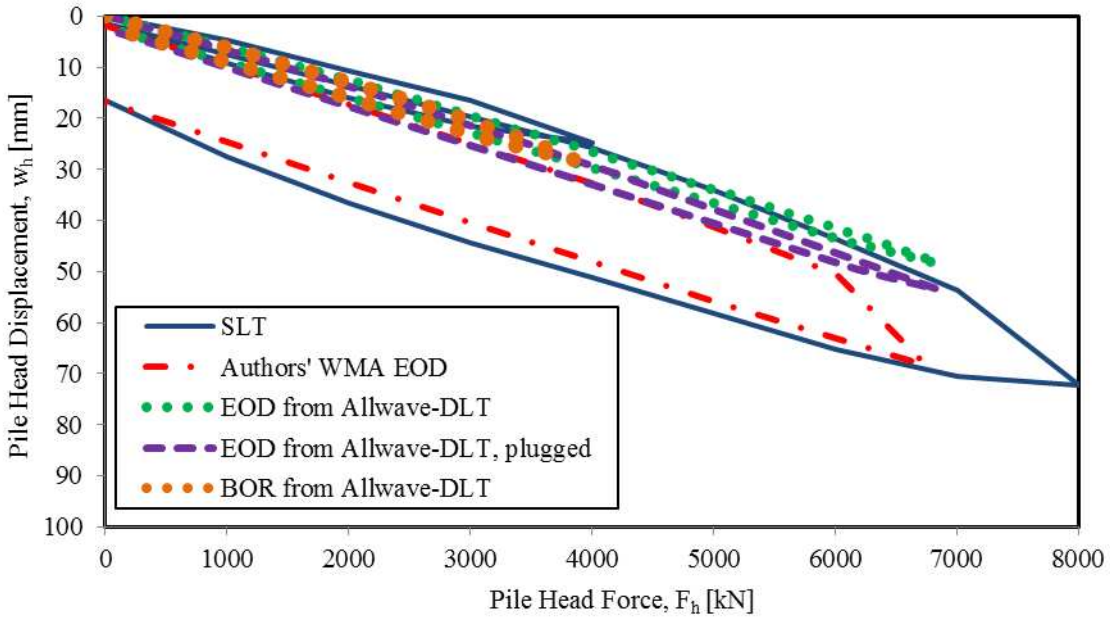


Figure 6. Comparison of static load displacement curves of TSP1.

DAMAGE ASSESSMENT

To better understand the extent of potential pile damage, AllWave-DLT was used to first replicate the change in impedance and then used to estimate the BOR capacity. The EOD results were initially used to estimate the impedance reduction above the pile toe. The WMA results showed that the impedance reduction was best modelled with the shaft wall thickness

reduced to 8.25 mm from 49.5 m below the instruments to the pile toe. The shaft and toe yield stresses and damping were then adjusted to improve the match quality. The results of the BOR WMA are shown in Figure 7. The WMA mobilized shaft, toe and total capacities are 3302, 553 and 3855 kN, respectively. The BOR results were then used to generate an equivalent static loading response that is also shown on Figure 6. By inspection, the stiffness is almost identical to the SLT results.

