

AN INNOVATIVE METHOD OF ASSESSING THE CAPACITY OF EXISTING WHARF PILES

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ABSTRACT

North Queensland Bulk Ports (NQBP) is upgrading the Mackay Outer Harbour wharves. The work involves a fender replacement and bollard upgrading programme. As a result of the proposed upgrade, the axial capacity of the fender piles at Wharves 1, 4 and 5 is increased from the original design requirements.

The existing piles beneath the wharf deck are driven steel tubular piles and they range in diameter from 0.457 m for Wharves 1 and 4, to 1.0 m for Wharf 5. Unfortunately, no construction records could be found in relation to as-constructed pile toe levels, final driving energy and sets or pile load testing. Therefore, the as-installed pile capacities could not be readily assessed. Using conventional theoretical assessment methods, the design consultant concluded the existing piles would not have adequate capacity to support the increased axial loads. The challenge was to investigate and assess whether the existing fender piles can support the increased axial loads associated with the wharf upgrade, and if not, what are the options for remediation.

This paper describes the investigation and assessment of the axial capacity of the existing piles using a combination of innovative downhole magnetic testing adjacent to the existing piles to assess the as-constructed pile toe levels, conventional coring to assess the founding materials, and dynamic pile load tests on new prototype piles driven close to the existing piles. Using the innovative investigation and assessment technique, the existing piles were assessed to have adequate capacity and successfully avoided the costly option of remedial works for the client.

1. INTRODUCTION

The Mackay Outer Harbour is situated in Northern Queensland of Australia, and was officially opened on 26 August 1939 (see Figure 1). The deep water port became an important facility for the thriving trade of a growing district. In 1957, the bulk sugar handling terminal was open, servicing the sugar industry that started in the region in 1865.



Figure 1: Official Opening of Mackay Outer Harbour in 1939 (source: Daily Mercury <https://www.dailymercury.com.au/news/mackay-harbour-70-today/304046/>)

Since its opening, various phases of harbor expansion took place under McKay Harbour Board. Wharf No. 4 was constructed circa 1967 and Wharf No. 5 followed later under McKay Harbour Ltd which changed to North Queensland Bulk Ports in 2009. The locations of Wharves 1, 4 and 5 are shown in Figure 2.



Figure 2: Site Plan showing Wharf and Borehole Locations

In 2017, NQBP commenced upgrade work on wharves 1, 4 and 5 involves a fender replacement and bollard upgrading programme. As a result of the proposed upgrade, the axial capacity of the fender piles at Wharves 1, 4 and 5 is increased from the original design requirements. The piles supporting the wharves are driven tubular steel piles ranging from 457 mm diameter at Wharves Nos. 1 and 4, to 1000 mm diameter at Wharf No. 5. The required pile capacities are compared to estimated pile capacities made in 2016 by another geotechnical consultant as summarized in Table 1.

Table 1: Pile Loads and Initial Estimate of Capacity

Statistics	Design Ultimate (kN)	Available Design Ultimate Capacity ⁽¹⁾ (kN)
Compression Row C	1,135	650
Compression Row D	1,364	975
Tension	426	225

⁽¹⁾ Estimated by another geotechnical consultant in 2016

Due to the lack of information on as-constructed pile toe levels, final driving energy and sets or pile load testing at the wharves in question, doubts were raised on the ability of the existing piles to carry the revised loads based on the theoretical assessment results shown in Table 1. Contrary to the theoretical assessment, however, a previous dynamic pile load test with CAPWAP assessment on a pile installed at Wharf 3 gave a compression capacity of a 1000mm diameter pile of 1,900 kN in shaft resistance and 5,380 kN end resistance, i.e. 7,280 kN total capacity (or a design ultimate capacity of 798 kN in tension and 5,096 kN using a tension to compression capacity ratio, R_{st} of 0.6 (based on De Nicola and Randolph (1993) and a geotechnical strength reduction factor ϕ_g of 0.7). This is significantly higher than the theoretical estimate made by the other consultant and therefore gave at least some confidence that the existing piles may be able to carry the revised loading associated with the upgrade without remedial works. To confirm this initially assessment, the following programme of investigation was carried out:

- Drilling of 3 boreholes, one at each of Wharf 1, 4 and 5 adjacent to existing piles at the locations shown in Figure 2.
- Conduct downhole magnetic testing in PVC access tubes installed at the completion of the boreholes to assess the toe levels of the adjacent piles.
- Install 457 mm diameter tubular steel test piles to the assessed pile toe levels.
- Conduct dynamic pile load testing and CAPWAP analyses (Rausche et al, 1985) to assess the ultimate pile capacity of the test piles.

