



UNIVERSITY OF
CAMBRIDGE

Department of Engineering

Suitability of Jacked-in Steel Piles as Tsunami Defences

Author Name: Andrei Dobrisan

Supervisor: Dr Stuart Haigh

Date: May 25, 2016

I hereby declare that, except where specifically indicated, the work submitted herein is my own original work.

Signed: _____ Date: _____

Summary

Seawalls are the first line of defence against tsunamis and typhoons. As such their failure usually leads to significant human and economic loss. The 2011 Tōhoku tsunami brought into question the adequacy of the pre-existing shallow foundation concrete caisson seawalls due to the large number of failures recorded. A new type of seawall made of adjoining large steel pipe piles embedded 10-15 m into the ground is currently installed along the Japanese coastline. Because a tsunami seawall is a very specialised structure, there is little research available on this topic. Hence the seawalls are currently designed using a combination of codes of practice which were not originally written with the seawall use case in mind. This report investigates the adequacy of using these codes for designing a pile wall against tsunamis. When dealing with a seawall design the major challenges are: estimating the wave loading, deciding whether the dynamic nature of the wave loading has any effect on the seawall and, finally, estimating the lateral capacity of the seawall itself. We refer to these challenges as the Fluids, Dynamics and Geotechnics problems. The Fluids and Dynamics problems were tackled through wave flume experiments on model test walls. Lateral loading tests on full scale piles were carried out to investigate the Geotechnics problem. The test data was analysed and a comparison between measurements and code predictions was made.

For the Fluids problem the Japanese standard (Okada et al., 2006) predicts tsunami wave loading accurately. At model scale the load predictions had significant scatter. A new model, named the Bernoulli Estimate, is introduced. It predicts wave load accurately at model scale and is equivalent to the Japanese standard for full scale tsunamis.

The results of the Dynamic analysis showed the wave loading causes dynamic amplification for seawalls. This amplification is however relatively small, on average 10% of the maximum wave load predicted by Okada et al. (2006). Even if the dynamic amplification is neglected, the seawall design does not change significantly.

Results of the Geotechnics analysis showed current codes such as API (2000) and DNV (1992) predict overall pile stiffness well. However the $p - y$ code predictions overestimated both soil stiffness and ultimate soil pressures. This is in line with results from recent research such as that of Kirkwood (2015). The ultimate pile lateral capacity is under-predicted by both API (2000) and DNV (1992) by a factor of 2. This is probably intentional to satisfy serviceability limits. However for seawalls displacement is not a concern and there would be benefit to be had in bringing the ultimate capacity predictions closer to the actual values.

Bringing the results from the Fluids, Dynamics and Geotechnical analyses together, a prediction is made on the highest tsunami the piles can withstand. Both piles tested can safely withstand tsunamis with height greater than their own, which is what the Japanese recommended after the 2011 Tōhoku tsunami. As such the new pile wall design seems to be a suitable replacement for concrete caissons for the purpose of tsunami defences.

Acknowledgements

I want to thank Kitamura-shachō-san for organising and continuing the Giken-Cambridge collaboration. I must also thank Prof. Malcolm Bolton for starting the Giken projects and for our engaging discussions while in Japan. I have learnt a lot over this past year from my supervisor, Dr Stuart Haigh, and wish to thank him for his support and enthusiasm. A special thanks goes to the great people at Giken. I must thank Ishihara-san not only for flawlessly organising the tests, but also for the great technical advice whilst in Japan. Okada-san deserves special mention for his great help in doing the experiments. I must thank Inomata-san and Karia-san for their hard work. Yan Zhuang's help in the data analysis done in Japan was much appreciated and I wish him good luck in this year's tests. I also want to thank everyone in Schofield for the stimulating atmosphere they created. Great thanks go to Srikanth Madabhushi, not only for his invaluable technical input and for proof-reading this report, but also for his friendship. Finally I wish to thank my father for his continued support this past year from 1500 miles away.

To my mom

Contents

1	Introduction	1
1.1	Concrete Caisson Seawall Failures	1
1.2	Steel Pile Walls as Tsunami Coastal Defences	2
1.2.1	Characteristics. Installation Method	2
1.2.2	Qualitative Appraisal. Project Aims & Challenges	3
2	Tsunami Simulator Tests	5
2.1	Goals	5
2.2	Current Codes of Practice	5
2.2.1	Maximum Tsunami Surge Force F_s	5
2.2.2	Equivalent Static Load	6
2.3	Experimental Setup	6
2.4	Results & Analysis	7
2.4.1	Pressure Readings	7
2.4.2	Wave Force. Comparison with Codes of Practice	8
2.4.3	Hydrostatic and Hydrodynamic Forces	8
2.4.4	Energy Approach	10
2.4.5	Surge Force Lower Bound	11
2.4.6	Scaling Issues. Comparison with Codes of Practice Revisited	11
2.4.7	Predictions of Full Scale Behaviour	13
2.4.8	Dynamic Analysis. Response Spectrum	14
2.5	Test Conclusions	17
3	Lateral Horizontal Tests on Steel Piles in Sand	17
3.1	Goals	17
3.2	Current Codes of Practice	17
3.2.1	General Theoretical Aspects	17
3.2.2	Ultimate Soil Pressure Predictions	19
3.2.3	$p - y$ Approach in Codes	21
3.2.4	Ultimate Horizontal Load H_{ult} Estimation	22
3.3	Experimental Setup	23
3.3.1	Loading System	23
3.3.2	Sensors. Monitoring	25
3.4	Results & Analysis	25
3.4.1	Soil Analysis	25
3.4.1.1	Grain Size Distribution	26
3.4.1.2	SPT	26
3.4.1.3	Triaxial Tests	27

3.4.2	Pile Analysis (Before Yielding)	29
3.4.2.1	Curve Fitting	30
3.4.2.2	Comparison with Codes of Practice	34
3.4.3	Plastic Regime	37
3.4.3.1	Material Law	37
3.4.3.2	Soil Pressures. Discussion	38
3.4.3.3	Comparison with Codes of Practice	39
3.5	Test Conclusions	40
4	Pile Performance as Tsunami Defence	40
4.1	Goals	40
4.2	Pile Frequency Estimation	41
4.3	Dynamic Amplification Factor	41
4.4	Overall Safety Factor Estimate	43
4.4.1	Fluids	43
4.4.2	Dynamics	44
4.4.3	Geotechnics	44
4.4.4	Total	44
4.5	Tsunami Predictions	44
5	Conclusions	45
	Bibliography	45
A	Health and Safety	48

1 Introduction

Coastal defences are vital structures in countries in the path of tsunamis and typhoons. The large waves generated by these natural events carry high energy, causing great damage and widespread floodings. This leads to large human and economic losses, the 2011 Tōhoku tsunami being a recent powerful example. Damage assessment after the tsunami (Suppasri et al., 2012) highlighted the need for a redesign of coastal defences which “should survive even if they are overtopped by a tsunami. For example, severe scouring should not occur at the foundations of seawalls, as seen in many examples investigated in this study, and the connection between seawalls and breakwater should be strengthened so that they will not slide or move and become floating debris that can increase damage.”

1.1 Concrete Caisson Seawall Failures

Suppasri et al. (2012) address issues observed with concrete caisson seawalls, many of which were overturned and swept away by the tsunami wave. Fig.1a shows a concrete seawall overturned by the 2011 tsunami. A schematic of the overturning process is presented in Fig.1b. One can notice the shallow foundations, the seawall’s design relying on its bulkiness to resist rotation. When the incoming wave was much higher than designed for, overturning was a common failure mode.

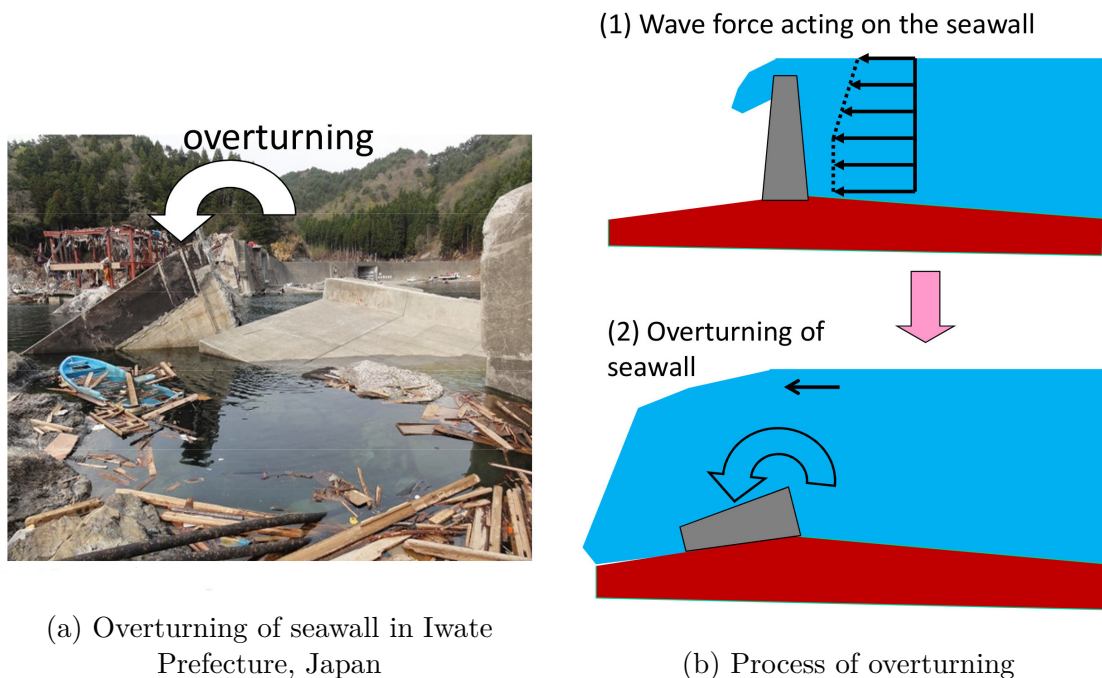


Figure 1: Seawall overturning (Kato et al., 2012)

Fig.2 shows the scouring failure mechanism for seawalls. This mode, as Suppasri et al. (2012) show, was the cause for many seawall failures. As with the overturning mechanism,

shallow foundations played an important role in making seawalls susceptible to scouring failure.

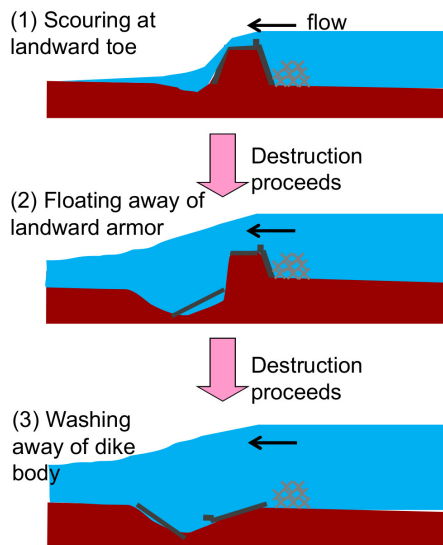


Figure 2: Scouring mechanism
(Kato et al., 2012)

embedded 10-15 m into the coastal soil (Fig 3). As can be seen in Fig 3, the coastal area in Japan is congested, with roads, railways and houses in the immediate vicinity of the coastal levees. Thus driving in the piles is not an ideal solution due to the high noise and vibrations involved. Furthermore pile driving could induce ground settlements that may affect the nearby infrastructure (especially the railways).

Alternative solutions for levee designs have been sought in Japan to address the shortcomings mentioned above. One of the new designs that is currently being installed along the Japanese coastline is a seawall made of adjoining large steel pipe piles. Steel pile walls, in contrast to concrete caissons, have greater embedment depth and display ductile behaviour at failure.

1.2 Steel Pile Walls as Tsunami Coastal Defences

1.2.1 Characteristics. Installation Method

The pile walls employed in Japan are made of large 1 m diameter hollow circular steel piles

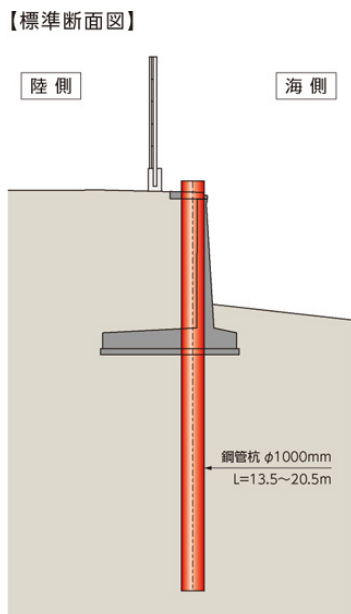


Figure 3: Steel pile walls. Elevation view (left) and Installation procedure(right)

Taking this into account, the Japanese instead jack the piles in using GRBS (Giken Reaction Base System). This pile installation method, developed by Giken Ltd., uses the uplift resistance of previously installed piles to provide the reaction force required to push the new pile in through a hydraulic jack. By relying on hydraulics the GRBS is virtually silent and vibration-free with a smaller impact on surrounding soil than driving procedures (Rockhill et al., 2003). Fig. 4 provides a schematic of the process and shows the Press-in Machine, the Silent Piler®.

1.2.2 Qualitative Appraisal. Project Aims & Challenges

The significant embedment (10-15 m) as well as the ductile behaviour a pile wall exhibit should ensure good lateral capacity against wave loads. For tsunamis greater than those designed for, the wall would deform plastically, dissipating energy. Furthermore due to the greater embedment depth, scouring should be less of an issue than for shallow foundation levees. The greater embedment also improves uplift-resistance, preventing the pile being pulled out of the ground and carried by the waves. Thus, qualitatively, pile walls improve on concrete caisson designs for all the failure mechanisms highlighted by Suppasri et al. (2012).

Measuring the piled wall performance quantitatively proves to be a greater challenge, with little research delving into this specific topic.

The problem is indeed complex, covering a range of engineering disciplines. Firstly the Fluids Engineering challenge is defining the pressures on the pile wall due to the tsunami wave in terms of the incoming wave parameters (tsunami wave height and velocity). The tsunami wave characteristics are estimated as soon as the generating earthquake has been recorded so being able to correlate these with expected impact force on the coastal defences is of great benefit. It allows authorities to determine the most vulnerable regions and to issue evacuation orders / set up relief operations in the vital minutes before the wave strikes. Secondly a Dynamics Engineering challenge comes from the varying wave input force. The flexibility of the wall may lead to Dynamic Amplification increasing the displacement of the pile wall. To further complicate matters, the wall movement itself might change the wave pressures so that the Fluids and Dynamic Problems become intercoupled. And finally the Geotechnical Engineering challenge deals with estimating the capacity and stiffness of the piles themselves. The stiffness of the pile-soil system is a



Figure 4: Pile wall jacked in by GRBS

varying quantity and it depends on the loading magnitude as well as the number of cycles the load is applied over and this couples the geotechnical problem with the other two described above. Cyclic softening would reduce the stiffness of the system which in turn affects the dynamic response which then modifies the pressure distribution on the wall. The pressure distribution gives the total force acting on the pile and influences stiffness / pile displacement, showing a full codependency between the three distinct engineering challenges.

The above paragraph highlights the complexity of the problem at hand. However admitting that an exact solution cannot be found, we can decouple the Fluids, Dynamics and Geotechnical problems and solve them separately. Then employing safe upper boundaries for each one of them we can arrive at a result that is a safe upper envelope of the possible scenarios. Hence a safe design can be constructed for a given expected tsunami wave. The 200-year return period tsunami is the current design recommendation in Suppasri et al. (2012).

Various codes of practice worldwide deal with each of the Fluids, Dynamics and Geotechnical problems respectively. Presently they are concurrently used in pile wall design. However most of these codes were written for other engineering use cases and are not specifically meant for the design of tsunami coastal defences. Thus questions arise on the adequacy of applying the codes for the design of tsunami pile walls. For example:

- To estimate the wave impact force on the piles the Japanese use the results of Okada et al. (2006). These were derived for the purpose of design of tsunami refuge buildings. Hence an assumption is that the incoming wave height is considerably smaller than the size of the structure. This does not hold for coastal walls, especially the ones built after 2011 meant to withstand tsunamis higher than their own height.
- Pile lateral capacity standards, both the American API (2000) and Norwegian DNV (1992), are based on the works of Broms (1964) and Matlock and Reese (1960). Their semi-empirical relationships are derived on piles 0.4-0.6 m in diameter. Since the piles installed in Japan are about twice as big a valid point is how well do these semi-empirical equations hold when scaled up?

This Project, through appropriately designed experiments, means to address these questions and see how well do codes suit the tsunami defence use case. Thus an assessment will be made on the suitability of currently installed Jacked-in Steel Piles as Tsunami Defences. Furthermore an estimate shall be given on the maximum tsunami wave height that current piles in Japan are expected to resist.

2 Tsunami Simulator Tests

2.1 Goals

The tsunami simulator tests were wave flume experiments meant to investigate the adequacy of current codes' predictions for the Fluids and Dynamics problems as defined in Section 1.2.2, namely how well the standards predict the wave pressures from larger-than wall tsunamis on a scaled system and whether there is any Dynamic Amplification that must be accounted for.

2.2 Current Codes of Practice

2.2.1 Maximum Tsunami Surge Force F_s

Even though tsunamis have been the cause of major damage in recent years (Indian Ocean 2004, Tōhoku 2011) there are still relatively few tsunami design codes worldwide, many of which base their recommendations on the same couple of papers. Among these papers the results of Camfield (1980) are notable, many of his formulae being present in recent design guidelines from America (Heintz and Mahoney, 2008) and Japan (Okada et al., 2006). Lloyd and Rossetto (2012) include a detailed study on the available guidelines on tsunamis and their origins.

Tsunamis are defined by their wave height and velocity. Even though the relationship between the two is complex and there is little measured data on tsunami velocity, Camfield (1980) suggests Eqn.1 as a good approximation for the purpose of initial surge force estimation:

$$v_{wave} = 2\sqrt{gh_d} \quad (1)$$

g in Eqn.1 is gravitational acceleration and h_d is the still water depth. Since Camfield (1980), Heintz and Mahoney (2008) and Okada et al. (2006) were only concerned with waves significantly smaller than the structures they interacted with, the hydrostatic force per unit width of wall takes the form of Eqn.2

$$F_{hs} = \frac{1}{2}\rho gh^2 \quad (2)$$

h in Eqn.2 represents the height of the water column just in front of the wall at any given time. Similarly the hydrodynamic force is defined in Eqn.3

$$F_{hd} = \frac{1}{2}C_d \rho hv^2 \quad (3)$$

C_d is the drag coefficient, taken to be 2 as suggested in the codes. To find the maximum tsunami surge force F_s , Camfield (1980) used the Chezy roughness coefficient to

estimate the dynamic component and then Eqn.1 to derive the result in Eqn.4. Several approximations were made during the derivation.

$$F_s = \frac{9}{2}\rho g h_{wave}^2 = \frac{1}{2}\rho g (3 \times h_{wave})^2 \quad (4)$$

The result in Eqn.4 suggests that the maximum force exerted by a tsunami on a seawall is equivalent to the hydrostatic force exerted on the wall by a water column three times bigger than the tsunami wave height. This is the interpretation Okada et al. (2006) adopts, limiting the methodology to walls with $h_{wall} \geq 3 \times h_{wave}$.

However for seawall design against tsunamis this limitation is removed. Thus assuming the maximum surge height is $3 \times h_{wave}$ and $h_{wall} < 3 \times h_{wave}$, the surge force F_s on the wall can be calculated by integrating a trapezoidal pressure distribution cut-out from the $3 \times h_{wave}$ hydrostatic pressure triangle (Fig.5).

