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The following paper is being presented at the DFI 34th Annual Conference on Deep Foundations on October 21-23, 2009 in Kansas City, MO, USA during the regular technical sessions.

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Expansion of the global piling industry has been coupled with significant research efforts investigating the mechanisms controlling pile capacity. Static pile design has transitioned from total stress approaches toward effective stress design, with associated improvements in predicting pile performance. As a result, the available literature is heavily skewed toward static capacity prediction. However, when considering piles offshore, the majority of loading is not static but highly variable and cyclic in nature. This paper outlines the additional factors that require consideration for cyclic pile design, including elevated loading rates, cyclic displacements and capacity degradation. A review of the literature identified a number of cyclic pile testing programs which are collated in this paper. General trends are established through comparison of the tests within the database and the conditions where cyclic loading is most likely to be a concern are identified. In particular, loading conditions which involve full plastic shear reversals are seen to cause the most extreme cyclic damage, with the degraded capacity as low as 31% of the precyclic static capacity. The underlying mechanisms which can result in cyclic damage are explored, with particular attention given to the pore pressure response, radial stresses, shear transfer and particle reorientation.

INTRODUCTION

Static pile design has undergone a revolution in recent years, as concentrated research efforts aimed at understanding pile behavior and establishing the underlying mechanisms have produced more reliable effective stress based approaches. However, when considering offshore piles, the majority of loading is not static but highly variable and cyclic in nature. The comparative reliability of piles subjected to cyclic axial loading is poor in comparison to static predictions. The poor predictive accuracy of cyclic axial capacity was demonstrated by Jardine et al (2001) through a class-A prediction competition for piles subjected to fully reversed cyclic loading on driven steel tubular piles. Jardine et al (2001) noted that “none of the competitors were able to offer convincing predictions for the cyclic load test” when asked to determine the number of cycles required to failure.

Piles subjected to cyclic loading are rarely designed to explicitly resist such loading conditions but are instead designed implicitly by applying a factor of safety against the expected extreme environmental loading condition. This usually results in the designed capacity being 1.3 times the maximum storm load predicted over the pile lifetime, of typically 100 years. Poulos (1989) highlights three aspects of pile response that need to be considered for cyclic loading conditions which are not a significant issue for static loading. These include (i) the degradation of pile-soil resistance (ii) the accumulation of displacements (either cyclic or permanent) and (iii) loading rate effects. While the first two considerations are associated with a negative impact on pile performance the loading rate effect will have a positive influence due to the short term nature of the storm loading. It is generally but not unanimously considered that positive rate effects can counteract and even overcome both degradation and excessive displacements. As a direct result, there is a general opinion that designing for the static loading condition is conservative (Rigden and Semple 1983). The limited number of cases involving offshore pile failure appears to support this viewpoint. An improved understanding of the mechanisms governing cyclic pile response could allow explicit consideration in design and potential economic savings. This paper reviews the available literature and provides details of the mechanisms influencing the axial capacity of piles founded in clay deposits, with a view toward explicit incorporation of cyclic loading effects within an effective stress framework. The review of the literature focuses on the results of experimental programs designed specifically to investigate cyclic tests on model piles and full
scale field load tests. Existing discrepancies in the literature are highlighted and a series of issues are identified which require further research.

**STATIC CAPACITY-RECENT DEVELOPMENTS**

A rational approach to pile design requires consideration of all stages in the pile lifetime, including the in-situ conditions, pile installation, consolidation and loading phases as illustrated schematically in figure 1. Advances in pile design have generally resulted from incorporation of all these stages into rational design approaches, instead of a purely empirical relationship developed from laboratory parameter-load test comparisons.

![Figure 1: Variation in pile-soil stress regime during foundation lifetime](image)

In order to fully understand the mechanisms controlling the development of shaft resistance, it is necessary to understand the radial effective stress regime at the pile-soil interface. In this regard, high quality testing using instrumented piles have provided a valuable insight. For example, the Imperial College Pile (ICP) which measured the radial stress and porewater pressure at a number of locations on the pile shaft has resulted in a more reliable and accurate design approach to estimate the axial static capacity of piles, IC-05 (Jardine et al. 2005).

