DEVELOPMENT OF SIMPLE GENERALIZED PSEUDO-STATIC P-Y CURVES FOR LIQUEFIED SANDS

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INTRODUCTION

Liquefaction has been associated with a variety of failures, including those of slopes, bridge abutments, building foundations and flotation of buried structures (e.g. Seed and Lee 1966; Seed and Idriss 1967). Based upon the increased need to understand liquefied soil behavior in laterally loaded pile foundations, small- and large-scale experiments have been conducted in the recent past using the centrifuge and 1-g shake table, as well as full-scale field tests. Several models to account for liquefied soil behavior have been proposed, ranging from assuming no lateral resistance in the sand, to using soil reduction factors (*p*-multipliers) with standard *p-y* curves (Wilson 1998; Weaver 2001), using the residual shear strength with Matlock's soft clay model (Wang and Reese 1998) and using site specific *p-y* models (Rollins *et al.* 2005). The use of the *p*-multiplier approach and Matlock's soft clay model, which are more generalized for design purposes, show too much resistance at small displacements and not enough resistance at large displacements. The small- and full-scale test results have shown that these models fail to capture the true behavior of the soil in the liquefied domain. Finite element models have been developed to account for these behaviors, but these are quite complicated for design purposes and require a substantial amount of geotechnical investigation.

This paper describes the details of the ongoing research to develop a simple generalized pseudo-static *p-y* model that would be based upon readily available parameters available by designers. This model will be calibrated against results from the centrifuge tests conducted at UC Davis (Wilson 1998) and the field tests conducted at Treasure Island (Ashford and Rollins 2000; Ashford and Rollins 2002).

CENTRIFUGE TEST RESULTS

A series of centrifuge tests for different soil properties (see Table 1) were conducted at the National Geotechnical Centrifuge at UC Davis (Wilson 1998). Different input motions were considered (Wilson 1998) including the 1995 Kobe earthquake record recorded at Port Island 83m deep in the NS direction scaled to 0.22g (named Event F for CSP2 and Event J for CSP3), scaled to 0.62g (named Event L for CSP2) and scaled to 0.60g (named Event O for CSP3). In the ongoing research the 16 tests conducted on soil profile CSP2 and the 16 tests conducted on soil profile CSP3 are taken into account. However, examples of the results obtained are outlined below.

Table 1: Soil properties for UC Davis Centrifuge Tests CSP2 and CSP3 (Wilson 1998).

	CSP2	**CSP3**
Upper Layer		
Depth	9 m	9 m
Relative density (D_r)	35-40%	55%
Buoyant unit weight (γ')	9.3 kN/m^3	9.7 kN/m^3
Specific gravity (G_s)	2.65	2.65
Void ratio (e)	0.743	0.677
Lower Layer		
Depth	11.4 m	11.4 m
Relative density (D_r)	80%	80%
Buoyant unit weight (γ')	10.1 kN/m^3	10.1 kN/m^3
Specific gravity (G_s)	2.65	2.65
Void ratio (e)	0.606	0.606

The excess pore pressure ratio (r_u) was around 80% through the upper loose layer (D_r*=35-40%)* in CSP2 Event F which confirms that the upper loose layer achieved partial liquefaction as opposed to an excess pore pressure ratio (r_u) of 100% which confirmed full liquefaction in CSP2 Event L. Excess pore pressures were not generated in the lower dense layer throughout the tests. In addition, the depth of the peak moment was deeper when liquefaction occurred compared to the same motion at a lower acceleration in which liquefaction had not occurred. The *p-y* curves calculated for CSP2 Event F (see Figure 1) clearly showed that after liquefaction, there is very little lateral resistance in the soil (<100 kN/m) even with large relative displacements (>25 mm).

For CSP2 Event L (see Figure 2), the soil shows an increase in lateral resistance with large relative displacements which are an order of magnitude larger than those obtained for CSP2 Event F. After liquefaction, there is a zone of almost no (or very little) lateral resistance followed by a stiffening in response occurring after a certain relative displacement is exceeded. In addition, when the soil was fully liquefied (r_u=100%), the relative displacements increased considerably (around 10 times compared to partial liquefaction with an average r_u of 80%).