The test results and subsequent interpretations are discussed below.

2. BOREHOLE AND DOWNHOLE MAGNETIC TESTING

Three boreholes were drilled at the approximate locations shown in Figure 2. It had originally been intended to drill the boreholes within 1 m of the external face of the existing pile for the downhole magnetic testing. Unfortunately, that was not possible due to site access constraints and the boreholes were drilled at distances ranging from 1.3 m to 1.6 m. Beyond 1 m distance, the accuracy of the magnetics testing reduces.

Downhole magnetics testing is a passive method based on the measurement of localised perturbations of the Earth's magnetic field measured in nano-tesla's (nT). These may be caused by geological features and buried ferrous targets (eg, pipes, cables, drums, iron sheets, steel reinforcement etc).

The magnetics testing was completed in-situ using pre-drilled, PVC cased boreholes. A Bartington 3-component fluxgate magnetometer was used to measure the magnetic field variations at 0.5m intervals to the maximum depth of the borehole. The magnetic data was acquired with Spectromag-6 software.

The borehole magnetometer was lowered from the top of each borehole (from the wharf deck) and repeated magnetic measurements were made at each depth to assist assessment of any external magnetic noise.

The borehole magnetometer measures magnetic field intensity in three dimensions (X, Y and Z) with the vertical (Z) component of the magnetic field typically providing the clearest indication of the base of steel piles and hence the approximate depth of the pile. Furthermore, the vertical first derivative of the vertical component of the magnetic field can also reliably indicate the base of a steel pile. Figure 3 shows a typical magnetic response of a pile as sensed from a proximal borehole adjacent to the pile, including the vertical component and vertical first derivative of the magnetic response. This shows that a magnetic anomaly indicates the approximate base of the pile.

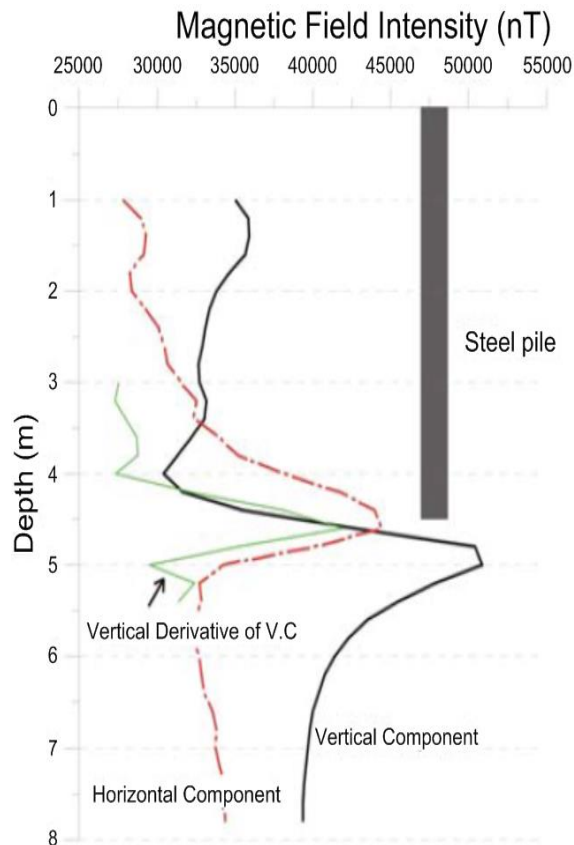


Figure 3: Typical Magnetic Response of a Steel Pile (from Jo. *et al.*, 2003)

3. SUBSURFACE PROFILE AND INFERRED PILE TOE LEVELS

The results and interpretation of the downhole magnetics testing are shown in Figure 4 below. It can be seen that the clearest magnetic response change was obtained at Wharf 4 where the borehole was at the least distance away from the face of the nearest pile.