The IC-05 method considers the effective stress changes as the pile is installed and undergoes subsequent consolidation. This is expressed through a semi-empirical function for the equalized radial effective stress which incorporates the yield stress ratio (YSR), soil sensitivity ($S_h$), interface friction angle and a geometric $h/D$ term. The $h/D$ term refers to the distance from the pile tip, $h$, normalized by the pile diameter, $D$ and is included in a power law form to capture the decay in radial effective stresses remote from the pile tip, termed friction fatigue. The post equalization radial effective stresses ($\sigma_{rc}'$) recorded for the Imperial College Pile tests, underwent reductions of approximately 20% during maintained load tests before peak shaft shear stresses were reached. The relative reduction in effective stresses showed no dependence on material type or soil properties. This was termed the loading factor ($f_L$) and was found to be in the range 0.8±0.05. The peak shear stress ($\tau_f$) mobilized during the IC load tests can be described by a mohr coulomb failure criterion with some tests indicating a strain softening effect with progressive failure. The resulting Imperial College method is shown in equation 4.

$$\tau_f = \sigma_{rf}' \tan\delta_f = f_L \sigma_{rc}' \tan\delta_f$$

where $\sigma_{rf}'$ and $\delta_f$ are the radial effective stress and interface friction angle at ultimate conditions. The improved reliability of the IC-05 approach was demonstrated through summary statistics presented by Jardine et al (2007), showing a mean ratio of measured to predicted capacity of 1.03 and a COV of 0.2, which compared favorably to a mean 0.99 and a COV 0.33, for the industry standard: American Petroleum Institute (API) method. The predictive capabilities of the IC-05 approach were subsequently verified by Overy and Sayer (2007) for full scale field installations in the North Sea.

What remains unclear following the ICP investigation is the impact of in service cyclic loading on the radial effective stress regime. This could be reflected in changes to the $\sigma_{rf}'$ value as a result of total and pore pressure fluctuations or through changes in $\delta_f$ stemming from structural changes within the clay.

**THE NATURE OF CYCLIC LOADING**

Geotechnical structures are commonly subjected to a variety of loading conditions from static building loads and short term construction loads to tidal waves and earthquakes. Many of these loads vary in magnitude above and below the design load of the structures and are termed cyclic in nature. These cyclic loads often
represent the most extreme loading conditions on a structure, for example the 100 year storm waves for which offshore platforms are commonly designed. The applied cyclic load can be defined by two components: the mean cyclic load ($Q_{av}$) and the cyclic amplitude ($Q_c$). The minimum cyclic load is simply calculated as the average load less the cyclic amplitude and similarly the maximum cyclic load ($Q_{cmax}$) is the sum of the average load and the cyclic amplitude. The long-term capacity of piles driven in clay is primarily resisted by the pile shaft, with the base usually contributing less than 10% of the capacity (Chow, 1997). Considering this, the discussion in this paper is limited to the influence of cyclic loading on the shaft resistance. The cyclic load applied to the pile head is resisted by shear stresses, $\tau$, mobilized along the pile shaft, as illustrated by figures 2a-c. These figures depict different possible stress conditions along the shaft. Figure 2a represents symmetrical cycling about a zero mean stress, whereas Figures 2b and 2c both have an initial average shear stress about which the cyclic shear stress is varied. The average stress is termed $\tau_{av}$ and the cyclic stress defined by the cyclic amplitude is termed $\tau_c$. The maximum stress, $\tau_t$, is defined as the sum of these two components. In cases where the mean stress is less than the cyclic amplitude, the shaft will undergo stress reversals with the shear stress alternating between compression and tension during each cycle, see figure 2b, however when the mean stress is larger than or equal to the cyclic amplitude no stress reversals occur and the loading remains unidirectional, see figure 2c. The response of piles to cyclic loading is heavily dependent on the nature of the applied load with each of the conditions represented in figure 2 resulting in a highly different response. It is worth noting that for unidirectional loading applied to the pile head long flexible offshore piles may develop shear reversals over the upper reaches of the pile shaft, in contrast to stiff incompressible model piles.