For CSP3 Event J (see Figure 3), the upper layer (D_r=55%) showed excess pore pressure ratios (r_u) that reached approximately 75%. Results similar to those observations for CSP2 were noted in terms of the lengthening of the fundamental period and the increased depth to the peak moment. The differences in peak moments and displacements between the soil when it liquefied and without liquefaction were not as considerable as those for CSP2, which is expected due to the fact that the upper layer in CSP3 is denser. For CSP3 Event O (see Figure 4) the upper layer showed excess pore pressures of approximately 85%. As expected for Event O the relative displacements were double those of Event J.

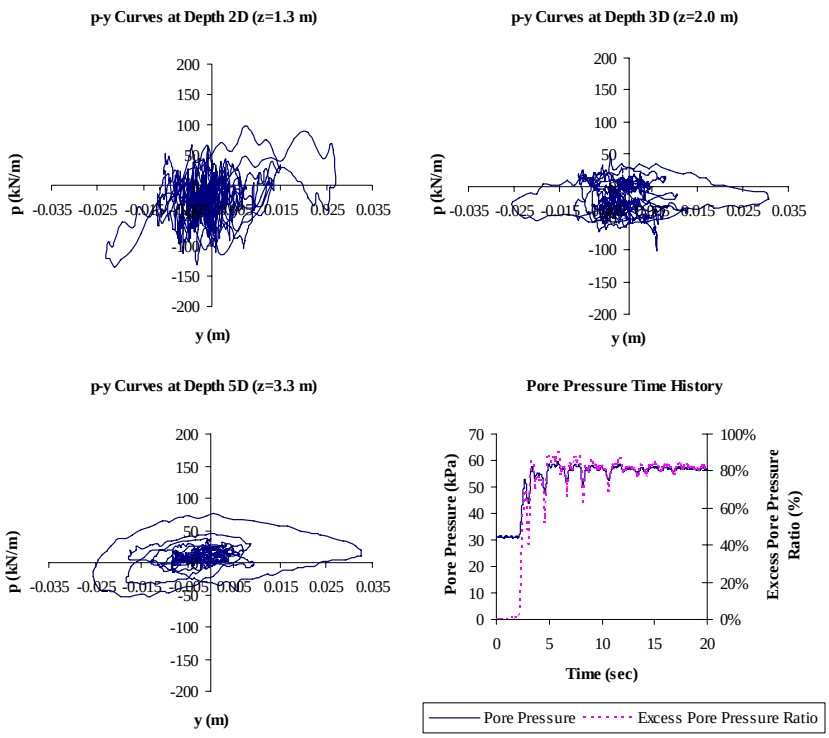

Figure 1: *p-y* curves and pore pressure time history for CSP2 Event F (Wilson 1998)

