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# Failure of a Barrette As Revealed in a Bidirectional Test

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**ABSTRACT.** A 1.5 x 2.8 m, 44.5 m deep barrette was constructed in weathered sedimentary rock of the Jurong formation in Singapore with a bidirectional-cell assembly of two cells installed at 33.5 m depth, 11 m above the barrette toe, chosen presuming that resistance above and below the cell level would balance. When the test was performed, a proof test at the end of the construction work, the shaft resistance for the section above the cell level was surprisingly low; the shaft failed at a load of only about 3 MN. In a re-test, at the maximum cell load of 12.5 MN, the upward movement exceeded 100 mm. The barrette was then tested in a conventional head-down test, which first (second test stage) was with the cell vented to validate the results of the O-cell test. In a third stage, the opening between the cell plates was grouted in order to transfer load across the cell level to mobilize the shaft resistance along the lower section of the barrette and the toe resistance. The results of Stage 1 bidirectional-cell upward test and Stage 2 head-down test on the pile with free-draining cell agreed well, as did the Stage 1 equivalent head-down test and Stage 3 head-down test. Stage 3 test confirmed a 12 MN working load, downgraded from the original 33 MN value. The test results are discussed and referenced to a numerical (FEM) analysis.

## 1. INTRODUCTION

The geology in Singapore requires that foundations supporting large load, such as from the many tall buildings constructed in the area, be supported on deep foundations, usually bored piles or barrettes (large rectangular cross-section units) constructed into the Jurong formation, a weathered sedimentary rock formation. In 2007, a 40-storey tower complex in western Singapore was designed on 33 barrettes constructed to depths of 45 to 50 m. As a part of the design, static loading tests to three times the desired working load were performed on special test barrettes of 2.8 x 0.8 m, 2.8x1.0 m, and 2.8x01.5 m cross section, and on one 2.0 m diameter circular cross section bored pile with desired working loads of 21.5 MN, 13.5 MN, and 21 MN "Ultimate Test Piles, UTP, to determine shaft and toe resistance responses. To validate the design and construction, one of the construction barrettes ("working piles") assigned a working load of 33 MN was selected for a proof test . All the tests were by means of the bidirectional-cell test.

Barrette BR15 cross section was 2.8 x 1.5 m and it was constructed to 44.5 m depth, about 25 m into the Jurong formation, as based on information from a borehole (BH-41) 3 m away. The bidirectional-cell test (Osterberg 1998) was carried out using two 670 mm diameter cells placed together at 33.5 m depth, 11 m above the barrette toe, where the shaft resistance along the 33.0 m length above the cell assembly level was expected to balance the combination of shaft resistance over the lower 11 m and toe resistance.

## 2. GEOLOGY AND SOIL PROFILE

The Jurong formation is a late Triassic to early Jurassic sedimentary deposit, and it covers the south, southwest, and western areas of Singapore. The formation has a variety of sharply folded sedimentary rocks, including conglomerate, sandstone, shale, mudstone, and dolomite (Rahardjo et al. 2004). Figure 1 shows the soil/rock log of two boreholes from the site, BH40 and BH41, where BH41 is about 3 m from the Test Barrette BR15. At BH-41, the soil profile consists of completely to highly weathered residual soils of the Jurong rock formation (Weathering Grade SVI to SIV) with SPT N-indices increasing linearly with depth. Below these soils, lies the Jurong sedimentary rock of mainly Siltstone RQD < 20 %, with weathering grades SIV to SIII (moderately to highly weathered) down to 25 m depth. Below 25m depth, lies mainly good sandstone with RQD ranging from 50 to 80% (slightly to moderately weathered — Weathering Grades SIII to SII). The groundwater table, GWT, lies about 2 to 3 m below ground level at Elev. +109 m.