The cyclic period, $T$ is defined as the time taken for a complete cycle of loading to occur and is the inverse of the cyclic frequency, $f$. Offshore structures will inevitably be subjected to cyclic loads over a wide periodic range, with tidal changes exerting load cycles with a period of hours and forcing storm waves with periods from 10-20 seconds. Extreme environmental cyclic conditions are of most concern, as they potentially represent the highest magnitude loading and as a result the high frequency storm waves occurring during a 100 year event are the cycles requiring primary consideration.

Cyclic pile load tests can be conducted under either displacement or load control, depending on the experimental procedure. Load control involves cycling the pile from a pre-specified minimum load to a predetermined maximum and observing the displacements that develop. In contrast, displacement control involves setting limits for the pile head displacement and recording the head load required to reach the set limits. The literature contains a combination...
of both type tests, with many of the smaller model piles subjected to displacement control, and the larger diameter piles load controlled. While both procedures have provided significant insight into cyclic behavior, the most representative testing should consider the nature of offshore loading. The most valuable observations are therefore achieved through cyclic tests of a load-controlled nature. Karlsrud et al (1986) also highlight the importance of considering how the pile transfers the applied cyclic load and comment that for long flexible piles subjected to high magnitude storm loads, the upper portion of the shaft may undergo displacement controlled loading.

OBSERVATIONS FROM CYCLIC FIELD TESTS

Pile Test Overview
While there is a dearth of cyclic pile tests in the literature when compared to static load tests, the limited number of cyclic tests conducted to date has significantly improved our understanding of cyclic behaviour. A review of these tests is provided in table 1, below. A combination of laboratory, model scale and full scale field tests were identified, with diameters ranging from 19 mm to 762 mm and maximum penetrations of 80 m. These tests included compression, tension and fully reversal cyclic tests under both displacement and load control over a range of cyclic periods in a variety of soil conditions.

LOADING MAGNITUDE

A review of the literature highlights a range of applied loading magnitudes. Direct comparison of these tests need to be considered cautiously, with due consideration given to the differences in testing procedure. Under ideal circumstances each case history would include a static load test conducted on an identical virgin pile at each site, which could be used to normalize the cyclic load test data. However this is rarely the case and the cyclic load is usually normalized by the precyclic ultimate static capacity ($Q_{ult}$) with no consideration given to the potential changes to the radial stress regime or the potential development of shear surfaces parallel to the shaft during pre testing. Normalising the cyclic load by the ultimate static capacity allows comparison amongst the literature.

Early field tests performed by Seed and Reese (1955), Sharman (1961) and Whitaker and Cooke (1961) involved multiple load applications on full scale piles. However the number of applied cycles was typically less than 8 and the applied loading rate was more representative of static conditions. The failure load following this small number of cycles ranged from 84% to 96% of the initial static capacity. A series of cyclic tests performed on reinforced concrete and timber piles at three Swedish sites were reported by Broms (1972). Cyclic loading magnitudes were varied and the pile head response indicated a relatively stable response prior to application of a particular high level load which caused rapid displacements and failure. Puech (1982) conducted cyclic field tests on a 273mm diameter steel pile driven into normally consolidated deposits at Cran, France and describe the presence of a load threshold, prior to which no cyclic damage occurred. While the observations from both Broms (1972) and Puech (1982) indicate a threshold exists, insufficient data was available to allow quantification of this loading threshold.

In contrast, cyclic load tests on two steel piles driven into boulder clay at Cowden, England were described by Gallagher and St. John (1980). Peak cyclic loads amounting to 50% of the precyclic static capacity resulted in no adverse effects over the 150 cycles applied, with post cyclic static load tests indicating an increase in ultimate capacity. Load tests on four 356 mm diameter steel open ended pipe piles were performed by Kraft et al (1981) to assess the combined impact of loading rate and cyclic effects. For loads above 80% of the static capacity, displacements began to accumulate and the displacement rate increased dramatically for loads amounting to 110% of the static capacity. In this case, the rapid post cyclic capacity was determined to be greater than the initial static capacity. McAnoy et al (1982) report experimental findings for a 193 mm diameter pile tested in Cowden boulder clay, where loads in excess of 80% of the static capacity caused significant degradation of ultimate capacity. Similarly a threshold of 91% was reported by Karlsrud and Haugen (1983) for tests conducted in overconsolidated clay at Haga by the Norwegian Geotechnical Institute. Cyclic tests conducted in San Francisco Bay Mud (Doyle...
and Pelletier, 1985) indicate that load magnitudes equal to the ultimate static capacity are required before permanent deformations occurred. For displacement controlled tests the skin friction reduced to between 61 and 85% of the precyclic values. Bogard and Matlock (1990) indicate a threshold equal to the static capacity with no degradation observed for one-way loading. In contrast 2-way loading at relatively low load levels can cause rapid degradation of capacity (Bogard and Matlock, 1990a, 1990b, 1991).