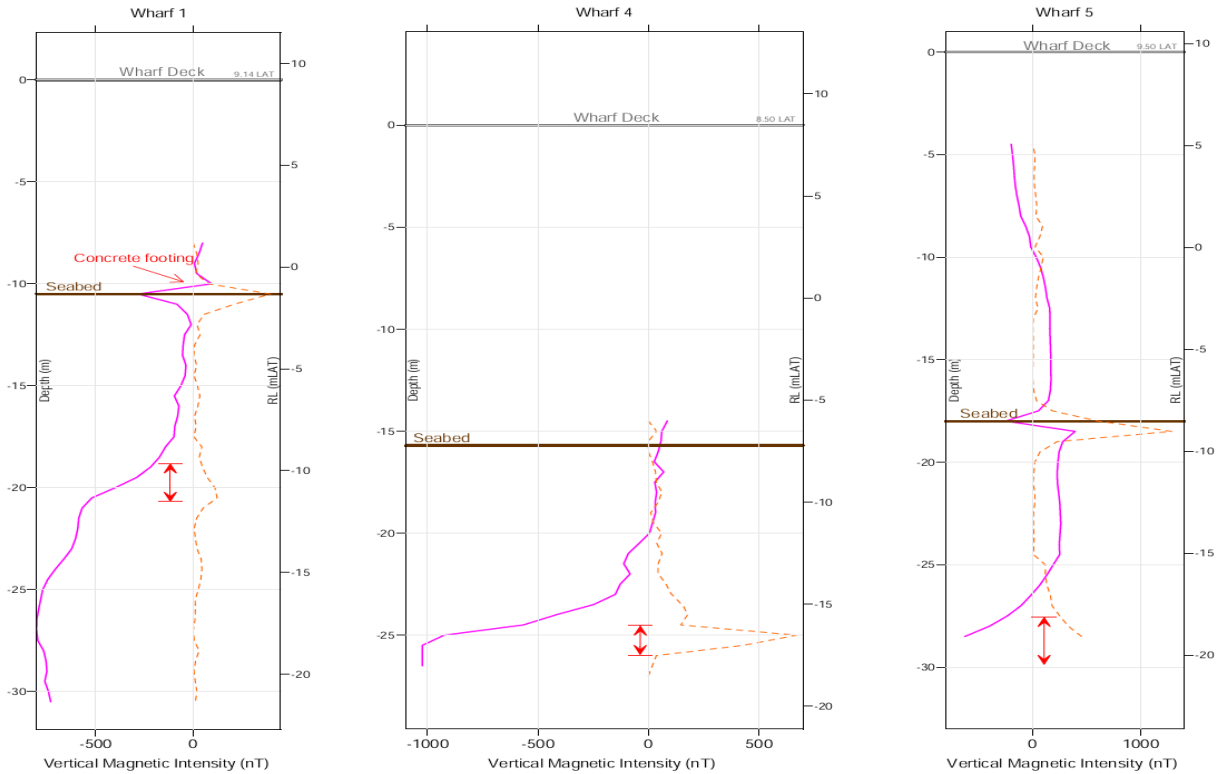


Figure 4: Results of Downhole Magnetics Testing

The results of the borehole drilling and inferred pile toe levels at each borehole location based on interpretation of the downhole magnetics testing are summarized in Table 2.

Table 2: Summary of Borehole and Downhole Magnetics Investigation Results

Borehole	Approximate Distance to Nearest Existing Pile (m)	Inferred Pile Toe Level Range ⁽¹⁾ from Magnetics Testing (mLAT)	Materials Recorded in Borehole and Standard Penetration Test Results
Wharf 1	1.4	-9.7 to -11.6 (-12.8) ⁽²⁾	Gravelly Clayey Sand and Clayey Sand, SPT-N = 21 and 26 above -11.6 m LAT SPT-N = 39 at -12.8 m LAT
Wharf 4	1.3	-15.0 to -17.4	Gravelly Sand and Clayey Gravelly Sand (Residual Soil), SPT-N = "R" (30 blows per 145 mm penetration)
Wharf 5	1.6	Below -18.1	Sandy Gravelly Clay and Sandy Gravel, SPT-N = "R" (30 blows per 100 mm penetration)

- (1) Definitive levels cannot be inferred due to attenuation of magnetic intensity with distance away from the pile, and potential influence from other piles that have different toe levels.
- (2) Based on SPT-N value, it is unlikely that the pile would have terminated in SP-N of 21 to 26 material. Therefore, the next SPT of N = 39 at 22 m depth (-12.8 mLAT) is considered to be a more reasonable estimate.

