Reinforcement of tailing dam foundation soil using jet grouting method

Mario Colil & Christian González Pulgar
Engineering Department, Pilotes Terratest, Chile-Perú, mcolil@terratest.cl, cgonzalezp@terratest.cl

ABSTRACT: The need to increase the height of the tailing dams, due to the increased volume of treated minerals has generated a significant geotechnical challenge, particularly concerning the high pressure that can occur in the foundation soil. This is especially relevant when the foundation soil consists of contractive saturated sand or fine deposits, capable of causing significant losses of resistance or stiffness due to increased pore pressure (ex. due to seismic loads). These factors can endanger the stability of the dam.

Jet grouting is a ground improvement technology used to provide strength and stiffness to soils. This technique uses high-velocity fluid jets to erode existing soil and mix the remaining cuttings with cement slurry to form soil-cement of varying geometries in the ground. In this theorical study, the use of the jet grouting method is described and some of the benefits of applying this technique to the foundation soil of tailing dams are evaluated through stability analysis. The analysis considers the improvement in the shear strength of the foundation soil, which initially had poor geotechnical capacity prior to the soil treatment.

KEYWORDS: jet grouting, shear walls, soil improvement, tailing dams

1 INTRODUCTION

The need to increase the heights of tailing dams, due to the increase of the volume of treated minerals, it has imposed a significant geotechnical challenge. This challenge stems not only from the characteristics and behavior of the dam's constituent materials but also from the high pressures that may develop in the foundation soil. This aspect becomes especially critical when the foundation soil consists of contractive saturated fine or sandy deposits, which can induce considerable reductions in resistance or stiffness. These changes are often associated with elevated pore pressures or a transition to a normally consolidated state, thereby potentially endanger the stability of the dam.

Tailing dams constructed on soft soils are non-uncommon. There are earth embankments and tailings dams in the Andean region of Peru and Chile founded on soft soils. The documented failure of Mount Polley in Canada in August 2014 serves as one example of a catastrophic foundation soil failure. Mount Polley, an open-pit mine for copper and gold, experienced a partial embankment collapse due to extensive strain in a significant portion of a thin layer of overconsolidated and sensitive clay, which approached the point of strain-weakening (Zabolotnii et al., 2022). This was aggravated by the subsequent addition of embankment material and the reduction of shear resistance in a small portion of the upper part of this unit to post-peak values (Zabolotnii et al., 2022). For tailings dams and earth embankments built on soft soils, which, as they grow, are suspected to develop stability problems due to foundation soil that can become normally consolidated and, therefore, contractive under shearing, thus becoming more susceptible to undrained failure, or due to liquefaction induced by seismic loads, there are different soil foundation treatment techniques (densification, consolidation, and in situ soil mixing methods).

Within the densification ground improvement method, mention can be made of the vibro-compaction (also known as vibro-flotation) method, the vibro-replacement (stone columns) method, and the dynamic compaction method. However, those techniques lose efficiency on their own or are not effective for soil containing more than 15% fine content (Kirsh & Kirsh, 2017).

Within the consolidation technique, mention can be made of the preloading method (with and without vertical drains). This method is typically used for silty and soft clayey soil deposits and relies on consolidating the ground by applying a designed preload (embankments, temporary water tanks, etc.). However, it is not easily adaptable for tailing remediation since the construction of a tailing dam implies a continuous increase in earth load. If remediation is needed, adding extra load would not be desirable (Burbano, 2021).

Finally, within the in-situ soil mixing methods, mention can be made of the soil mixing and jet grouting techniques. With these methods, soil-cement discrete columns can be created (isolated within an area of improvement), rectangular panels with the Cutter soil mixing tools, and panels with overlapping columns. Additionally, with the jet grouting method, other geometries can be created (see Figure 19). The use of panels as grids can improve liquefaction resistance (Namikawa et al, 2007; Nguyen et al., 2013).

For tailing dams and earth embankments, the use of shear walls parallel to the embankment axis built with rectangular panels (Cutter sol mixing) or secant columns has been found to be more effective (Filz et al., 2012). In the national market, Chile, the use of secant columns is more common due to the available equipment. These secant columns can be constructed using either deep cement mixed (DCM) columns, where the deep soil mixing process forms columns of cemented material in the ground by mechanically mixing the in situ soil with an introduced binder agent such as cement, or jet grouting columns (JGC), which involve cutting the soil with a high-velocity jet of fluid(s) and mixing this eroded soil with a self-hardening grout to form columns.