**IMPACT OF LOADING RATE**

Clays exhibit viscous behavior, such that their strength is dependent on the rate at which they are sheared. The typical cyclic period of 10s for offshore storm waves contrasts with traditional maintained load tests which usually involve loads applied over several hours and possibly days. As a direct result, the pile capacity available offshore may be significantly higher than the static capacity.

The rate effect in soils is not a new concept; the positive impact of reducing the time to failure on the soil strength began with an account by Collin (1846). The rate effect was thus recognised for many years before the first systematic investigation of its effect on soil strength was conducted. This involved a series of triaxial tests on both cohesive soils and sands by Whitman (1957) and later by Yong and Japp (1967). These investigations confirmed that the compressive strength consistently increased as the rate of loading increased. Researchers such as Whitman (1957) examined different soil types under different rates and determined that the strain rate effect was much more significant in clay than sand, a result that was verified by many researchers seeking to quantify the strain rate effect for incorporation into design (Matesic and Vucetic 2003). Briaud (1985) describes the origins of this viscous component, including the pore water, the particle contact and the soil skeleton/water particle interaction response. Essentially under slow loading the soil/water particles can reorientate slightly to shear along the path of minimal resistance, however for rapidly applied loads the particles do not have time to relocate and as a result the soil strength increases. This dilatant response will be accompanied by decreasing pore pressures and hence the soil permeability plays an important role in soil viscosity. Briaud (1985) also showed the viscosity to increase with water content and plasticity index and decrease with undrained shear strength.

This rate dependent behavior has been observed to apply to pile capacity, with relatively fast constant rate of penetration (CRP) tests exhibiting higher pile capacities than identical piles subjected to slow maintained load tests. $Q_{ult}$ in clays increases between 5% and 20% for each order of magnitude increase in loading rate, with the greater increases typically applying to weaker and more plastic clays (Dunnavant, et al., 1990). An example of this behavior is presented in figure 3, illustrating the strong rate dependence of friction piles founded in high plasticity Mexico City clay (Jaime et al, 1989). The positive viscous effect is represented by a substantially higher stiffness over the measured range of displacements and hence a higher capacity at both the ultimate and serviceability limit states.

![Figure 3: Pile Rate Dependence (adapted from Jaime et al, 1990)](image)

Kraft et al (1981) considered a range of load tests under maintained, CRP and cyclic conditions and reported an increase of between 40-75% over a three order of magnitude rise in loading rate. In fact, the primary conclusion was that the impact of loading rate counteracted any negative cyclic effects. The tests were also influenced by varying soil conditions, different set up times and stress histories which probably accounts for the large range in viscosity observed. The maximum loading rate impact was observed on a pile which had undergone a significantly less severe loading history, indicating that the positive rate effect is most influential for untested piles. Examination of the load-displacement relationship indicated a less dramatic impact of loading rate on the stiffness response with two of the pile exhibiting a 10% increase in secant stiffness per log cycle.
increase in loading rate. Chow (1997) points out that post-peak increase in loading rate from 0.001 to 1.5 mm/min had little or no effect on shaft capacity. However, increases in rate from 1.5 to 15 mm/min produced a rise of ~8%. Karlsrud and Haugen (1985) described fast static tests which failed in 10 to 20 seconds giving capacities which were 20-25% larger than a subsequent slow static test which corresponded to a 9% capacity increase per order of magnitude. It has been postulated, but not uniformly accepted, that the influence of such positive rate effects would override any negative effects resulting from cyclic loading.