4. TEST PILE AND CAPWAP ANALYSES

The proposed methodology of testing originally proposed was that the test piles should have been installed to a similar pile toe level as assessed by the magnetics testing. However, due to logistics reasons, the test piles were installed before the magnetics testing took place, and they were generally driven to a level lower than the pile toe levels inferred from the magnetics testing.

The test piles comprised 457 mm OD steel tubular piles with a wall thickness of 15.9 mm, and they were driven using a Junttan HHK7A hammer. The average final sets under a final drop height of 1.2 m and the corresponding materials at the end of drive toe level from the nearest boreholes are tabulated in Table 3 and the CAPWAP analysis results of the test piles are summarized in Table 4.

Table 3: Test Pile Average Final Set and Founding Material

Test Pile	Installed Pile Toe Level (mLAT)	Average Final Set (mm/blow)	Founding Material Inferred from Nearest Boreholes
TP1 – Wharf 1	-16.553	1.6	Clayey Sandy Gravel, Residual Soil, SPT-N = “R” (30 blows / 60 mm penetration)
TP4 – Wharf 4	-17.558	2.1	Extremely Weathered Rock, SPT-N = “R” (30 blows / 60 mm penetration)
TP5 – Wharf 5	-19.169	2.2	Sandy Gravel, SPT-N = “R” (30 blows / 100 mm penetration)

Table 4: Results of Test Pile CAPWAP Analysis

Test Pile	Inferred EOD Capacity ⁽¹⁾ (kN)			Inferred Restrike Capacity ⁽²⁾ (kN)		
	Shaft	Base	Total	Shaft	Base	Total
TP1 – Wharf 1	-	-	-	2273	2685	4958
TP4 – Wharf 4	1674	2053	3727	1716	2304	4020
TP5 – Wharf 5	2169	2385	4554	1984	2338	4332

It can be seen from Table 4 that the test pile capacities are sufficiently high (even after accounting for geotechnical strength reduction and tension to compression reduction factors) to deal with the increased loads associated with the wharf upgrade work. However, because the test piles are driven into rock and founded below the existing pile toe levels inferred from the downhole magnetics testing, direct application of the dynamic pile load testing to assess the existing pile capacities was not possible. Further assessment of the pile capacities by correlation of shaft friction and end bearing capacities for the soil and rock layers intersected by the boreholes was necessary as discussed in the following section.

5. ASSESSMENT OF EXISTING PILE CAPACITIES

As a cautious approach, an assumption was made to assess the situation where the pile toes of the existing piles are not founded into or close to rock level. The assessment was made as follows:

- Assume the pile toe is located at the average of the range assessed by the magnetics testing, or the best estimate for TPI as shown in Note 2 of Table 2.
- For driven steel tubular piles, the phenomenon of “friction fatigue” as described by White and Lehane (2004) results in the pile shaft capacity mainly derived from about 10 x pile diameters (i.e. say 4.5 m) above the pile toe. This is the section where adjustment of SPT-N value and shaft capacity will take place. The ultimate shaft friction in this section were back-analysed from the CAPWAP results using the correlation of SPT-N values by DeCourt (1982) who suggested $f_s = K_s N + 10$ (kPa) where K_s is typically about 3.3 for displacement piles, although this was later reduced to 2.8 in Decourt (1995). Based on the CAPWAP results, however, we inferred a K_s value of about 3.4 at this site as shown in Figure 5.