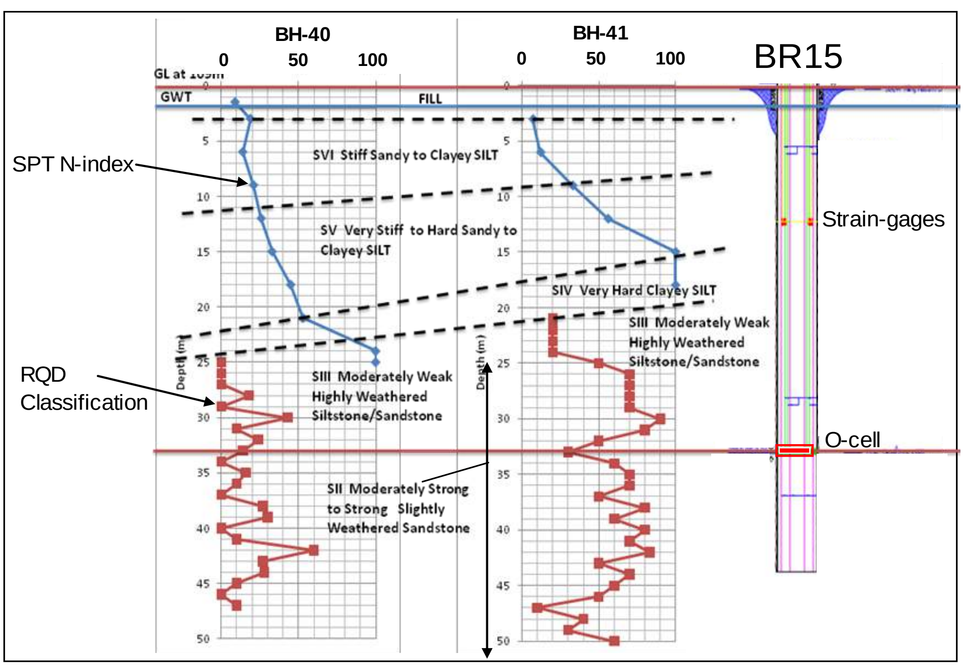



**Fig. 1 Jurong soil/rock profile near BR15 test barrette**

Field observations and geophysical surveys in several past projects show that due to the humid tropical condition in Singapore with high annual precipitation of more than 2 m per year, the thickness of the highly weathered soils typically ranges from 10 m to 30 m. For this particular site, the depth of soil weathered layers ranges from 20 m through 25 m (Figure 1). Figure 2 shows photos of a typical Jurong formation rock coring and the exposed face of a 30 m deep subway excavation about 5 km east of the test site.

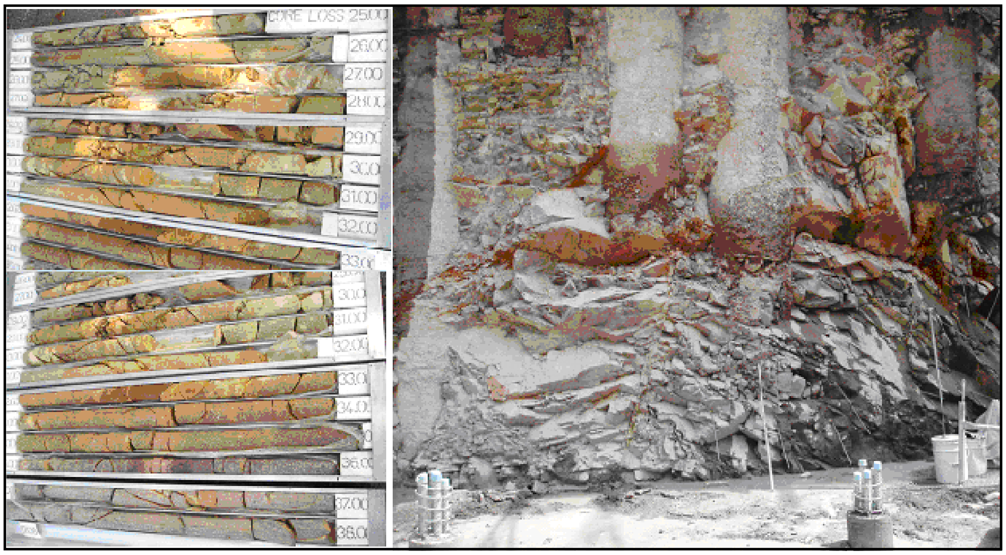


**Fig. 2 Typical coring log and exposed excavation into Jurong Siltstone/Sandstone**

## 3. BARRETTE DATA AND TEST ARRANGEMENT

The test barrette BR15, intended to be a working pile for the building, was constructed from September 15, 2007 through October 8, 2007 (a period of 23 days or 552 hours). The long time was due to the very difficult task of cutting through 20 m of very hard Sandstone rock. The barrette hole was excavated to a depth of 44.5 m with a 2.8 x 1.5 m cross-section diaphragm wall cutter. The hole walls were supported with bentonite slurry. After completing de-sanding operations and final cleaning with a conventional mechanical cleaning bucket, the reinforcing cage with Bidirectional assembly attached at 33 m depth was lowered into the slurry. Concrete was then delivered by tremie, displacing the slurry.

The Bidirectional assembly consisted of two 670 mm cells, calibrated special hydraulic jacks. The cell assemblies, with top and bottom plates in common, were placed at 11.0 m above the barrette toe. The load from the cell is obtained by means of hydraulic pressure from a pump at the ground surface using water as fluid. The load acts in two opposing directions, resisted by the shaft resistance of the pile above and combined shaft and toe resistance below. Theoretically, the cell does not impose an additional upward load or compression in the pile until the expansion force exceeds the buoyant weight of the pile above the cell plus any residual load (locked-in load) present at the cell assembly level. For the subject test pile, the buoyant weight above the cell level was 1.67 MN.

One set of 4 sister-bar vibrating wire strain gages (VWSG) was installed at the cut-off level (top of future pile cap) at 12 m depth to allow for an estimate of the average unit shaft resistance below the cap to the cell level. A set of 4 telltales (8 mm steel rods in 13 mm GI pipes) extensometer were installed near the pile toe, and at the cut-off level. An additional set of 4 telltales were installed at the top plate and another set at the bottom cell plate. The telltales provide adequately redundant measurements of the pile relative shortening or lengthening between the top plate and the cut-off level, and between the bottom plate and the pile toe level. All telltales were referenced by LVDTs at the pile head.

Four lengths of pipes were installed through the top plate to the bottom plate to vent the break in the pile during cell expansion. The pipes were also intended to be used for post-test grouting of the void between the cell plates created in the test to ensure that the barrette will serve in supporting the building. The opening in each cell was grouted by replacing the hydraulic fluid (water) with cement grout.

When evaluating a static loading test for large diameter bored piles and barrettes, the piling practice in Singapore applies the acceptance criterion of a “maximum pile head movement of 25 mm at an applied load of 1.5 times the desired sustained axial load — the "working load”. This rule also applies to the equivalent head-down load-movement curve of a bidirectional test. The equivalent curve is produced by adding and plotting the cell loads at equal upward and downward movements and considering the larger pile shortening occurring in a head-down test as opposed to that in a bidirectional test (Fellenius et al. 1999, Schmertmann 2000).

