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Numerical simulations on unsaturated soil experiments using tri-phase seismic response analysis

K. Kitade
Chuden Engineering Consultants Co., Ltd., Hiroshima, Japan

O. Ozutsumi
Meisosha Corporation., Tokyo, Japan

N. Orai
Chuden Engineering Consultants Co., Ltd., Hiroshima, Japan

S. Iai
General Incorporated Association FLIP Consortium, Kyoto, Japan

ABSTRACT: Generally, it is recognized that liquefaction occurs for saturated soil. Besides, it is also confirmed in experiments that liquefaction can occur even for unsaturated soil. In order to evaluate seismic response of unsaturated soil which can be liquefied, responses of soil skeleton, pore water, and pore air should be modelled properly. As a method to express the behavior of unsaturated soil at the time of earthquake, Iai (2018) proposed a method combining a constitutive law "cocktail glass model" for sand, with "generalized fluid pressure vector" for expressing the behavior of multiphase system of pore fluid. In this study, numerical simulation of cyclic triaxial test and shaking table test on unsaturated soil were carried out in order to verify the validity of this method. As a result, it was confirmed that it captured well responses of cyclic triaxial test and shaking table test on unsaturated soil.

1 INTRODUCTION

1.1 *Background*

Generally, it is assumed that liquefaction can occur only for the ground layer which is located deeper than the groundwater level. It also means that the ground layer which is located shallower than groundwater level (unsaturated soil) can never be liquefied.

However, in the actual situation, it is a concern that the degree of saturation of unsaturated soil is in high state due to the effect of rainfall or infiltration of ground water, and it also can be liquefied even it is not fully saturated. In fact, in the Niigata Prefecture Chuetsu Earthquake 2004, damage on many earth structures including embankments was observed. It is reported that the degree of saturation inside the embankments had risen due to the influence of rainfall caused by a typhoon just before the earthquake (National Institute for Land and Infrastructure Management, 2006). Hence, it is important to consider the effect of liquefaction of unsaturated soil properly for the sake of mitigation of seismic damage.

1.2 *Method*

In order to simulate the behavior of saturated sand deposits at the time of earthquake, an equation of motion with the constitutive law "cocktail glass model" (Iai et al., 2011) for sand, and a mass balance equation of flow (Zienkiewicz & Bettes, 1982) for pore water, have already been implemented in the FLIP program (FLIP Consortium, 2018) and we can solve both as a coupled problem. Unknown variable in the equation of motion is displacement vector **u**, and in the mass balance equation of flow unknown variable is pore water pressure p_f. Iai (2018) applied the idea of the mass balance equation of flow to pore air and showed that it is possible to express the behavior of unsaturated ground deposit by solving these three equations as a coupled problem.

The unknown variable newly added is pore air pressure p_a. Iai (2018) proposed the generalized fluid pressure vector that consists of pore water pressure p_f, pore air pressure p_a, and so on. By introducing the generalized fluid pressure vector, the governing equations of unsaturated sands of the same form as the governing equations of saturated sands are obtained. The mass balance equation of flow using the generalized pore pressure vector was incorporated into the FLIP program (hereinafter called FLIP-TRIP). The pore fluid element of the FLIP-TRIP program handles this equation. Every nodal point of the pore fluid element has a degree of freedom of pore water pressure p_f and pore air pressure p_a in this study.

In order to confirm the applicability of the cocktail glass model and the generalized pore pressure vector, authors conducted simulation analysis for two different tests using the FLIP-TRIP program. One target test was a cyclic triaxial test of unsaturated soil (Matsumaru et al., 2014a), and the other target test was a shaking table test for embankment (Matsumaru et al., 2014a).