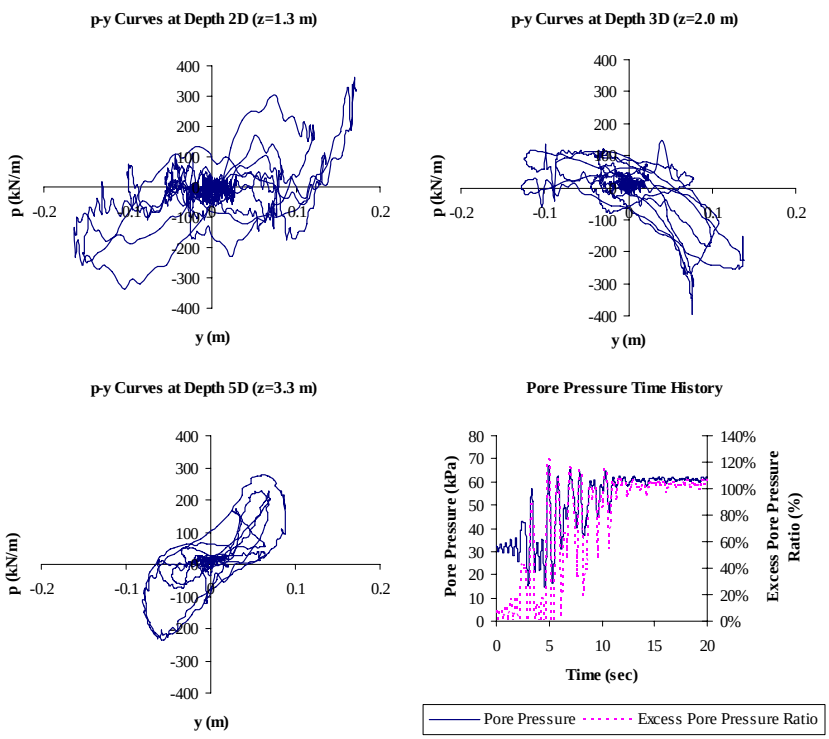

Figure 2: *p-y* curves and pore pressure time history for CSP2 Event L (Wilson 1998)

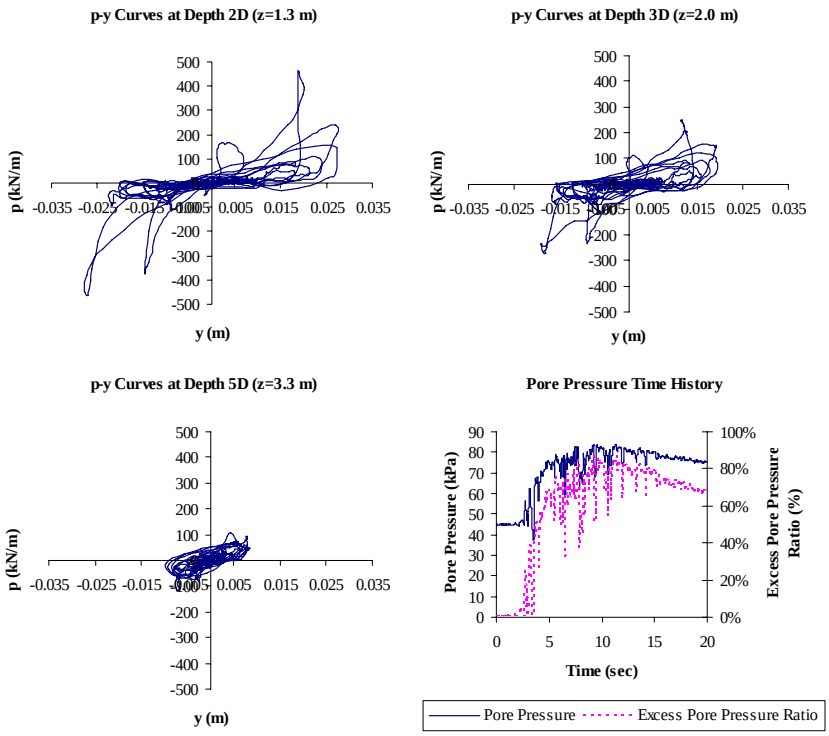

Figure 3: *p-y* curves and pore pressure time history for CSP3 Event J (Wilson 1998)