## 4. O-CELL TEST

The test programme was meant to be a single stage, bi-directional test to assess the upper shaft resistance and the combined lower shaft and toe resistances. The test was carried out as a quick maintained loading test applying a 1.67 MN load increment every 15 minutes to reach 25 MN maximum cell load in 15 load steps. A data acquisition unit recorded all values at intervals of two minutes.

The bidirectional test started on October 26, 2007. When attempting to increase to the third level of load, progressive upward movements developed, indicating shear failure along the barrette shaft above the cell assembly. The cell load was a mere 4.8 MN (3.1 MN after subtraction of the buoyant weight, 1.67 MN). The cell was then unloaded in a single step. Seven hours later, the test was restarted, showing non-abating upward movement occurring at every applied load increment. The barrette was unloaded when the upward movement had exceeded 100 mm. The cells were then unloaded in four decrements. The recorded load-movement curves are shown in Figure 3.

An estimate of the barrette stiffness based on a 7-day strength of Grade 35 concrete and the steel reinforcement indicated a stiffness, EA, of 110,000 MN. The barrette shortening determined from the telltale records showed that the concrete did not fail, and the observed upward shaft movement was indeed due to soil failure. The induced strains were too small, about 30 με, to be useful in the analysis.

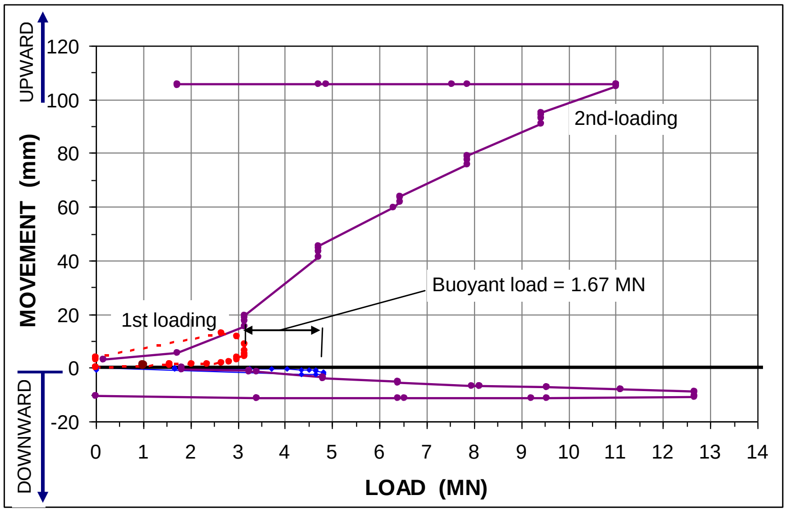



**Fig. 3 Results of bidirectional test on Barrette BR15**

It is not clear why the length above the cell assembly still managed in the re-loading to accept, albeit now with large movement, the increasing load without similar plunging response as that occurring in the virgin loading. Possibly, protrusions of the barrette along the shaft added some resistance as the barrette moved increasingly upward.

The mobilized shaft resistance over the upper shaft in the re-loading of the barrette ranged linearly from about 20 to 100 MN, which corresponds to an average unit shaft resistance of 30 to 60 kPa, is far below what one expects from a 25 m length of the barrette in the Jurong formation. The response of Barrette BR15 to the test is typical for shaft failure in a soil-bentonite cake formed on barrette shaft wall. This may be due to the very long standing time (553 hours; 23 days) required to construct the barrette socket in the very hard rock conditions. On the other hand, evidence from other barrette tests has shown barrettes to perform well despite long standing time. This suggests that BR15 shaft failure may be an accidental event.

Plainly, the results of the test, virgin phase test, cannot be used to produce an equivalent load-movement curve to use for assessing the response of the barrette to load. The engineers decided to downgrade the expected capacity for the barrettes and selected a working load of 11.77 MN (12 tonne). To determine if this would be a reliable working load, it was decided to grout the opening between the cell plates and inside the cell and then carry out a conventional head-down proof test on the barrette, which now would respond also along its lower length to the applied load. The test load reaction was obtained by jacking against a loaded kentledge.

Performing the head-down re-test in two phases, one before the grouting of the cells and one after, provided an opportunity to compare the results of the cell tests (virgin test and re-test) to that of conventional head-down tests. The bidirectional test engaged the soil shear along the upper length in negative direction, while a head-down test engages it in positive direction.

Stage 2, the head-down test on BR15 with the cell vented was performed on April 24-25, 2008. After grouting the cell opening, Stage 3 head-down test with the cell void grouted was performed on May 2-7, 2008. On April 11-16, 2008, a supplementary head-down test was carried out on an adjacent barrette, BR12, of the same cross section. Both head-down tests were limited to a maximum load of 1.5 times the downgraded Working Load (WL) of 11.77 MN.

About ten months before the BR15 proof-test, in December 2006 and January 2007, prior to finalizing the foundation design and construction of the working piles, bidirectional tests had also been performed on two specially constructed, 2.0 m x 0.8 m cross section barrettes named PTP LBWD, and PTP BR, and on a 2.0 m diameter bored pile named UTP4, constructed to depths of 30 m, 28 m, and 35 m, respectively. The maximum gross cell loads were 36 MN, 32 MN, and 37 MN, respectively, and the movements at equivalent head-down loads of 65 MN, 52 MN, and 75 MN were 12 mm, 30 mm, and 30 mm, respectively. The tests indicated succesful performance of the piles. Figure 4 shows the location of the three pre construction cell tests at the site in relation to test barrettes BR12 and BR15.