DMS and JGC can increase the strength and decrease the compressibility of soft soils, as well as reduce permeability, thereby improving the stability and reducing settlement of tailing dams or earth embankments. The soil mixing and jet grouting methods utilize rotary drilling or milling, which inherently do not introduce significant vibration into the subsurface during the drilling and mixing process. This paper focuses on the use of jet grouting columns as a technique for soil foundation improvement. It is important to highlight that this technique has been available in our Chile for some years ago.

2 THE JET GROUTING TECHNIQUE

Jet grouting is a ground improvement technology in which fluid is radially injected with the aid of high energy (pumped at 400 to 500 bar), transforming this pressure into high kinetic energy at the nozzle outlets. The high shear speed disaggregates the soil, breaking its structure and mixing it with a cement agent to form a new material in the form of columns or panels (as shown in Figure 1). This solid body is composed of soil-cement with higher resistance and lower permeability than the original soil. The finest soil content is then swept out of the well, together with leftover grout (resurgence).

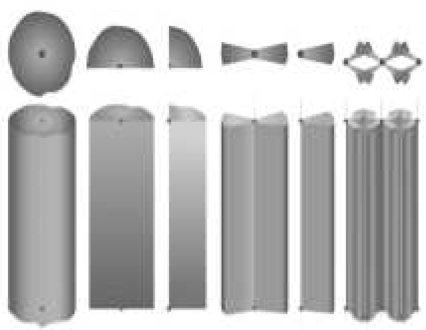

Figure 1. Different geometries of jet grouting bodies.

There are three traditional system of jet grouting: single fluid or monojet (one fluid: cement grout), double fluid (cement grout plus air), and triple fluid (cement grout plus air plus water) as shown in Figure 2.

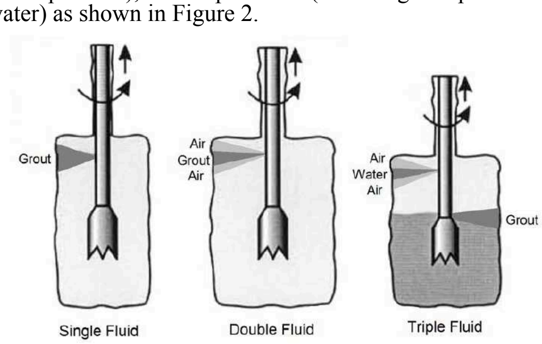

Figure 2. jet grouting: single (left), double (middle), and triple (right) fluid systems (Burke, 2004).

Figure 3 illustrates a jet of water emanating from the nozzle in the double fluid jet grouting method.

Figure 3. Jet of water coming from the nozzle.

Jet grouting technology is capable of treating soils ranging from clays to gravel. Optimal results are achieved when treating cohesionless or soft cohesive soil of low plasticity and loose sand. In all cases, it is necessary to calibrate the parameters (rotational and extraction speed of the rod, injection flow, etc.) according to the soil type and the required column diameter. The execution process begins with drilling the soil to the desired depth in one step, without injecting cement grout. Next, the injection step occurs with the simultaneous rotation and extraction of the rod, with the speed of rotation and extraction monitored and controlled by sensors. This speed is predetermined to achieve the required column diameter. Once a column is completed, the next column overlaps the previous one, forming a continuous wall.
Table 1 illustrates the expected column diameters for different systems and soils. Larger diameters are typically achieved in sands and gravels, while smaller diameters are observed in cohesive soils such as silts and clays.

Table 1. Jet grouting column diameter expectation (Burke, 2004), in meters.

System	Soft clay	Silts	Sands or gravels
Single fluid	0.40 – 0.90	0.60 – 1.10	0.80 – 1.20
Double fluid	0.90 – 1.80	0.90 – 1.80	1.20 – 2.10
Triple fluid	0.90 – 1.20	0.90 – 1.40	0.90 – 2.50

Croce et al., (2014) proposed Eq. 1 based in efficiency to predict the mean diameter of JGC.

$$D_{mean} = 1,128\sqrt{PV_g\lambda_E} \quad (1)$$

In which, for dimensional consistency, pressure P expressed in MPa, the volume of injected grout V_g in m^3/m, and the energetic efficiency λ_E in m^3/MJ.