2 SIMULATION OF CYCLIC TRIAXIAL TEST

2.1 *Outline*

The objective of analysis was to simulate the cyclic triaxial test on unsaturated Inagi sand (Matsumaru et al., 2014a) and verify the applicability of the proposed method. As for the procedure of the test, suction was given under unloading and drained conditions. And then, cyclic axial strain was applied with displacement control under loading and undrained conditions. Figure 1 shows the time history of the axial strain. The initial state of the specimen of the

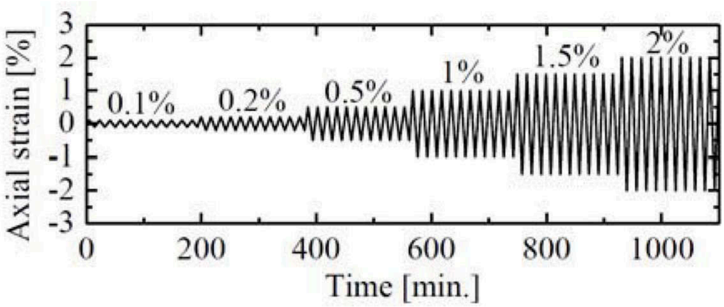


Figure 1. Time history of axial strain(Matsumaru et al., 2014a)

Table 1. Initial state of cyclic loading (based on Matsumaru et al., 2014a)

Case	Pore air pressure p_a kPa	Pore water pressure p_f kPa	Suction s kPa	Degree of saturation S_r	Effective saturation S_e	Pore pressure p kPa	Net stress σ_{net} kPa	Total stress σ kPa	Effective stress σ' kPa
Case 1	18	5	13	0.437	0.328	13.74	25	43.0	29.26
Case 2	5	0	5	0.538	0.452	2.74	25	30.0	27.26
Case 3	1.5	0	1.5	0.792	0.763	0.36	25	26.5	26.14

*1 Pore pressure is determined by the method of (Ohno et al 2007).

Table 2. Parameters of Inagi sand (based on Matsumaru et al., 2014a)

Specific gravity	Fine fraction content	Dry density	Initial porosity	Saturated permeability	Air permeability	Bulk modulus of pore water	Bulk modulus of pore air
ρ_s	F_c	ρ_d	n_0	k_f	k_a	K_f	K_a
t/m^3	%	t/m^3		m/s	m/s	kPa	kPa
2.723	23.6	1.108	0.596	1.86E-04	1.86E-05	2.20E+06	9.80E+01

*1 The values of K_f and K_a are based on Iai (2018)

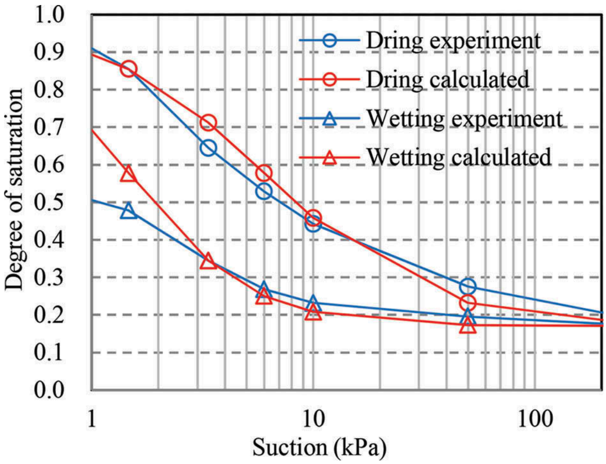


Figure 2. Water retention curve (based on Matsumaru et al., 2014a)

cyclic triaxial test is shown in Table 1. Table 2 shows the parameters of Inagi sand. Figure 2 shows the water retention curve of the Inagi sand. The test values in the figure are referred from Matsumaru et al (2014a). The simulation results in the figure were obtained by FLIP-TRIP using parameters shown in Table 3.

Table 3. Parameters for water retention curve in FLIP-TRIP

Maximum degree of saturation	Minimum degree of saturation	Parameters of upper limit of water retention curve					
S_{rx}	S_m	s_{aU}	p_{nfU}	p_{mfU}	s_{aL}	p_{nfL}	p_{mfL}
0.985	0.17	6	1.182	1	1.47	1.576	1

Table 4. Parameters of Inagi sand employed in cocktail glass model in FLIP-TRIP

ρ	n	p_a	G_{ma}	m_G	K_{La}, K_{Ua}	n_K	H_{max}	H_{maxL}	q_{us}	φ_f
t/m^3	%	kPa	kPa	m/s	kPa				kPa	degree
0	0.596	100	29500	0.5	76930	0.5	0.24	0.24	0	28.7

φ_p	$-\varepsilon_{dcm}$	$r_{\varepsilon dc}$	$r_{\varepsilon d}$	r_k	q_1	q_2	q_4	r_K''	c_1	l_k	r_γ	r_{mtmp}
degree												
25.1	0.228	3.45	0.4	0.5	10	1	1	0.3	2	1.2	0.1	0.5

*1 For the meaning of the symbols, see (Iai et al., 2011).