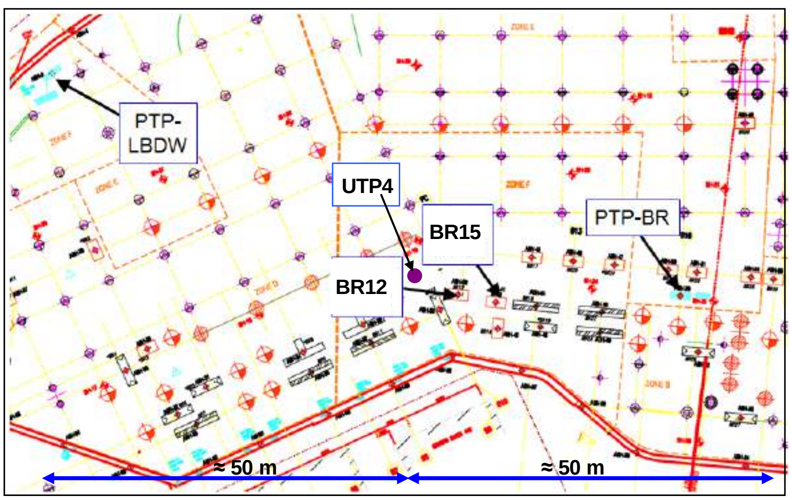



**Fig. 4 Test locations**

## 5. HEAD-DOWN TEST

As mentioned, the head-down test on Barrette BR15 was performed in two stages, Stages 2 and 3. Stage 2 was to test the upper shaft of the barrette with the cells opening created in Stage 1 vented. Following Stage 2, the void between the cell plates and opening inside each cell were pressure-grouted to enable the axial load to be transferred across the cell assembly. A wait period of 17 days (rapid strength gain cement was used) between the test stages was imposed to obtain adequate strength and stiffness before commencing Stage 3.

The BR-15 head-down tests were performed with manually operated hydraulic pumps and manual recording of loads and movements. Stage 2 head-down test started on April 24, 2008, six months after the bidirectional test. Increments of 100 KN were applied every 15 minutes. For Stage 3, the loads were applied in differing magnitude increments of 0.4 MN to 1.1 MN, which were maintained for 15 minutes with readings taken at start and end, only.

In Stage 2, when the sixth increment was being applied to increase the load from 3.4 to 4.4 MN, the barrette shaft failed, and the barrette started to unload rapidly on its own. The total head movement was then 1.1 mm, and the next reading (taken 15 minutes later) showed a movement of 12.7 mm at a remaining load of 2.0 MN.

Figure 5 shows the measured load-movement response of BR15, length above the cell level, in the head-down test together with the results of the virgin loading cycle of the cell test. In conformity with conventional mode of display, the cell load-movement curve is plotted after adjustment for the buoyant weight of the barrette, while the head-down curve does not include an adjustment for buoyant weight. The results of the two tests mobilizing shaft shear in opposing direction agree quite well considering the different method of testing and data collection.

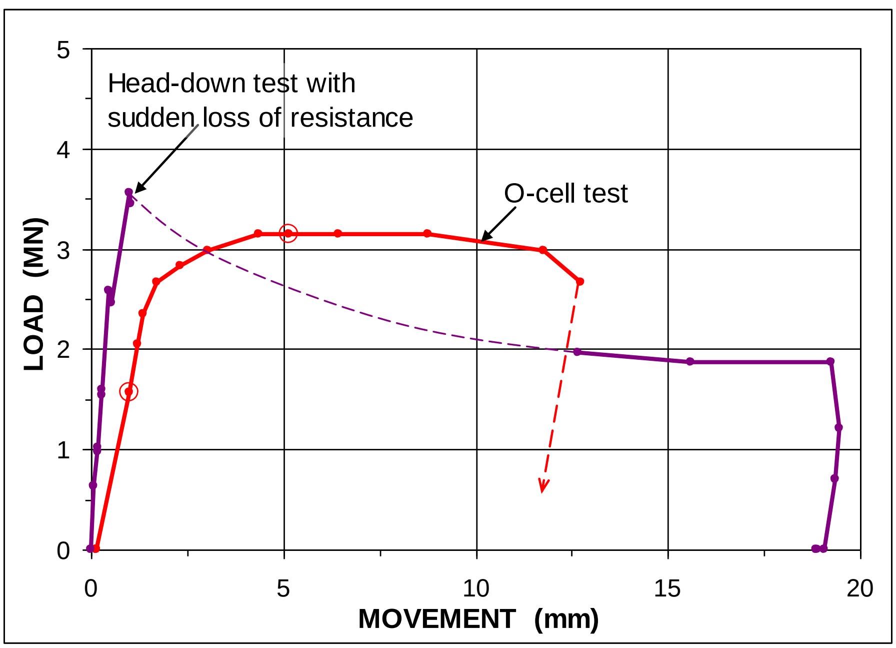



**Fig. 5 First Cycle bidirectional test and Stage 2 head-down test on BR15**

Stage 3, performed 7 days after grouting the cell void, consisted of loading the barrette in increments of about 3 MN to 100 % of WL, which load level was maintained for 24 hours to observe "creep" effects. Then, additional increments of load were applied until 150 % of WL (17.65 MN; 1,800 tonne), which load was maintained for 48 hours. After unloading to zero load, the barrette was reloaded to 17.65 MN and the load then held for 14 hours. Unloading records from this load are not available. Figure 6 shows the results of the Stage 2 head-down test, indicating that the Stage 2 head-down load-movement curve of BR15 has a much softer response to loading as opposed to that by the head-down test on BR12.