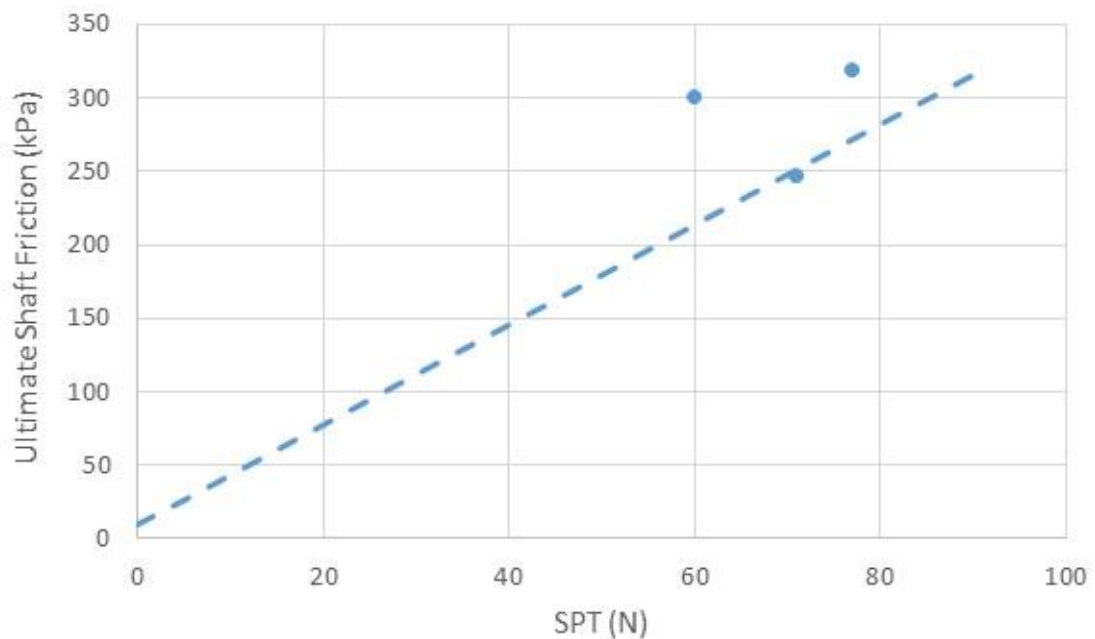


Figure 5: Back-analysed Relationship between Ultimate Shaft Friction and SPT-N

- For the shaft above 4.5 m above the test pile toe, the shaft friction from the CAPWAP testing was relatively low, and based on the SPT results, average values of 28 kPa to 45 kPa was adopted for the pile assessment for the upper part of the pile shaft, ignoring any shaft friction in soft to firm soils.
- For the ultimate end bearing pressure, the CAPWAP results were back-analysed using the correlation of SPT-N values by Decourt (1982) who suggested $f_b = K_b N$ (kPa) for driven piles where $K_b = 250$ for sandy silt and 400 for sands, although the value for sands was later reduced to 325 for sands in Decourt (1995). From the test results, we adopted a correlation of $f_b = 300 N^{0.77}$ as shown in Figure 6. This relationship was then applied to the inferred existing pile toe levels where SPT values may be lower than those below the toes of the test piles.
- Finally, reduction factors, ϕ_g and R_{st} as described in Section 1 above were applied to compute the design ultimate pile capacity presented in Table 5.