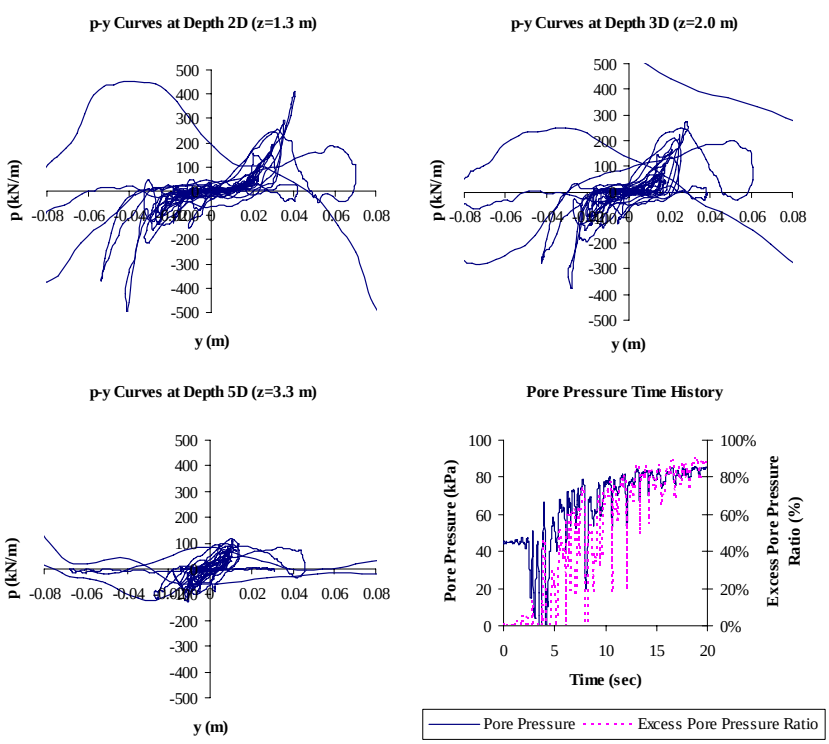

Figure 4: *p-y* curves and pore pressure time history for CSP3 Event O (Wilson 1998)

FIELD TEST RESULTS

Full scale lateral loading tests were conducted at Treasure Island, a man made island located in the San Francisco Bay. Several tests were conducted and the ones of interest here were the tests conducted on 0.6-m and 0.9-m cast in steel shell (CISS) single piles. The soil profile consists mainly of loose to medium dense sands with zones of clay from the ground surface to a depth of 9.6 meters. The 0.6-m pile consists of a 0.6 meter outside diameter steel shell with a 13 mm wall thickness with a total length of 13.8 meters below the ground surface. The rebar cage consisted of 2% longitudinal reinforcement. The 0.9-m pile consists of 0.9 meters outside diameter steel shell with an 11 mm wall thickness with a total length of 12 meters below the ground surface. The piles were fully instrumented including strain gages. The elastic flexural stiffness (*EI*) for the 0.6-m CISS pile is 291,800 kNm^2 and for the 0.9-m CISS pile is 1,019,358 kNm^2.

The average relative density for the upper sand layer in the 0.6-m CISS pile test area was 50%, while the average relative density for the sand in the 0.9-m CISS pile test area was 40-45%. These tests clearly showed that the soils are susceptible to liquefaction. Liquefaction was induced by blasting consisting of 0.5 kg of trinitrotoluene (TNT). Pilot liquefaction blasts were performed to determine the blasting layout to ensure that the excess pore pressure ratios could reach 100% (i.e. initiation of liquefaction) and sustained for a considerable time. These were successful as excess pore pressure ratios reached 100% approximately five minutes after blasting. In addition, soil boils started 5 minutes after the blast and continued for around 20 minutes. More details on the tests could be found in Ashford and Rollins (2000, 2002) and Weaver (2001).

The *p-y* curves at a depth of 3-diameters (3D) resulting from the tests are shown in Figure 5. These curves show the same trend as the UCD centrifuge tests where there is a zone of almost no (or very little) lateral resistance followed by a stiffening in response occurring after a certain relative displacement is exceeded. The curves also show that the zone of no lateral resistance decreases as the excess pore pressure ratio decreases. As expected, the zone of no lateral resistance is also smaller for the larger diameter pile as opposed to the smaller diameter pile.

