Slope failure in complex volcanic terrain, Opito Bay, New Zealand

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ABSTRACT: The Ohinau Drive ground movement affected multiple lots adjacent to a volcanic hill and was a complex variable depth failure. Following disturbance to infrastructure in 1996, localised artesian pressures were encountered under the slide mass. Groundwater drainage was undertaken by an inclined large diameter bored drain, later complemented by a second drain. Inclinometers, and multi-level piezometers, have provided over 10 years of data. Geotechnical models have been subject to stability analyses for measured and assumed worst case groundwater levels, and seismic shaking. Uncertainties were present in the geotechnical model due to geological and hydrological complexity. These were addressed in sensitivity analyses, land monitoring and engineering measures. Model robustness and stability measures were disputed by Council, leading to an independent hearing process. The geotechnical expert for the hearing controllers agreed the monitoring of observed ground performance since 1996 proved the site was likely to remain stable provided drainage remained operational.

1 INTRODUCTION

The Ohinau Drive slide is located at Opito Bay, north-eastern Coromandel Peninsula, New Zealand, as shown in Figure 1. The slope failure developed largely beneath vacant lots of a mid-1990s subdivision, extending southward into Tahanga Hill, as shown in Figure 2.

Earthworks were undertaken in the late 1980s, removing up to 5m from an existing ridge line at the base of Tahunga Hill, to form the subdivision and road (Ohinau Drive).

In 1996 significant ground movement occurred with a headscarp developed on sloping ground above the subdivision southern boundary. Ground heave was noted to the north within one of the lots, along with damage to services including the road, footpaths and stormwater reticulation. Lot boundary pegs placed in 1993 suffered lateral movement by up to 0.5m. This movement followed a period of prolonged rainfall over several months, with up to 220m of rain falling is less than a week.

Only limited development has occurred on the subdivision since the 1996 land movement, following local council imposition of encumbrances on the affected and nearby lots' property titles. Such encumbrances detrimentally affect property values and insurance cover.