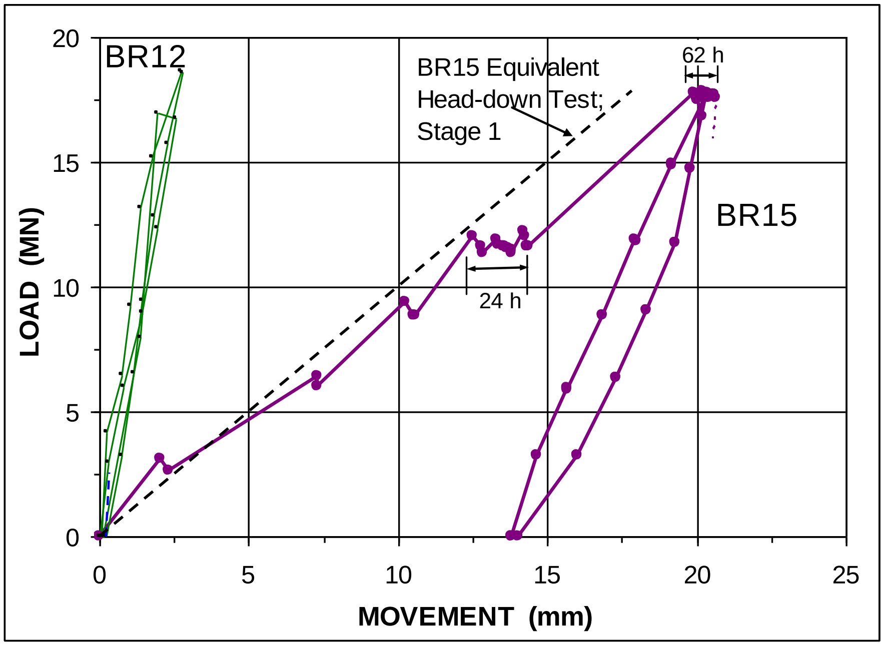



**Fig. 6 Results of Stage 3 head-down test and equivalent head-down curve from bidirectional test on BR15, and head-down test on BR12.**

Figure 6 also demonstrates a good agreement between the BR15 Stage 1 re-test equivalent head-down curve and the BR15 Stage 3 actual head-down response.

The results of the head-down test on BR15 contrast to the results of the mentioned two earlier barrette bidirectional tests at the site performed on 2.8 x 0.8 m section barrettes and on the bidirectional test on the 2.0 m diameter bored pile. In Figure 7, the equivalent head-down curves of BR12 and BR15 are shown together with the equivalent load-movement curves of the bidirectional tests on Barrettes BTP BR and PTP LBDW and on Pile UTP4. The BR12 curve has been extrapolated. All equivalent head-down curves plot above the 25 mm limit for 50 MN load.

Figure 7 shows comparisons between the results which imply that the soft response of Barrette BR15 may be an anomaly at the site. It would probably have been more advantageous to counteract the potential inadequacy of other barrettes by imposing construction and de-sanding controls rather than reducing the assigned working load on the barrettes and adding barrettes as then required. However, there was little choice, because Barrette BR15 was the 33rd and last barrette to be constructed at the site of the 33 barrettes of the original design. (Barrette BR12 was the 27th). The downgrading required adding a total of sixty-six 1.5-m diameter bored piles constructed to bearing in the Jurong formation.