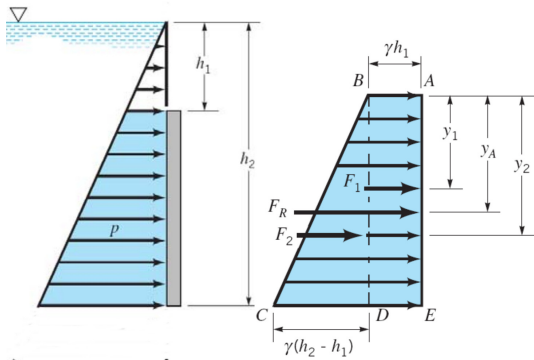


Figure 5: Trapezium pressure distribution for $h_{surge} > h_{wall}$

$$\begin{aligned} F_s &= \frac{h_{wall}}{2} [\rho g \times 3h_{wave} + \rho g \times (3h_{wave} - h_{wall})] \\ &= \frac{1}{2}\rho g h_{wall} (6h_{wave} - h_{wall}) \end{aligned} \quad (5)$$

2.2.2 Equivalent Static Load

Palermo et al. (2009) reviewed codes around the world to determine the maximum equivalent static force to be used in the design of coastal defence seawalls. They concluded that most codes just recommend taking the Tsunami Surge Force F_s as the design static loading. Essentially the codes use a unity factor of safety. Furthermore no explicit provisions are made to account for the dynamic nature of wave loading. It seems that the factor of 3 on wave height is thought to encapsulate all the wave behaviour. This is also the case for the Japanese standard (Okada et al., 2006).

2.3 Experimental Setup

The experiments were carried out in the Giken Tsunami Simulator, a 14m long by 2m wide wave flume. Two model walls, 10 cm and 15 cm high respectively, were used in these tests. Pressure transducers were fitted along the height of the walls to quantify the wave-wall interaction. The general dimensions of the walls as well as the position of the pressure transducers can be seen in Fig.6.

A wave height sensor and 2 velocity sensors were fitted in the flume just in front of

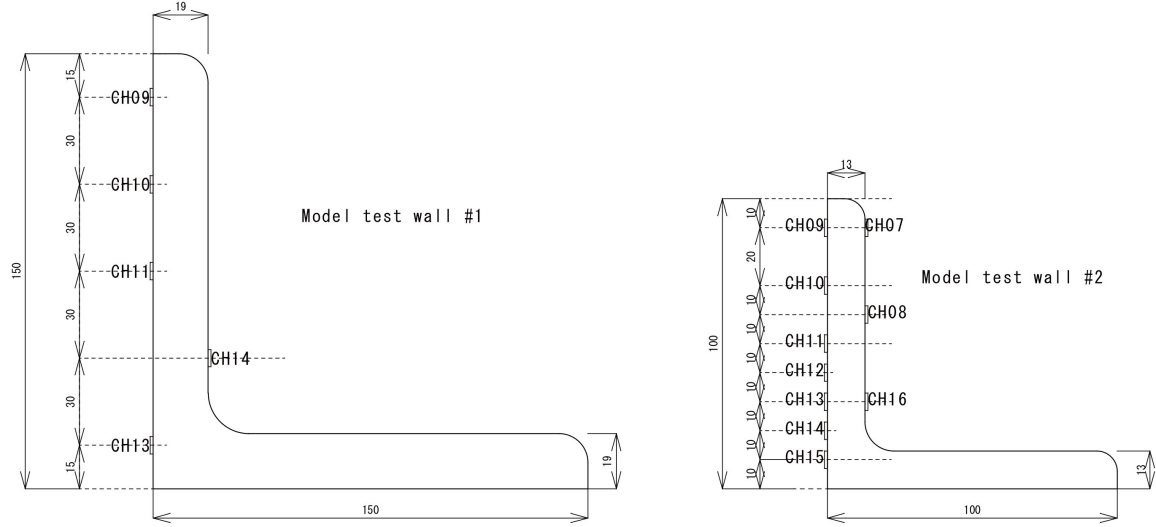


Figure 6: Model test walls. General dimensions (mm) and sensor placement

the test wall. One of the velocity sensor was lowered to the base of the wall, while the other measured water velocities just above the wall as the water passed.

Another height and velocity sensor pair was positioned 4m in front of the wall to measure the incoming wave characteristics. The sensors can be seen in Fig.7 A total of 47 experiments were carried out in the Tsunami Simulator, with wave heights both greater and smaller than the height of the walls. The sampling frequency for all sensors was 1 kHz.



Figure 7: Giken Tsunami Simulator. View from behind the 10 cm model wall. Wave sensors in view

2.4 Results & Analysis

2.4.1 Pressure Readings

The transducers readings were first normalised to reflect only the excess water pressure due to the incoming wave. Fig.8 shows full unfiltered results for one of the tests. One can notice the high maxima recorded when the bore hit the wall (highlighted by horizontal bars in Fig.8). These high pressures presented large variability, with durations between 5-100 ms. Furthermore the peaks did not happen concurrently for all sensors.

These maxima were also hard to reproduce reliably, repeating a test with exactly the same wave input yielding different peak distribution and magnitudes. Lugni et al. (2006) and Kihara et al. (2015) encountered the same phenomena in their wave flume

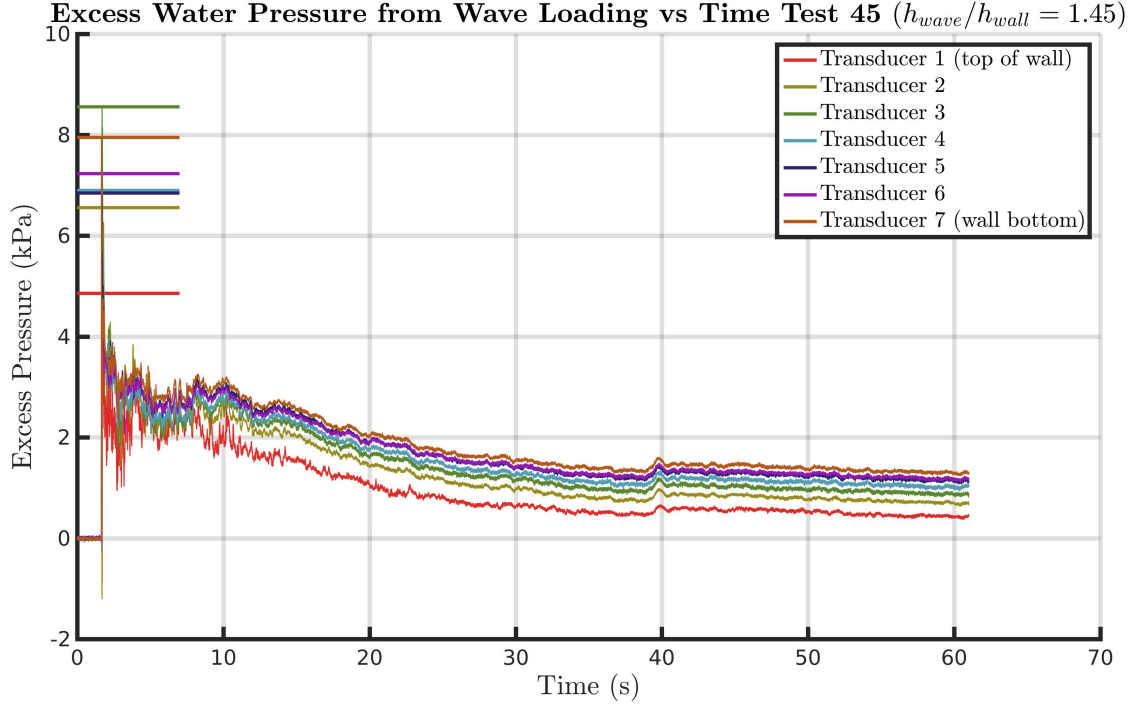


Figure 8

experiments and attributed their irregular behaviour to the adiabatic process of a large amount of air bubbles being mixed in the turbulent bore.

2.4.2 Wave Force. Comparison with Codes of Practice

A different way to plot the results is to look at the total force acting on the wall by integrating the pressure values. Since the pressure peaks are asynchronous this reduces their perceived overall effect on the structure. Furthermore standards of practice deal with total wave force acting on structures and as such we can compare our force results to the code values. Okada et al. (2006) consider a maximum wave loading equivalent to a hydrostatic distribution $3 \times h_{wave}$ high. Since for many of our tests $h_{wall} < 3 \times h_{wave}$ the trapezoidal integration in Eqn.5 was used (see Fig. 5).

As can be seen in Fig.9 there was large variation in quality of agreement between code and experiments. Generally for tests in which $h_{wave}/h_{wall} < 1.5$ the Japanese code under-predicted the maximum surge force while for tests with $h_{wave}/h_{wall} > 2$ it overpredicted the maximum force by as much as 70%.

2.4.3 Hydrostatic and Hydrodynamic Forces

To further investigate whether it was possible to get more accurate predictions of measured maximum force, the hydrostatic and hydrodynamic components of the wave force were plotted separately on top of the experimental data (Fig.10a). These force components

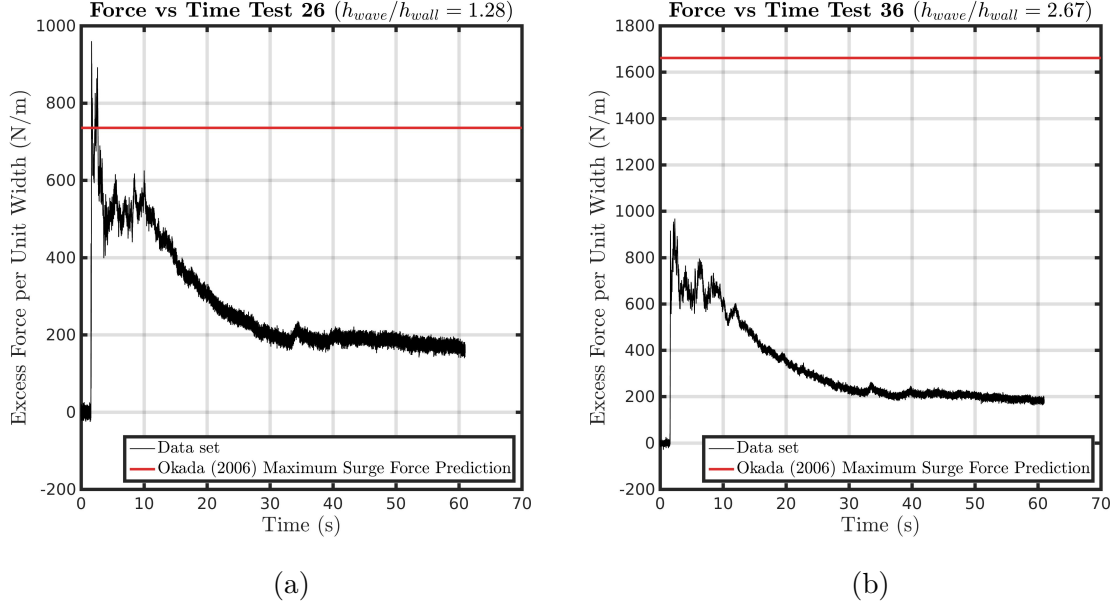


Figure 9

were evaluated using the definitions given in Eqns.2 & 3. As input, the hydrostatic force calculation used the wave height as measured by the height sensor in front of the wall. The hydrodynamic force was evaluated twice, once with the wave velocity read from the sensor at the bottom of the wall and another time using the sensor just above the wall. These two values should define suitable boundaries for the actual hydrodynamic force experienced by the wall during the tests.

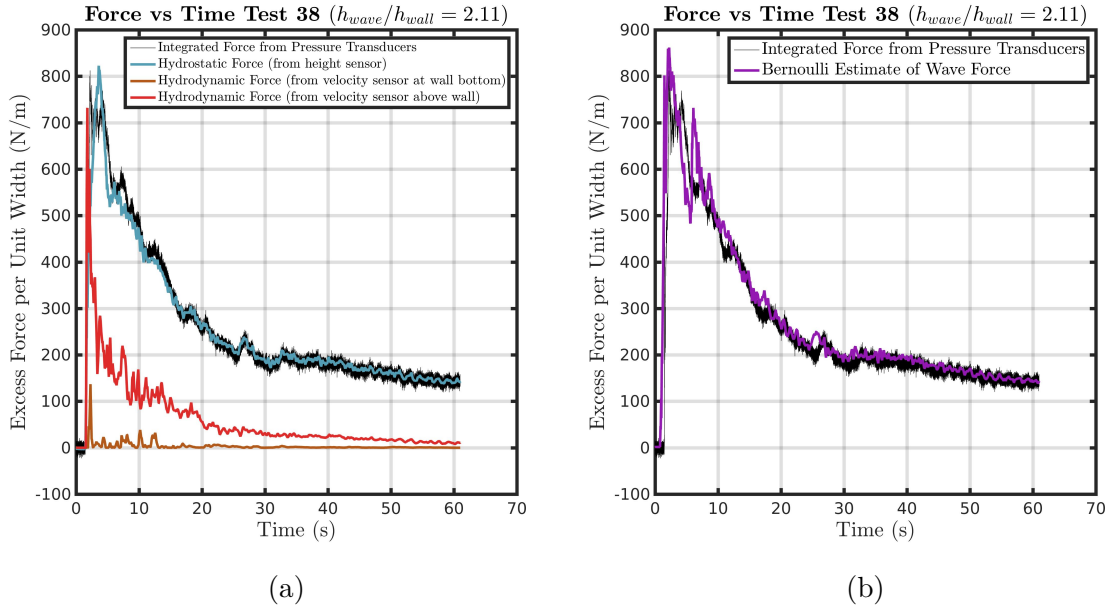


Figure 10

Fig.10a shows that both the hydrostatic and upper hydrodynamic forces offer accurate predictions of the maximum wave force measured in the experiment. This was found to

be true for all tests irrespective of wave height. However each by its own predicts the wave force well, which is at odds with standards like FEMA (Heintz and Mahoney, 2008) which take the sum of the hydrostatic and hydrodynamic forces as the surge force. Lloyd and Rossetto (2012) noticed the same discrepancies in their experiments.

2.4.4 Energy Approach

The fact that the height and velocity data near the wall can be used to yield accurate predictions of the wave force is encouraging. However it would be beneficial if the wave height h_{wave} and wave velocity v_{wave} data from the sensors far away from the wall could somehow be used to find the surge force. Those sensors simulate wave parameters away from the coast and are usually known for real tsunamis as soon as the generating earthquake has been recorded.

Since the Hydrostatic Force close to the wall gives a correct estimate of surge force (see Section 2.4.3) we shall try and predict it from initial wave parameters. Assuming little energy is lost by the wave as it travels through the flume we can write Bernoulli's energy conservation along a streamline (Eqn.6).

$$\rho g h_{wave} + \frac{\rho v_{wave}^2}{2} = \rho g h + \frac{\rho v^2}{2} = \text{const.} \quad (6)$$

Assuming the hydrostatic force on the wall is maximum when wave velocity is low (mass conservation; increase in height of the bore comes with decreased wave velocity) setting $v = 0$ in Eqn.6 yields an upper bound for the maximum wave height h_{max} (Eqn.7).

$$h_{max} = h_{wave} + \frac{v_{wave}^2}{2g} \quad (7)$$

To calculate force from h_{max} Eqn.5 was used, substituting surge height, $3 \times h_{wave}$, with h_{max} .

Fig.10b plots this approximation, named the Bernoulli Estimate, against measured force. As can be seen the Estimate matches the surge force well. It is an upper envelope of measured force, while staying close to it.

Having the new Bernoulli Estimate approximation, it is interesting to see how this compares to the Japanese standard's predictions.

This comparison can be found in Fig.11. Measured maximum wave force was plotted against predictions from both Okada et al. (2006) and the Bernoulli Estimate. Since tests were carried out on two different model test walls, the plot features non-dimensional expressions of force to accommodate both data sets.

As can be seen in Fig.11 linear fits through both data sets predict the measured wave forces very well. The gradients of the line fits, which equivalently represent factor of safety (FoS) in prediction, are close to unity. To find the better fit in a quantifiable manner, the

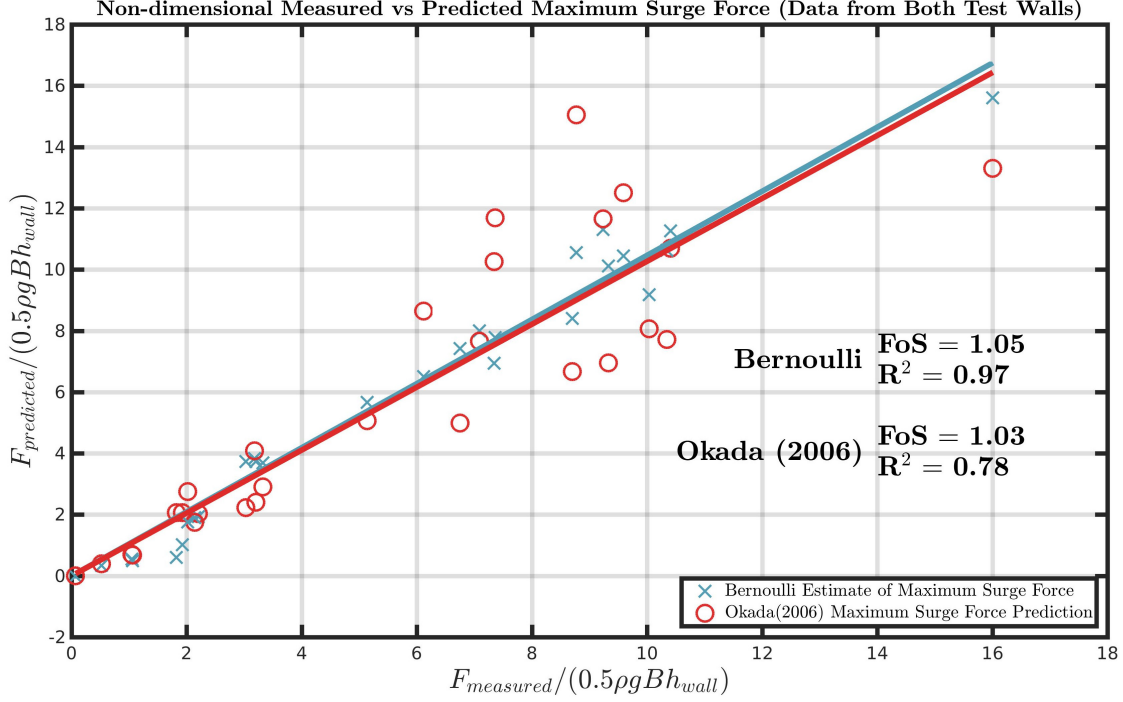


Figure 11

coefficient of determination, R^2 , was evaluated. R^2 takes values from 0 to 1; the higher the value, the better the model is at predicting the data.

With $R^2 = 97\%$ the Bernoulli Estimate gives more accurate predictions of Surge Force than the Japanese code for our scaled wave flume tests.

2.4.5 Surge Force Lower Bound

Fig.12 shows normalised plots of measured maximum wave force against wave height. The red line in the plot is the Japanese standard prediction.

There is a noticeable increase of force with increasing wave height for $h_{wave} < 1.5 \times h_{wall}$. For larger waves it seems the gradient reduces considerably. To try and explain this trend one can think of the case when $h_{wave} \gg h_{wall}$. Then, as the wave passes, its height and velocity are not altered by the presence of the wall and hence the pressure distribution is hydrostatic of height h_{wave} . The green line in Fig.12 represent this force and as expected it is a lower bound for the measured forces. We can reasonably expect the force on the wall to asymptote to the green line as $h_{wave} \rightarrow \infty$.