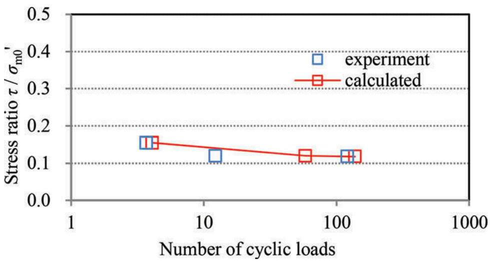


Figure 3. Liquefaction resistance curve ofsaturated Inagi sand

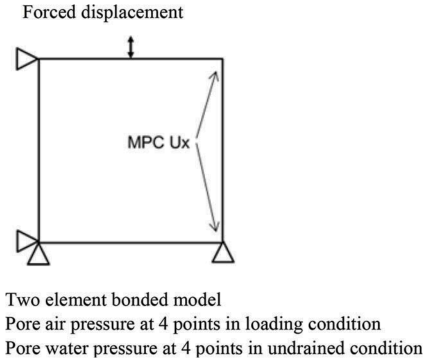


Figure 4. Model for element simulation

2.2 *Parameters of cocktail glass model*

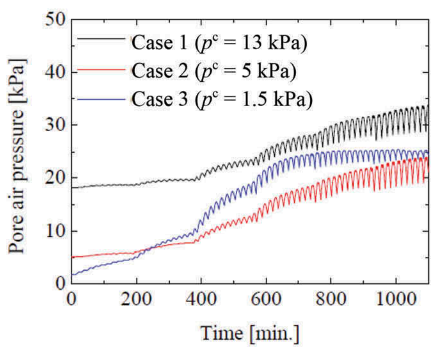
The specimen was modeled by a dual element composed of a cocktail glass model element and a pore fluid element overlapped by node sharing. The parameters of the cocktail glass model element were determined by calibration targeting the liquefaction test results (Matsumaru, et al., 2014b) on the saturated Inagi sand. Table 4 shows the determined parameters. Based on these parameters, a simulation of the liquefaction test was conducted. An example of the analysis result is shown in Figure 3.

2.3 *Element simulation of initial state setting and cyclic triaxial test*

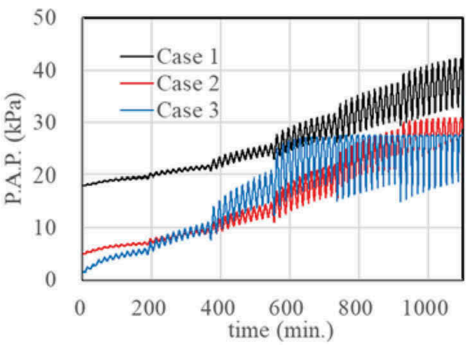
We used the dual element shown in Figure 4 for the element simulation. The parameters of the pore fluid element were determined based on the parameters shown in Tables 2 and 3. In the initial state setting, the pore air pressure "p_a", the pore water pressure "p_f", and the total stress "σ" shown in Table 1 were applied to the model. The procedure of loading was as follows; First, air pressure and pore water pressure were applied, and then total stress was loaded. After that, a simulation of cyclic triaxial test was conducted as a dynamic analysis.

2.4 *Results*

Figures 5 to 7 show the time history of pore air pressure, pore water pressure, and volumetric strain that are obtained in cyclic triaxial tests. In each of these figures, the test results (Matsumaru et al., 2014a) are shown on the left side and the simulation results are shown on the right side. As a result, the proposed method and test results agreed well. Hence, it was shown that FLIP-TRIP with the cocktail glass model for unsaturated sand can express well the basic behavior of unsaturated soil.