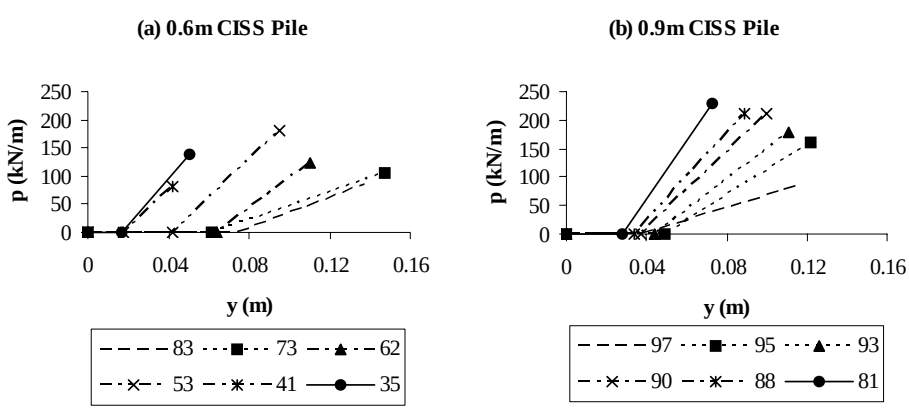

Figure 5: *p-y* curves for different excess pore pressure ratios for (a) 0.6-m CISS pile and (b) 0.9-m CISS pile at a depth of 3D (Weaver 2001).

P-Y CURVES CHARACTERISTICS

The current standard of practice with liquefied soils has included the following:

1. *Assuming no resistance in the liquefied sands.* This can be quite conservative for cases where the relative density is greater than 50% especially as in many cases the soil only partially liquefies during the strong motion portion of the earthquake record and as a result, the soil does actually provide some form of lateral resistance especially at larger displacements. The maximum moment calculated using this approach could be significantly (2.5-3.5 times) greater than the moments measured during testing.
2. *Using soil resistance reduction factors (p-multipliers) with standard p-y curves for sand (API 1993) to account for reduced soil resistance (Wilson 1998; Weaver 2001).* These curves have been shown to overestimate the lateral resistance at smaller displacements and underestimate the lateral resistance at larger displacements at smaller depths. At larger depths, the *p*-multiplier method tends to over-predict the soil strength as the *p-y* curves develop increased strength as the effective strength increases. This may lead to an under-prediction of the bending moment.
3. *Using Matlock's soft clay model (Matlock 1970) with the ultimate undrained shear strength of the soil defined by the residual strength of the liquefied sand (Wang and Reese 1998).* The residual strength of the liquefied sand could be determined using the SPT blow counts (Seed and Harder 1990). This method tends to provide a reasonable prediction of the soil resistance at small displacements, but under-predicts it at larger displacements.

As a result, the current standard of practice methods fail to account for the concave upwards shape of the *p-y* curves that have been obtained from small scale and full scale tests. In the non-liquefied sand curves, the resistance increases with strain, while with liquefied soils, the resistance increases with increased strain. As the soil shears, the surrounding soil dilates which effectively reduces the excess pore pressure. This reduction in the excess pore pressure results in an increase in the effective normal stress and would in turn results in increased shear strength of the sand.

The results of the centrifuge tests for single pile for UC Davis (Wilson 1998) and for the field tests at Treasure Island (Ashford and Rollins 2002) were reviewed in order to better understand the relationship between the tests. Several common elements of the results from both tests were observed.

1. All the curves showed an increased in resistance with increased strain (Figure 1-5).
2. All the results clearly showed that very little lateral resistance was available at the ground surface when the soil liquefied, but as the depth and confining pressure increased, the zone of little lateral resistance decreased and the corresponding stiffness developed increases progressively. The depth of the liquefied layer which shows no lateral resistance in all the full-scale and small-scale tests was found to be equal to 6-10 pile diameters. As a result we could assume that the soil deforms in simple shear (see Figure 6) where $u(x) = (g/2)\times(H_{liq} - x)$ and H_{liq} is the depth of the liquefied layer which has a zone of no (or very little) lateral resistance. As the diameter increased, H_{liq} decreased, which could be the result of a soil wedge developing behind the pile.

