Embankment stabilisation on Van Reenen's Pass, Kwa-Zulu Natal

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ABSTRACT: The National Route 3 highway is the main artery between the hubs of Johannesburg and Durban. This route passes through the Van Reenen's Pass on the border of Free State and Kwa-Zulu Natal, an area which has historically been noted for slope instability, and with little alternatives for diverting traffic. One such area of historic slope instability below the roadway showed clear signs of settlement and cracking in the 2021-22 period. This paper discusses the investigation, design, and construction of a piled solution intended to protect the roadway from progressive slip caused by the downslope instability. Special attention is paid to the underground morphology, subsurface water regime, and strength of fill, colluvium and weathered rock layers which lie at the root of the instability.

1 INTRODUCTION

National Route 3 highway (N3) serves as the primary connection between South Africa's largest economic hub in Gauteng and Africa's largest port in Durban. A significant portion of the route is operated and maintained by the N3 Toll Concession (N3TC), spanning a 415 km stretch from the Cedara Interchange near Pietermaritzburg in Kwa-Zulu Natal to the Heidelberg South Interchange in Gauteng. This route passes through the Van Reenen's Pass on the border of Free State and Kwa-Zulu Natal, an area which has historically been noted for slope instability with little alternatives for diverting traffic.

A number of roadside slopes are annually assessed as part of N3TC's ongoing geotechnical management system. One of these slopes, located on the Van Reenen's Pass showed evidence of instability after heavy rains in the 2021-22 period.

This paper aims to present a case study concerning the investigation, design, and construction of mitigating measures on this challenging site.

2 SITE CHARACTERISATION

2.1 *Geological Setting*

The site lies at an elevation of ~1650 m MASL at the foothills of the escarpment. This area is characterised by rolling hills, small streams, and erosion gullies towards the upper escarpment.

Sedimentary rocks of the Adelaide Sub-group underlay the site, this sub-group is represented by a single formation in this area, known as the Normandien Formation. This forms part of the Karoo Supergroup which has further been intruded by dolerites of the Karoo Dolerite Suite, occurring in the form of dykes and sills. Due to the mountainous topography, colluvium and talus have been deposited on the residual layers. The residual horizons formed through chemical decomposition of the bedrock during the Cenozoic period when the historical escarpment existed closer to the current coast line (Knight & Grab 2015). A gulley is present in the middle of the site, running westwards under both the N3 roadway and downslope area. Based on previous experience, early- and mid-20th century roadbuilders did not always take care to clear these loose colluvium/talus layers before placing fill, which has caused problems on similar slopes in the past.

2.2 *History and Site Layout*

The current N3 route was built in the 1970s as a multiple carriage roadway, replacing the earlier single lane roadway that had been in use for much of the early 20th century. Different generations of infrastructure development therefore lie on variable mountainous topography that have been shaped through the ages. These factors contribute to the complexity of understanding and working on this site, as records are not available prior to the 1970s. A cross section of the site is presented in Figure 1. Of note in this section is the current N3 roadway, as well as the old roadway below it.

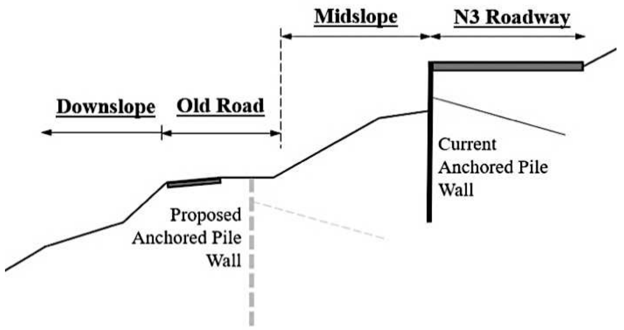

Figure 1. Named areas of site, in section.

Cracking in the old road in 2021 (Fig. 2) necessitated reviewing the history of both the current N3 roadway and the old road itself. There records of slope stability problems on this site dating back to the early 1990's as the Department of Transport commissioned in-depth investigations on the causes of cracking and settlements in both the current N3 roadway and old roadway. As a result of the deteriorating situation, N3TC undertook stabilisation of the N3 roadway once they had been awarded the Concession Contract. Stabilisation and rehabilitation of the N3 roadway was achieved with an anchored pile wall system, completed 2003.