DEGRADATION

As discussed previously a load threshold exists prior to which no degradation will result. This threshold ranges from 60% to 100% of the precyclic static capacity. A number of researchers report no degradation until the maximum cyclic load becomes larger than Q_{ult} (e.g. Jaime et al, 1990, Kraft et al 1981 and Doyle and Pelletier, 1985). A slightly lower threshold of 60 to 80% of Q_{ult} has been identified for low plasticity clays. For example McAnoy (1982) reported on a test conducted immediately after cyclic loading at 60% of Q_{ult}, where 6% degradation occurred, and a subsequent 15% reduction in Q_{ult} following loading at 80% of the static capacity. However Bouckovalas (1996) suggests that this reduction is in line with the expected decrease when shearing to the residual interface friction angle.

The large diameter pile tests (LDPT) conducted at Pentre and Tilbrook (Cox et al, 1993) provide an interesting case study with respect to cyclic degradation. The near identical pile geometries and testing procedures allow for direct comparison with respect to the soil type. Considering the static capacity before and after cyclic loading, demonstrates no reduction in strength for the less plastic normally consolidated deposits at Pentre despite cycling at 100% of the static capacity, which contrasts with the 8% reduction observed at the highly overconsolidated Tilbrook test after cycling at only 80% of the capacity. The slightly higher plasticity index of the Tilbrook soils conflicts with traditional opinion that higher plasticity soils are more resistant to cyclic degradation and also suggests that higher OCR soils may be more susceptible to degradation. Jardine (1995) postulates that higher plasticity soils are less susceptible to cyclic influences because of the more extensive elastic recoverable region and lower interface friction angles. While this point seems to be validated by some load tests involving low numbers of cycles to failure, the degree of overconsolidation seems to be a highly influential factor that requires further consideration.

For 2-way loading conditions involving fully reversal shear transfer, the pile capacity rapidly decreases towards a residual value, with typical reductions between 50% and 75% of Q_{ult}. This is demonstrated by Holmquist and Matlock (1976), who indicated that unidirectional cycling at 90% of the static capacity resulted in no degradation, however increasing the load magnitude to 100% of Q_{ult} resulted in a 20% reduction in static capacity, which stretched to a 66% decrement when considering 2-way conditions. Bogard and Matlock (1990) conducted 2-way displacement controlled tests on model pile segments and reported shear transfer reductions as low as 40% of the initial shear capacity following 21 cycles. Similar observations were noted by Karlsrud and Haugen(1985) and Grosch and Reese(1980),with ultimate capacity decreasing by 58% and 61% respectively. Karlsrud and Haugen (1985) comment that the minimum shear transfer following cyclic degradation is approximately equal to the shear transfer directly following installation or the remolded shear strength measured in direct simple shear tests.

The difference in behavior under unidirectional and 2-way loading is best described by the shear transfer response shown in figure 4, where the static load tests before and after one way cyclic loading are presented alongside the cyclic minimum shear transfer following two way cycling (Bogard and Matlock, 1990). Fully reversed plastic slip resulted in capacity degradation of 32% from the pre cyclic static capacity to a minimum shear transfer value for load tests conducted 69hours after installation. Prior to plastic slip occurring the failure was only mildly non linear. Initial static load demonstrated a peak value which rapidly decreased toward a residual value as displacements continued. Repeated cycles to failure caused a transition toward simpler elasto-plastic behavior, with severe temporary losses in strength occurring as a direct result of full reversal cycles to failure.
Following cyclic failure, consolidation can lead to partial recovery in the static capacity. However Karlsrud et al (1986) observed that the pile capacity could undergo significant increases following failure and subsequent reconsolidation. The gain in capacity was typically 23% but could be as large as 60% after several failure-recovery periods. It was observed that this increase was related to the pore pressure changes occurring in the interface zone. This contrasts with observations from Kraft et al. (1981) who found that static capacities following initial failure and subsequent consolidation were up to 25% lower than the capacity measured in initial static tests. Residual shear surface formation was suggested as the primary mechanism behind this capacity decrease.