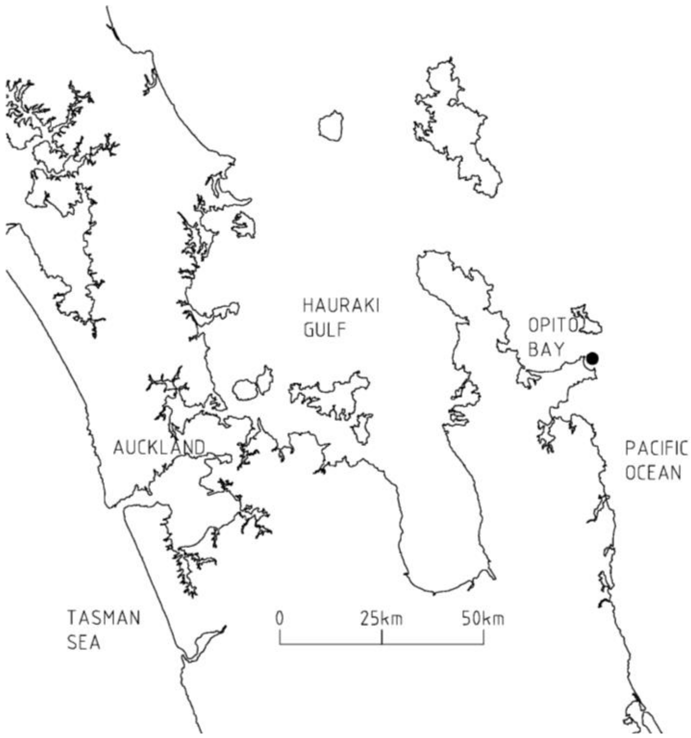

Figure 1

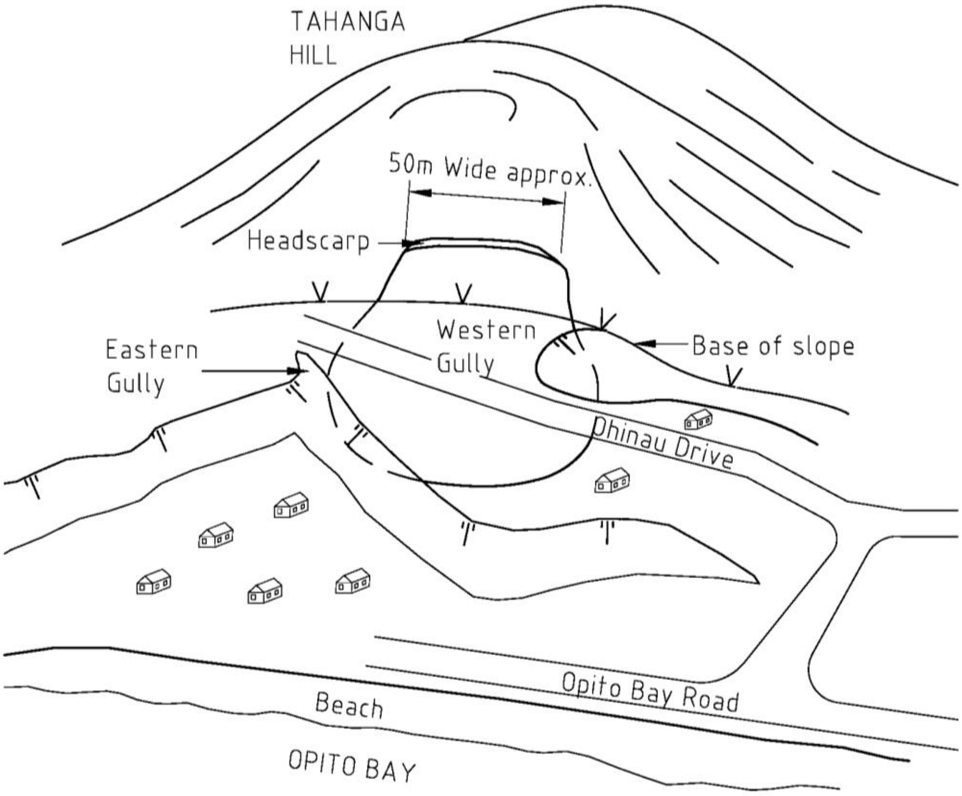

Figure 2

2 LANDSLIDE INVESTIGATION

A geological / geotechnical model was developed with a reasonable degree of confidence, although a complete definition of lateral extent, depth and interfaces between various units was not achieved.

Three phases of investigation were undertaken. An initial investigation in 1997 was performed by Worley Consultants Ltd (1997). In 2000 and 2005, second and third investigation stages were undertaken by Riley Consultants Ltd (RILEY) (2001, 2006).

Investigation comprised a review of existing information dating back to the 1970s, including aerial photographs, topography plans, drilling of nine machine boreholes (both for exploration and installation of monitoring equipment), four hand auger boreholes, and excavation of five test pits over the three stages.

Installed monitoring equipment included multi-level Casagrande type piezometers along with inclinometers extending to a depth of 31m.

A block sample and two push tube samples from the failure plane at the slip toe were taken for laboratory testing, including evaluation of Atterberg limits, along with effective stress parameters from direct and ring shear tests. A walkover appraisal and a review of pre-earthworks topographical plans were also undertaken. No subsurface investigation was undertaken adjacent to the headscarp, as permission from the neighbouring landowners was not forthcoming.

Information from these investigations was supplemented by a subsequent investigation on a neighbouring property immediately downslope of the subdivision undertaken by RILEY, where three test pits were excavated in 2011.

3 GEOLOGY / SUBSURFACE CONDITIONS

The geology of the general area has been described by Skinner (1976) and, in more recent times, Hawthorn (1996). The area generally consists of the basaltic Tahanga Hill surrounded by a mixture of basalt lava / plugs generally overlying and intruded through andesite lava deposits (Mahinapua andesite).

Geology of the investigated area was found to be more complex than shown on available geological plans with the inclusion of fill, recent alluvium, colluvium (debris), and lacustrine (old lake) deposits.