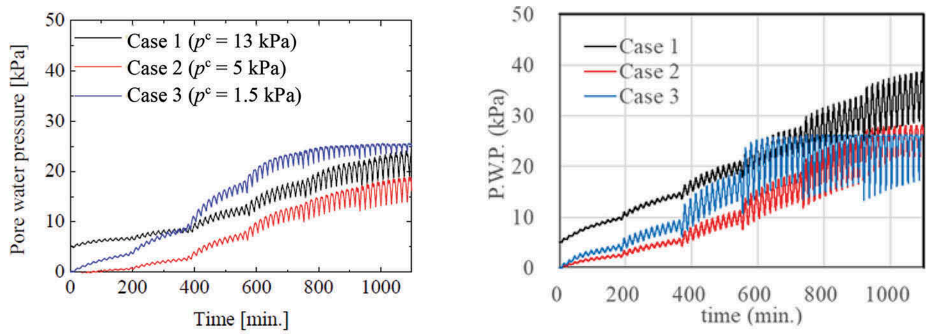


(a) Experiment



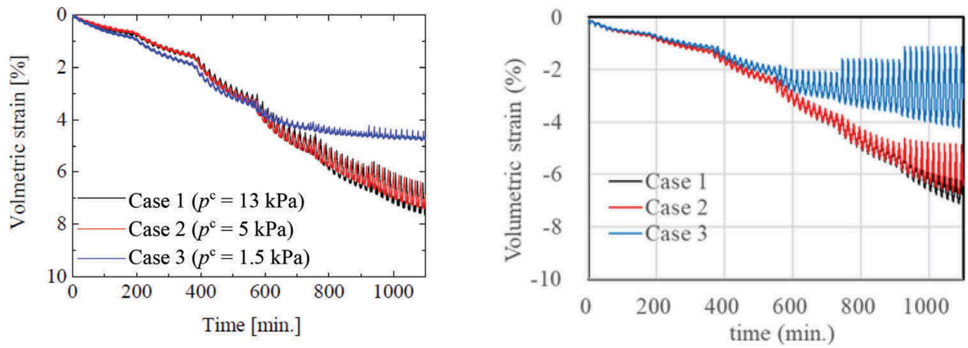
(b) Simulation

Figure 5. Time history of pore air pressure



(a) Experiment (b) Simulation

Figure 6. Time history of pore water pressure



(a) Experiment (b) Simulation

Figure 7. Time history of volumetric strain

3 SIMULATION OF SHAKING TABLE TEST

3.1 *Outline*

A schematic diagram of the shaking table test is shown in Figure 8. Excitation of the model was conducted under 1G gravitational field. The objective of this test is to understand the seismic responses of the embankment under different condition of rainfall and water infiltration (Matsumaru et al., 2014a). Table 5 shows the case conditions of the tests. In this paper, the results of the simulation on Case 1 are described.

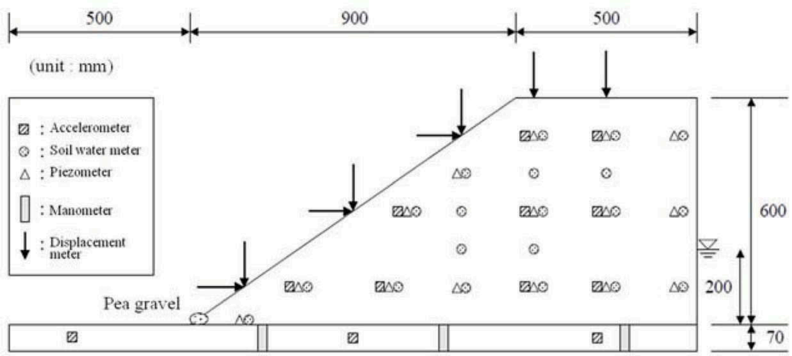


Figure 8. A schematic diagram of shaking table test (Matsumaru et al., 2014a)

Table 5. Case list of tests

Case	Infiltration and rainfall condition	Input motion and its magnitute
Case 1	Infiltration from back side, water level 20 cm, 4 hours	Sine wave 200 Gal, 400 Gal
Case 2		Hyogo ken Nambu earthquake motion of 1995 400 Gal, 600 Gal
Case 3	Rainfall, 30 mm/hr, 4 hours	Hyogo ken Nambu earthquake motion of 1995 600 Gal, 800 Gal, 1000 Gal

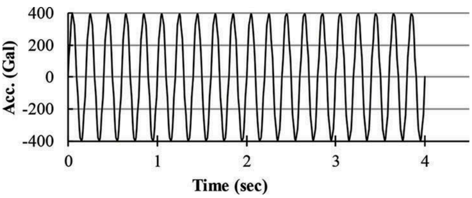


Figure 9. Input motion (400 Gal)