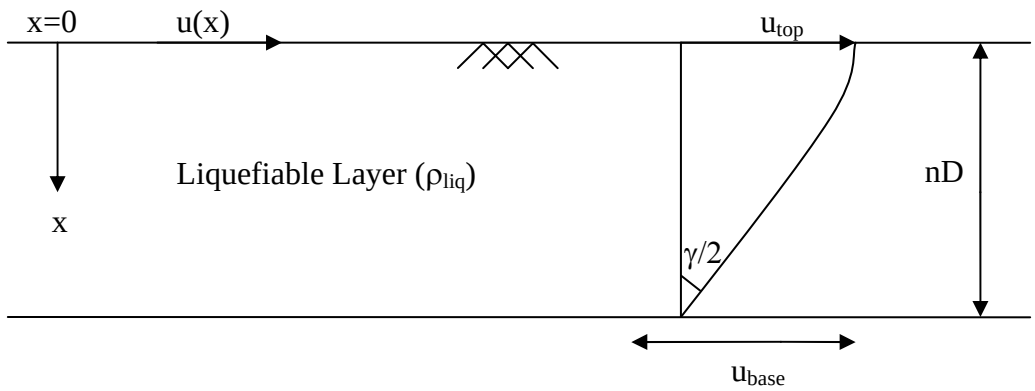

Figure 6: Soil wedge in a fully liquefiable soil

3. As the diameter increases the zone of no (or very little) lateral resistance decreases. This could be the result of the larger size soil wedge developing behind the pile.
4. As the excess pore pressure decreases, the zone of no resistance decreases exponentially (see Figure 7).

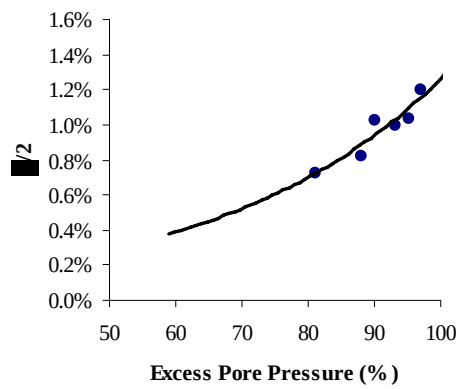

Figure 7: Exponential increase in shear strain at which the soil starts to stiffen as the excess pore pressure increases for 0.9-m diameter pile (based on results from Ashford and Rollins 2002)

5. The stiffening of the *p-y* curves (i.e. the slope of the *p-y* curves after the zone of no resistance) increases progressively with the reduction in excess pore pressure.
6. The zone of no lateral resistance decreases as the soil is denser (i.e. greater relative density).
7. The slope of the *p-y* curves after the zone of no resistance is greater for denser soils.

SUMMARY AND CONCLUSIONS

Based on the observations above and using the matrix of test results which includes results from more than 30 tests, a generalized model is under development (see Figure 8). This model will attempt to be more generalized and not site specific. It will be applicable to soils with relative densities 35-60%, diameters ranging from 0.324-m to 0.9-m and pore pressure ratios ranging from 40-100%. In addition, the model will be based on liquefaction occurring near the ground

surface. The model could be further improved as more test results become available. To avoid the shortcomings of the current standard of practice, the newly proposed curves with have the same concave upwards shape as observed. The zone of no resistance y_o will be based on estimating the shear strain in the soil at which the soil stiffens. This would be presented as a function of the relative density of the soil, pile diameter and excess pore pressure ratio. The excess pore pressure ratio is the value which would account for the earthquake motion (i.e. magnitude, acceleration, etc.). The slope of the stiffening portion of the curve (E_{sp}) would be presented as a function of the relative density of the sand, the pile diameter and the excess pore pressure ratio. Below the depth of H_{liq}, y_o is equal to zero even if the soil is liquefied, however the value of E_{sp} calculated at that depth would be used in the analysis.

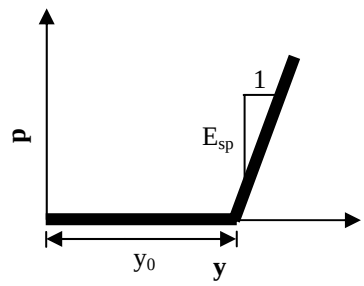

Figure 8: Generalized *p-y* model for liquefied sands under development

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