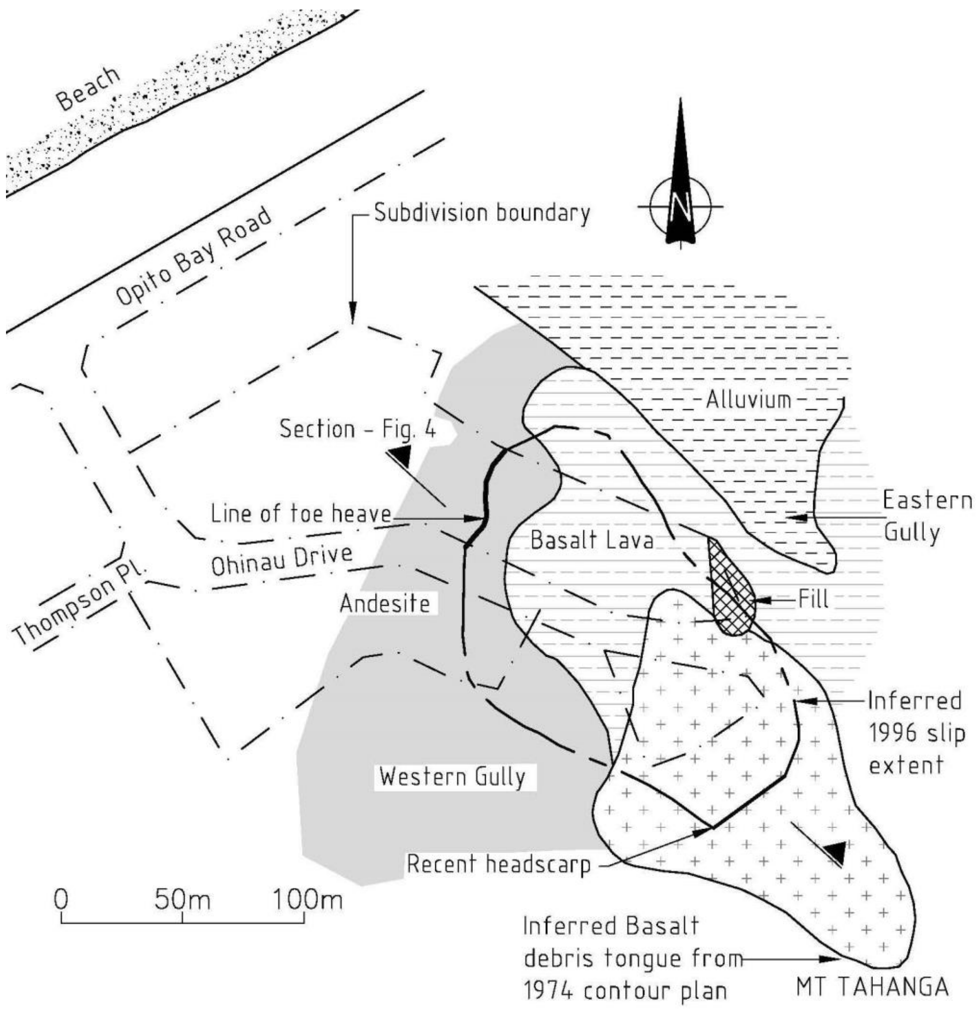

Figure 3

Brief descriptions of the main units are presented below with areal distribution shown in Figure 3:

1 Fill: Adjacent to the eastern gully, inferred remnants of a stockpile created during earthworks. Typically firm to very stiff silt, with topsoil layers.
2 Alluvium: Firm to very stiff silt, found beneath the fill within the eastern gully.
3 Basalt debris (colluvium): Basalt gravels, cobbles and boulders in a silt and clay matrix encountered to a depth of 11m at the base of Tahanga Hill and interpreted to extend beneath Ohinau Drive.
4 Weathered basalt lava: Highly weathered products of basalt lava flow, very stiff to hard soil inferred to be filling an ancient palaeovalley.
5 Weathered basalt tuff: Beneath the basalt debris to the east, this hard silt is inferred to be part of the original tuff ring that surrounded Mt Tahanga.
6 Lacustrine deposits: Typically comprising hard silt and clay, with layers micaceous and carbonaceous included rounded siliceous gravels. Upper horizon comprising mudstone, inferred to have been baked by andesite lava flow above. These deposits are inferred to be intercalated with the andesite and be in excess of 5m thick.
7 Hydrothermally altered andesite: Underlying the basalt and outcropping north of Ohinau Drive, the Mahinapua andesite varies from very stiff soil to weak rock strength, with varying degrees of hydrothermal alteration. In places the andesite appears sheared. Artesian water pressures were encountered in this material in 1997, underlying a portion of the slip mass.
8 Failure surfaces: Although not easily detected in recovered core, a distinct failure surface was encountered at the basaltic debris base where it contacts the underlying andesite consisting of a striated slickenside. A failure surface comprising clayey silt was also observed in a test pit excavated in the toe heave zone, with movement of hydrothermally altered andesite over similar residually weathered andesite. A failure surface, involved in the recent movement, is also inferred to have been encountered in the lacustrine deposits encountered in an older Worley borehole immediately below the basalt. Other potential failure surfaces were encountered in deeper lacustrine deposits, and these are inferred to have been possibly involved with past ancient movement.