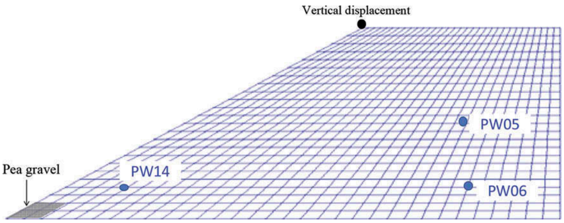


Figure 10. Mesh model and output nodes

3.2 *Analysis condition*

The shaking table test was carried out as follows: First, the water level on the back of the embankment was kept at 20 cm from the ground level, and water infiltration was carried out. Then, after the seepage line was formed at the foot of the slope, excitation was carried out with 20 oscillations of sine wave at 5 Hz.

As for the procedure of simulation, seepage analysis was conducted first, and then, initial stress analysis and the seismic response analysis were carried out continuously. Figure 9 shows the input wave which was used in the seismic response analysis in the case of amplitude of 400 gal. The parameters for the analysis were the same as previous test as shown in Tables 3 and 4. Note that, "ρ" in Table 4 was changed to 1.696 t/m^3, and horizontal and vertical permeability was newly added as 1.86×10^{-4} m/s. Figure 10 shows the output nodes, which correspond to the sensor location of the test.

The computer code we used was the same as previous simulation, which means the cocktail glass model and the generalized fluid pressure vector of FLIP-TRIP were used. In order to make the analysis stabilized, Rayleigh damping "βe" of the pore fluid element in case of input amplitude 200 Gal was set to 400 times ($\beta e = 6.0 \times 10^{-5}$) of Iai method (FLIP Study Group, 2011). For the case of 400 Gal, it was set to 100 times ($\beta e = 1.5 \times 10^{-5}$) of Iai method. In the shaking table model, in order to prevent the collapse of the embankment only by the seepage flow, pea gravel whose permeability is higher than the embankment material was arranged in the foot of the slope. In the simulation, parameters of pea gravel was basically same as the parameters of embankment, only changing the permeability into 1.0×10^{-3} m/s, and the air permeability into 1.0×10^{-4} m/s. Regarding the boundary conditions, in the seepage flow analysis and the initial stress analysis, the embankment surface was set to zero air pressure boundary, and the pea gravel was set to zero boundary for both air pressure and water pressure. In a seismic response analysis, both the embankment surface and pea gravel were provided with zero air pressure and zero water pressure boundary.

3.3 *Results*

Both Figures 11 and 12 show the distribution of the degree of saturation before shaking. Figure 11 corresponds to the test results, and Figure 12 the analysis results. Focusing on the

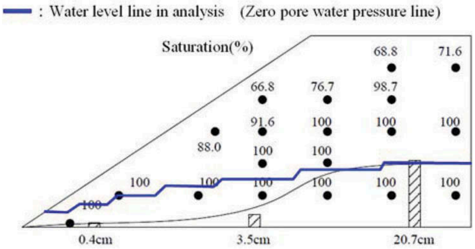


Figure 11. Water level line and distribution of degree of saturation in the test (Matsumaru et al., 2014a), (Added a water level line as zero pore water pressure in the simulation)

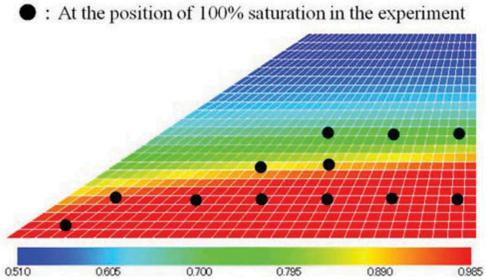


Figure 12. Distribution of degree of saturation obtained in the analysis (Measurement results of Sr=100% in experiment are plotted)

region with the saturation degree of 100%, though the observed region spreads toward the shallower part than the region obtained by the simulation, as a whole the two are almost the same. On the other hand, the degree of saturation in the most upper part of the embankment do not agree, which means 60 to 70% in the test, and 50% in the simulation. In addition, as for the ground water level in the test and the simulation shown in Figure 11, though the differences can be seen around the foot of the slope, from the center to the back side of the embankment, both lines agree well with each other.