Table 2. Typical values of λ_E (m^3/MJ) (Croce et al., 2024).

Soil type	Single fluid	Double fluid
Sandy gravel	0.067 – 0.100	-

From gravelly sand to silty sand	0.033 – 0.067	0.077 – 0.125
From sandy silt to clayey silt	0.020 – 0.033	0.077 – 0.025

Burke (2004) presents a range of typical soilcrete strengths for single fluid systems, as shown in Table 3. Regardless of the system (single fluid, double or triple fluid), the strength is a function of the cement content in the final product (Bruce et al., 2013).

Table 3. Range of typical soilcrete strengths – single fluids system (Burke, 2004).

Soil type	UCS, MPa
Clean sands and gravel	5.17 to 8.62
Silts and silty sands	3.45 to 5.17
Clays	1.72 to 3.45
Organic silts and peats	Less than 1.72

The prediction of the diameter of JGC is a fundamental step for designing jet grouting applications. However, the margin of uncertainty stemming from empirical or theoretical correlations is still relatively large. Therefore, in practice, most of the time, the final treatment parameters (such as the number and diameter of nozzles, injection pressure and/or flow rate, and lifting speed of the rod) are obtained from field tests to confirm the actual column size, shape, verticality, homogeneity, and strength that can be achieved. From Figure 4 to Figure 6 shown JGC from a production field test in Constitución and northern Santiago, Chile, used to confirm the size of the columns in silty sands and silts.

Figure 4. Exhumation of Jet Grouting Columns in silty sands, C (ref. Pilotes Terratest).

The results from these field test show mean diameters of the JGC ranging from 1.50 to 2.50 m, depending on the execution parameters utilized. Unconfined compressive strength values of samples extracted from these JGC ranged from 4 to 11 MPa, for cement quantities ranging from 168 to 228 kg/m^3, exhibiting an exponential increase in strength for higher cement quantities.

Figure 5. Exhumation of Jet Grouting Column in silty sands, Constitución (ref. Pilotes Terratest).

Figure 6. Exhumation of Jet Grouting Column in silts, Northern Santiago (ref. Pilotes Terratest).

Figure 7 depicts a JGC from the other production field test in Santiago, used to obtain core samples to measure the compressive strength of the JGC material.

Figure 7. Exhumation of JGC, Diamond Drilling, and Jet Grouting Core Sample (ref. Pilotes Terratest, field test in Santiago Gravel).

The typical range of jet grouting parameters for conventional jet grouting systems can be obtained from Burke (2004) and from the knowledge of the companies that apply this technic.

3 SHEAR STRENGTH OF JET GROUTING PANELS

For the design of shear wall panels made of secant jet grouting columns, Bruce et al. (2013) suggests a typical arrangement of columns, as shown in Figure 8.

Bruce et al. (2013) recommends using a total stress characterization of the soilcrete strength for design, with a total stress friction angle of Ø = 0 and no tensile strength considered. Therefore, the shear strength of a column made of jet grouting, Sjg can be estimated from Eq. 2 where qu is the unconfined compression strength of the soilcrete.

$$S_{jg} = \frac{1}{2} q_u \tag{2}$$

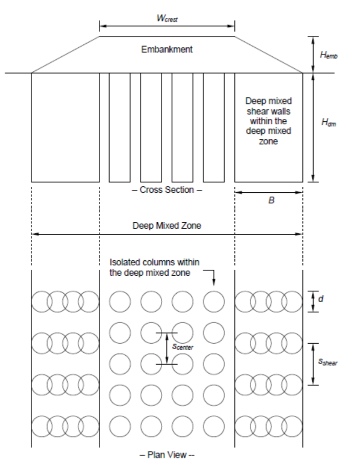

Figure 8. Illustration. Typical arrangement for deep-mixed zone beneath an embankment (Bruce et al., 2013).