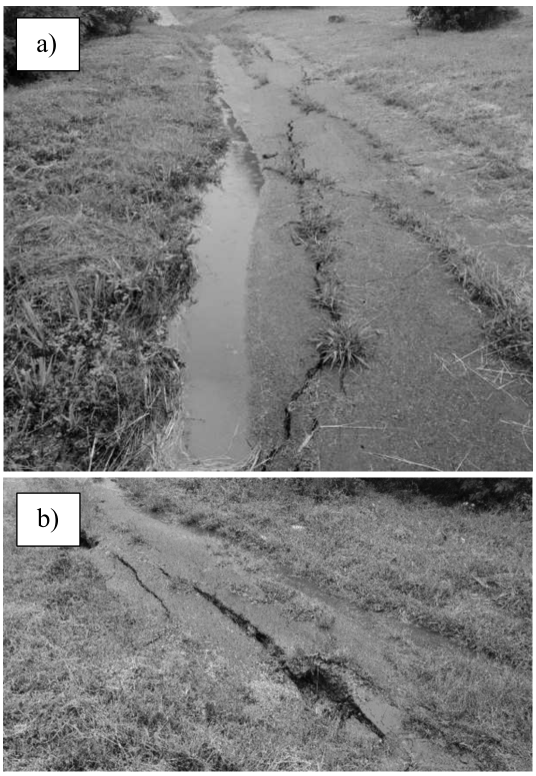

Figure 2. Old road in 2021 after heavy rain a) longitudinal cracking and settlement, b) open cracks and voids.

The cracking and settlement in the old road (now simply a cattle path) downslope from the N3 roadway continued to be monitored, but was not addressed at this time as downslope movements appeared to have stopped, and the N3 roadway had been secured by the installation of the pile wall.

The anchored pile wall has been successful in preventing settlements in the N3 roadway, but in 2021 the renewed cracking was noted on the old road after particularly intense rains. The distance between the old and current roadways meant that this did not immediately affect the current roadway, but this cracking was nevertheless treated, sealed and inspected regularly. Cracks on the old road subsequently reopened after heavy rains in 2022, whereupon it was decided that further investigation and structural intervention was required.

2.3 *Investigation Findings*

A total of nine boreholes were drilled to complement the historical borehole data from the 1993 investigation. The boreholes in the 2023 investigation were primarily focused on establishing ground conditions on the downslope area, as this area had not been investigated during the previous investigations. Figure 3 presents a general site layout with boreholes.

Boreholes confirmed the presence of a deep fill and colluvium (~20 m to very soft rock) on the old road towards the north of the site. However, the deep fill and colluvium layers in the downslope were not as extensive across the downslope area as initially anticipated. Both boreholes on the northern and southern ends of the downslope area found much shallower depths to bedrock (10 m - north, 3 m - south).

The borehole investigations were supplemented with geophysical investigations using three traverse paths across site (along the old road, downslope parallel to the cattle path, and perpendicular to the first two).

Depth to bedrock from geophysics was found to differ substantially across the cattle path and downslope areas. The following two zones of interest were identified by the geophysical survey:

Zone 1: Thicker colluvium and a greater depth to bedrock.

Zone 2: Linear zone/feature which appears to be voided, theorised to function as a subterranean drainage path.

Laboratory tests were carried out on all soil and rock strata identified. For the purposes of this study, only the three most relevant to the design and construction are discussed here, and in later sections:

- Fill (old road): Sandy silt and clayey silt with pebbles and cobbles, 12 m at centre of gulley.
- Colluvium: Clayey silt with minor traces of sub-angular gravels and pebbles of mixed origin, 4 m deep at centre of gulley.
- Residual mudrock: Clayey silt with sub-angular and angular mudstone gravels to pebble sized fragments.