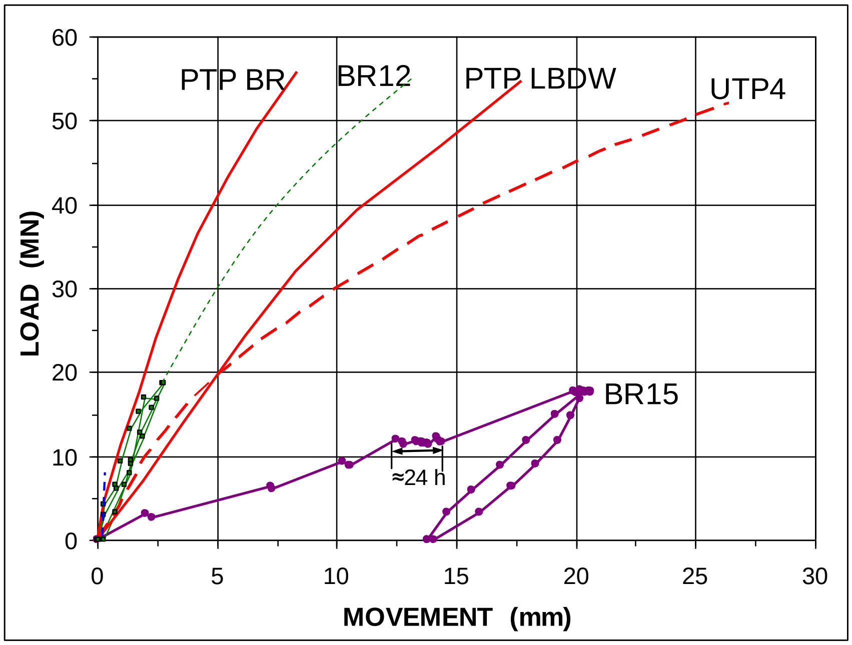


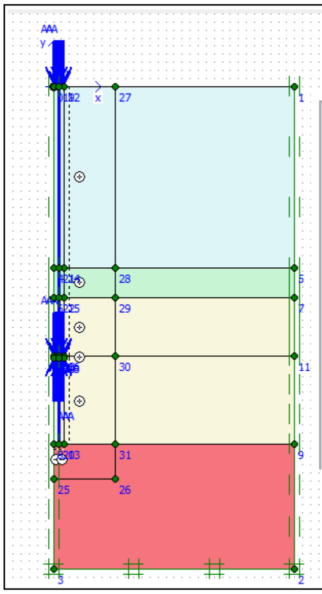

**Fig. 7 Equivalent head-down load-movement curves from bidirectional tests on Barrettes PTP BR and PTP LBDW, and bored pile UTP4 compared to results of head-down curves on Barrette BR12 and BR15-Stage 3. The curve for BR12 is shown extrapolated.**

## 6. FEM MODEL AND INTERPRETATION

### 6.1 FEM model of BR15 barrette

For further insight into the nature of this failed barrette, a detailed FEM analysis of the whole sequence of the barrette test was attempted. The barrette was modeled by an equivalent circular pile in axi-symmetry to capture the 3-dimensional nature of a pile. The axi-symmetric model maintains the same perimeter as the rectangular barrette (so that unit shaft resistance would be correctly estimated) and the same axial stiffness, EA, as the barrette. The soil model uses the Mohr-Coulomb elasto-plastic soil with parameters that have been shown suitable for modeling pile loading tests in head-down tests in Jurong formation, as shown in Figure 8. The model has been calibrated and used successfully for bidirectional-cell loading-tests in previous Singapore projects, e.g., Bui et al. (2005).

The stages of analysis are as follows: Stage 1 bidirectional test (initial and re-test), Stage 2 head-down test with the cell level vented, Stage 3 head-down test assuming that there is no opening in the barrette, i.e., cell opening is grouted.



| ID | Name | Type | $\gamma_{unsat}$ [kN/m$^3$] | $\gamma_{sat}$ [kN/m$^3$] |
|---|---|---|---|---|
| 1 | 1 Residual Soil | Drained | 17.0 | 18.0 |
| 2 | 2 Siltstone | Drained | 20.0 | 20.0 |
| 3 | 3 Sandstone 1 | Drained | 21.0 | 21.0 |
| 4 | 4 Sandstone 2 | Drained | 21.0 | 21.0 |

**Fig. 8 FEM model of BR15 for bidirectional and head-down tests**

## 6.2 FEM model results of O-Cell test

An interface element between the barrette and the soil represents the shaft resistance transfer from barrette to soil. The strength of the interface at which plastic soil slip will occur is given by the interface factor (R_inter) times the Mohr-Coulomb strength of the soils (c' + σ' tan φ). The stiffness of the interface is governed by the soil stiffness modified by R_inter-value, allowing for lower shear and compression stiffness, so that soil slip relative to the barrette will be computed in the interface elements.

For typical pile response in stiff soils and rocks, a value of R_inter equal to 0.5 would usually apply as in the "alpha" method of pile analysis. However, in the case of a debonded shaft, one can assume that the soft soil cake must have reduced the R-inter-values to very low numbers, e.g., 0.05 to 0.15. This is tested in the FEM model by comparing the pile response with the results calculated from a range of R_inter-values: 0.50, 0.30, 0.15, 0.10, and 0.05.

Figure 9 shows the results of the upper shaft movement versus the imposed cell loads for various R_inter values. From the plot, it appears that the failed BR15 barrette would have an interface factor reduced from the expected response of 0.50 to a very low value of 0.05 to 0.10.

A plot of the interface distribution of shaft resistance with depth is shown in Figure 10. When R_inter-value is reduced to a range of 0.05 to 0.10, the average unit shaft resistance over the upper shaft is about 30 to 60 kPa, which compares well with values back-calculated from the actual Stage 3 head-down loading test.

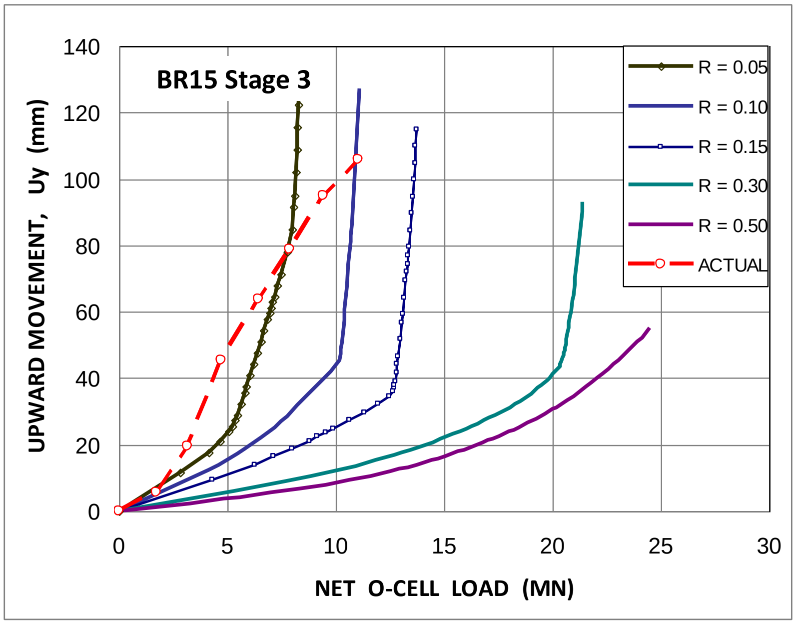



**Fig. 9** **FEM simulations of BR15 Stage 3 test with range of R_inter values. Applied cell load versus upward movement of upper cell plate.**