According to Bruce et al., (2013) the composite shear strength of the improved soil with deep soil mixing can be obtained from Eq. 3. This expression is found to be suitable for jet grouting as well.

$$S_{jg,wall} = f_v a_{s,shear} S_{jg} \tag{3}$$

Where f_v is a coefficient of variation accounting for the greater variability that typically exist in the strength of deep mixed ground compared to the variability that typically exist in the strength of deposited clay soils. According to Bruce et al., (2013) for slope stability, an F_v value of 1.0 is obtained for analysis with factor of safety equal or less than 1.20. In Eq 3 $a_{s,shear}$ correspond to the replacement ratio, defined as the ratio of the area of the shear wall to the tributary soil area surrounding the shear wall.

$$a_{s,shear} = \frac{b}{S_{shear}} \tag{4}$$

In Eq. 4, b corresponds to the average shear wall with according to Figure 9. The replacement ratio typically ranges from about 0.2 to 0.4. Because the shear walls are constructed of overlapping columns, the extend of the overlap influences the minimum and average widths of the shear walls.

To conserving the overlapping between the jet grouting columns, the replacement ratio can be calculated as shown in Eq. 5

$$a_{s,shear} = \frac{\pi d(1-a_e)}{4S_{shear}\left(1-\frac{e}{d}\right)} \tag{5}$$

Where a_e corresponds to the overlap area ratio. Typical values of e/d range from about 0.2 to 0.35.

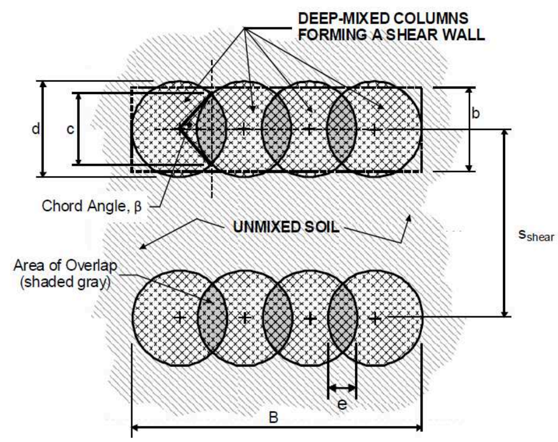

Figure 9. Illustration. Definition sketch for column overlap calculations (Bruce et al., 2013).

4 CASE STUDY

The case of study involves a hypothetical tailing dam founded on fine soils, which, as it grows to a certain level, is expected to cause the foundation soil to become normally consolidated, with a shear strength insufficient to resist a global failure under seismic conditions with an appropriate stability safety factor. The tailing dam is built by cyclone sand, using the downstream method, with a maximum height of 100 m, a crest width of 10 m, a downstream slope of 1:3.5, an upstream slope of 1:2, and a freeboard of 2 m. This configuration is typical in Chile (Quiroz, 2021).

The foundation soil is composed of 15 m of non-plastic silt of medium consistency overlaying bedrock. The tailing consists of an initial wall made of compacted gravel with a height of 10 m and downstream and upstream slopes of 1:3 and 1:2, respectively. The tailing is extended with compacted cyclone tailings sand. The configuration of the analysis section without soil foundation improvement is illustrated in Figure 10.

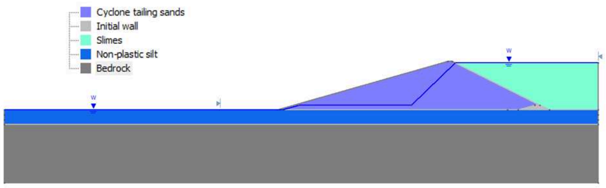

Figure 10. Section of analysis in Slide 2.

Two sets of calculation models are conducted. The first set includes static analysis, pseudo-static analysis with drained parameters, and pseudo-static analysis with undrained parameters. The second set of calculation models involves soil foundation improvement with JGC forming shear panels to achieve an adequate factor of safety for slope stability problems in the same scenarios as the previous one.
The soil improvement design involves shear panels that extend to cut the shear failure plane occurring in the pseudo-static analysis for both drained and undrained scenarios. Figure 11 presents the case of foundation soil improved by jet grouting shear panels with a horizontal extension of 60 m from the toe of the current growth stage, to increase the overall safety factors of the future growth stage of the tailing dam.
The problem addressed in this paper focuses on slope stability, simulating that the improvement of the ground foundation occurs in an existing dam that needs to be raised in height, so only shear wall panels below of the downstream foot of the tailings dam in a certain length are considered.