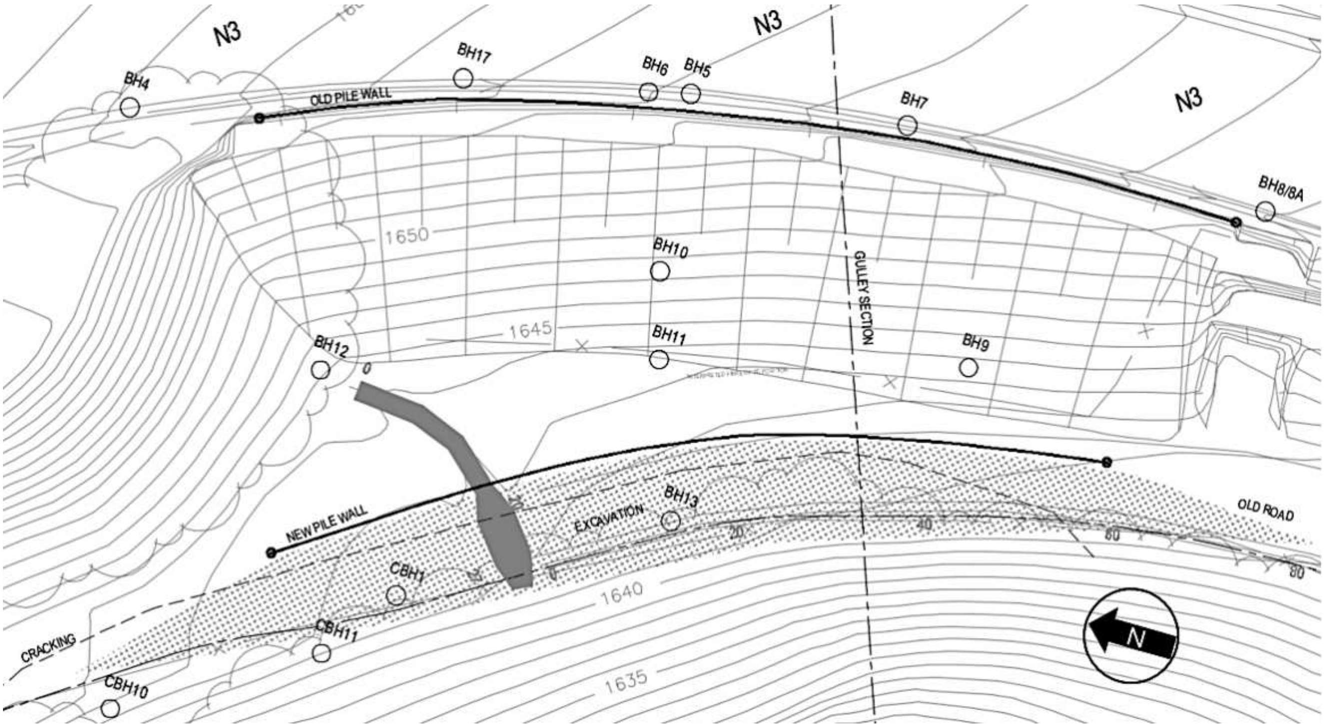

Figure 3. Site layout

3 STABILISATION DESIGN

3.1 *Design Options*

A holistic assessment of the slope stability indicated the following design problem: Cracking and further settlement of the old road did not immediately affect the stability of the N3 roadway. Furthermore, the existing anchored pile wall at the N3 was designed assuming that a slope slip on the old road would lead to some loss of downslope support. However, it is likely that a slip of the old road would be the first in a series of progressive slips that would slowly remove the midslope as well, to a point that the current N3 wall was not designed for.

Preliminary limit equilibrium models were set up in Rocscience's Slide2 program of the most critical slope stability sections at the gulley (deepest fill and colluvium layers).

Different solutions were trialled, starting with solutions that would stabilise the downslope and old road areas, and then looking at solutions that would prioritise protecting the midslope from progressive slip. Factors such as risk of failure, cost effectiveness and construction time were considered in the assessment of the solutions. It was found that downslope stabilisation solutions were more costly and would likely take longer than comparative midslope protection solutions. This was primarily due to constructability (heavily vegetated downslope, little access) and ground conditions (deep soil strata required more and longer soil nails to stabilise). Another factor that was also considered was that should a shorter-term downslope solution be implemented, and a midslope failure does occur, structural intervention at that stage would be extremely difficult and costly. It was therefore decided to proceed with midslope protection solutions for final design.

A comparison of solutions considered is presented in Table 1.

Table 1. Design options considered.