2.4.6 Scaling Issues. Comparison with Codes of Practice Revisited

Even though the results in Fig.11 show good agreement between measured values and predictions there are several issues with using them as validation for accuracy of the methods in predicting real-scale tsunamis. Firstly, as highlighted by Lloyd and Rossetto

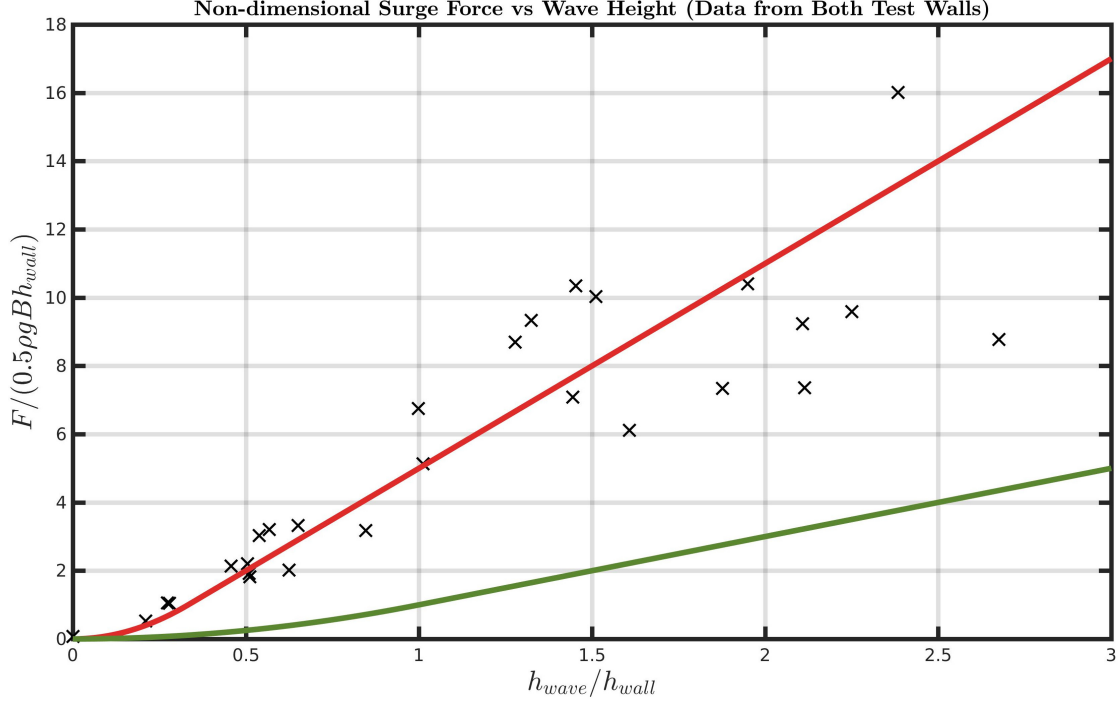


Figure 12

(2012), real tsunamis have very long wavelengths which can reach hundreds of kilometres in deep water. Waves generated in the wave flume have very short wavelengths and it is still an open issue whether this affects the accuracy of the results. Secondly the wave generation method is different between the tsunami and the wave flume. The tsunami is a gravitational wave formed by the sudden movement of the Earth's crust in an earthquake. In deep water the tsunami does not transport mass and it collects water only when near the shoreline. The Tsunami simulator generates the wave by opening the tank gate and adding water into the wave flume. The two mechanisms are distinct and this constitutes a big hurdle in the face of scaling. However we can attempt to quantify this difference and check if we get sensible results.

The fact that the generating mechanism is different means Eqn.1 does not apply in the Tsunami simulator. Indeed an easy check of v_{wave} against h_{wave} shows our incoming wave velocities are greater than both $2\sqrt{gh_d}$ and $2\sqrt{gh_{wave}}$. This means Eqn.4 no longer holds true and hence there is less physical basis sustaining the $3 \times h_{wave}$ approach in the Tsunami Simulator experiments. This explains why the Japanese code force predictions in Fig.11 have a relatively high scatter compared to the Bernoulli Estimate.

On the other hand it would be interesting to do the reverse and see what becomes of the Bernoulli Estimate when applied to full scale tsunamis. For this we will use the same approximation used by Palermo et al. (2009) in deriving Eqn.4:

1. Eqn.1 holds true

2. The tsunami wave breaks into a turbulent bore near the shore, when $h_{wave} = h_d$

Then the Bernoulli Estimate in Eqn.7 becomes:

$$h_{max} = h_{wave} + \frac{v_{wave}^2}{2} = h_{wave} + \frac{(2\sqrt{gh_d})^2}{2g} = h_{wave} + 2h_d = 3 \times h_{wave} \quad (8)$$

And hence the Bernoulli Estimate and the Japanese standard (Okada et al., 2006) become equivalent when dealing with real scale tsunamis. The advantage of the Bernoulli estimate was that by decoupling the 2 parameters, wave height and wave velocity, it gave accurate results for model tests for which the two are not linked by the same relationship as tsunamis are.

2.4.7 Predictions of Full Scale Behaviour

The Bernoulli Estimate applies well in our model tests and is equivalent to established results in literature for full scale tsunamis. Thus by corroborating experiment findings highlighted in Section 2.4.4 with previous research as discussed in Section 2.2.1 we make a prediction on the general shape of the relationship between maximum surge force and wave height (Fig.13).

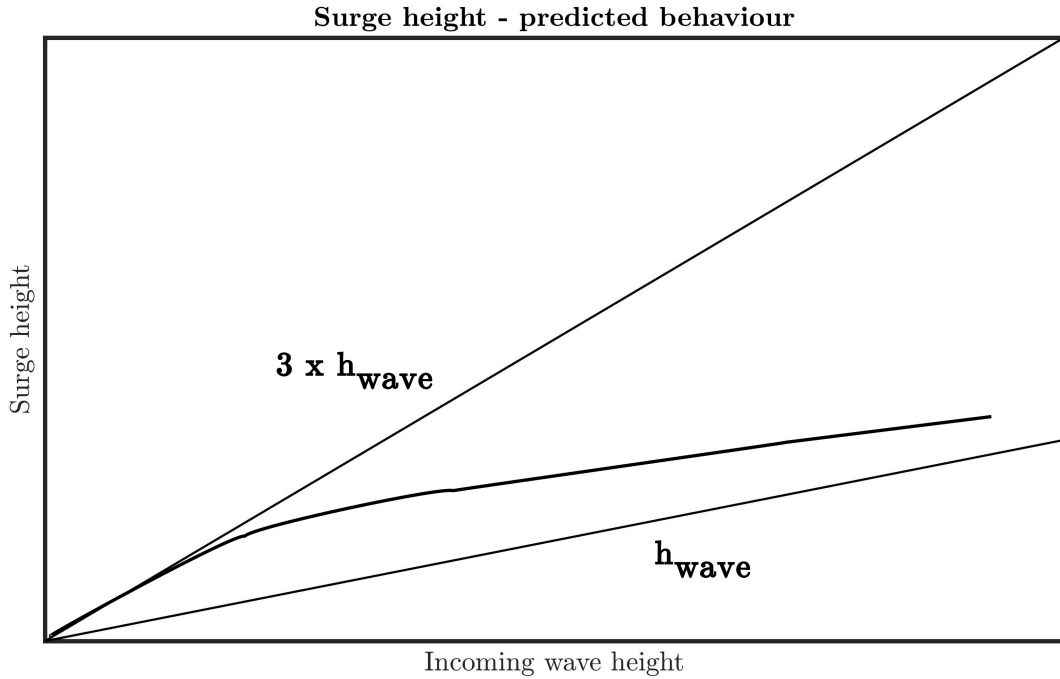


Figure 13

For smaller tsunami waves, the maximum surge force is well approximated by the trapezoidal cut-out from the hydrostatic force of a water column of height $3 \times h_{wave}$ (Eqn.5). Very large waves are not affected by the presence of the wall and hence the

maximum water height above the wall base will be h_{wave} (see Section 2.4.5). The maximum force on the wall is calculated then by substituting $3 \times h_{wave}$ with h_{wave} in Eqn.5.

Fig.13 shows that the current Japanese code's $3 \times h_{wave}$ approach should be a safe upper boundary of surge force for any wave height. Hence, even though it was designed for structures considerably higher than incoming waves, the standard should provide safe design forces for the case of seawalls smaller than the tsunami too.

However for very high waves the code approach is expected to overestimate the surge force by up to 200%. Thus it would be beneficial to know when the curve in Fig.13 breaks off the initial $3 \times h_{wave}$ gradient. In Section 2.4.5 we make the observation that surge force results plotted in Fig.12 show reduction in gradient for waves $1.5 \times$ bigger than the model walls. However in our tests Eqn.1 does not hold so such observations cannot be directly translated to real scale waves. Further research possibly conducted in long wave generators to accurately simulate tsunami wave characteristics (Lloyd and Rossetto, 2012) could investigate the correctness of the shape prediction in Fig.13 and potentially find the deviation point in a manner applicable to real scale tsunamis and seawalls.

2.4.8 Dynamic Analysis. Response Spectrum

The maximum tsunami surge force on a seawall can be predicted well by current standards of practice. However wave loading is dynamic in nature. Using the wave force time histories from our flume experiments we shall investigate whether there is any significant dynamic amplification in the typical frequency range of seawalls and buildings.

To do so we carry out a simplified 1D analysis in which we model the mass, stiffness and damping of the pile-soil system by parameters m, k and ζ respectively. We can then express the resultant force as in Eqn.9.

$$ma = F_{total} = F_{wave} - ku - C_v \dot{u} \quad (9)$$

where u is pile lateral displacement and C_v is the damping constant, $C_v = 2\zeta\sqrt{km}$. Substituting the angular frequency $\omega = \sqrt{k/m}$ in Eqn.9 we arrive at the standard form of the equation of motion:

$$\ddot{u} + 2\zeta\omega\dot{u} + \omega^2u = \frac{F_{wave}}{m} \quad (10)$$

Codes of practice do not offer explicit guidance on dynamic calculations for waves. On the other hand dynamics is vital in Earthquake Engineering. For seismic calculations Eqn.11 represents the equation of motion (Kramer, 1996).

$$\ddot{u} + 2\zeta\omega\dot{u} + \omega^2u = -\ddot{u}_g \quad (11)$$

where $\ddot{u}_g$ represents the ground motion. As can be seen mathematically Eqns.10 & 11

are equivalent, the only difference being the driving force. In the case of tsunamis this is the wave force, while for earthquakes it is the ground motion driving the system dynamics. Because the equations are analogous we can use methods and results from Earthquake Engineering to quantify the dynamics of our system. Hence we shall numerically integrate Eqn.10 for a range of natural periods $T = 2\pi/\omega$ to evaluate the spectral acceleration $S_a = \omega^2 S_d$ (Kramer, 1996). However the system mass m is an unknown constant and is needed in Eqn.10. Since the equation of motion is a linear second order differential equation multiplying the input by a constant value results in the output being scaled by the same amount. We can use this property to simplify the mass term:

$$\text{Input : } \frac{F_{wave}}{m} \xrightarrow{\times m} F_{wave} \quad \text{Output : } S_a \xrightarrow{\times m} mS_a \equiv F_{dyn} \quad (12)$$

Eqn.12 shows that by setting the wave force time history $F_{wave}(t)$ as input to the equation of motion we will obtain the spectral maximum dynamic force as output. The maximum surge force (F_s) was then used to normalize results so as to compare spectra between different wave force data sets:

$$F_s = \max(F_{wave}(t)) \quad \text{DAF} = \frac{F_{dyn}}{F_s} \quad (13)$$

where DAF is the Dynamic Amplification Factor. Using the Newmark- β method for the numerical integration the DAF spectrum was obtained for each test.

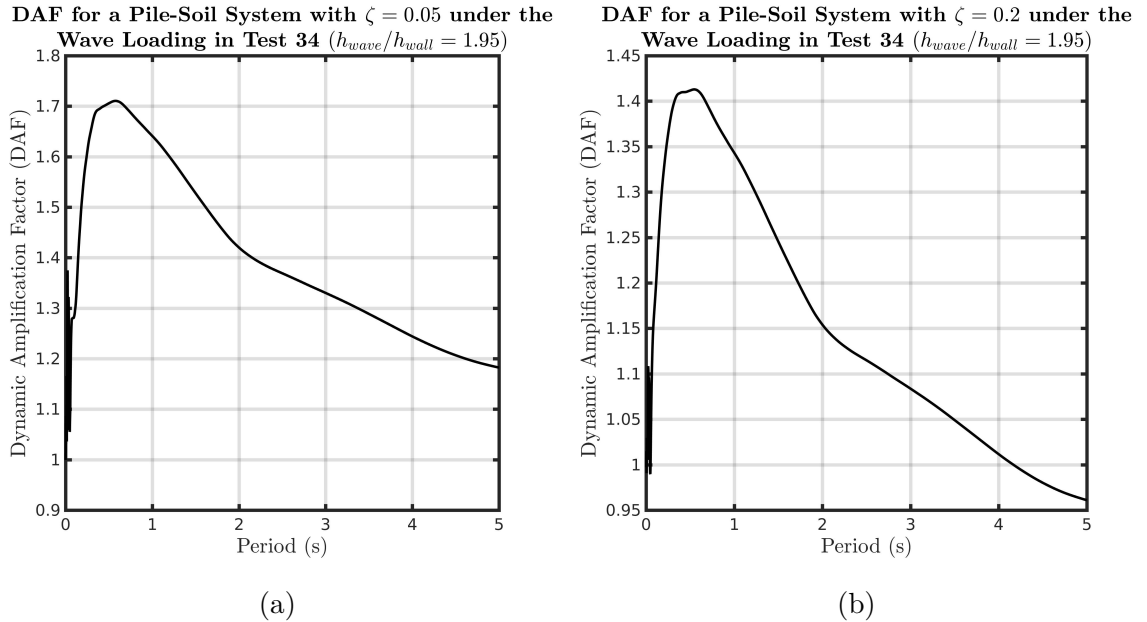


Figure 14

Fig.14a shows the DAF spectrum for a test with a wave height about twice that of the model wall and considering $\zeta = 0.05$ (common in Earthquake Engineering). Dynamic

amplification is noticeable across all periods, going as high as 70% for systems with a natural period $T_n \approx 0.5$ s. Buildings have periods of about 1-2 s and steel pile walls are more elastic and display even lower periods $T_n \approx 0.1$ -0.3 s (see Section 3). This puts them in the range of high dynamic amplification. Fig.14b shows the results of the same tests for a system with high damping. Even if the peak amplification is reduced, the peak dynamic force is still 40% higher than the maximum surge force.

The same behaviour was encountered for experiments with wave heights smaller than the model wall (Fig.15)

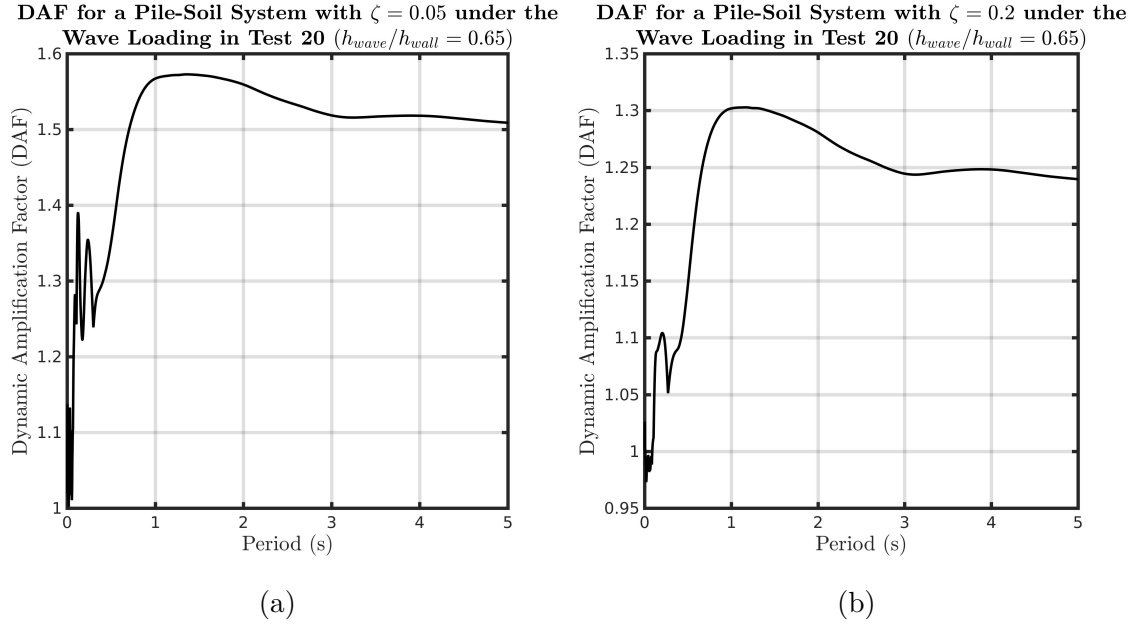


Figure 15

The amplification for these tests is of the same order as that for the high wave experiments. As mentioned in Section 2.2.2, current codes of practice make no explicit provisions for the dynamic loading and recommend using F_s as the equivalent maximum static loading. This seems unsafe, however it might be that the pile wall design itself features enough safety margins that adding a dynamic safety factor on top would constitute over-design. The issue will be revisited in Section 4.3 after the results on the pile tests have been discussed.

Furthermore there are a few points to make on the applicability of the dynamic results. Firstly this is only a simple 1D analysis, more involved modelling is required to get accurate design spectra. Secondly, the dynamic wave histories were measured in the Tsunami Simulator, at scale. This does not guarantee the same frequency content will be present in a true scale tsunami. However the waves generated in the Simulator were turbulent bores like broken tsunami waves so there might not necessary be a great difference. The issue requires further research for clarification.

2.5 Test Conclusions

The Japanese standard provided safe surge force predictions for our tests. However the Bernoulli Estimate improved on the accuracy of the predictions while being equivalent to the Japanese standard for full scale tsunamis. Hence its factor of safety (Fig.11) shall be considered as relating measurements and predictions in the fluids problems. This will be used in Section 4.4.

Wave dynamics appear to increase the force on a seawall, however they are not explicitly taken into account in codes of practice. For the purpose of the pile tests setup it shall be assumed a trapezoidal cut-out from the hydrostatic pressures of a water column $3 \times h_{wave}$ high acts on the piles.

3 Lateral Horizontal Tests on Steel Piles in Sand

3.1 Goals

Two tests were carried out on full scale instrumented steel piles to measure their lateral capacity. The tests were designed to investigate the issues raised by the Geotechnical problem as defined in Section 1.2.2. Namely how accurately do the standards predict:

1. the maximum horizontal load H_{ult} the piles can carry
2. the soil pressures p acting on the pile, resisting the movement
3. the load-displacement $(H - y)$ curve

3.2 Current Codes of Practice

3.2.1 General Theoretical Aspects

When a horizontal load is applied to a pile, movement will be resisted by its own stiffness combined with that of the soil. The soil displaces around the pile to accommodate movement in a complex, 3D manner (Fig.16a).