Figure 4: Comparison between Shear Transfer occurring under one-way cyclic loading, Static testing and extreme two way cyclic strength at Empire (Bogard and Matlock, 1990a)

**CYCLIC SETTLEMENT AND STIFFNESS RESPONSE**

The load-displacement response and accumulated settlements exhibit two distinct responses; specifically cyclic shakedown and incremental collapse which correspond to stable and unstable behavior respectively. Cyclic shakedown is characterized by permanent displacements that develop according to a linear logarithmic relationship with respect to the number of cycle, which ultimately stabilizes as the test progresses. Incremental collapse involves an initial accumulation of permanent displacements, while the cyclic stiffness remains approximately constant and a subsequent degradation in this stiffness accompanied by increased displacement rates and failure. As discussed previously the settlements are related to the applied load and may act progressively for loads in excess of the damage threshold but otherwise remain stable or exhibit meta-stable behavior following sufficient cycles.

Figure 5: Incremental Collapse compared to Cyclic Shakedown and the positive impact of preshearing (data from Bouckovalas, 1996)

Tension tests on model lab scale piles performed by Bouckovalas (1996) indicated that for maximum loads of 0.67Qult the cyclic stiffness increased with respect to the number of cycles and ultimately reached shakedown. In contrast for tests with Qcmax of 0.81Qult cyclic stiffness progressively reduced, with stiffness degradation accelerating after the initial few cycles, and finally reaching incremental collapse. These trends are illustrated in Figure 5, using data from Bouckovalas (1996) where the loading magnitudes in each test are indicated by $Q_{av} +/- Q_c$. 

Static testing, following unidirectional cyclic loading (e.g. Chow, 1997), have demonstrated significant strain hardening, which resulted in an initial stiffness in excess of the original static curve, with reduced pile head movements prior to peak load development. Similar behavior was observed by Bouckovalas where stiffness
increases occurred for cyclic tests following cyclic preshearing. So for piles which had been previously cycled at a specified load and then re-cycled the permanent displacements are smaller at a specified cycle. This behavior extended to both cyclic shakedown and incremental collapse, and is illustrated in figure 5 through comparison of cyclically presheared and virgin load tests.

Karlsrud and Haugen (1985) demonstrated both shakedown and collapse at different cyclic magnitudes. Following close inspection of the displacement response for both individual cyclic loops and long term plastic displacements, Karlsrud et al (1987) pointed out that the accumulated displacement was directly related to the effective time that the peak load is maintained on the pile. Essentially, this suggests that the displacements during incremental collapse are a creep response under high level loads and not stiffness degradation.

The long term load test performed by Peuch et al (1980) indicated that for extended cyclic tensile tests with peak applied loads of 52% of the failure load in the static test, the accumulated displacements continued progressively over 1500 cycles. They also comment that short term cyclic tests would have indicated stability after a limited number of cycles, which is not the case.

**CYCLIC MECHANISMS**

**Pore Pressures**

Highly instrumented pile tests have successfully established the failure mechanisms during static loading and the underlying factors influencing such mechanisms. The primary conclusion of the research being that static capacity is controlled by the effective stress response and follows a Mohr Coulomb failure path. The mechanisms that could potentially lead to cyclic degradation are discussed within this effective stress framework and as such the radial stresses, pore pressures and structural changes need to be considered.

Measured pore pressures (u) at the pile shaft interface in a number of tests indicate a range in behavior, even in a qualitative sense. A number of researchers report pore pressures that undergo minimal fluctuations during cycling and no net changes from the ambient conditions. Puech and Jezequel (1980) recorded no pore pressure build up during loading at 0.62 Qult, for periodic cycles of 14 s. Similarly, Chow (1997) reports one-way tension loading under a 60s period in silty clay that developed no excess pore pressures over 40 cycles at 0.77 Qult. McAnoy et al. (1982) observed no pore pressure generation in Cowden till during applied loading of 0.6Qult, however slight increases were observed over the first 40 cycles when the cyclic magnitude was increased to 0.8Qult. The excess cyclic pore pressure was observed to dissipate once significant displacements occurred and pull-out commenced. In contrast, a long term low magnitude cyclic test performed by Karlsrud and Haugen (1985) indicated progressive pore pressure generation over the initial 500 cycles followed by a return to the hydrostatic conditions over the remaining 10,000 cycles. This is explained as a radial consolidation response coupled with a parallel reduction in the cyclic generation of pore pressures.