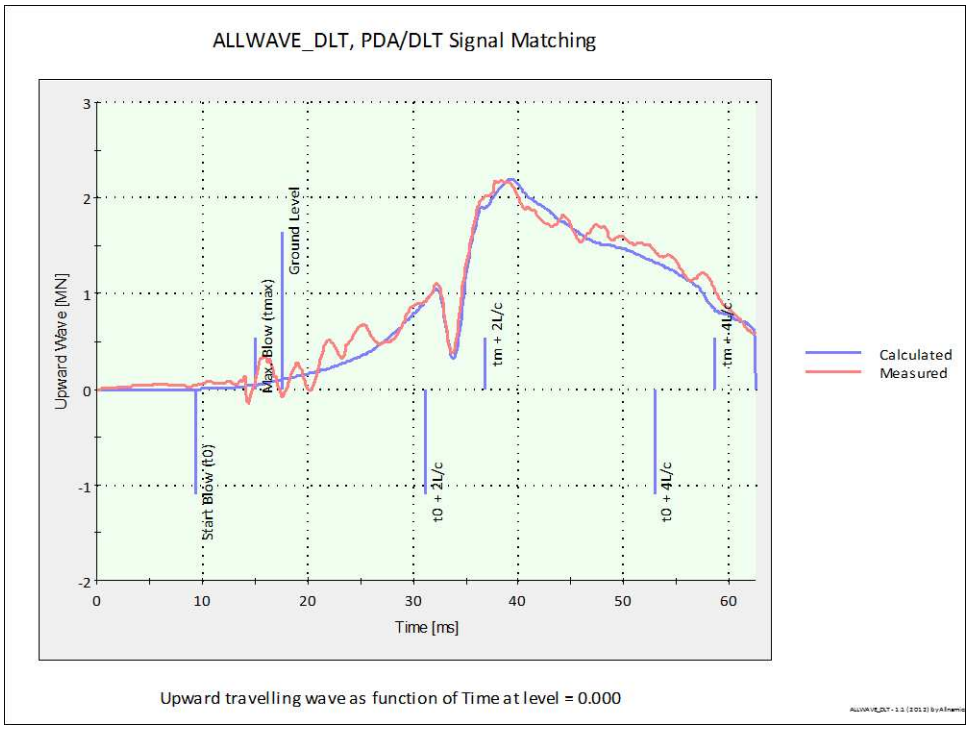


Figure 7. Upwave, U↑, of TSP1 at BOR assuming pile damage below 49.5 m..

DISCUSSION

By inspection, the quality of the EOD and BOR WMAs is quite reasonable. Our analyses suggest that TSP1 appears to have been damaged due to hard driving, possibly at EOD, but more likely at BOR, and that the BOR mobilized capacity may be less than estimated by the authors. The discussers' requested additional information regarding the restrike test. Unfortunately, the authors were unable to confirm how many blows were applied during the restrike test and did not have access to the raw data obtained during the restrikes to share with the discussers. Given that TSP1 was statically tested to about 8000 kN after the BOR DLT, it is likely that the damage does not seriously affect the integrity and long term performance of the pile. Further, with a steel yield strength of 360 MPa, it is expected that TSP1 could be loaded statically to in excess of 9000 kN without risk of structural failure. Nonetheless, as the damage (reduction in pile impedance) likely occurred during restrike, the discussers recommend that this be confirmed by reviewing the raw data in detail.

Normally, it has been the discussers experience that both the pipe pile dimensions and impact hammers used for installation and dynamic testing should be selected to safely advance the pile to the desired embedment so that the required capacity can be achieved without unnecessarily risking the integrity of the pile due to damage. The guidelines included herein to help select appropriate pile dimensions and hammer ram weight are intended for screening only. More work is required to develop comprehensive guidelines. Only with sharing case histories as has been done by the authors and Mostafa (2011) will the geotechnical community be able to move forward on this important issue.

REFERENCES

American Petroleum Institute (2000). "Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design". 21 Edition. API.

Canadian Foundation Engineering Manual (CFEM). (2006). Canadian Geotechnical Society, BiTech Publishers, Vancouver, B.C., Canada.