Directly measuring soil stress distribution around the pile or finding a closed analytical solution for the stresses have proven to be complex tasks without any considerable results until the present day. Hence instead of focusing on the exact stresses in the ground, research and codes use the soil pressure p to define ground response to movement. p represents the total force exerted by the soil on a section of the pile at given depth z below ground. Eqn.14 defines p for a circular pile.

$$p(z) = \int_0^{2\pi} \sigma(z, \theta) R d\theta + \int_0^{2\pi} \tau(z, \theta) R d\theta \quad (14)$$

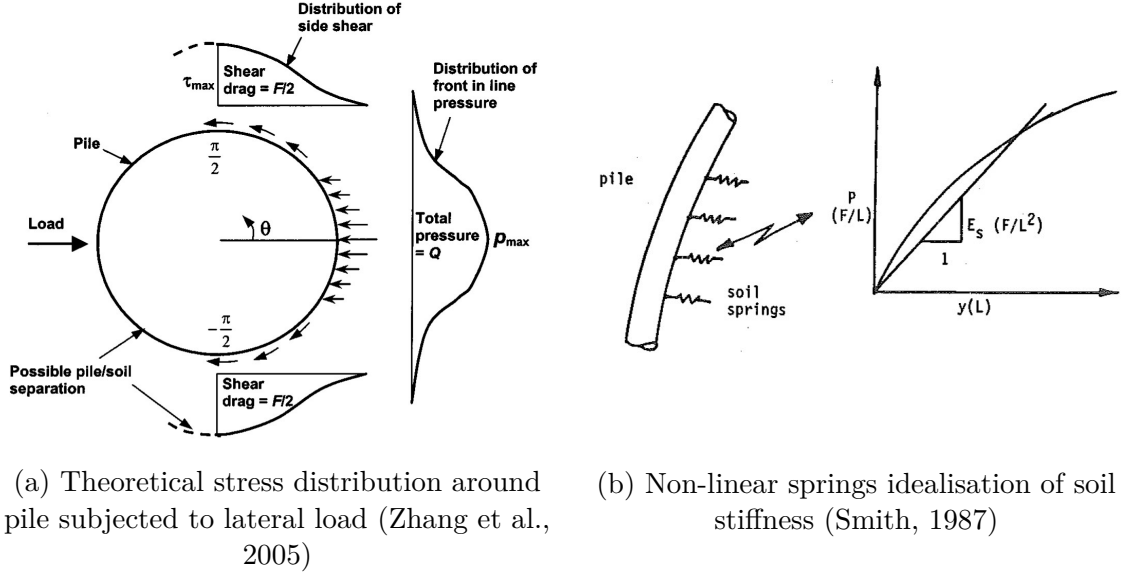


Figure 16

where θ is the integrating angle as shown in Fig.16a, (σ, τ) represents the soil stress state around the pile and R is the pile radius.

Denoting the lateral displacement of the pile at depth z by $y(z)$ the soil stiffness at any given displacement can be defined as:

$$E_s = \frac{p}{y} \quad (15)$$

$p(y)$ is inherently non-linear and dependent upon number of loading cycles; for a detailed analysis see Kirkwood (2015). Hence a common way to approach pile lateral deflection problems is to consider E_s a non-linear spring constant which varies with displacement y , depth z , ground conditions and number of cycles. Such springs are known as Winkler springs. If $E_s(y, z, grd, cyc)$ is known we can proceed to integrate $p = E_s \times y$ numerically and solve equilibrium on the whole pile. The analysis results are moment, shear, soil pressure and displacement distributions along the length of the pile. This is commonly known in literature as the $p - y$ method and is shown schematically in Fig.16b.

With the $p - y$ relationship established the equilibrium of a small pile element can be expressed through the forces and moments in Fig17. Moment equilibrium then yields:

$$(M + dM) - M + Sdz + Vdy = 0 \quad (16)$$

For our lateral loading tests $V \simeq 0$ so Eqn.16 reduces to the standard beam theory result: shear is the derivative of moment. From force equilibrium we find another established result: pressure on the beam is the derivative of shear. Hence from beam theory

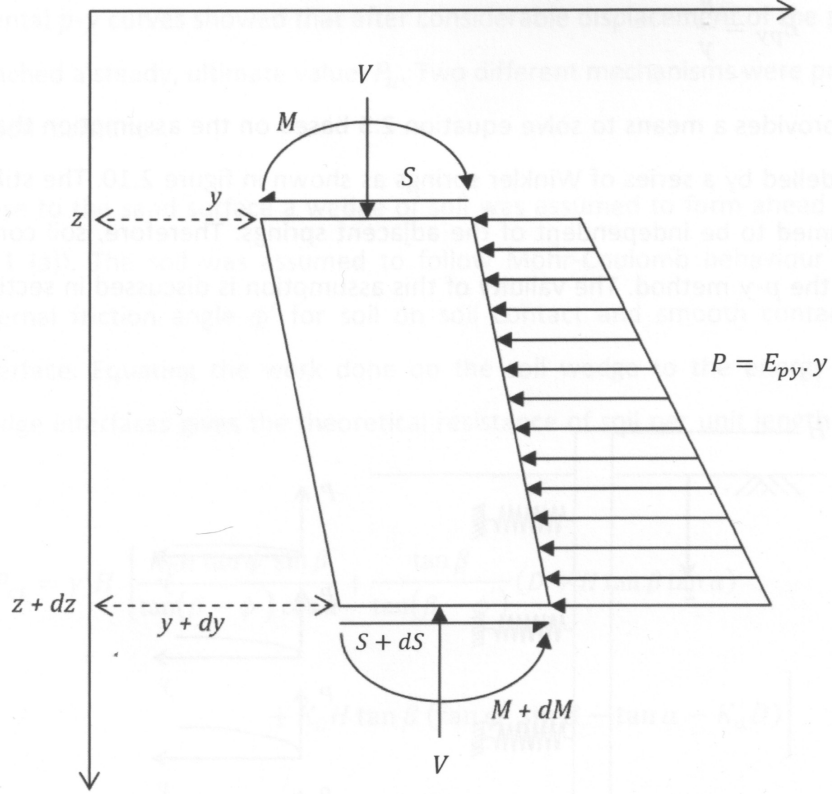


Figure 17: Equilibrium of a pile element (Kirkwood, 2015)

we can write:

$$\kappa = \frac{d^2 y}{dz^2} = \frac{M}{EI} \quad p = -\frac{d^2 M}{dz^2} \quad (17)$$

where κ is pile curvature and EI represents bending stiffness (for elastic response). Hence if moment distribution along the pile $M(z)$ is known differentiating twice will give earth pressures while integrating twice will yield displacement. Combining results from Eqns.15 & 17 the differential equation governing pile response becomes (for constant EI):

$$\frac{d^4 y}{dz^4} + \frac{E_s}{EI} y = 0 \quad (18)$$

3.2.2 Ultimate Soil Pressure Predictions

An important requirement for the $p - y$ method is a good prediction of the ultimate soil pressure p_u also known as ultimate lateral resistance. p_u is the soil resistance beyond which increasing y will not lead to an increase in p .

Hansen and Christensen (1961) suggested Eqn.19 as a method to approximate the ultimate resistance of sands.

$$p_u = K_q \gamma' z D \quad (19)$$

where K_q is an earth pressure coefficient depending on Φ' (friction angle), γ' is the effective unit weight of soil and D is pile width or diameter. Broms (1964) based on tests on 0.4 m diameter flexible piles proposed the following expression for K_q :

$$K_q = 3 \times K_p \quad (20)$$

in which K_p is the passive earth pressure coefficient, $K_p = (1 + \sin \Phi') / (1 - \sin \Phi')$. Based on distinct tests on 0.5-0.6 m diameter flexible piles Reese et al. (1974) proposed a different equation for K_q , a semi-empirical relationship derived with the help of numerical modelling.

$$K_q = K_p - K_a + \frac{z}{D}(K_p - K_0)\sqrt{K_p} \tan \alpha + \frac{z}{D}K_0\sqrt{K_p} \left(\frac{1}{\cos \alpha} + 1 \right) \tan \Phi' \sin \beta \quad (21)$$

near the surface and

$$K_q = K_p^3 + K_0 K_p^2 \tan \Phi' - K_a \quad (22)$$

well below ground.

K_a is the active earth pressure coefficient, $K_a = (1 - \sin \Phi') / (1 + \sin \Phi')$, K_0 is the in-situ earth pressure coefficient, $\beta = 45^\circ + \Phi' / 2$, α is wedge angle. The reason for the difference in formulae with depth is due to the fact that Reese et al. (1974) consider two distinct soil failure mechanisms. The first one assumes soil near the ground surface will fail through a wedge mechanism in front of the pile. This yields Eqn.20. The second mechanism is a planar one, soil at depth being assumed confined enough to only displace in plane resulting in the K_q formula of Eqn.21.

Bogard and Matlock (1980) noticed some of the terms in Eqns.21 & 22 were essentially constant for naturally occurring soils so they provided a simplified form for K_q :

$$K_q = C_1 \frac{z}{D} + C_2 \quad (23)$$

near the surface and

$$K_q = C_3 \quad (24)$$

well below ground.

C_1, C_2 and C_3 are functions of Φ' only. DNV (1992) uses this approach and Fig.18

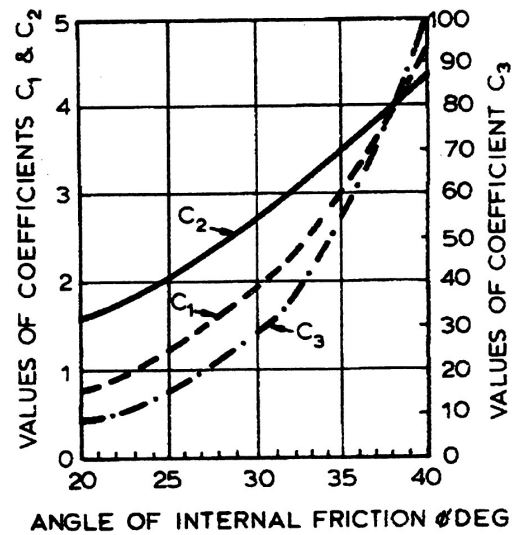


Figure 18: Coefficients as functions of Φ' (DNV, 1992)

plots the three coefficients against friction angle. However in DNV (1992) the right-hand side of Eqns.23 & 24 are multiplied by a factor A to give K_q :

$$A = \left(3 - 0.8 \frac{z}{D}\right) \geq 0.9 \quad (25)$$

Research by Fleming et al. (1992) suggests a new approximation for K_q :

$$K_q = K_p^2 \quad (26)$$

For most soils $K_p > 3$ so Eqn.26 will give higher K_q values than Eqn.20. The soil is assumed to be stronger in Fleming et al. (1992) than it is in Broms (1964). Furthermore as Eqns.23 & 24 are used in DNV (1992) to estimate ultimate lateral resistance so is Eqn.26 used by API (2000) for the ultimate pressure estimate. Fleming et al. (1992) points out that these different methods to estimate K_q can produce significantly different results. As such one of our goals will be to check ultimate pressures in our tests against the predictions given by Eqns.20-26.

3.2.3 $p - y$ Approach in Codes

The $p - y$ method is the approach most codes of practice use for lateral loading of piles. They generally adopt Eqn.19 for the ultimate lateral resistance p_u calculation with K_q

taken from one of Eqns.20-26. However the general shape of the $p - y$ curve up to failure is estimated through a mostly empirical formulation based on fitting a wide range of full scale pile results:

$$p = Ap_u \tanh\left(\frac{kz}{Ap_u}y\right) \quad (27)$$

where coefficient $A = \left(3 - 0.8 \frac{z}{D} \geq 0.9\right)$ and k is the initial modulus of subgrade reaction. Eqn.27 is used in DNV (1992) with k being read off the graph reproduced in Fig.19.

As Haigh (2013) points out, there are some open issues with the $p - y$ method as currently used by standards. Firstly the data set the $p - y$ curves were calibrated against was mostly made up of small 0.4-0.6 m diameter flexible piles. However the

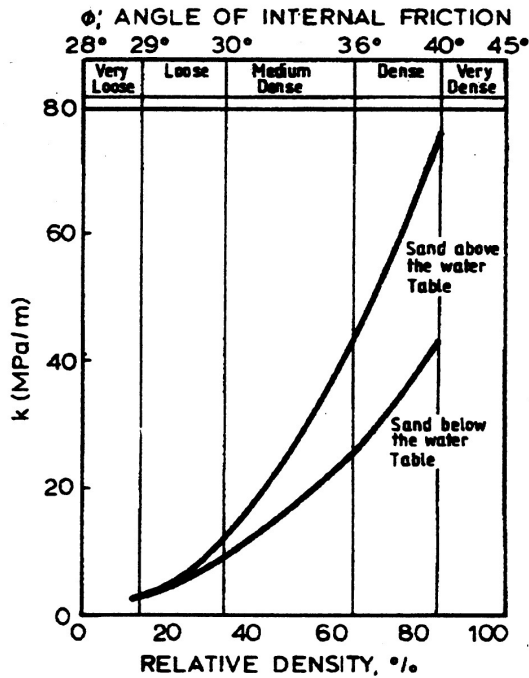


Figure 19: k as a function of I_D and Φ' (DNV, 1992)

method is extrapolated to large diameter rigid piles like those used in the tsunami seawalls

or monopile offshore wind turbine foundations. This direct extrapolation may lead to errors in predicting the true pile response. Secondly most pile designs focus on limiting displacement. This conservative behaviour can lead to a considerable underestimation of the expected maximum load H_{ult} the pile can carry. For structures like seawalls for which displacements are not as important, this might lead to over-design. Finally cyclic loading has a loosening effect on the Winkler springs. This degradation has yet to be incorporated clearly in Eqn.27.

3.2.4 Ultimate Horizontal Load H_{ult} Estimation

API (2000) features Eqn.27 as a way to find soil pressures p . On the other hand it also includes a simple straightforward way to estimate ultimate lateral load H_{ult} for both short and long pile failure mechanisms. For brevity only the long pile mechanism load calculation is presented below as it was the one relevant for our pile tests.

To evaluate H_{ult} , API (2000) assumes the pile is failed when the first plastic hinge forms along the length of the pile. A second assumption is that above the plastic hinge the soil has mobilized its full lateral capacity p_u . API (2000) uses Eqn.26 for ultimate soil pressure. At the hinge point the shear force is zero and hence we can write force and moment equilibrium about the hinge (see Fig.20):

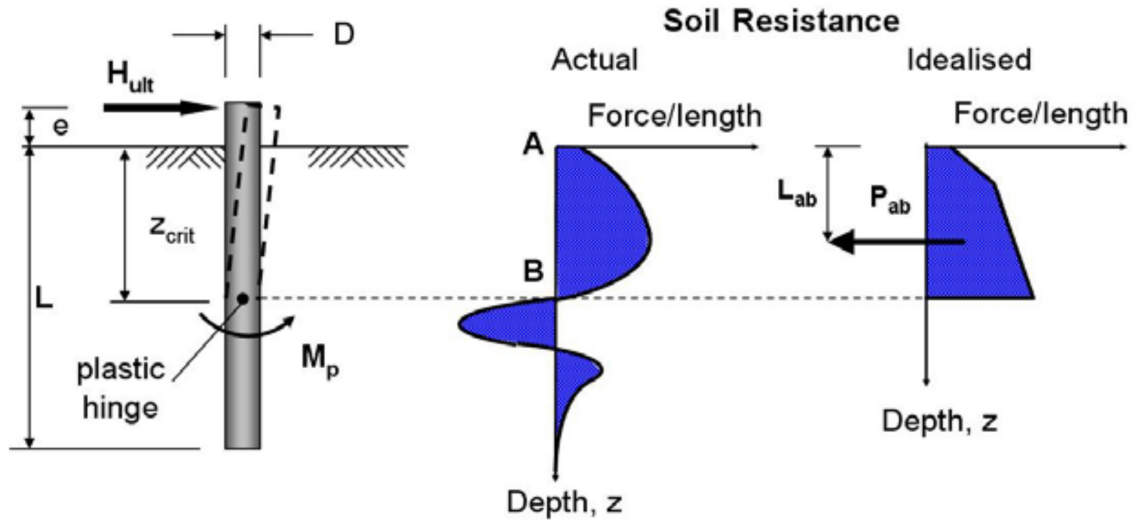


Figure 20: Lateral Failure Mechanism for Long Piles (from 4D5 Handout 8)

$$\begin{cases} H_{ult} = P_{ab} \\ M_p = P_{ab}(L_{ab} + e) \end{cases} \Rightarrow \begin{cases} H_{ult} = \frac{\gamma' K_p^2 D z_{crit}^2}{2} \\ M_p = H_{ult} \left(\frac{2z_{crit}}{3} + e \right) \end{cases} \quad (28)$$

Solving the equation system for z_{crit} we get:

$$z_{crit}^3 + \frac{3e}{2}z_{crit}^2 - \frac{3M_p}{\gamma'K_p^2D} = 0 \quad (29)$$

From Eqn.29 z_{crit} can be obtained and then substituted back in Eqn.28 to get H_{ult} .

3.3 Experimental Setup

Two full scale lateral pile tests were carried out in Japan in Summer 2015. Table 1 highlights the key properties of the two steel piles. From here on onwards we shall refer to the two piles as Pile 1 and Pile 2 as described in Table 1.

Table 1: Test Piles Key Data

	Diameter (m)	Section Thickness (mm)	Pile Length (m)	Embedment (m)	Yield Stress σ_y (MPa)	Ultimate Stress σ_{ts} (MPa)
Pile 1	1.0	22	15	10	315	490
Pile 2	1.0	12	15	10	235	400

3.3.1 Loading System

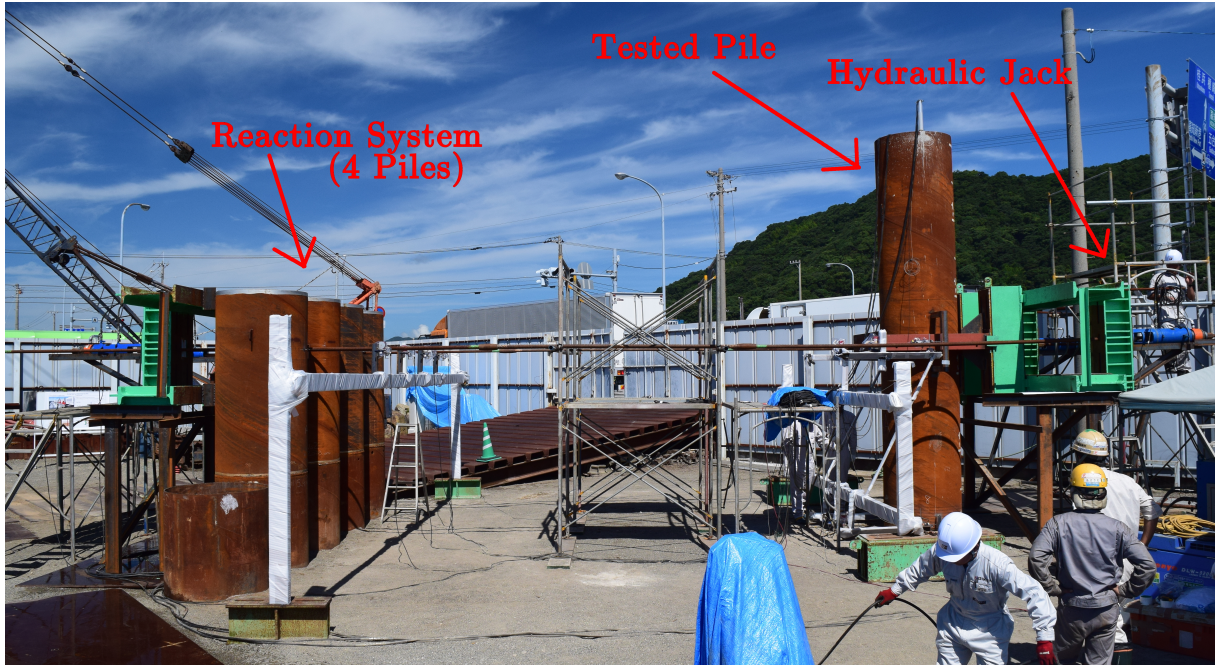


Figure 21: Loading System Configuration

The piles were loaded horizontally through a powerful hydraulic jack. To provide reaction a system of 4 steel piles was used. These had similar properties to the tested

3.3.2 Sensors. Monitoring

An assortment of sensors were used to monitor the pile response during testing: strain gauge pairs at 10 distinct sections, inclinometers at the same 10 sections and 2 LVDT's (linear variable displacement transducer) and 4 wire sensors to measure pile displacement. The displacement sensors were positioned at ground level and at the height the load was applied at. Fig.22 shows the pile in the vertical plane together with sensor positions and type.