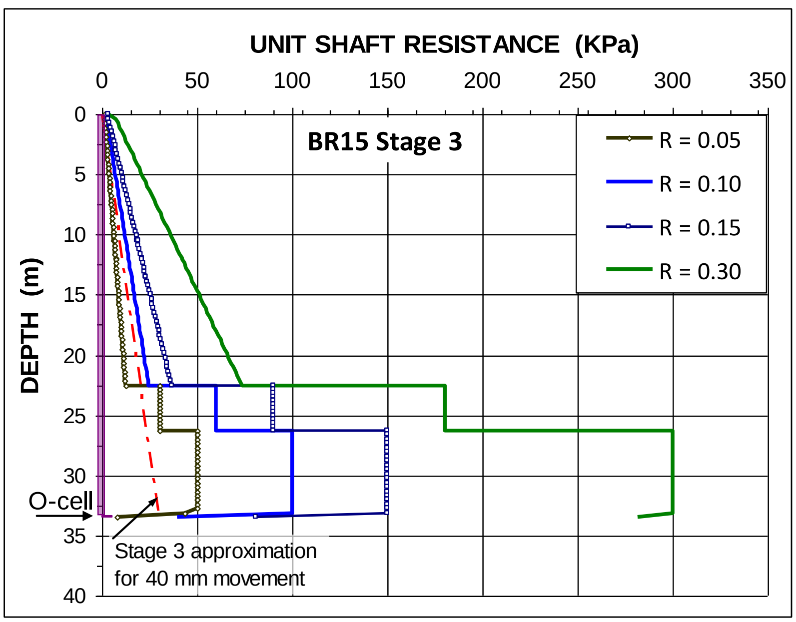



**Fig. 10** **Unit shaft resistance distributions for a range of R_inter values simulating BR15 Stage 3 test.**

Figure 11 shows calculated load distribution in the barrette at the maximum barrette head movements of about 120 mm and 40 mm, respectively, as simulated for R-inter-values of 0.05 through 0.30. [It is unfortunate that strain gages were installed at the cut-off level only (12m depth from pile head) and that due to the small strains in Stage 1 and the non-abating movements in Stage 2, the strain data are not useful].

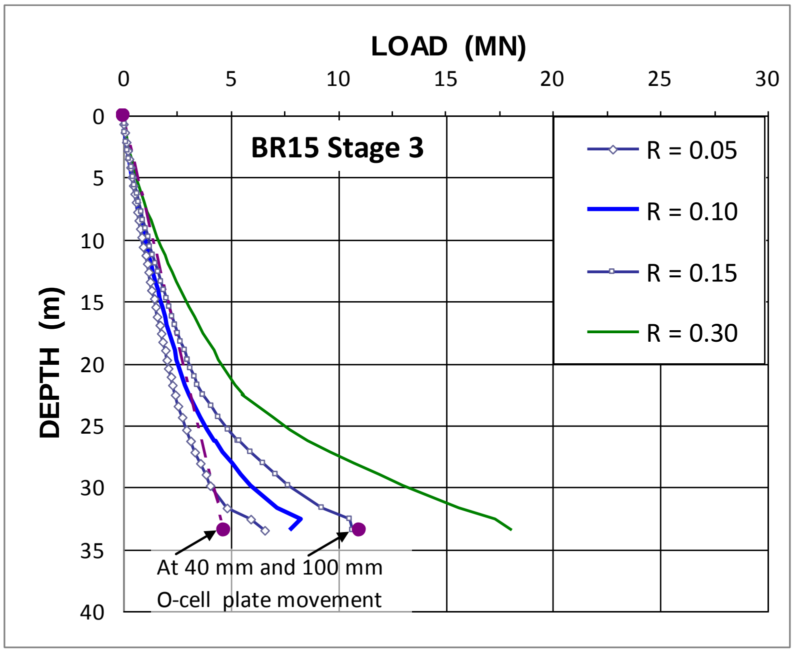



**Fig. 11 Axial force distribution for a range of R_inter values simulating BR15 Stage 3 test.**

### 6.3 FEM model results of head-down retest

Figure 12 shows that the FEM modeling of the results of the Stage 2 test did not manage to simulate the small relative pile/soil displacements needed to mobilized the upper shaft resistance. However, it does show that the upper shaft is likely to fail between 2.3 and 4.8 MN applied load, which range encompasses the measured actual load of about 3.5 MN.