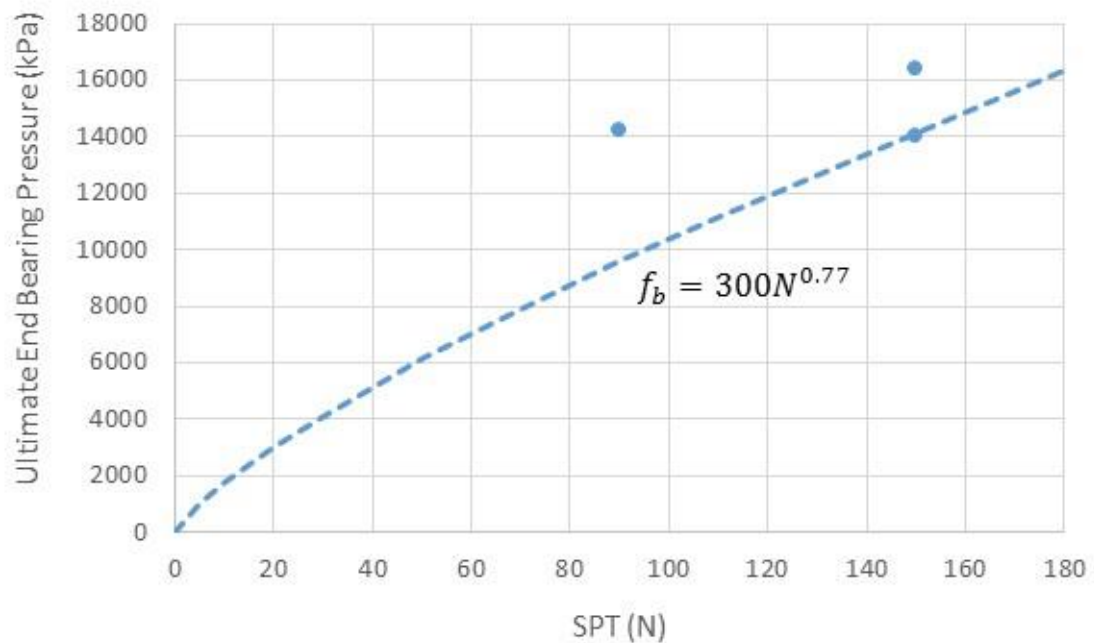


Figure 6: Back-analysed Relationship between Ultimate End Bearing Pressure and SPT-N

Table 5: Pile Capacity Assessment Results

Statistics	Wharf 1	Wharf 4	Wharf 5
Length of bottom 4.5 m	4.5	4.5	4.5
Length of upper pile shaft	5.5	3.2	4.5
Ultimate End Bearing Pressure (kPa)	4,255	5,804	6,514
Ultimate Compression Shaft Friction in upper section of pile shaft (kPa)	28	34	45
Ultimate Compression Shaft Friction in bottom 4.5 m of pile shaft (kPa)	120	185	202
Assessed Ultimate Shaft Compression Capacity (kN)	997	1,351	2,382 ⁽²⁾
Assessed Ultimate Compression Base Capacity (kN)	698	952	5,116 ⁽²⁾
Assessed Ultimate Compression Pile Capacity (kN)	1,695	2,304	7,498 ⁽²⁾
Assessed Design Ultimate Compression Capacity ⁽³⁾ (kN)	1,187	1,613	5,249 ⁽²⁾
Assessed Design Ultimate Tension Capacity ⁽³⁾ (kN)	419	568	1,000 ⁽²⁾

⁽¹⁾ Based on restrrike CAPWAP analysis result (see Table 4)

⁽²⁾ Based on 1,000 mm diameter existing pile at Wharf 5. Pile diameter = 457 mm for all other piles

⁽³⁾ Design ultimate values including a ϕ_g value of 0.7 and a R_{st} ratio of 0.6 for tension capacity

Based on the assessment results shown in Table 5, only the Wharf 1 Row D piles were found to have design ultimate capacities slightly less than the required capacity of 1,364 kN for compression and 426 kN in tension. Following a review of the results with the marine/structural engineer for the project, it was considered that the existing piles are considered to be acceptable based on the following reason:

- The test piles were installed off the existing wharf and therefore slightly more seawards compared to the existing piles and the boreholes which were drilled from the wharf decks. Therefore, it is feasible that the bedrock is slightly deeper at the test pile locations compared to the existing piles which may also have been driven to rock.
- The downhole magnetics testing may be influenced by other piles further in-shore and which are shorter due to shallower depth to bedrock.
- The ultimate static end bearing capacity of the test piles are likely to be higher than those inferred from the dynamic pile load testing because during dynamic testing, the steel tubular pile will behave in an “unplugged” manner. Under static loading conditions, the steel tubular pile is likely to behave in a “plugged” manner thereby giving a much greater end bearing area.
- The inferred ultimate shaft capacity of the test piles will provide a reasonable estimate of the lower-bound unit shaft resistance of existing piles because the piles would not have sufficient time to “set up” with the restrike testing conducted only 24 hrs to 48 hrs after end of drive. Furthermore, minor rusting/aging of the steel surface over time has been shown to increase the ultimate shaft resistance of tubular piles.

As a result, no remedial works were deemed to be required for the proposed wharf upgrade works.

6. CONCLUSIONS

The innovative approach of combining conventional investigations with geophysical techniques using downhole magnetics testing, and dynamic pile load testing has enabled the toe levels of existing wharf piles to be assessed. Through the interpretation of unit shaft and base resistance derived from the test piles and adjustments made according to standard penetration test values at the inferred pile toe levels, the capacity of the existing piles were able to be made and compared to the increased load requirements associated with the wharf upgrade. In doing so, the existing piles were assessed to have adequate capacity and successfully avoided the costly option of remedial works for the client.

7. ACKNOWLEDGEMENT

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