3.4 Results & Analysis

3.4.1 Soil Analysis

To interpret our pile test results and compare them against code values we need to know the soil parameters at the test site first. Of interest are the effective unit weight of the sand γ' , its relative density I_D , the peak internal friction angle Φ'_{peak} and the critical state friction angle Φ'_{crit} .

To determine these parameters three SPTs (Standard Penetration Tests) were carried out in Japan at the site the piles were tested at. Furthermore soil from 1 m depth was extracted, sealed and brought back to the UK for further analysis. Two triaxial tests were carried out on this soil and agreement with SPT predictions was tested.

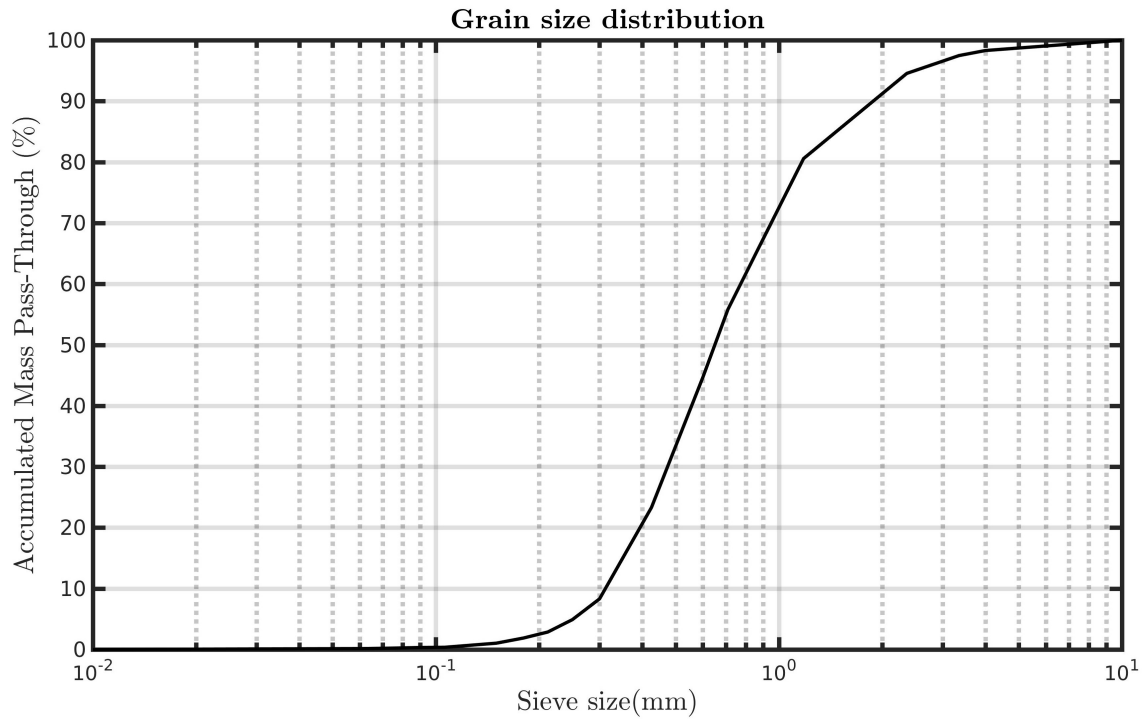


Figure 23

3.4.1.1 Grain Size Distribution The sand from Japan was put through gradation tests to check Particle Size Distribution (PSD). A traditional sieve analysis in the mechanical shaker as well as a modern automated sieving using Single Particle Optical Sizing (SPOS) were carried out. White (2003) includes a detailed description of the SPOS testing procedure. The results between the two sieve tests were fairly consistent, within acceptable deviation. Fig.23 shows the grain size distribution plotted from the results of the traditional sieve analysis.

The particle size distribution curve was used to grade the soil. This was done according to the ASTM-D2487 (2011) standard of soil classification. The result was a silty sand with 20% fines by mass bearing the SM designation.

3.4.1.2 SPT Data from three SPTs from the tests site was available for analysis. Fig.24 shows the SPT for the location nearest to where Pile 1 was installed. The test also classified the soil up to 4.8m deep as sand, while for greater depths it reported a mix of sand and gravel. The water table was located at 7.45m depth. The other two SPT Tests gave results similar to the one shown in Fig.24. Among the three tests N -values, stratigraphy and water table location agreed within a 10% error margin suggesting a uniform site as far as can be inferred from the SPT data.

From the SPT we wish to infer relevant soil parameters. To do so we shall use empirical correlations found in geotechnical literature which are commonly used in industry. However we first need to make an initial assumption on the value of γ' . It shall be taken to be 20 kN/m^3 which is a reasonable assumption for dense sand. The 4D5 Supplementary Databook gives the correlation between I_D and N -value:

$$I_D \approx \sqrt{\frac{N}{20 + 0.2\sigma'_v(\text{kPa})}} \quad (31)$$

The piles are embedded 10m into the ground hence it is reasonable to take the effective σ'_v at 5 m depth. Hence $\sigma'_v = \gamma' \times 5 = 100 \text{ kPa}$. Then:

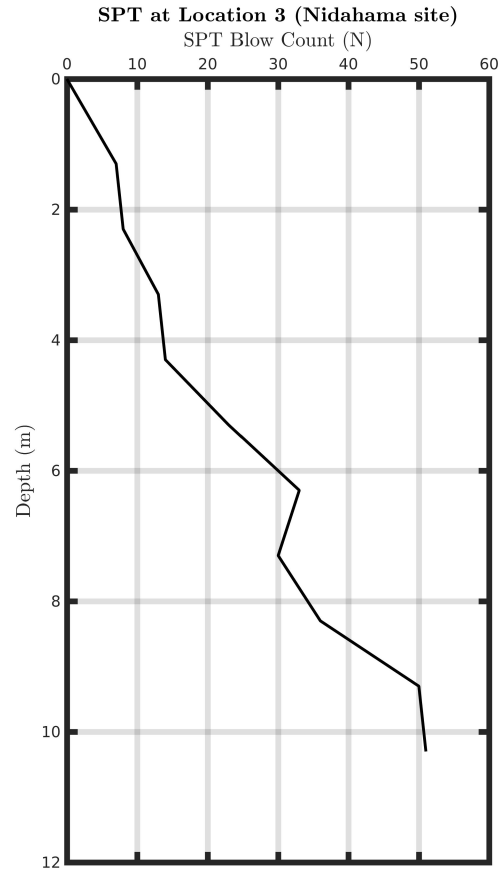


Figure 24

$$I_D \approx \sqrt{\frac{N(5m)}{20 + 0.2 \times 100}} = \sqrt{\frac{20}{40}} = 0.7 \quad (32)$$

Hatanaka and Uchida (1996) offer a way to estimate Φ'_{peak} from SPT data:

$$\Phi'_{peak} = \sqrt{20N} + 20 = \sqrt{20 \times 20} + 20 = 40^\circ \quad (33)$$

Then using the dilatancy formulae developed by Bolton (1987) we find $I_C = \ln(\sigma_c/\sigma'_v) = \ln(20000/100) = 5.3$ and hence $I_R = I_D I_C - 1 = 2.71$. We then compute the critical state friction angle as:

$$\Phi'_{crit} = \Phi'_{max} - 3 \times I_R \approx 32^\circ \quad (34)$$

A Φ'_{crit} of 32° is typical of uniform, rounded sand. The location of the test site was very close to the beach hence washed, rounded particles are a believable answer.

3.4.1.3 Triaxial Tests Before preparing samples for triaxial testing, preliminary measurements were done to determine sand water content w , as well as minimum and maximum voids ratios e_{min} , e_{max} . Since the soil sample was brought back from Japan in a hermetically sealed bag the water content at sample collection time could be determined by simple weighing and drying. The result was $w = 3\%$. e_{max} and e_{min} were found through standard procedures ASTM-D4254 (2015) and ASTM-D698 (2007) respectively. They were $e_{max} = 0.65$ and $e_{min} = 0.19$.



(a) Compacted sample in triaxial cell.
Notice thin yellow membrane



(b) Triaxial sample failure by bulging

Figure 25

Hence to replicate the relative density $I_D = 0.70$ derived from the SPT data a voids ratio of $e = 0.33$ would be required for the triaxial samples. Since the sand from Japan

presented very little cohesion it had to be compacted in the triaxial device itself (Fig.25a). When compacting care had to be exercised not to pierce the isolating membrane otherwise water could leak in and the test would be compromised. Given these restrictions the relative density of the two triaxial samples tested ended up about $I_D = 0.54$. Even so the triaxial tests enable us to measure Φ'_{crit} and check dilatancy results against Bolton (1987).

The two tests were successfully carried out. One of the samples was subjected to a confining stress $\sigma_{conf} = 100$ kPa while the other was confined by $\sigma_{conf} = 150$ kPa. Both samples failed by bulging (Fig.25b). Fig.26a shows the Stress-Strain curve for one of the tests. From the recorded data we found $\Phi'_{crit} = 34^\circ$ in both tests.

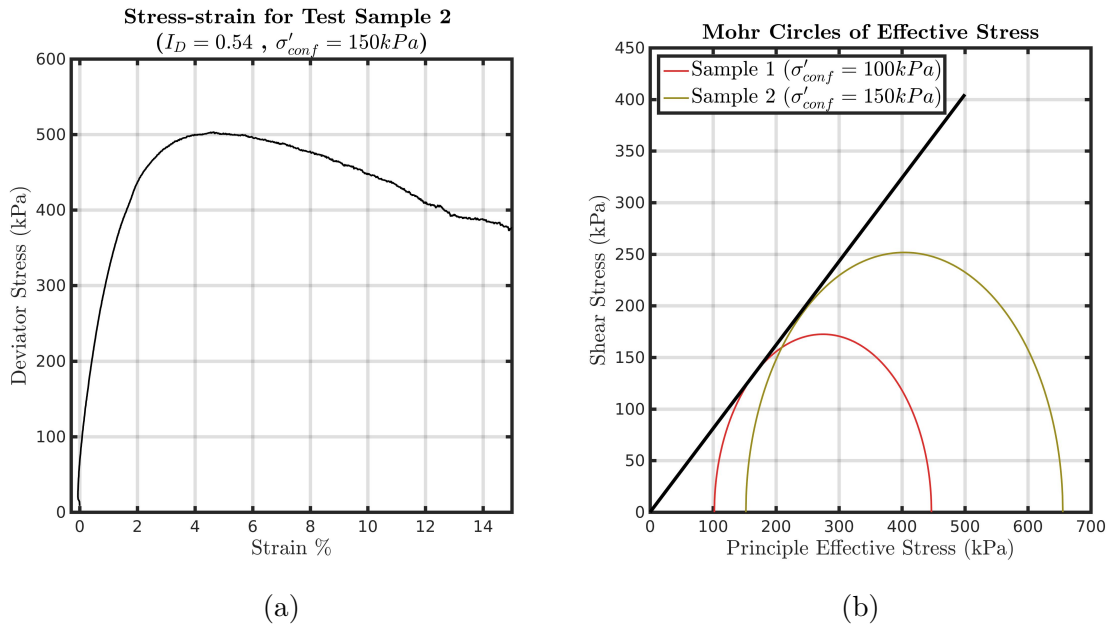


Figure 26

Firstly let us check the Φ'_{max} prediction from Bolton (1987). For the test shown in Fig.26a this is:

$$I_R = I_D I_C - 1 = 0.54 \times \ln\left(\frac{20000}{150}\right) - 1 = 1.64 \implies \Phi'_{max} = \Phi'_{crit} + 3 \times 1.64 = 38.9^\circ \quad (35)$$

The measured peak friction angle in the test was 38.8° which shows good agreement. Since the confining stress appears in the logarithmic term in Eqn.35 the prediction for the other test with 100 kPa confining stress is virtually the same, i.e. $\approx 39^\circ$. Fig.26b plots the Mohr circles corresponding to peak stress for both tests and it is visible Bolton (1987) predicts Φ'_{peak} very well.

The SPT results gave a predicted $\Phi'_{crit} = 32^\circ$ while the triaxial tests resulted in $\Phi'_{crit} = 34^\circ$ which is within reasonable tolerances. Staying conservative, for further calculations $\Phi'_{crit} = 32^\circ$. Other soil parameters will be taken as follows: $\Phi'_{peak} = 40^\circ$, $I_D = 0.7$, $\gamma' = 20$ kN/m³ above water table and $\gamma' = 12.4$ kN/m³ below it (see Section 3.4.1.2).

3.4.2 Pile Analysis (Before Yielding)

The two tested piles were subjected to a small number of unload reload cycles, each cycle with increasing maximum loading. Pile 1 was subjected to 2 cycles, the maximum force overall being 1836 kN, while Pile 2 resisted horizontal action over 6 cycles with an input force smaller or equal to 1314 kN. The recorded Force-Deflection responses are shown in Fig.27 for the two piles.

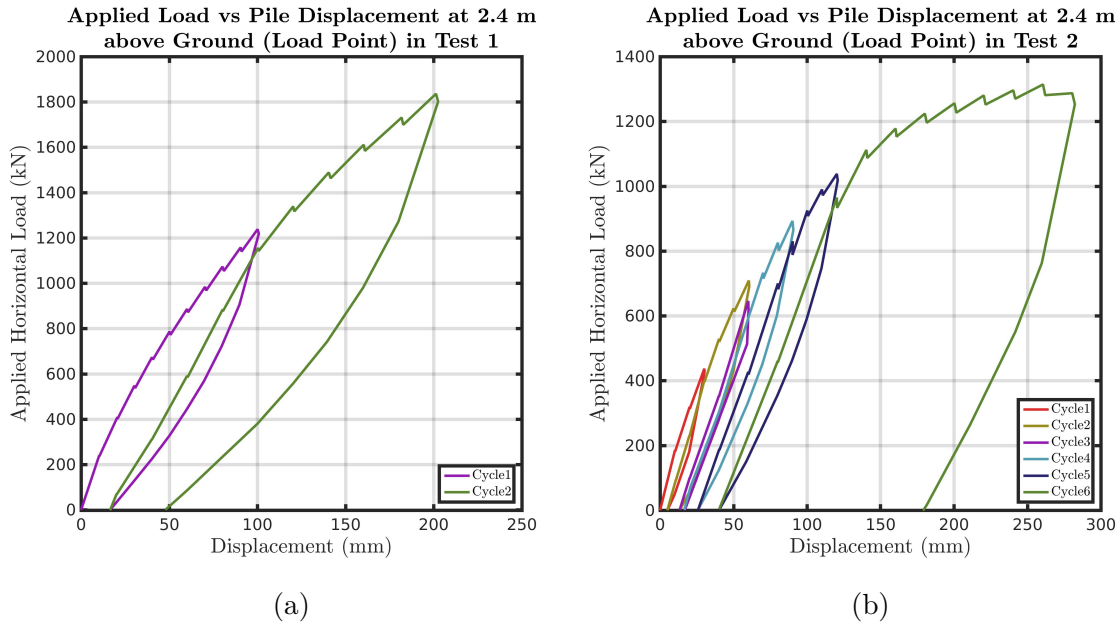


Figure 27

As shall be explained in Section 3.4.3, the analysis method depends on whether the pile had yielded or not. Thus we only discuss pile response up to yielding in this Section. Codes of practice generally limit design forces to the formation of the first plastic hinge so the range of loadings considered should still be relevant for the problem at hand.

Calibrated strain gauge and inclinometer data was used to obtain curvatures and rotations at the query points. Since we are investigating the elastic regime Eqn.17 holds and moments can be obtained from curvature points simply by multiplying with bending stiffness EI . To find the moment distribution along the pile $M(z)$ a 6th order polynomial was fitted through the data set. The resulting bending moment diagram can be seen in Fig28a.

The general shape of the fit looks good, with the polynomial approximating the data set well. The height the force was applied at by the jack (2.4 m) physically constitutes the point of zero moment above the ground. One can see in Fig.28a that the fits do not agree on the same jack position, instead there being a spread of zero moment points between fits. As shown by Eqn.17, differentiating moment twice yields soil pressures p . Differentiating our polynomial fit two times gives the pressure distribution shown in Fig.28b. The general shape matches well with soil pressures shown in other works like Broms (1964). However

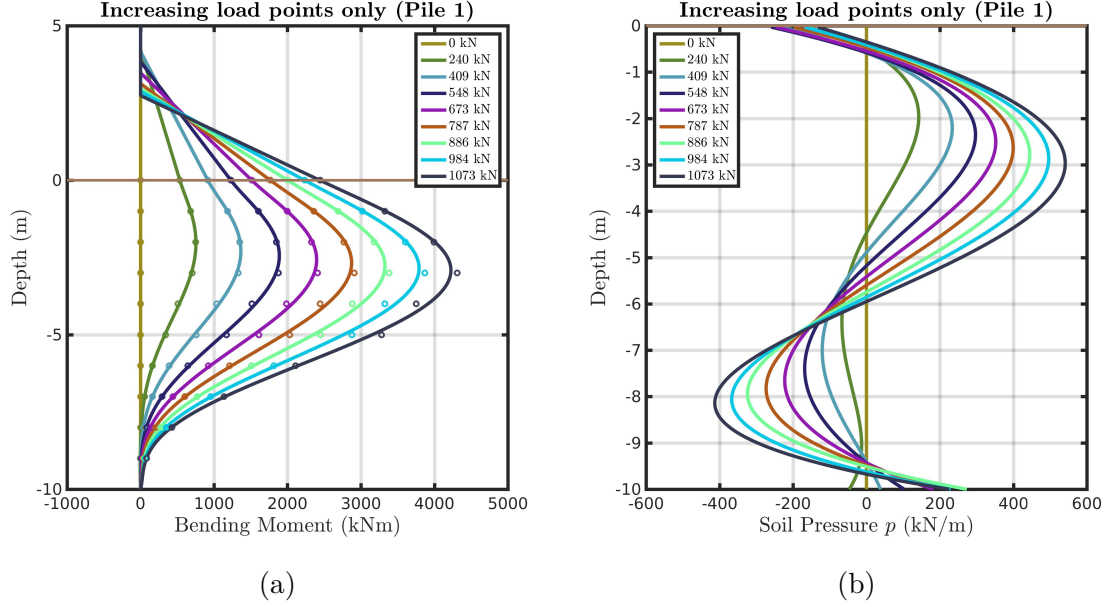


Figure 28

the plot presents several physical inconsistencies. Firstly it shows high negative pressures at the ground level. This is clearly untrue since completely unconfined soil has virtually zero horizontal capacity. Secondly it predicts a pressure sign reversal at the pile bottom. This would imply the bottom soil is acting with the applied force instead of resisting it which again is physically inconsistent.

As such the polynomial fitting gives promising results, but there are inconsistencies that need be addressed to be consistent with both experimental data and physical restraints.

3.4.2.1 Curve Fitting Currently there is no standardized methodology in geotechnical literature dealing with how to fit moment data to get consistently good predictions of earth pressures. In this section we shall present a method developed to solve this problem for the two pile tests, but which might be generalizable to other experiments as well.

Firstly we classify the conditions our polynomial fit has to obey into two categories: hard and soft restraints. Hard restraints are criteria our polynomial fit has to fulfil exactly. These are physical observations/measurements we have a very high confidence in, e.g. the fact that lateral capacity of surface soil should be zero or that the moment at the pile base should be zero. Soft restraints are conditions we want our fit to meet as well as possible, but not necessarily exactly. A good example would be test data which we want to fit as closely as possible, but also want to leave room for experimental errors.