Grosch and Reese (1980) observed a net pore pressure decrease during cyclic failure under two way displacement controlled loading. The drop in pore pressure occurred as the displacement limit increased to +/-0.02in corresponding to approximately 0.5mm after which no significant changes in u were observed despite individual cyclic fluctuations.

A comprehensive pile testing program was undertaken by Bogard and Matlock (1990a, 1990b, 1990c, 1991) to investigate the behavior of friction piles driven in clay during both static and cyclic loading conditions. Initial testing was conducted at Harvey, Louisiana at a soft high plasticity clay site. No disturbance of pore pressure was observed at loads and displacements below full slip condition. Close inspection of the response to two-way cyclic loading indicated rapid degradation of the shaft capacity as indicated by measured shear transfer curves and changes in the pore pressure regime. A complex pore pressure response was identified for tests with fully reversed plastic slip. The rapid degradation during cyclic loading was accompanied by a general tendency for pore pressures to increase which is explained by a shear induced remolding of a cylindrical band of clay adjacent to the shaft. Superimposed on the rising pore pressure trend was a local tendency for pore pressures to rapidly decrease during plastic slip prior to a
rapid recovery during load reversal. Experiment 2 conducted at Harvey is presented in figure 6 as a typical example of the previously described behavior. The test is displacement controlled with both the displacements and the rates indicated throughout each phase of the experiment. The highly dilative behavior upon displacement limit increase from ±0.05 inches to ±0.2 inches is particularly evident as the pore pressures decrease dramatically. It should be noted that this phase of testing also involves a displacement rate increase which could in part be responsible for the depressed pore pressure profile. Subsequent testing at slower rates shows pore pressures returning to the previously elevated levels, leaving questions remaining as to the relative contributions of rate effects and cyclic displacements during interface dilation/elevated effective stresses.

Figure 6: Displacement Controlled 2way cyclic test at Harvey (Bogard and Matlock, 1990)

Additional tests involving low level cyclic loading without shear reversal were also performed to investigate the threshold for pore pressure generation. Based on these test it was concluded that fully reversed plastic slip was required before degradation could occur. Cyclic tests were conducted 5 mins and 53 hours after driving as primary consolidation was occurring and the rapid return to the equalization profile indicates a very thin shear zone is involved in the pore pressure response to cyclic loading. The post cyclic static shear capacity showed rapid recovery with time indicating that cyclic degradation is a temporary phenomenon.

Radial Stress and Shear Behavior

Limited information is available describing the total stress response, however comparison of the net pile behavior alongside recorded pore pressure distributions indicates that the effective radial stress can reduce through cyclic loading with limited changes in pore pressure occurring. This cyclic total stress relaxation could be a more severe phenomenon than pore pressure generation as the recovery will not be as rapid or as complete. An example of this behavior is described by Karlsrud and Haugen (1985) who indicate that the degradation in radial effective stresses observed in Haga clay were associated with decreases in the total stress.

Additional tests were also conducted by Bogard and Matlock (1990b) at Empire, where an initial load was applied with a tension bias of 50% of the capacity and cycling around this load level was applied through incrementally increasing the load magnitudes. An example of this loading procedure is given by Figure 7. The radial stress response was also shown to be relatively unaffected by single directional loading and pore pressure development was only significant following full shear reversal, as previously observed at Harvey. Fully reversed plastic slip resulted in capacity degradation of 32% from the pre cyclic static capacity to a minimum shear transfer value for load tests conducted 69 hours after installation.

Additional offshore testing was performed at West Delta platform in Mississippi where the clay is stratified into three zones specifically very soft, medium plasticity soft clay and high plasticity stiff clay. These tests followed similar procedures to the onshore tests and indicated very similar pile responses in terms of both shear stresses and lateral stress response during both cyclic and static tests. The cyclic minimum shear stress transfer following cyclic degradation was shown to be independent of the degree of primary consolidation and therefore also independent of the time after installation. This minimum stress transfer was shown to be approximately equal to the remolded shear strength of the clay and involved a very narrow...
zone of clay explaining the temporary nature of the degradation.