Design Option	Acceptable Factor of Safety*			Duration	Cost	
	N3 Road	Midslope	Downslope			
Do-Nothing	✓	-	X			
Soil nail downslope	✓	-	✓	+ +	+	*Downslope Stabi-*
Soil nail & micropile downslope	✓	✓	✓	+ + +	+ + +	*lisation options*
Excavate old road, nail excavation face	✓	-	X	+	+	*Midslope Protec-*
Pile old road	✓	-	X	+	+ +	*tion Options*
Pile, excavate, anchor old road	✓	✓	X	+ +	+ + +	

* Factor of Safety given for primary slip through road/midslope/downslope zone, progressive slip not indicated.
✓ indicates permanent stability, - indicates temporary stability, X indicates instability.

3.2 *Final Design*

The critical design section was identified as crossing through the cracked area overlying the natural gulley with fill. Given the outcomes of the options study, the stabilisation analysis would focus on achieving a permanent Factor of Safety (FoS > 1.4) through the midslope and N3 roadway areas.

At this stage it was necessary to refine the inputs used in the analyses using data returned from the laboratory. Triaxial testing was carried out on a colluvium sample as this material was considered a weak layer primarily at fault for the slip. The shear strength parameters for the colluvium material were interpreted from the triaxial as $\phi' = 28.5°$ and $c' = 7.2$ kPa. These strength parameters agree with the expected range of shear strength parameters for a silty clayey (CL) soil.

As final confirmation of these and other adopted parameters, a back analysis was run. This model showed a verification of the adopted parameters, as current slip through the old road fill was recreated with a FoS ~1.0.

This slip surface primarily occurs in the fill layers, with some of the slip passing through lower colluvium layers. This allowed calibration of the fill parameters, which were not measured in the laboratory, unlike the colluvium parameters.

While the results of the critical section indicate the slip primarily through the fill material, it is still believed that the colluvium layer forms the critical slip plane as the following should be considered:

The geology and the topography of the gulley section is unlike adjacent sections, where colluvium is typically nearer to the surface and drives the slip. Due to its depth in the gulley section, the colluvium becomes the key weak layer in triggering a follow up slip through the midslope (FoS ~ 1.1).

The critical section was then modified to allow analysis of a stabilised solution. A pile wall was created with ground anchors drilled into the soft rock mudstone and medium hard rock sandstone layers. A 2 m excavation was then created in front of the pile wall to relieve load on the underlying slip surface. Pile shear and anchor tensile capacities were iteratively tested to find a solution that satisfies the permanent stability criteria through the midslope. For the critical section it was found sufficient to use 700 kN shear capacity piles (0.9 m diameter) and 600 kN tensile capacity anchors (5-strand anchors) at a spacing of 1.25 m c/c. Figure 4 provides the analysis of critical section with a FoS of 1.4 through the midslope.

Further analyses focused on optimising the solution across the cracked area of the old road. Different zones were demarcated with pile spacing varying from 1.5 – 2.0 m c/c for sections outside of the critical section, and midslope FoS varying between 1.5 – 2.0 for these sections.