Generally speaking let us assume we have m hard conditions on moment $M_1, M_2, \dots, M_m$ at depths $z_1, z_2, \dots, z_m$. We can write this in matrix form:

$$\mathbf{M}_h = [M_1 \ M_2 \ \dots \ M_m]^T \quad \mathbf{Z}_{\mathbf{M},h} = [z_1 \ z_2 \ \dots \ z_m]^T \quad (36)$$

We shall denote the number of elements in $\mathbf{M}_h$ by $\overline{\mathbf{M}}_h \equiv m$. Similarly to Eqn.36 we define the hard restraints on shear $\mathbf{S}_h$ and soil pressure $\mathbf{P}_h$ at locations $\mathbf{Z}_{\mathbf{S},h}$ and $\mathbf{Z}_{\mathbf{P},h}$ respectively. Soft restraints bear the same notations as hard conditions but with the index h substituted by s , i.e. $\mathbf{M}_s, \mathbf{S}_s$ and $\mathbf{P}_s$ at $\mathbf{Z}_{\mathbf{M},s}, \mathbf{Z}_{\mathbf{S},s}$ and $\mathbf{Z}_{\mathbf{P},s}$.

Let us assume we wish to fit a polynomial of order n through the above defined conditions. Firstly the order of the polynomial has to allow for all the hard conditions to be met, ie:

$$n \geq \overline{\mathbf{M}}_h + \overline{\mathbf{S}}_h + \overline{\mathbf{P}}_h - 1 \quad (37)$$

The general moment, shear and pressure equations for a polynomial fit are:

$$M = \sum_{k=0}^n a_k z^k \quad S = \sum_{k=0}^n k a_k z^{k-1} \quad P = \sum_{k=0}^n k(k-1) a_k z^{k-2} \quad (38)$$

We shall reserve $\overline{\mathbf{M}}_h + \overline{\mathbf{S}}_h + \overline{\mathbf{P}}_h$ coefficients from the polynomial to satisfy the hard conditions, leaving the rest ($\mathbf{A}_v$) to satisfy the soft restraints:

$$\mathbf{A}_M = [a_0 \ a_1 \ \dots \ a_{\overline{\mathbf{M}}_h-1}]^T \quad (39)$$

$$\mathbf{A}_S = [a_{\overline{\mathbf{M}}_h} \ a_{\overline{\mathbf{M}}_h+1} \ \dots \ a_{\overline{\mathbf{M}}_h+\overline{\mathbf{S}}_h-1}]^T \quad (40)$$

$$\mathbf{A}_P = [a_{\overline{\mathbf{M}}_h+\overline{\mathbf{S}}_h} \ a_{\overline{\mathbf{M}}_h+\overline{\mathbf{S}}_h+1} \ \dots \ a_{\overline{\mathbf{M}}_h+\overline{\mathbf{S}}_h+\overline{\mathbf{P}}_h-1}]^T \quad (41)$$

$$\mathbf{A}_v = [a_{\overline{\mathbf{M}}_h+\overline{\mathbf{S}}_h+\overline{\mathbf{P}}_h} \ a_{\overline{\mathbf{M}}_h+\overline{\mathbf{S}}_h+\overline{\mathbf{P}}_h+1} \ \dots \ a_n]^T \quad (42)$$

To ease reading the following notations shall be used:

$$\alpha = \overline{\mathbf{M}}_h - 1 \quad \beta = \overline{\mathbf{M}}_h + \overline{\mathbf{S}}_h - 1 \quad \gamma = \overline{\mathbf{M}}_h + \overline{\mathbf{S}}_h + \overline{\mathbf{P}}_h - 1 \quad (43)$$

Eqn.38 needs to be written for all heights at which there are either hard or soft conditions. To manage and group the large number of equations the following $\square$ operators are introduced, acting on the $\mathbf{Z}$ vectors:

$${}_a\Box_b^M \mathbf{Z} = \begin{bmatrix} \vdots \\ z_k^a & z_k^{a+1} & \dots & z_k^b \\ \vdots \end{bmatrix} \quad (44)$$

$${}_a\Box_b^S \mathbf{Z} = \begin{bmatrix} \vdots \\ az_k^{a-1} & (a+1)z_k^a & \dots & bz_k^{b-1} \\ \vdots \end{bmatrix} \quad (45)$$

$${}_a\Box_b^P \mathbf{Z} = \begin{bmatrix} \vdots \\ a(a-1)z_k^{a-2} & (a+1)az_k^{a-1} & \dots & b(b-1)z_k^{b-2} \\ \vdots \end{bmatrix} \quad (46)$$

Using these operators the hard restraints on moment $\mathbf{M}_h$ can be expressed as follows:

$${}_0\Box_\alpha^M \mathbf{Z}_{M,h} \mathbf{A}_M + {}_{(\alpha+1)}\Box_\beta^M \mathbf{Z}_{M,h} \mathbf{A}_S + {}_{(\beta+1)}\Box_\gamma^M \mathbf{Z}_{M,h} \mathbf{A}_P + {}_{(\gamma+1)}\Box_n^M \mathbf{Z}_{M,h} \mathbf{A}_V = \mathbf{M}_h \quad (47)$$

Equations similar to Eqn.47 can be written for the shear and soil pressure hard conditions too. Hence we have three distinct equations with 4 unknowns $\mathbf{A}_M$, $\mathbf{A}_S$, $\mathbf{A}_P$ and $\mathbf{A}_V$. We can assemble these in the following manner:

$$\overbrace{\begin{bmatrix} {}_0\Box_\alpha^M \mathbf{Z}_{M,h} & {}_{(\alpha+1)}\Box_\beta^M \mathbf{Z}_{M,h} & {}_{(\beta+1)}\Box_\gamma^M \mathbf{Z}_{M,h} \\ {}_0\Box_\alpha^S \mathbf{Z}_{S,h} & {}_{(\alpha+1)}\Box_\beta^S \mathbf{Z}_{S,h} & {}_{(\beta+1)}\Box_\gamma^S \mathbf{Z}_{S,h} \\ {}_0\Box_\alpha^P \mathbf{Z}_{P,h} & {}_{(\alpha+1)}\Box_\beta^P \mathbf{Z}_{P,h} & {}_{(\beta+1)}\Box_\gamma^P \mathbf{Z}_{P,h} \end{bmatrix}}^{\mathbf{D}_h} \overbrace{\begin{bmatrix} \mathbf{A}_M \\ \mathbf{A}_S \\ \mathbf{A}_P \end{bmatrix}}^{\mathbf{A}_t} = \overbrace{\begin{bmatrix} \mathbf{M}_h \\ \mathbf{S}_h \\ \mathbf{P}_h \end{bmatrix}}^{\mathbf{T}_h} - \overbrace{\begin{bmatrix} {}_{(\gamma+1)}\Box_n^M \mathbf{Z}_{M,h} \\ {}_{(\gamma+1)}\Box_n^S \mathbf{Z}_{S,h} \\ {}_{(\gamma+1)}\Box_n^P \mathbf{Z}_{P,h} \end{bmatrix}}^{\mathbf{E}_h} \mathbf{A}_V \quad (48)$$

$\mathbf{D}_h$ is a square matrix of dimensions $(\gamma+1) \times (\gamma+1)$. Hence we can express the hard coefficients in terms of the soft ones, provided $\mathbf{D}_h$ is not singular. This should be the case if the hard conditions in moment, shear and pressure are independent of each other.

$$\mathbf{A}_t = \mathbf{D}_h^{-1} \mathbf{T}_h - \mathbf{D}_h^{-1} \mathbf{E}_h \mathbf{A}_V \quad (49)$$

For the soft conditions we can write an equation similar to Eqn.48:

$$\mathbf{D}_s \mathbf{A}_t = \mathbf{T}_s - \mathbf{E}_s \mathbf{A}_V \quad (50)$$

However in Eqn.50 $\mathbf{D}_s$ is not a square matrix and cannot be inverted. Substituting the result for $\mathbf{A}_t$ from Eqn.49 into Eqn.50 and rearranging terms we obtain:

$$(\mathbf{E}_s - \mathbf{D}_s \mathbf{D}_h^{-1} \mathbf{E}_h) \mathbf{A}_v = \mathbf{T}_s - \mathbf{D}_s \mathbf{D}_h^{-1} \mathbf{T}_h \quad (51)$$

which can now be solved for $\mathbf{A}_v$ through QR decomposition. Substituting $\mathbf{A}_v$ back into Eqn.49 we obtain $\mathbf{A}_t$ and thus find all the coefficients of the sought n^{th} order polynomial.

Using this curve fitting procedure new polynomial fits were generated for the two pile tests. zero moment at the base of the piles, zero p at the ground surface, zero shear at the base of the pile and shear at the ground level equal to the force applied by the jack were the hard conditions chosen for the fit. The bending moment data at the query points formed the soft restraints. The polynomial order was taken to be 6. Figs.29 & 30 show the new predicted earth pressures next to the initial ones.

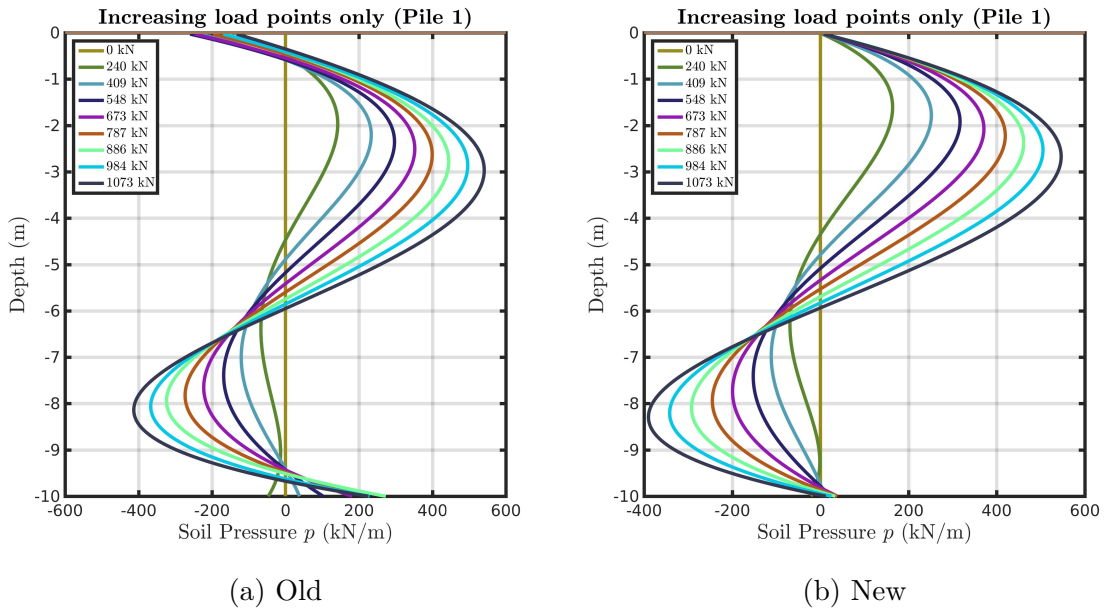


Figure 29

The new fits are a noticeable improvement from the initial ones. They correct the issues raised in relation to the first fits while maintaining the same general shape which matches the findings of Broms (1964) and Matlock and Reese (1960). Furthermore close to the ground surface we expect the soil to have reached its ultimate capacity. In uniform soil like the one found in our test site Eqn.19 predicts this condition manifests as an initial gradient on the p -depth curves which only depends on soil properties and not on the pile tested or loading profile. This feature can be spotted in our plots and is more apparent in Fig.29b. One can notice that soil pressures corresponding to high load values start off with the same gradient as curves from lower load cases. Measuring this gradient from the two graphs in Figs.29b and 30b yields approximately the same value $n \approx 400$ kPa. The agreement with the predictions of Eqn.19 is another argument in favour of the improved fits.

The new bending moment diagram can be seen in Fig.31a. The moment plots converge

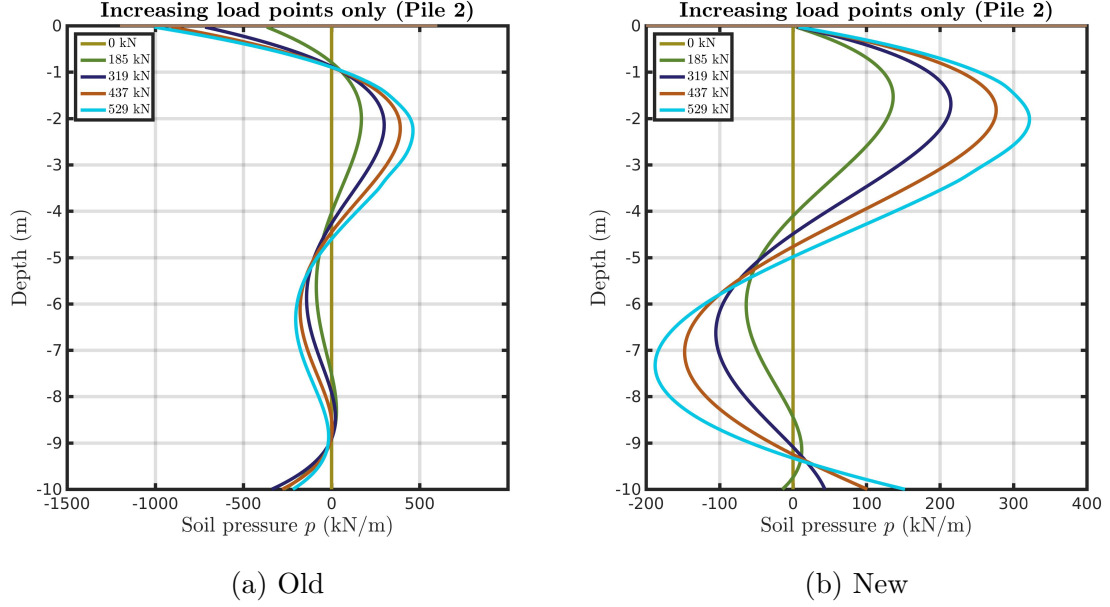


Figure 30

better now towards a single nul point above ground surface (the jack position).The pile deflection, obtained through integration of rotation data from the fitted inclinometers, is plotted in Fig.31b. The results are in good agreement with the LVDT data shown as circular markers on the plot.

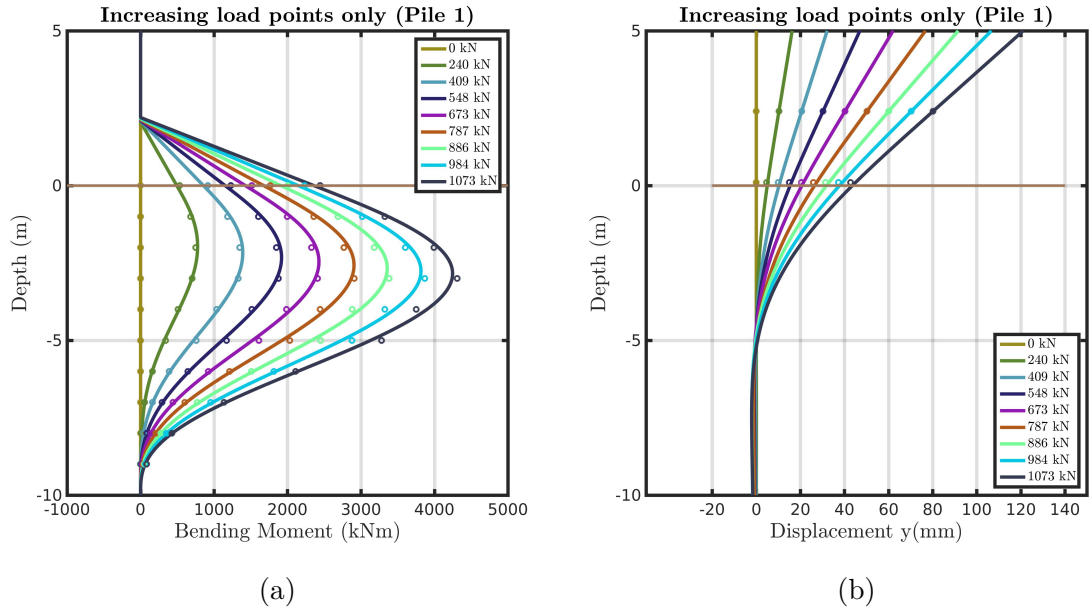


Figure 31

3.4.2.2 Comparison with Codes of Practice Using the soil parameters determined in Section 3.4.1, ultimate soil resistance estimates were computed through Eqn.20-26 in Section 3.2.2. Fig.32 shows the predictions plotted on top of the pressure results from the

two tests.

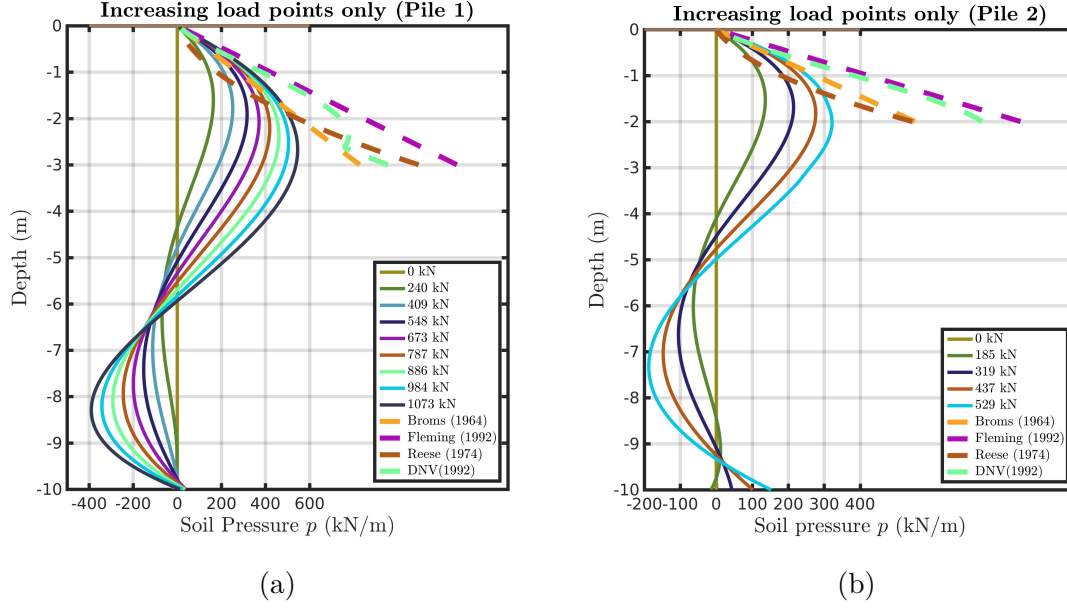


Figure 32

Both DNV (1992) and Fleming et al. (1992) predict the ultimate soil pressures well. Broms (1964) somewhat under-predicts the gradient, but is, however, a safe result. Reese et al. (1974), with the wedge failure mechanism equation, is the furthest away from measured results. Interestingly, however, DNV (1992) is directly based on Reese et al. (1974), but amplifies the result by a factor A defined in Eqn.25 which for our test case is ≈ 2 . Thus it might be that the wedge mechanism under-predictions were observed in other tests too and hence the codes correct for it.

Combining the pressure and deflection data sets we can plot $p - y$ curves at any depth along the pile and check these curves against the DNV (1992) predictions (Eqn.27). Results are plotted in Fig.33.