4 LANDSLIDE CHARACTERISTICS

4.1 *Geomorphology*

Topography near Ohinau Drive is dominated by the 212m elevation basaltic peak and associated lava flows. A concave depression is evident on the steep sided slopes of the peak with a hummocky landscape below. The 1974 topographical plan indicates a large "tongue" of material below (north) of the concave depression and most recent headscarp. This "tongue" has been obliterated by recent earthworks and is inferred to have comprised basaltic debris.

Ohinau Drive and north consists of a flat to gentle graded area created by earthworks where up to 5m depth was cut from a pre-existing ridgeline. A heave zone of approximately 150mm was evident in this area. Below this is a steeper slope leading down to alluvial flats associated with an earlier subdivision, just above beach level.

4.1.1 *Failure surface*

Initial investigations, six months following the slope movement, postulated two failure mechanisms: a shallow failure involving just movement of the surface debris, and a deep failure through the underlying basalt. However, failure along just one of these surfaces was not entirely consistent with surficial expressions, such as surface peg movement and cracking. The combination of surface and subsurface observation did not indicate any plausible alternative failure scenarios.

Following subsequent investigations by RILEY in 2000, it was concluded the observed movement could be attributed to both postulated failure mechanisms.

1 Surface debris failure: Reactivation of ancient slip debris moving over andesite to a depth of approximately 11m, which explains relatively large peg movements (in the order of 0.5m) in lots adjacent to Tahanga Hill. However, this failure does not explain the andesite toe heave north of Ohinau Drive, or the moving of boundary pegs in the western gully embedded in andesite.

2 Andesite failure: Deep failure in the andesite and also along planes in the lacustrine deposits, to a maximum depth of 20m. This failure mechanism is believed to have been active in the winter of 1996, is consistent with all the observed movement indicators, and is considered likely to be the primary cause of the observed damage.

Both failure mechanisms are shown in Figure 4.

Subsequent subsurface investigations in 2005 for the installation of further piezometers and inclinometers provided supporting evidence for the 1996 failure being deep-seated, occurring approximately 15m to 20m in the andesite and lacustrine deposits.

The more recent on-site drilling also indicated an ancient failure surface within lacustrine deposits at 30m depth, although there was no evidence of recent movement on this surface, nor surface expressions consistent with such a depth of movement. In addition, computer aided stability analyses did not indicate such a depth of failure was critical to the present day topography.

4.1.2 *Recent movement and groundwater*

The 1996 mass movement appeared to have been strongly influenced by highly localised artesian water pressures in combination with subdivision earthworks that removed approximately 5m of earth from the toe area. Two boreholes drilled into the andesite in 1997 encountered artesian water pressures, although three others did not. It is inferred the location of such water pressures is localised, being dependant on fracturing and faulting within the andesite and potentially an old filled palaeo-valley combined with capping of relatively impermeable lacustrine deposits.

Water levels within the surface debris were also found to be high, within 2.5m of ground surface during winter months and probably at or close to ground surface at the time of failure.