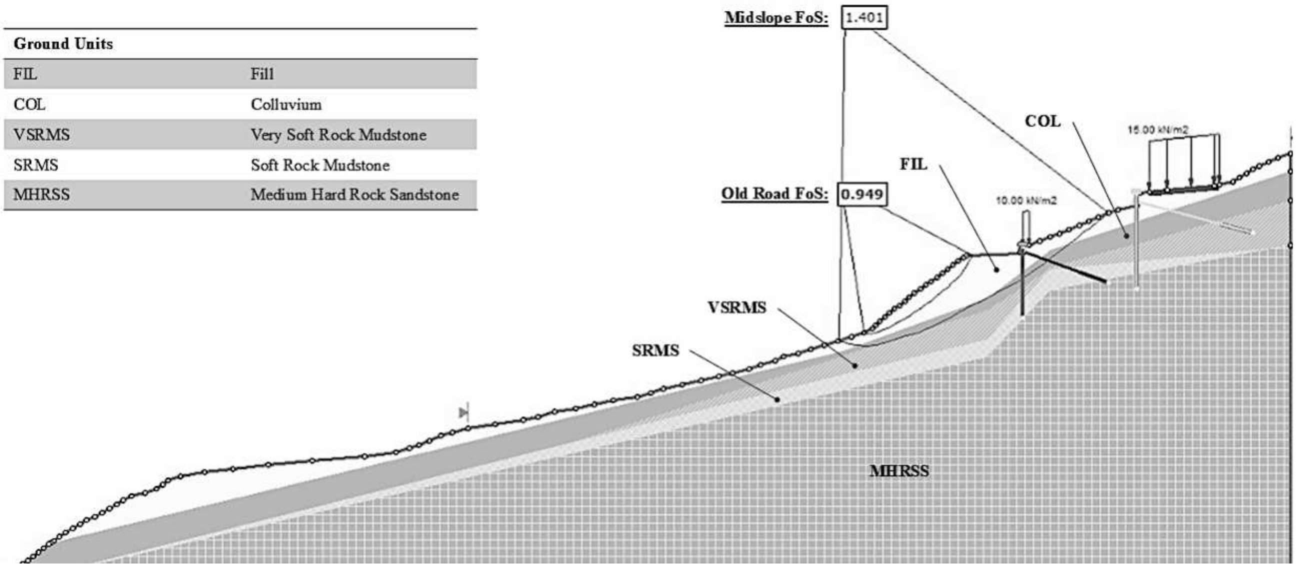

Figure 4. Gulley section stabilisation analysis

4 CONSTRUCTION

4.1 *Piling*

Construction of the anchored pile wall commenced in June 2024. The dry season was deliberately chosen for work as significant underground water flow had been interpreted during the investigation phase earlier in the year. It was noted that if this was true, drilling could be complicated substantially.

Five initial test holes were drilled across the site to confirm that the open hole auger piling method would be suitable for use considering the ground conditions. Importantly, testing found that hole side walls were stable, but areas of rocky talus were found towards the centre-north areas of the site. While these boulders did slow down the drilling procedure considerably, it would still be manageable to drill in these conditions, and the talus was believed to be limited to a certain area of the site.

Pile installation commenced at the southern end of the pile wall with Pile 60 and proceeded well towards the centre of the pile wall length (Fig. 5). Talus boulder horizons were occasionally found as 1-2.5 m thick on the southern centre side of the wall, however drilling became significantly more challenging at the centre with thicker boulder horizons in more pile holes.

Holes at Piles 16-26 would prove to be the centre of the boulder deposits, with boulder horizons of 6-10 m deep in this zone. Furthermore, these horizons would prove to be voided in areas, which further complicated drilling. Holes at Piles 21-23 would prove the most challenging drilling conditions, as voids were substantial enough to form a joined cavity. Concrete was poured into Pile hole 23 while Pile hole 21 was open. Concrete was observed to rapidly sink in Pile hole 23, while Pile hole 21 partially filled. Pile hole 21 was loosely backfilled immediately after the observed concrete loss in Pile hole 23. Thereafter, Pile hole 23 was successfully cast with approximately 136% additional concrete due to the voided nature of the talus.

Pile installation continued at the northern end of the pile wall with Pile 1, and proceeded without problems until Pile hole 17, to aid project progress and identify the extent of the thicker talus zone.

In Pile hole 17 a large boulder at 9.0 m below ground level (BGL) deviated the auger path. Consequently, the reinforcement could not be placed within the skewed hole. The hole was loosely backfilled to 12.0 m BGL, with a further 3.0 m^3 of concrete poured to cover the boulder at 9.0 m BGL. The concrete was allowed to cure for three days. Augering resumed with careful control to maintain alignment while penetrating through the concrete and large boulder. This method proved successful.

With an understanding of the extent of the thicker talus zone, the remaining Piles 18-22 were drilled successfully by reducing the drilling rate, ensuring adequate auger traction along the flight, and maintaining proper alignment. These factors facilitated more stable drilling operations and assisted the auger in penetrating the voided talus zone.

Figure 5. Piling work.

4.2 *Anchoring*

Despite the setbacks due to talus in the piling stage, all piles were installed in good time by August 2024. Excavation in front of the piles followed this, and anchoring commenced in late August (Fig. 6a). The anchoring was also started on the southern end of the wall and progressed well. However, rainy weather started in October 2024 when about half of the anchors had already been installed.

Theories about groundwater recharge and subterranean water flows were to be proven correct as it was observed that drilling anchor holes mere hours after rain would often cause water to spurt out the front. The presence of moisture also led to an increase in hole collapses and re-drills required.