Fig.33a shows the $p - y$ plot at shallow depths. DNV (1992) seems to predict higher stiffness and ultimate capacity than those from our test data. This was observed for both piles at shallow depths. At higher depths the soil does not move enough to reach ultimate capacity, however we can still compare stiffness results. Fig.33b is a high depth test and the same observation holds: the codes overpredict stiffness. Recent centrifuge tests by Kirkwood (2015) found the same stiffness overprediction. Kirkwood (2015) argues that the effect of pile stiffness on the $p - y$ curves is only significant for rigid piles. Since the codes were developed on test data from flexible pile tests, it might be that existing formulae need tweaking to cover rigid piles accurately. Klinkvort (2012) conducted centrifuge tests on rigid piles under monotonic loadings. The same overprediction was found and a different equation for the parameter A was proposed:

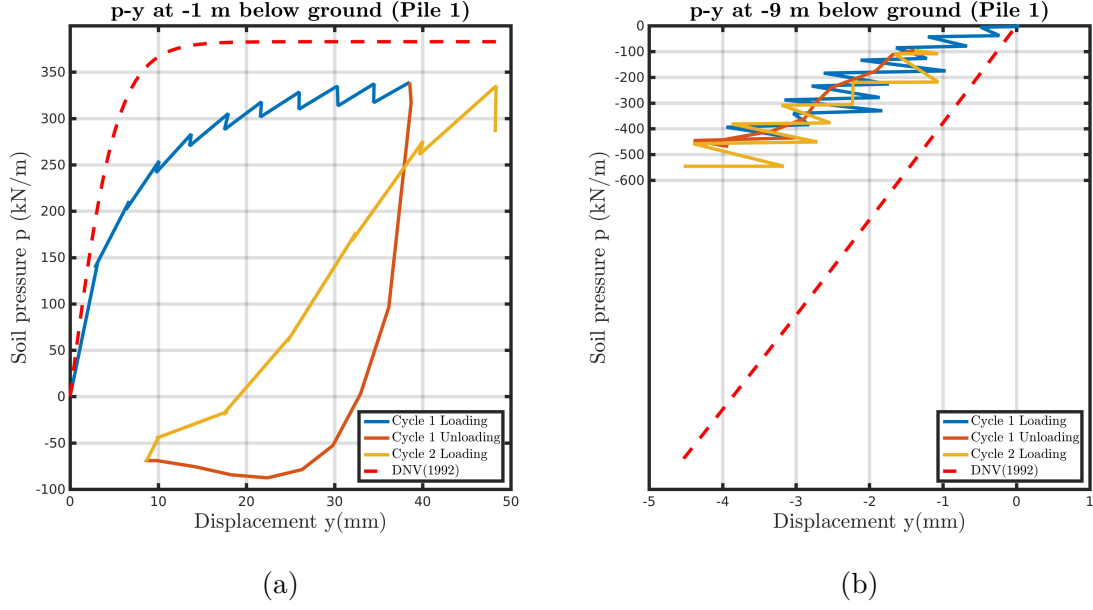


Figure 33

$$A = 0.9 + 1.1 \times \frac{1 + \tanh\left(9 - 3\frac{z}{D}\right)}{2} \quad (52)$$

Fig.34 shows the results of Klinkvort (2012) against our own data sets.

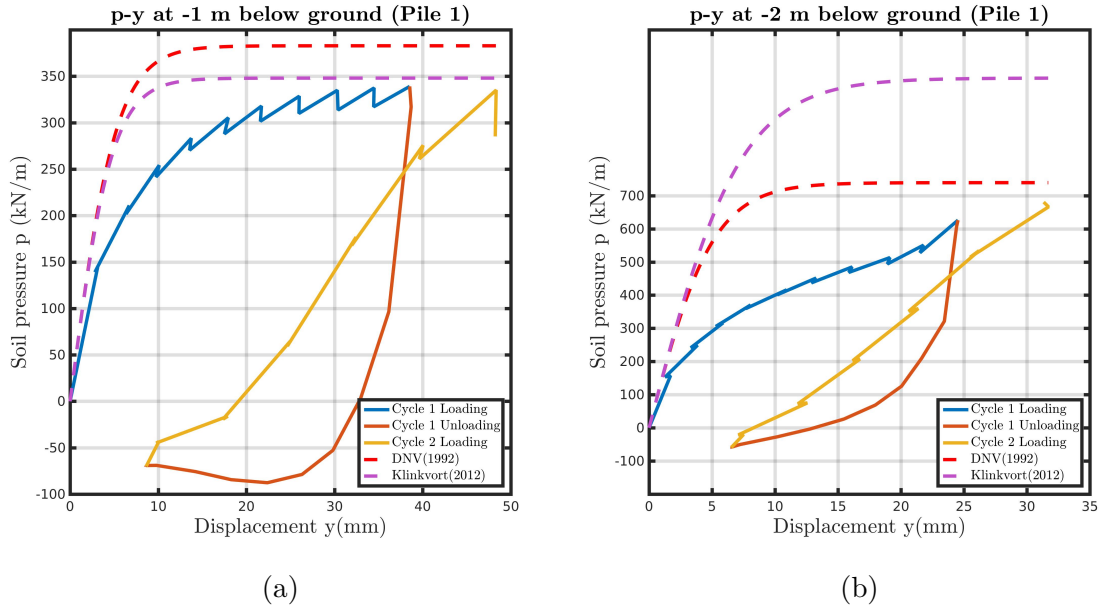


Figure 34

Even though the Klinkvort (2012) coefficient change gave better prediction of ultimate behaviour at 1m depth, the stiffness results are identical to DNV (1992). The predicted lateral soil capacity at 2m depth is further from test data than DNV (1992).

In conclusion there is an increasing amount of research showing current codes over-

predict the $p - y$ response for rigid piles. The same was observed in our tests, however good new formulae to account for the discrepancies has yet to emerge.

3.4.3 Plastic Regime

The analysis thus far has only included load levels for which the entire pile remained elastic. However the tests in Japan continued further into the plastic regime. Since the walls are meant to resist tsunamis, deflection constraints are not an issue and it would be beneficial to know how much extra capacity is in the pile plastic deformation regime.

3.4.3.1 Material Law From the curvature data we cannot transition to moment the same way as before since Eqn.17 does not hold any more.

Hence an elasto-plastic material law is required. Knowing steel stiffness E as well as yield stress of pile steel σ_y an elastic-perfectly plastic model could be used. But since such a model features a gradient discontinuity at σ_y this results in abrupt soil pressure changes with depth. To ensure a smooth transition between the elastic and plastic regimes a transition fit curve was sought that, at its ends, fitted the elastic law as well as the plastic in value, first and second derivative. This proved to be a non-trivial task. However noting that the function $\tan(x) - x$ at $x = 0$ is 0 in value, first and second derivative by appropriate transformations the following fit curve was found:

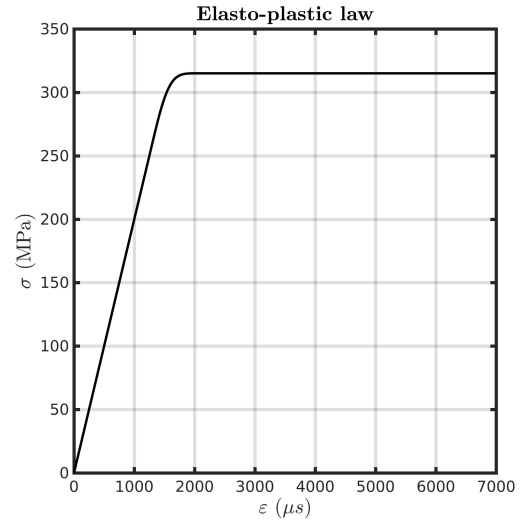


Figure 35

$$\begin{aligned}
\sigma(\varepsilon) = & \sigma_y + \frac{E\varepsilon_i - \sigma_y}{-\tan \alpha + \alpha - \frac{\alpha^4}{2}} \left\{ -\tan \left[\frac{\alpha}{\varepsilon_i - \varepsilon_f} (\varepsilon - \varepsilon_f) \right] + \frac{\alpha}{\varepsilon_i - \varepsilon_f} (\varepsilon - \varepsilon_f) \right. \\
& \left. - \frac{1}{2} \times \left[\frac{\alpha}{\varepsilon_i - \varepsilon_f} (\varepsilon - \varepsilon_f) \right]^4 \right\} + \left[E - \frac{E\varepsilon_i - \sigma_y}{-\tan \alpha + \alpha - \frac{\alpha^4}{2}} \times \frac{\alpha}{\varepsilon_i - \varepsilon_f} \right. \\
& \times \left(-\frac{1}{\cos \alpha^2} + 1 - 2\alpha^3 \right) \times \frac{\varepsilon_f - \varepsilon_i}{2\pi} \times \left\{ \sin \left[\frac{\pi}{\varepsilon_f - \varepsilon_i} (\varepsilon - \varepsilon_i) \right] \right. \\
& \left. \left. + \frac{1}{2} \sin \left[\frac{2\pi}{\varepsilon_f - \varepsilon_i} (\varepsilon - \varepsilon_i) \right] \right\} \right\} \quad (53)
\end{aligned}$$

where ε is in microstrain, Young's modulus E is in MPa and $\varepsilon_i, \varepsilon_f$ are the begin and

end strain of the transition curve. α is the middle root of:

$$2 \frac{\sin x}{\cos x^3} + 6x^2 = 0 \quad (54)$$

The resulting elasto-plastic law can be seen in Fig.35.

3.4.3.2 Soil Pressures. Discussion With the elasto-plastic law in Eqn.53, curvature readings can be translated to bending moment for all load tests. Curve fitting was done in curvature space and then through the material law converted to moment results. Fig.36 shows the full bending moment diagram for Pile 1.

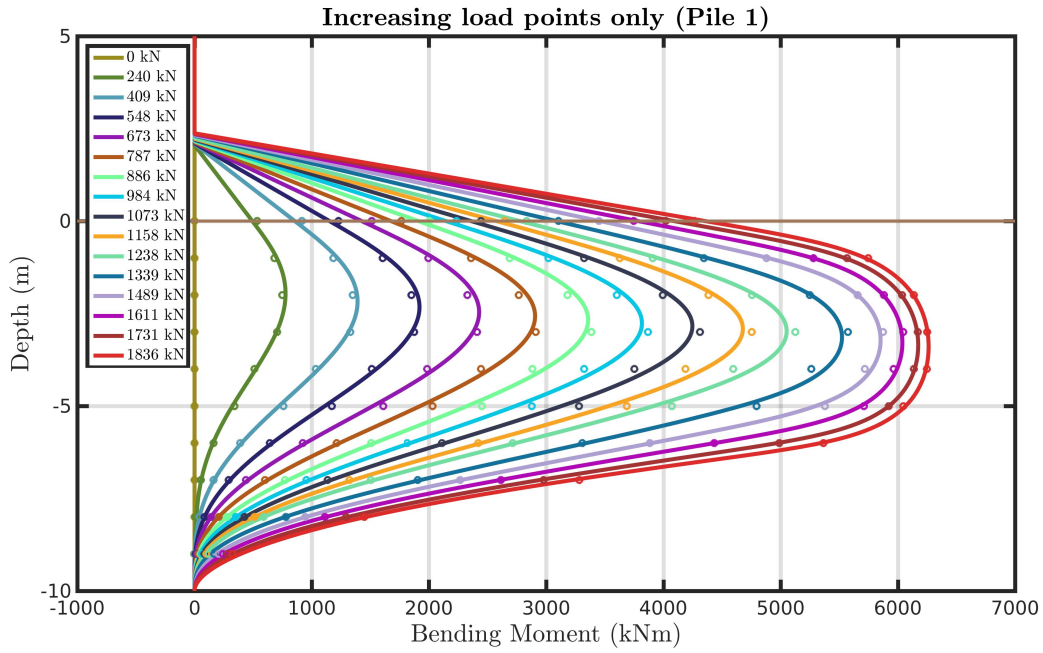


Figure 36

Differentiating twice we get the predicted earth pressure distribution (Fig.37a). Even when using a very smooth material law the pressures become unrealistic as the pile moves into the plastic regime. The large drop in soil resistance seen between -1 and -6 m is mathematically explained by the bending moment shape since soil pressure is the curvature of the moment plot. When the pile goes plastic, it cannot rotate freely about the first plastic hinge because of the soil confinement. Hence load is transferred to nearby sections and a plastic region develops rather than a single hinge. Inside this region the moments are close to the plastic capacity M_p and hence the curvature of the bending moment plot is small (the plastic region tends to a flat plateau). This leads to the drop in pressure seen in Fig.37a. At the limit moment would be constant in the plastic region ($= M_p$) hence no increase in shear force can be present and then $p = 0$. However physically we do not expect the soil pressure profile to change shape inside the plastic region. p is a function

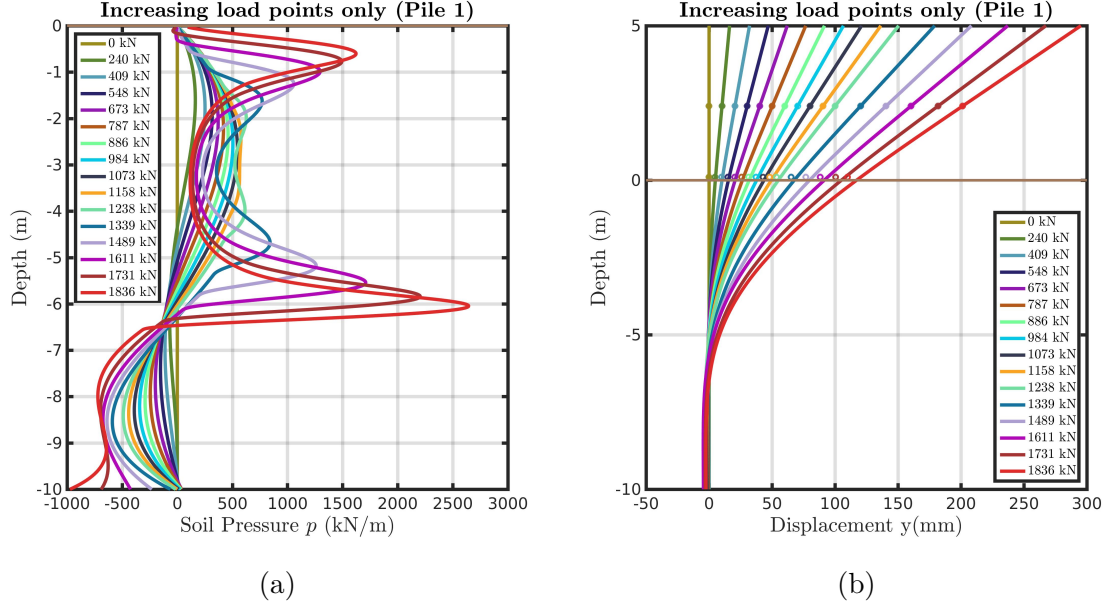


Figure 37

of pile displacement y and as can be seen in Fig.37b there is no visible change in the y profile as the pile transitions to the plastic regime.

Because section moment cannot increase past M_p , it might be that other forces come into play to provide further capacity. Hence we might need to add skin friction forces to Fig.17 and other possible components for an accurate representation of equilibrium in the plastic regime. This changes the relationship between section moment M and p and explains why differentiating M twice no longer leads to good soil pressure profiles.

Currently there is little research on the behaviour of piles in the plastic regime so the above statements cannot be verified against existing work. Further research could delve into the topic since there is considerable capacity past the yield point.

3.4.3.3 Comparison with Codes of Practice Even though $p - y$ curves could not be derived in the plastic region, we can still compare our force-deflection response against the ultimate horizontal load prediction H_{ult} detailed in Section 3.2.4. The H_{ult} values are plotted on top of measured force and deflection data in Fig.38. Also using Eqn.18 the $p - y$ DNV (1992) equation can be integrated along the pile length to give force-deflection predictions for load values up to the formation of the first plastic hinge. These predictions are plotted in Fig.38 as well.

As can be seen in both plots DNV (1992) predicts the pile stiffness in the elastic regime very well. Even though issues concerning the DNV (1992) $p - y$ results were raised in Section 3.4.2.2 the fact that there are four integration steps between soil pressures and displacement means some irregularities are potentially smoothed out. The H_{ult} prediction, although derived through a different approach than the $p - y$ method gives similar results. They both estimate the hinge formation load fairly well.

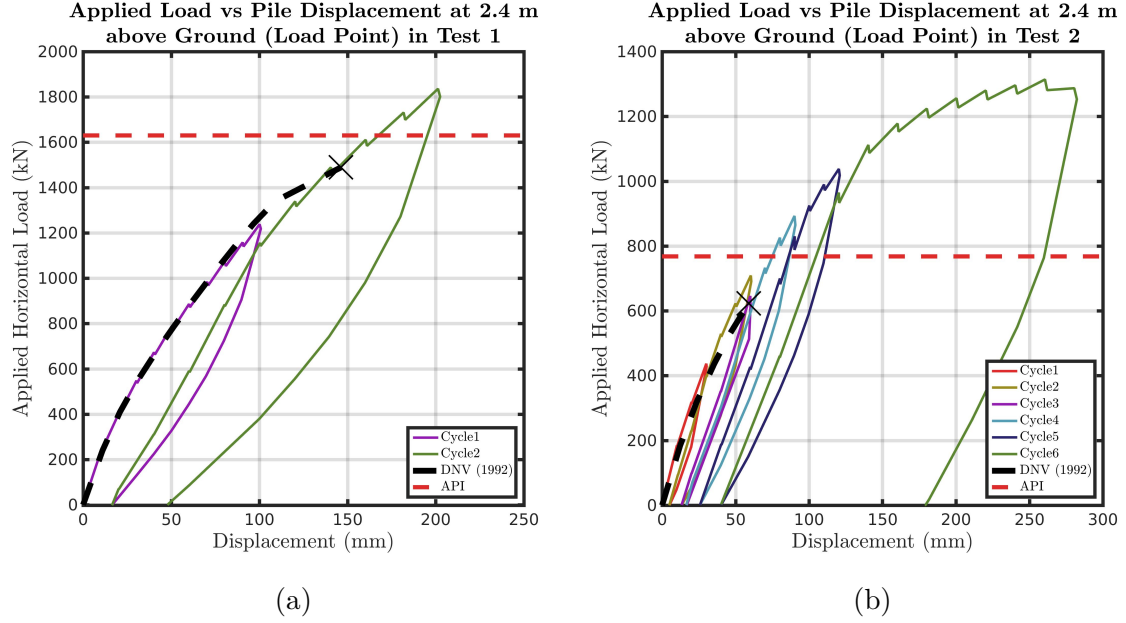


Figure 38

Pile 2 was tested up to its maximum lateral capacity. As can be seen in Fig.38b, the ultimate lateral capacity is roughly twice the code estimates. This confirms the earlier statement of large capacity reserves in the plastic regime.

3.5 Test Conclusions

The two pile tests were very consistent between themselves, observations inferred from one pile usually applying for the other as well. They showed the current $p - y$ code formulae overpredict stiffness and lateral capacity which is in line with what other research has also concluded. However the codes predict the force-deflection shape very well in the elastic regime. The ultimate pile lateral capacity is under-predicted by the same standards by about a factor of 2. Since most codes were developed with serviceability concerns in mind, it is likely their results willingly stay away from the plastic region to ensure large deformations do not occur. However for seawalls displacement is not a concern and there would be benefit to be had in increasing H_{ult} predictions into the plastic regime.

4 Pile Performance as Tsunami Defence

4.1 Goals

Through the Tsunami Simulator and Pile Tests, the Fluids, Dynamic and Geotechnical Problems (as defined in Section 1.2.2) were analysed and the questions raised in Section 1.2.2 tackled. In this section I shall try and bring the results from the two tests together to provide a safe estimation of the highest tsunami our two test piles could carry.

4.2 Pile Frequency Estimation

As detailed in Section 2.4.8 we expect dynamic amplification to occur due to the wave loading. This is dependent on frequency and hence we need suitable estimates for the natural frequencies of the pile-soil systems.