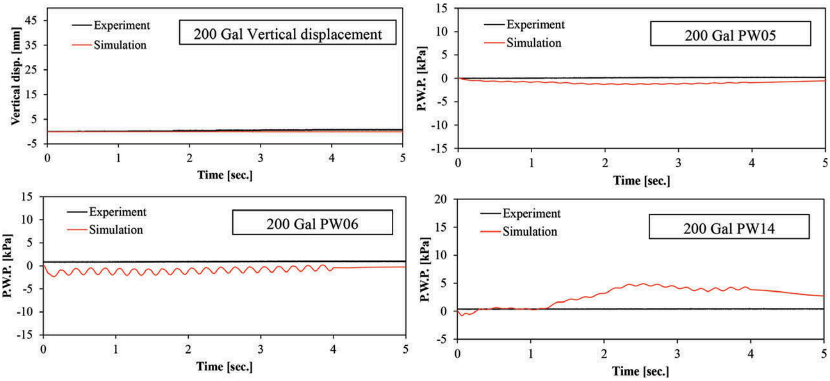


Figure 13. Comparison of time history of experiment and simulation (200Gal)

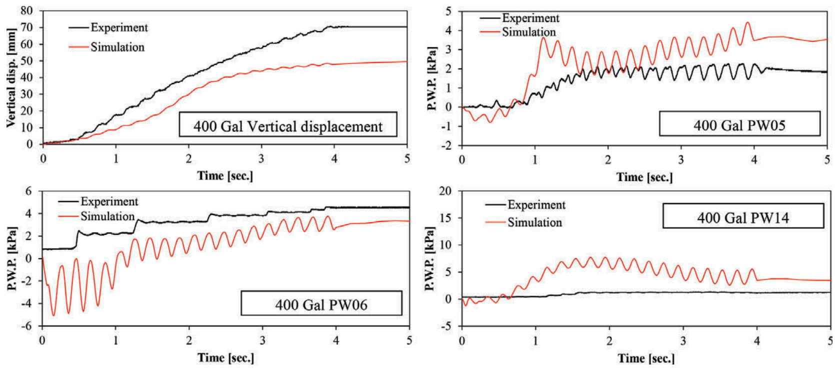


Figure 14. Comparison of time history of experiment and simulation (400 Gal)

Comparisons of time histories of the observed and the simulation results are shown in Figures 13 and 14. In case of input amplitude 200 Gal, the vertical displacement of both hardly occurred, then both results agree well. Regarding the time histories of water pressure, the simulation result at PW05 reproduced the test result. As for PW06 and PW14, though the fluctuation can be seen for the simulation results only, overall tendencies agreed.

For the case of 400 Gal, the residual settlement of the simulation was slightly smaller than that of the test. Regarding the time histories of the water pressure, the simulation result of PW05 reproduced well the fluctuation and the overall tendencies. For PW06 and PW14, the same as the case of 200 Gal, though fluctuation can be seen in the simulation results, overall tendencies that pore water pressure rises as time goes by agreed.

4 CONCLUSION

In this study, simulation analyses for two different tests, using the cocktail glass model (Iai et al., 2011) and the generalized pore pressure vector (Iai, 2018), were carried out. One target test was cyclic triaxial test of unsaturated soil (Matsumaru et al., 2014a), and the other target test was shaking table test for embankment (Matsumaru et al., 2014a).

As for the cyclic triaxial test, the results of the simulation for pore air pressure, pore water pressure, and volumetric strain agreed very well with test results. Hence, it was shown that proposed method can simulate the basic behavior of unsaturated soil in terms of pore pressure and deformation in a specimen level.

As for the simulation analysis of the shaking table test, the region with the saturation degree of 100% before shaking based on the test results and the region based on the simulation results, the two were almost the same. Regarding the settlement, the simulation reproduced well the residual displacement for the case of 200 Gal, but underestimated it for the case of 400 Gal. Regarding the pore water pressure, the simulation reproduced the overall tendencies of the test results, though the simulation results showed the fluctuation of pore water pressure, which was not observed in the test for the case of 200 Gal and 400 Gal. Hence, it was shown that the proposed method is applicable to the evaluation of the settlement when the magnitude of input motion is less than 200 Gal. Besides, the method can be applicable to estimation of the overall tendencies of the pore water pressure.

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