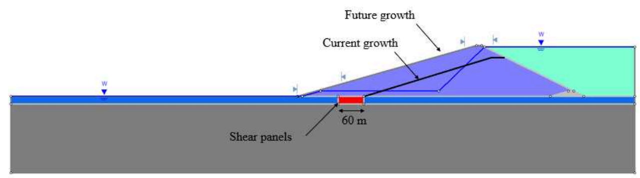

Figure 11. Improvement section of analysis in Slide

Table 4 shown the geometric parameters for the configuration of the shear panels.

Table 4. Geometric parameters for the shear panels.

Parameter	Value
d (m)	1.00
e (m)	0.20
e/d	0.20
a_e	0.11
S_{shear} (m)	4.25
$a_{s,shear}$	0.21

5 GEOTECHNICAL CHARACTERIZATIONS

In Table 5 the geotechnical parameters of the varied materials are shown, adopting the undrained shear strength only for the seismic conditions.

Table 5. Geotechnical parameters adopted.

Parameter	Non-plastic silt	Initial wall	Tailing dam	Slimes	Bedrock	JGC
Total unit weight (kN/m^3)	17.5	20	19	15	23	17.5
Saturated unit weight (kN/m^3)	19	21	20,5	17	23	19
Internal friction angle (°)	28	38	35	30	35	0
Cohesion (kPa)	50	0	0	9	200	---
Undrained shear strength (kPa)	$0.4\sigma'_v$	---	$80+0.3\sigma'_v$	$0.05\sigma'_v$	---	2000
Unconfined compression strength (kPa)	---	---	---	---	---	4000
Composite shear strength of panels (kPa)	---	---	---	---	---	371

The initial wall's typical parameters are considered for the loan material. The geotechnical parameters for the tailing dam and slimes are taken from Figueroa et al. (2017) and Gonzalez et al. (2016). For the non-plastic silt, geotechnical parameters that can be found in Chilean soils. Finally, the Jet Grouting Columns (JGC) are characterized according to Table 3 for Silts.

6 SEISMIC COEFFICCIENT

According with de Chilean practice, the horizontal seismic coefficient K_h for conducting a pseudo – static analysis is obtained from a seismic hazard study. However, for preliminary analysis it can be obtain from the expression develop by Saragoni (1993) represented by Eq. 6

$$K_h = \{0.3\,\frac{a_{max}}{g},\ a_{max} \leq 0.67\,g \quad 0.2\left(\frac{a_{max}}{g}\right)^{0.2},\ a_{max} \geq 0.67\,g \qquad (6)$$

Where a_{max} corresponds to the maximum effective horizontal acceleration. This equation is normally used when there's a seismic hazard study defining amax. A value of Kh = 0.16 g is used in the analysis which corresponds to a PGA = 0.53 g according to Eq. 6.

7 LIMIT EQUILIBRIUM ANALYSIS – FACTORS OF SAFETY

According to the supreme decree N°35 248 (2021) for the analysis of tailing dams using limit equilibrium methods, the minimum factors of safety are shown in Table 6.

Table 6. Minimum factors of safety for tailing dam using limit equilibrium methods (DS 35 248, 2021).

Scenario	Soil behavior	Factor of Safety
Static	drained	≥1.50
Static	undrained	≥1.20
Pseudo-static	drained	≥1.20
Pseudo-static	undrained	>1.00

8 RESULTS AND DISCUSSION

The results show that for the static case of the study case without ground improvement a factor of safety of FS = 2.21 (using the Bishop simplified method). For the pseudo-static analysis, the factors of safety for the drained and undrained analyses are FS = 1.29 and 0.93, respectively.

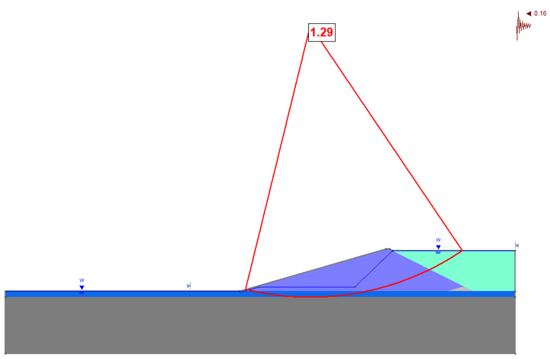

Figure 12. Slope stability, analysis seismic loading, and drained behavior.