In this analysis we shall assume a simple 1D model in which pile stiffness, damping and mass are represented by the scalars m , k and ζ . Then natural frequency f is:

$$f = \frac{1}{2\pi} \sqrt{\frac{k}{m}} \quad (55)$$

The mass/unit width is known for the 2 piles from their catalogues and hence pile mass m_{pile} can be found by multiplying by the pile height (15 m):

$$m_{pile,1} = 531 \frac{\text{kg}}{\text{m}} \times 15\text{m} \approx 8.0 \text{ tonnes} \quad m_{pile,2} = 292 \frac{\text{kg}}{\text{m}} \times 15\text{m} \approx 4.4 \text{ tonnes} \quad (56)$$

The mass of the soil participating in the vibrations is unknown. However by taking $m = m_{pile}$ we will find an upper boundary for the system natural frequency.

The system stiffness k was estimated by the initial gradient of the force - displacement plots in Fig.27, $k_1 = 16 \text{ MN/m}$ and $k_2 = 14.5 \text{ MN/m}$. These estimates do not take subsequent softening due to increasing load/ load cycles into account. Together with the assumption on mass this should result in frequency estimates that are upper bounds for the true system's natural frequency. The estimates are:

$$f_1 = 7 \text{ Hz} \quad f_2 = 9 \text{ Hz} \quad (57)$$

Damping shall be taken at 5%, i.e. $\zeta = 0.05$. This value is commonly used in Earthquake Engineering for civil structures, and is probably a lower bound for the pile-soil system.

4.3 Dynamic Amplification Factor

The Dynamic Amplification Factor (DAF) was computed for each stage through the procedure described in Section 2.4.8. Figs.39 and 40 show the normalised pile dynamic response to a wave twice as high as itself.

The response plots are similar between the two piles. However one can notice the significant amplification in dynamic force F_{dyn} : 1.4 times the wave surge force for Pile 1 and 1.3 for Pile 2.

Figs.39 and 40 only plot the response spectra for a single test. Repeating the procedure for all wave flume experiment we get a cloud of DAF data. Fig.41 plots this data as a

Normalised Response of Pile-Soil System with $f = 7\text{Hz}$ and $\zeta = 0.05$ to Wave Loading in Test 34 ($h_{\text{wave}}/h_{\text{wall}} = 1.95$)

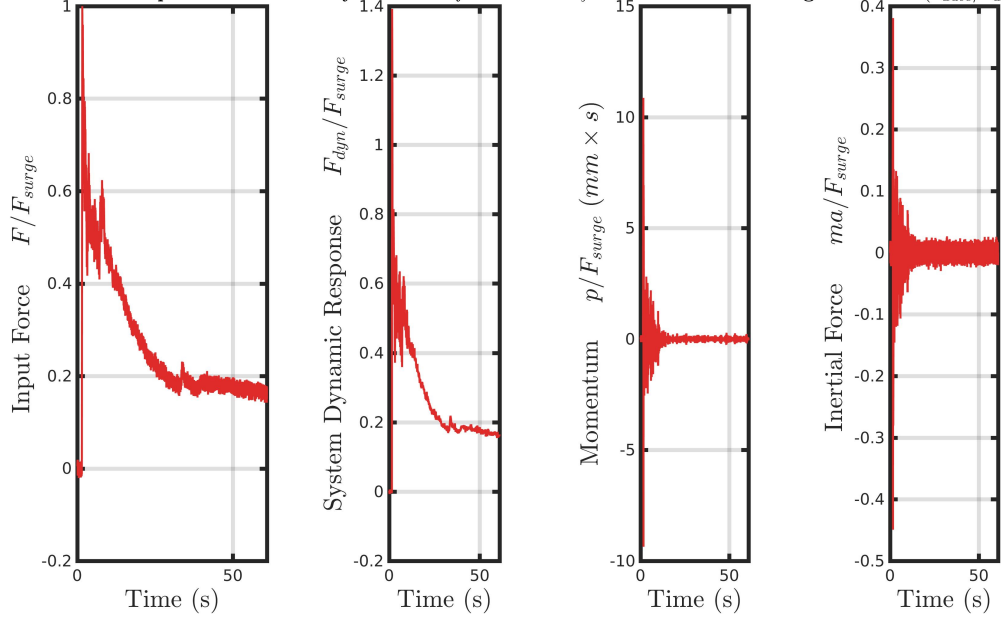


Figure 39

Normalised Response of Pile-Soil System with $f = 9\text{Hz}$ and $\zeta = 0.05$ to Wave Loading in Test 34 ($h_{\text{wave}}/h_{\text{wall}} = 1.95$)

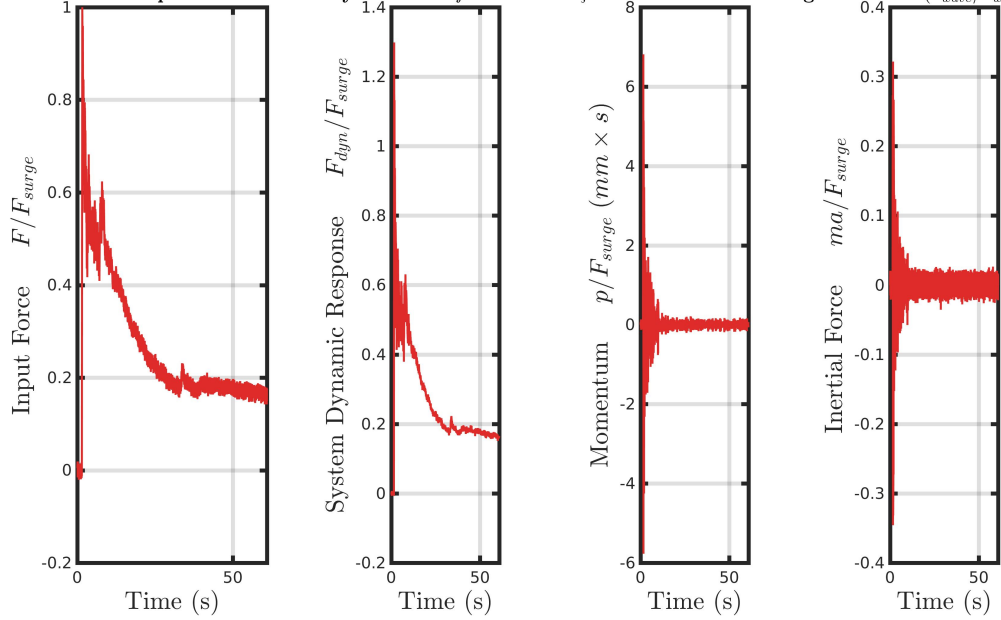


Figure 40

function of normalized wave height for the 2 test piles. The horizontal violet line represents the mean of the DAF results across all wave flume tests. A red line is shown at $DAF=1$. All tests above it displayed dynamic amplification, while those below recorded an attenuated response.

The average DAF for both piles was about 1.1 which is close to no amplification. On the other hand all points below the $DAF=1$ line come from waves smaller than the pile height. Larger waves caused amplification only and if we consider their data exclusively the average increased to ≈ 1.2 .

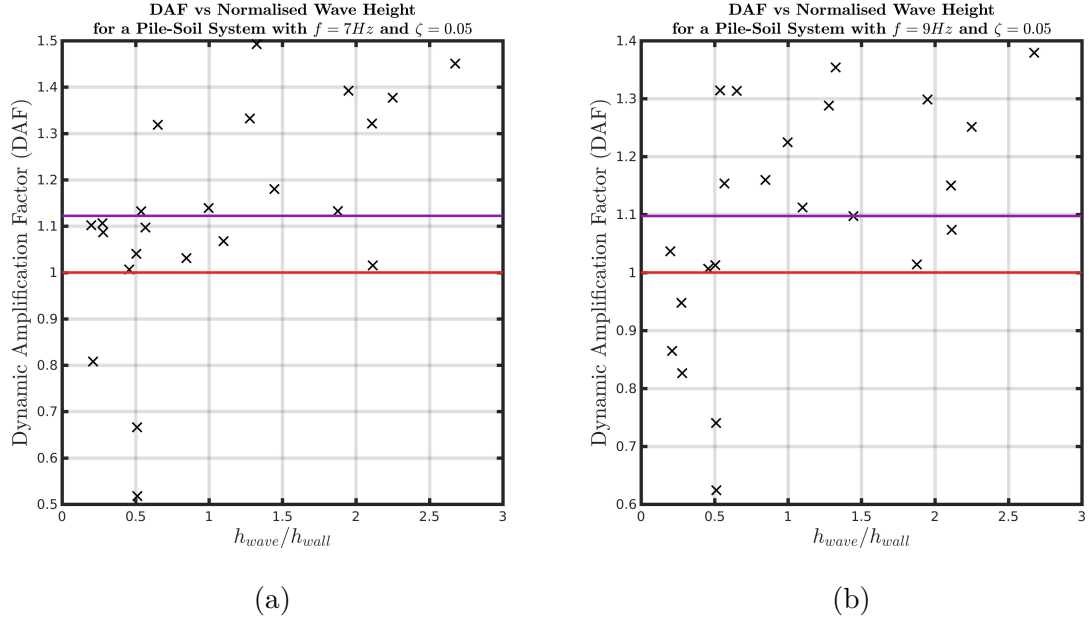


Figure 41

4.4 Overall Safety Factor Estimate

Aggregating data from the Fluids, Dynamics and Geotechnical problems(Section 1.2.2) we shall estimate the safety factor for each problem and then bring the results together into an overall factor of safety. These factors shall be calculated for the maximum force the pile is subjected to in a tsunami F_{wave} and shall be denoted by γ_f, γ_d and γ_g respectively.

4.4.1 Fluids

The Japanese standard uses the $3 \times h_{wave}$ approach to estimate the maximum wave surge force (Section 2.2.1). Our tsunami simulator experiments have shown that the factor of safety (FoS) in this approach is close to unity, the method giving an accurate prediction for the surge force. As discussed in Section 2.4.6 the FoS for the fluids problem will be taken as $\gamma_f = 1.05$.

4.4.2 Dynamics

Section 2.2.2 mentions the codes recommend using the maximum surge force F_s as found through the $3 \times h_{wave}$ approach (Okada et al., 2006) as the equivalent static maximum load on the piles. Our dynamic analysis (Section 4.3) showed there is dynamic amplification present and an average amplification factor was given as DAF=1.1 . Hence $\gamma_d = 1/DAF = 0.91$.

4.4.3 Geotechnics

Two code methods, $p - y$ (Section 3.2.3) and H_{ult} (Section 3.2.4) were used to predict the measured pile behaviour. They both predicted the load at which the first plastic hinge formed well, however the piles still had plenty of capacity in the plastic regime. As discussed in Section 3.5 the FoS for these tests was $\gamma_g = 2.0$.

4.4.4 Total

As explained in Section 1.2.2 if we find upper boundaries for each problem then we can treat the three as being decoupled. Hence the total factor of safety in current design can be taken as the product of the three partial ones:

$$\gamma_t = \gamma_f \times \gamma_d \times \gamma_g \quad (58)$$

$$\gamma_t = 1.05 \times 0.91 \times 2.0 = 1.9 \quad (59)$$

4.5 Tsunami Predictions

We can express the result of Eqn.59 in a different way, appropriate for tsunami design. Namely the predicted highest tsunami the piles can withstand. Eqn.5 links wave height h_{wave} to maximum surge force F_s predicted by Okada et al. (2006). Rearranging terms we arrive at:

$$h_{wave} = \frac{1}{6} \left(h_{pile} + \frac{2F_s}{\rho g D h_{pile}} \right) \quad (60)$$

where D is pile diameter and h_{pile} is pile height above ground. Firstly using the current code procedure, F_s is taken to equal the maximum pile lateral capacity predicted by codes H_{ult} (Section 3.2.4). Table 2 shows the calculated tsunami height values obtained through this procedure.

Using the data from the Tsunami simulator and the Pile tests we make our own prediction on maximum tsunami height. Instead of using H_{ult} for F_s we shall use the full horizontal pile capacity measured in the tests (Fig.38) H_{cap} divided by DAF to transition from the static to the dynamic problem. Then Eqn.60 becomes:

$$h_{wave} = \frac{1}{6} \left(h_{pile} + \frac{2H_{cap}}{DAF \rho g D h_{pile}} \right) \quad (61)$$

Results are summarized in Table 2 below:

Table 2: Maximum Tsunami Height Predictions

	Pile 1	Pile 2
Code	12 m	6 m
Tests	12 m	9 m

For Pile 1 the results are essentially identical since it was only tested up to plastic hinge formation. Pile 2 was tested up to its full horizontal capacity and there is a 33% difference between measurement and prediction. This means there is a factor of safety of 1.5 on the tsunami height.

It is important to note however that the ultimate capacity H_{cap} is derived from tests on single piles in sand. If the pile is part of a wall (as is usually the case), the capacity reduces since the failure mechanism for a wall is 2D. A pile in a wall does not benefit from the added capacity from the sand on its sides. API (2000) deals with the reduction in capacity by changing K_q in Eqn.26 from K_p^2 to K_p . Thus for a wall the maximum tsunami heights would be lower than the values in Table 2. Ongoing research at the Schofield Centre by Madabhushi (2015) investigates the response of dual-row steel walls to both tsunami and earthquake loadings through centrifuge tests on model walls.

5 Conclusions

The Fluids, Dynamics and Geotechnics problems were analysed through Tsunami Simulator experiments and full scale pile tests. In seawall design, codes are used which were not written with this specific use case in mind. The test results showed that even so, the overall predictions are safe by a factor of 1.9. Furthermore both piles tested had the capacity to withstand waves with heights greater than their own. This is a requirement the Japanese set after the 2011 Tōhoku tsunami (Suppasri et al., 2012). As such the jacked-in pile wall seems to be a suitable design for tsunami coastal defences.

Bibliography

- API, 2a-WSD. *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms-Working Stress Design*, 21(2000)
- ASTM-D2487, 2011. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

- ASTM-D4254, 2015. Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density. Technical report
- ASTM-D698, 2007. Standard test methods for laboratory compaction characteristics of soil using standard effort. Technical report
- D. Bogard and H. Matlock, Simplified calculation of p_y curves for laterally loaded piles in sand. (*unpublished report*), *The Earth Technology Corporation, Inc., Houston, TX*, (1980)
- M. D. Bolton, The Strength And Dilatancy of Sands. *Géotechnique*, 37(1987)(2)
- B. B. Broms, Lateral Resistance of Piles in Cohesionless Soils. *Journal of the Soil Mechanics and Foundations Division*, 90(1964)(3):123–158
- F. E. Camfield, Tsunami Engineering. Special Report No. 6. *US Army Engineer Waterways Experiment Station, Coastal Engineering Research Center, Vicksburg, Mississippi*, 222p, (1980)
- DNV, 1992. Foundations. Technical Report 30.4, Det Norske Veritas
- W. G. K. Fleming, A. J. Weltman, M. F. Randolph, and W. K. Elson, Piling Engineering, 2nd. Edition. *Blackie, Glasgow & London*, (1992)
- S. K. Haigh, 2013. Foundations for offshore wind turbines. In *Proceedings of the 8th International Conference on Physical Modelling in Geotechnics 2014 (ICPMG2014)*, pages 153–159. CRC Press
- J. B. Hansen and N. H. Christensen, 1961. *The ultimate resistance of rigid piles against transversal forces: Model tests with transversally loaded rigid piles in sand*. Bulletin 12. Danish Geotechnical Institute, Copenhagen
- M. Hatanaka and A. Uchida, Empirical Correlation between Penetration Resistance and Internal Friction Angle of Sandy Soils. *Soils and Foundations*, 36(1996)(4):1–9
- J. A. Heintz and M. Mahoney, 2008. *Guidelines For Design Of Structures For Vertical Evacuation From Tsunamis*. Federal Emergency Management Agency and the National Oceanic and Atmospheric Administration ATC-64 Project, FEMA P646 Report
- F. Kato, Y. Suwa, K. Watanabe, and S. Hatogai, Mechanisms of coastal dike failure induced by the Great East Japan Earthquake Tsunami. *Coastal Engineering Proceedings*, 1(2012)(33)
- N. Kihara, Y. Niida, D. Takabatake, H. Kaida, A. Shibayama, and Y. Miyagawa, Large-scale experiments on tsunami-induced pressure on a vertical tide wall. *Coastal Engineering*, 99(2015):46–63

- P. Kirkwood, 2015. Cyclic lateral loading of monopile foundations in sand. PhD thesis, University of Cambridge
- R. T. Klinkvort, Paper I. *Centrifuge modelling of drained lateral pile-soil response*, (2012):111
- S. L. Kramer, 1996. *Geotechnical earthquake engineering*. Pearson Education India
- T. O. Lloyd and T. Rossetto, 2012. A comparison between existing tsunami load guidance and large-scale experiments with long-waves. In *15 World Conference on Earthquake Engineering*
- C. Lugni, M. Brocchini, and O. M. Faltinsen, Wave impact loads: The role of the flip-through. *Physics of Fluids (1994-present)*, 18(2006)(12):122101
- S. C. C. Madabhushi, 2015. Multi hazard modelling of dual row retaining walls. First Year Report, University of Cambridge
- H. Matlock and L. C. Reese, Generalized Solutions for Laterally Loaded Piles. *Journal of the Soil Mechanics and Foundations Division*, 86(1960)(5):63–94
- T. Okada, T. Sugano, T. Ishikawa, S. Takai, and T. Tateno, Tsunami loads and structural design of tsunami refuge buildings. *the Building Centre of Japan*, (2006)
- D. Palermo, I. Nistor, Y. Nouri, and A. Cornett, Tsunami loading of near-shoreline structures: a primer. *Canadian Journal of Civil Engineering*, 36(2009)(11):1804–1815
- L. C. Reese, W. R. Cox, and F. D. Koop, Analysis of laterally loaded piles in sand. *Offshore Technology in Civil Engineering Hall of Fame Papers from the Early Years*, (1974):95–105
- D. J. Rockhill, M. D. Bolton, and D. J. White, 2003. Ground-borne vibrations due to press-in piling operations. volume 1, pages 743–756
- T. D. Smith, Pile Horizontal Soil Modulus Values. *Journal of Geotechnical Engineering*, 113(1987)(9):1040–1044
- A. Suppasri, S. Koshimura, K. Imai, E. Mas, H. Gokon, A. Muhari, and F. Imamura, Damage Characteristic and Field Survey of the 2011 Great East Japan Tsunami in Miyagi Prefecture. *Coast. Eng. J.*, 54(2012)(01):1250005
- D. J. White, PSD measurement using the single particle optical sizing (SPOS) method. *Géotechnique*, 53(2003)(3):317–326
- L. Zhang, F. Silva, and R. Grismala, Ultimate lateral resistance to piles in cohesionless soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 131(2005)(1):78–83

A Health and Safety

The Tsunami Simulator and full scale pile tests took place in Japan in summer 2015, therefore no risk assessment for them was submitted to the Department. Health and Safety was a major concern though in Japan. Everyone on-site adhered to specific and clear H&S standards. Every morning the work area was tidied to avoid tripping or slipping hazards. A clear plan for the day was set out each morning and everyone had to sign they understood their role and had been briefed accordingly. The biggest concern was the heavy machinery being operated to load the piles. This was done by qualified staff only. The pile tests took place on very hot and humid days so refreshments were provided. The site office was kept cool and used to freshen up.

The tests were carried out in Kochi, Japan where there is an earthquake and tsunami risk. The contingency procedures were explained to us carefully and the sites of the evacuation points nearest to the workplace and the accommodation were located. A small earthquake occurred during the stay in Japan, but without causing any damage.

During the rest of the project in Cambridge, the main H & S concerns were related to repetitive strain and eye-strain due to computer use. A PC was used instead of a laptop for better ergonomics and regular breaks were taken. No problems occurred during this stage of the project.