Figure 7: Example Loading History of tests at Empire (Bogard and Matlock, 1990a)

Reviewing the experimental evidence compiled by Bogard and Matlock (1990, 1990a, 1990b, 1991) suggests that no cyclic degradation occurs under unidirectional conditions. However when examining the applied loading regimes, it appears that the number of cycles were typically limited to less than 30. Therefore caution needs to be applied when extrapolating these observations to more extreme loading conditions. Essentially, questions remain regarding the impact of continued high magnitude unidirectional loading.

**Particle Reorientation**

The platy nature of clay particles make them amenable to rotation during shearing as the slip zone develops and ultimately the particles align parallel to the direction of shear strain (Atkinson and Bransby, 1978). This realignment has been observed in a range of lab tests including direct simple shear (Notwatzki, 1966) and cyclic triaxial tests (e.g. Taylor, 1971). The mechanical reorientation of the clay will result in different shearing characteristics during subsequent loading and hence changes in $\delta_f$ could ensue.

Grosch and Reese (1980) attributed the degradation observed in their pile tests to particle realignment. As the pile was loaded to failure, shearing was concentrated on a narrow plastic slip zone. The negative pore pressures observed during failure indicate dilation within the slip zone as the clay particles realign. The destructuring of the clay interparticle bonds and the subsequently highly orientated shear zone leads to reductions in the pile load-transfer capacity. This indicates that the reduction in load transfer and the potential degradation is a function of the soil mineralogy. Essentially the increased parallelism could be captured by a reduced interface friction angle, possibly even lower than the large strain residual value. The minimum load transfer during 2-way displacement controlled cyclic loading ranged between 39 and 46% of the maximum load transfer. Grosch and Reese comment that the reduction in pore pressure observed during cyclic loading are inconsistent with effective stress or critical state concepts; however the experimental evidence was not sufficient to validate such proposals.

The observed pore pressure fluctuations that occur as failure develops are attributed by Bogard and Matlock to two sources. (i) Structural changes within a band of clay undergoing plastic deformation consisting of reorientation of the clay platelets and an increase in the degree of parallelism, ultimately causing pore pressure increases. This is a temporary phenomenon within the shear zone and upon cessation of loading the pore pressures return to previously established equalization profiles. (ii) As slip surfaces develop, a marked reduction in pore pressure occurs; this is associated with shear slip dilation. Following plastic slip the pore pressures return to values higher than the previous slip conditions. However some structural changes may remain leading to a permanently reduced interface friction angle and hence a portion of the cyclic degradation may be irrecoverable.

**SUMMARY**

A review of the existing literature was conducted with respect to cyclic axial pile tests in cohesive soils and the primary observations are summarized below:

- A range of cyclic magnitudes were applied, however testing focused primarily on high
level unidirectional loading in excess of 60% of $Q_{ult}$ and 2-directional loading.

- Positive rate effects were seen to elevate ultimate capacity between 5-20% per log cycle increase in rate.
- Degradation was shown to be extremely rapid and dramatic in the case of 2-directional loading.
- A threshold loading magnitude exists prior to which no damage occurred, which ranges from 60-100% of $Q_{ult}$.
- The degraded capacity can be reduced to 31% of the pre cyclic capacity during 2-way testing.
- Degradation has largely been attributed to particle reorientation and pore pressure changes.
- Pore pressure increases were primarily observed during fully reversal plastic strains.

**RECOMMENDATIONS/FUTURE CONSIDERATIONS**

While the existing literature provides a broad spectrum of cyclic data, existing uncertainties remain, specifically:

- The literature primarily focuses on high level loading and therefore the response of piles under typical in-service loading conditions at 33-50% $Q_{ult}$ (corresponding to traditional factors of safety of 2-3) have been neglected. Additional testing in this range is required.
- The scatter in the degradation threshold should be investigated further by performing instrumented pile testing under high level unidirectional cyclic loading. The testing should be load controlled and extended to large cycles ($100<N<1000$) to assess whether effective stress changes are in fact limited to 2-way loading conditions.
- The loading rate mechanisms also require separation from the cyclic influences, which can be achieved by performing a range of static/CRP/cyclic tests on instrumented piles in uniform soil conditions.
- Finally the influence of soil type should be investigated further, particularly in light of the LDPT testing program.

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