The minimum factor of safety in undrained pseudo-static analyses is lower than the minimum factor shown in Table 6. Additionally, the deep failure plane passes through the non-plastic silt, indicating the necessity for soil foundation improvement. The shear wall panels made of overlapping JGC are evaluated, with the geometric properties shown in Table 4 and the geotechnical resistance shown in Table 5.

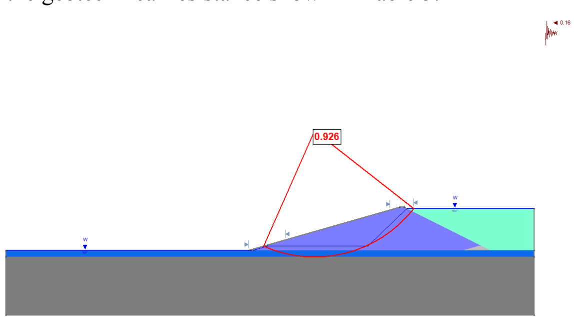

Figure 13. Slope stability, analysis seismic loading, and undrained behavior.

The shear wall panels made of overlapping JGC are introduced from the toe of the slope extending 60 m in downstream direction (see Figure 14 and Figure 15) and covering the entire thickness of the non-plastic silt layer. It is considered that the panels are executed in a stage prior to the future growth of the tailing dam.
The values of factor of safety obtained from the soil improvement range from 1.28 to 1.02 for the seismic scenario and drained and undrained soil behavior, respectively, which are greater than those shown in Table 6. Showing that soil improvement carried out by shear panels formed by JGC would be effective in this analysis case.

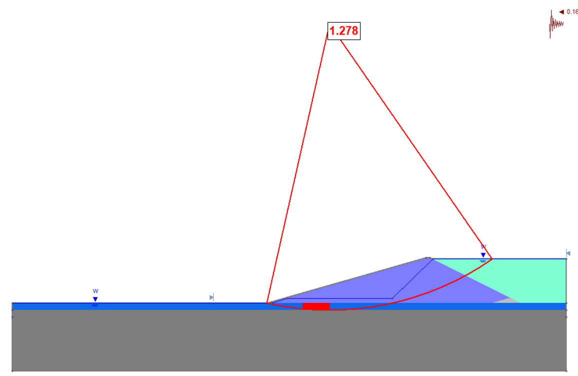

Figure 14. Slope stability, JGC shear panels improvement, analysis seismic loading, and drained behavior.

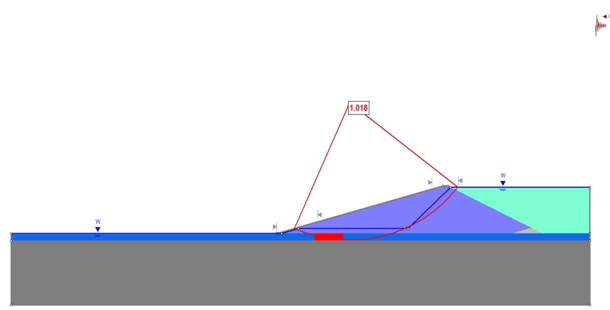

Figure 15. Slope stability analysis, JGC shear panels improvement, seismic loading, and undrained behavior.

It is important to note that the stability analysis is the primary factor controlling the design of this ground improvement. However, according to Bruce et al. (2013), there are other failure mechanisms that need to be revised. These include for the internal case of failure, the crushing of the center isolated jet grouting column, shearing on vertical planes through the improvement zone, and soil extrusion through the shear walls, etc. These failure mechanisms must also be analyzing for a definitive design.
On the other hand, a cost-benefit analysis of the solution must be carried out. This alternative may be more applicable to limited sectors under the dam, of a few hundred meters, than to those that require treatments of several kilometers.

9 CONCLUSIONS

Ground improvement techniques using soil-cement mixtures (soil mixing and Jet Grouting) have been used successfully in the stabilization of embankments and slopes.

In this paper, an alternative of tailing dam soil foundation improvement was theoretically developed using the Jet Groutig technique, evaluating its static and pseudo-static stability. To determine the properties of the improved soil, a calculation method based on recommendations associated with the soil mixing technique was considered.

The applicability of the use of shear walls built with JGC as a method of ground improvement and stabilization for global slope failures that compromise the foundation soil was technically demonstrated.

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