Talus was once again encountered in several holes. Anchor holes 17-28 proved very difficult to drill due to the influence of voids and water. The following was observed in this area:

1. Hole collapse due to water.
2. Loss of air and grout in voids.

With ever increasing probability and quantity of rainfall on site, it was decided to install Anchors 17-28 with full length metal casing up to rock level

(Fig. 6b). While this did have a time and cost impact, the full-length casing was an effective intervention and allowed all of the anchors to be installed and made operational before the close of the year.

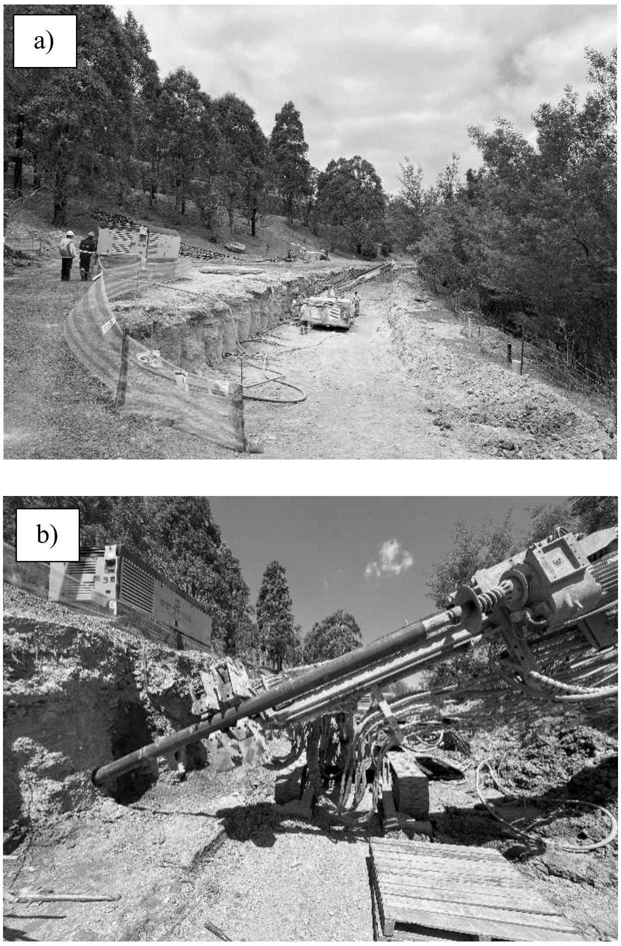

Figure 6. a) Anchor drilling and installation, b) drilling of cased anchors.

5 DISCUSSION AND CONCLUSIONS

Despite the complexities involved with installation on this site, the anchored pile wall was delivered within time and budget.

Two geotechnical constraints identified during the investigation phase, namely talus boulders and subterranean water flow, were to prove the greatest challenges to the installation of the anchored pile system.

It is however also worth reflecting on how these factors played into the initial cracking and settlement on the old path:

- Boulders deposited in the colluvium layer were first identified in boreholes located on the old road, with some voiding being apparent as well. It was however unknown how commonplace these boulders were present across the site.
- Geophysics encountered deep fill and colluvium layers across the centre of the proposed pile wall. In particular, a zone of apparent voiding was found at depth which corresponded to the area of worst cracking. A traverse further downslope also intercepted the zone, and it seemed that this was a linear feature that was theorised to convey water.
- The area described in the point above roughly coincided with the problematic anchored and piled area (Piles 16-26 and Anchors 17-28). Experience with the anchor holes in particular during the rainy season confirmed the rapid recharge of the groundwater and underground flow in the area.

Taking the above points together seems to point towards the existence of a voided linear zone comprised of talus boulders. It is possible that the talus boulders were deposited in the gulley before the construction of the fill, and were naturally packed in a way that left many discrete voids. Years of rainfall likely connected these voids together as a path of least resistance to flow, which formed as fines were washed away.

The location of this voided flow path above the worst of the old road cracking implies a connection, possibly due to the seasonal saturation of that area of slope, or due to progressive internal erosion due to the subterranean flow.

The site continues to be monitored on a biannual basis for both deformation (survey beacons, inclinometers), and moisture state (water content, soil suction).

This project demonstrates the importance of spending proper time and money on the investigation first, to identify possible constraining factors, and then remain ready and flexible to accommodate further challenges posed by the complex sites.

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