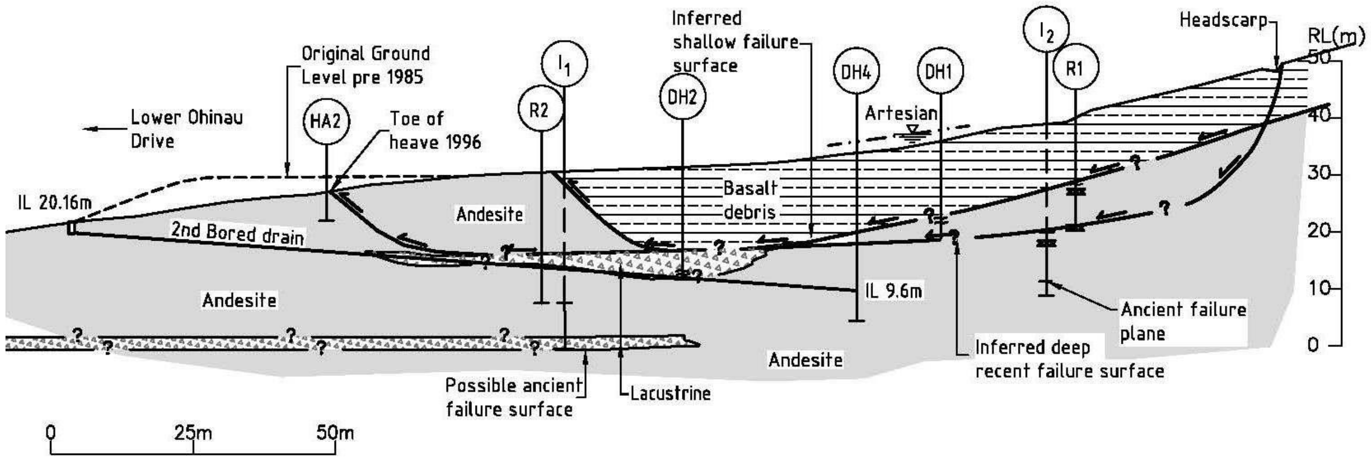

Figure 4

5 DRAINAGE AND GROUND MOVEMENT

A single bored drain was installed in August 2007 with an outlet in the eastern gully. The 65m long 100mm diameter bored drain targeted the zone of andesite artesian water pressure located centrally below the landslide mass. The effect of this drain was a substantial reduction in groundwater levels: approximately 1.7m and 8.7m drawdown within the surface debris and andesite respectively. However, the effect was localised to the central area and other areas showed no significant drop in groundwater level that could be directly attributable to the bored drain. The amount of drawdown was limited by the outlet elevation, as the bore was installed on a slight downward angle of approx 12º.

The bore discharge rate has been regularly monitored by the site owner since 1997, as have the piezometer groundwater levels. Initial flow rate of approximately 7,000 l/hr was observed.

A second large diameter bored drain was installed in December 2004 targeting the same former artesian area, with an outlet some 3.5m lower in elevation than the first bored drain. The installation of this second drain further lowered groundwater levels in the andesite by some 2m to 3m in the central area. This immediately stopped flow from the first bore.

A series of subsoil drains (counterfort drains) were installed through the surface debris to a maximum depth of 6m in April 2005.

Since the installation of the first bored drain in 1997 no discernable ground movement has occurred. The inclinometers, installed since 2005 show no deflection consistent with downslope ground movement. However, the lower inclinometer did within one year of installation display, an ‘S’ shaped bend, with no total cumulative displacement from 21m to 25m depth. This is inferred to be the result of incomplete grouting around the inclinometer tube. Similar bends have been observed in inclinometer installations on other sites.

6 STABILITY ANALYSES

6.1 *Static stability*

A series of computer assisted stability analyses were undertaken with respect to both failure surfaces. The computer programme (Rocscience SLIDE) was allowed to ‘free search’ for the most critical non-circular failure surface, the only constraints being fixity at the headscarp and toe.

The critical failure plane identified by the model closely aligned with that inferred from the investigations, i.e. deep failure in the andesite.

Two groundwater regimes were adopted, one for the shallower basalt and debris, and a deeper regime for the andesite and lacustrine layers.

A back-analysis of the 1996 slope was undertaken assuming a 2m artesian head beneath the centre of the landslide. Such an artesian head is only assumed based on driller’s reports from 1997, as no well head was installed to measure the pressure. The analysis indicated c'=2kPa and $\varphi'=15^{\circ}$ for the failure plane at time of movement in 1996 for a Factor of Safety (FoS) of approximately 1.0. Such parameters were considered appropriate to the material encountered in the subsurface investigation. Direct shear and ring shear tests of the failure plane material gave results of c'=25kPa and $\varphi'=20^{\circ}$ and c_r'=0kPa and $\varphi_r'=11^{\circ}$ respectively. Combining Atterberg results with common PI v φ relationships, such as from Fell et al (2005) gives $\varphi'=22^{\circ}$ and $\varphi_r'=11^{\circ}$. Shear strength testing of the failure plane at the toe indicated the material was not at residual strength, with peak shear strengths of 56kPa to 90kPa recorded, with a sensitivity of 1.9 to residual. Based on this evidence, a friction angle of 15º was consider appropriate for forward analyses, with values as low as c'=0kPa and $\varphi'=12^{\circ}$ adopted for sensitivity analyses, assuming artesian pressure was only 1m head or in a very confined area beneath the slip.