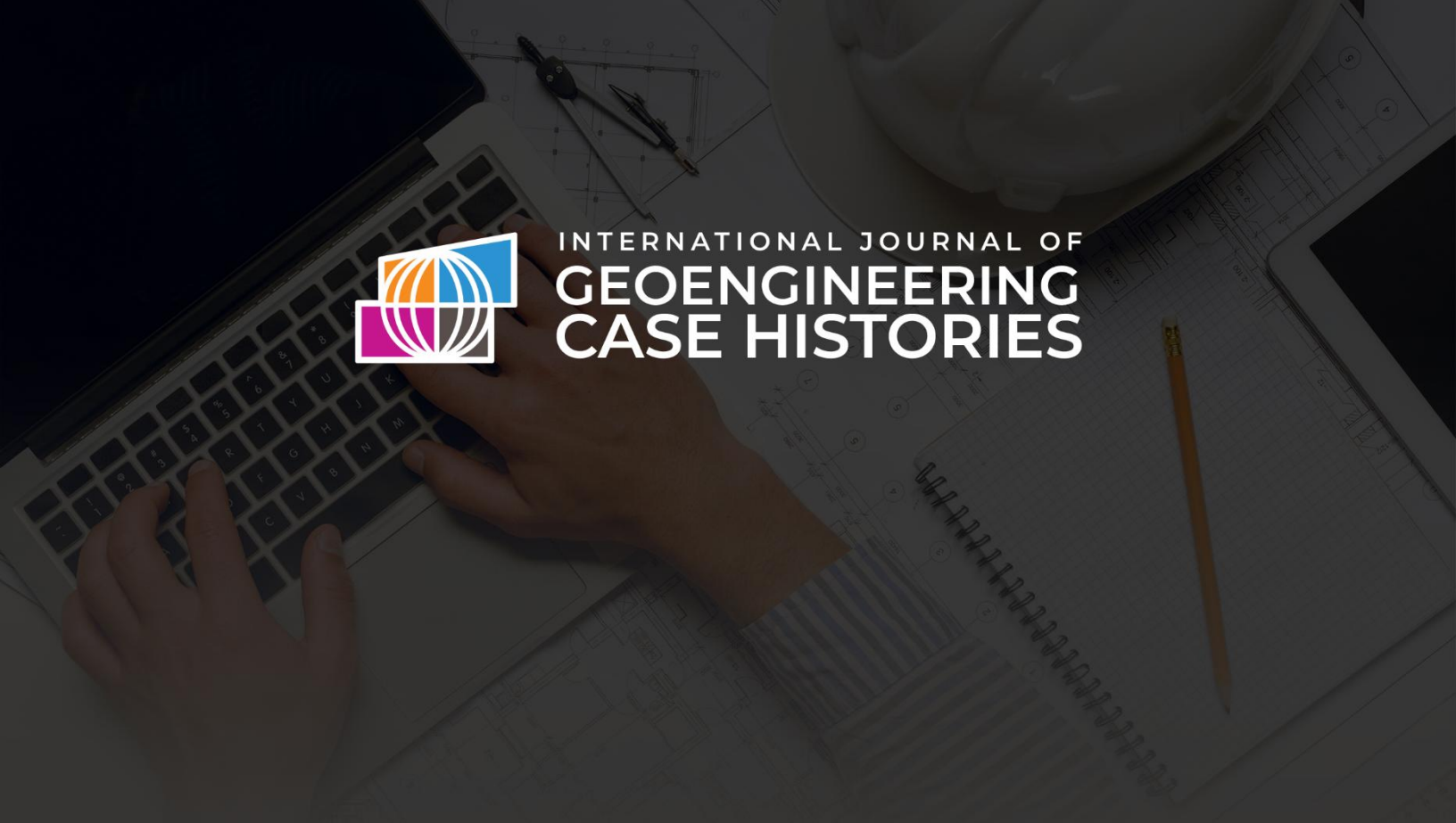
Crapps, D.K., (2004). "The role of ram momentum in pile driving". Current Practices and Future Trends in Deep Foundations, GSP125, ASCE, edited by DiMaggio and Hussein, 276-309.

Middendorp, P. and Verbeek, G.E.H. (2005). "30 Years of Experience with the Wave Equation Solution Based on the Method of Characteristics." Proceedings of the 30th Annual Conference on Deep Foundations, Deep Foundations Institute, Chicago, Illinois, USA, Vol. 1, 13-22.

Mostafa, Y.E. (2011). "Onshore and Offshore Pile Installation in Dense Soils". Journal of American Science, Vol. 7, No. 7, 549-563.

Phan, T.L., Matsumoto, T. and Nguyen, H.H. (2013). "Comparison of Static and Dynamic Pile Load Tests at Thi Vai International Port in Viet Nam". International Journal of Geoengineering Case Histories, http://casehistories.geoengineer.org, Vol.3, Issue 1, p.36-66. doi: 10.4417/IJGCH-03-01-04.

Tara, D.J. (2012). "Pitt River Bridge 2007 Static Pile Loading Test". Full-Scale Testing and Foundation Design, GSP227, ASCE, edited by Hussein, Massarsch, Likins and Holtz, 289-306.



INTERNATIONAL JOURNAL OF
GEOENGINEERING
CASE HISTORIES

The Journal's Open Access Mission is generously supported by the following Organizations:

dar
a Sidara company

Geosyntec consultants
engineers | scientists | innovators

CONETEC
MUD BAY DRILLING
LANKELMA
Geotech DRILLING

ENGEO
Expect Excellence

Access the content of the ĪSSMGE Īnternational Journal of Geoengineering Case Histories at:
www.geocasehistoriesjournal.org

Downloaded: Wednesday, November 18 2026, 19:57:22 UTC