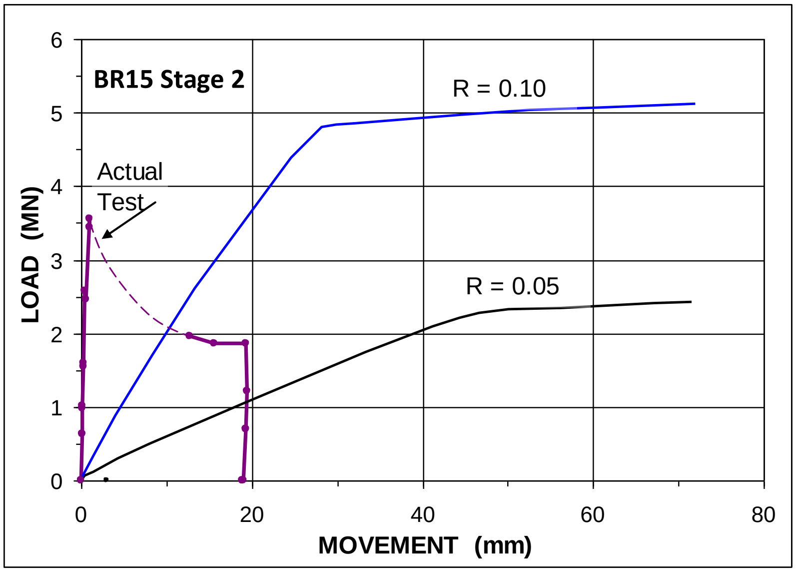



**Fig. 12 Head-down load-movement response for R_inter values of 0.05 and 0.10 simulating BR15 Stage 2 test and actual test values.**

## 6.4 FEM model results of conventional head-down test

When the Stage 3 head-down test was modeled, it produced the load-movement response shown in Figure 13. At a load of 20 MN, the estimated barrette head movement would be between 12 and 17 mm for R_inter-values between 0.50 and 0.05. In the actual Stage 3 test, the movement response to 20 MN applied load was 20 mm.

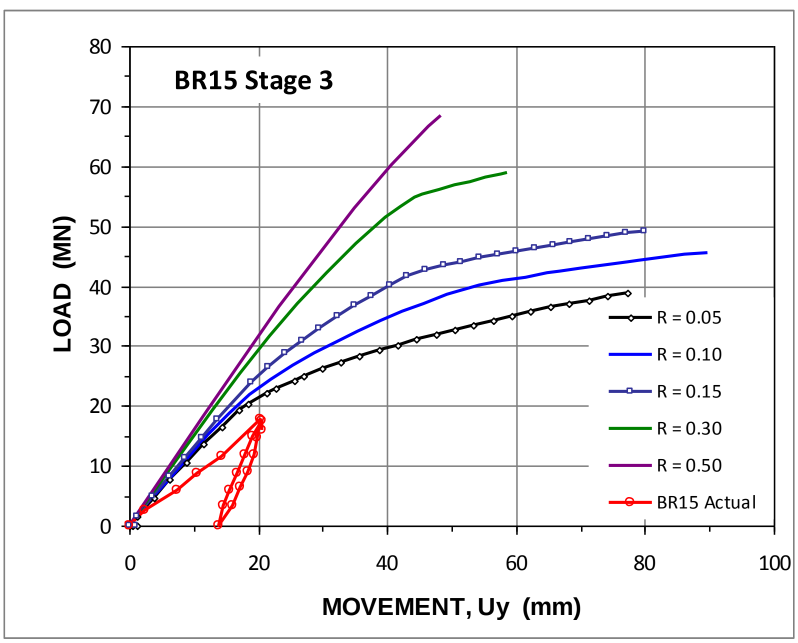



**Fig. 13 Head-down load-movement response for R_inter values of 0.05 and 0.10 simulating BR15 Stage 3 test and actual test values.**

The results of the equivalent head-down curve of Stage 1, re-test, the Stage 3 actual head-down test, and the FEM simulations agree well. It would appear that Barrette BR15 could support a working load of up to 20 MN as opposed to the conservative value of 12 MN actually adopted.

## 7. CONCLUSIONS

1. A large barrette was accidentally de-bonded during construction likely due to failure of the maintenance of the de-sanding process for the bentonite slurry. As a result, a weak soil cake layer was formed around the upper barrette shaft leading to premature failure of the upper shaft in the bidirectional test.

2. The bidirectional tests showed that the upper shaft could mobilize only between 30 and 60 kPa unit shaft resistance in very stiff residual soils and hard siltstones/sandstones.

3. Retest of the failed barrette in head-down loading with the bidirectional opening vented confirmed barrette de-bonding.

4. After grouting bidirectional gap, a head-down proof-test to 1.5 times the assigned new (reduced) working load of 12 MN proved acceptance of the new load.

5. FEM analysis of the complete sequence of tests showed that the barrette would have been de-bonded to between 10 to 15% of its interface shear force to produce the barrette response observed in the bidirectional test.

6. The FEM analysis together with the Stage 3 test results and the equivalent head-down curve from the Stage 1 bidirectional test suggest that the failed barrette could have been accepted for a working load of 20 MN as opposed to the actually assigned 12 MN.

7. The FEM analysis and the proof test on Barrette BR12 indicates that the failure of Barrette BR15, may be an anomaly at the site and the barrette foundations did not have to be downgraded had it only been possible to instead implement an improved construction verification of the de-sanding process. However, this requisite was discovered too late in the construction process.

8. The bidirectional test on Barrette BR12 did not have to be terminated at the proof-test load of 1.5 times the presumed working load. Had the test been continued, which would have required neither additional time nor expense, the assessment of the barrettes would have been further enhanced.

## ACKNOWLEDGEMENTS

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