Stability analyses indicate that for the highest groundwater levels recorded since the second bored drain installation a FoS=1.5 to 1.6 was available against both shallow and deep movement. Sensitivity analyses with c'=0kPa and $\varphi'=12^{\circ}$ provide a FoS=1.1 to 1.2. However, as previously discussed, $\varphi'=12^{\circ}$ is not considered representative of the failure material recovered.

6.2 *Seismic stability*

New Zealand is generally a seismically active area being located across the Australian and Pacific plates. Stability and displacement analyses were undertaken for a peak ground acceleration of 0.2g (equivalent to a 1 in 500 year event).

Assuming an average undrained shear strength on the failure plane of 50kPa, FoS<1.0 was calculated leading to an estimated 2mm displacement using Ambraseys & Menu (1988) equations. A sensitivity analysis was performed assuming Su=39kPa (average measured residual shear strength) with again a FoS<1.0 computed and an estimated displacement of some 29mm. By comparison with other studies e.g. Miles & Keefer (2001), the site was categorised as low to moderately low hazard in the seismic case.

7 FUTURE DEVELOPMENT

Following the movement in 1996, the local council (Thames Coromandel District Council) had not precluded development of the affected lots, however, has required an encumbrance be registered against each property. Such an encumbrance warns of the land being affected by instability and has implications to the property value and insurances. As a result, only one of the lots has been developed to date.

The subdivision owner has argued the subject land has been stabilised since the 1996 movement and there is no need for the encumbrances. Analysis of geotechnical information for the site by RILEY in 2001 confirmed the apparent stability. Evidence supporting this view was then prepared by RILEY, peer reviewed by two external consultancies, and submitted to council. It was argued the slope currently had a suitable FoS and the drainage system installed comprising two large bored drains was sufficiently robust. In the unlikely event both stopped functioning there would be sufficient time for action to be taken before stability became critical (such as drilling an additional drain). However, the council and its advisors would not remove the encumbrances, citing uncertainties in the geological model (given the complexities), in addition to the stabilisation system (being the two bored drains) not being sufficiently robust in their view.

RILEY acknowledged that, as with any geotechnical model, there were uncertainties in the model for the site. However, the monitoring observed performance of drainage measures over many years, and subsequent significant rainfall events, provided sufficient evidence to positively off-set the level of uncertainty in the model. The long term monitoring and observed performance of land at risk of instability provides information that is considered, by the author, to be of greater relevance than attempting to achieve high levels of certainty in the 3D representation of a slide mass.

The property owner appealed the council's decision to the Department of Building and Housing (DBH), the body for building controls in New Zealand. Following a 2009 hearing, the DBH ruled development of the subdivision could proceed without encumbrances provided a robust and responsive monitoring and maintenance programme was established. The DBH independent expert was satisfied that, provided the bored drains continued to operate satisfactorily, the slope would be unlikely to move.

A monitoring and maintenance programme has been proposed including monitoring of rainfall, piezometers, drain flows, seismicity, and inclinometers with data transferred to web hosting. Trigger levels have been established, at which defined actions need to be taken.

The proposed programme is currently under review by the DBH and its advisors.

8 CONCLUSIONS

There is no such thing as certainty in a geotechnical model developed to model a slope, or a site as there will always be some uncertainty.

The key issues in respect of what constitutes an acceptable level of uncertainty are:

1. Consequences of variances in the model.
2. Observed performance and monitoring of the land and to what degree these provide a positive offset to uncertainty in a model.
3. The options / systems available to monitor the site / slope, report changes in a timely manner, and engineering measures to address any changes.

The geotechnical profession needs to convey to regulating authorities and the public that the assessment of geotechnical hazards and risk for development of land should consider these three factors together. Failure to do this will mean that geotechnical hazards affecting land developed are not appropriately assessed.

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