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SOIL LIQUEFACTION POTENTIAL STUDYING IN THE SOUTH OF URMIA PLAIN

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ABSTRACT

Liquefaction is one of the main topics of seismic geotechnics. The effects of liquefaction on structures and installations during an earthquake can be very destructive. In the two earthquakes Alaska and Niigata Japan in 1964, spectacular examples of earthquake-induced failures, such as rupture of slopes, deformation of foundation in buildings and bridges, and the floating of buried structures a result of the flow of soil bed occurred. In liquefaction in general, soils tend to be denser when non-adherent, saturated and loose soils are exposed to earthquake vibrations. However, in certain granulation ranges, drainage is somewhat slow, as the rapid fluctuations in the soil due to lack of drainage will necessarily increase the pore pressure. Increasing pore water pressure results in a reduction in effective stress, which results in reduced shear resistance of the soil. Based on the relationship between the geostatic stresses in the soil, the increase in pore pressure may reduce the effective stress in the soil, which results in low or even zero shear strength and, consequently, Soil flows into fluid state, which is called liquefaction. In this research, we will study this phenomenon in the East of Urmia plain using the Japanese specifications for highway bridges 1999. The basis of this method is the evaluation of the liquefaction potential, by comparing the liquefaction resistance assessed from the soil SPT with the shear stress ratio induced by the earthquake to the normal effective stress from the earthquake. Of the 6 study boreholes, only one of the boreholes with high-liquefaction risk was evaluated. So the risk of liquefaction is very low. In general, the southern part of Urmia region requires more geotechnical studies due to the density of existing industries and roads, and also geophysical studies such as the use of shear waves.

Keywords: Liquefaction, Standard Penetration Test, Southern Urmia Plain, Soil, Seismic Geotechnics, Japan highway bridges 1999

مطالعه پتانسیل روانگرایی خاک در جنوب دشت ارومیه با بررسی آئین نامه پل های بزرگ راهی ژاپن1999

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چکیده: روانگرایی یکی از عناوین اصلی ژئوتکنیک لرزه ای است. اثرات روانگرایی بر روی سازه ها و تاسیسات در هنگام زلزله می تواند بسیار مخرب باشد. در دو زلزله آلاسکا 1964 و نیگاتا ژاپن 1964، نمونه های جالبی از گسیختگی شیبها، گسیخگی شالوده ساختمان ها و پلها و شناوری سازه های مدفون بر اثر روان شدن خاک بستر اتفاق افتاد. هر گاه خاکهای غیر چسبنده، اشباع و شل در معرض ارتعاشات زمین حین زلزله قرار گیرند خاکها تمایل به تراکم پیدا می کنند این پدیده را روانگرایی گویند. بر اساس رابطه تنش های ژئواستاتیکی در خاک، ممکن است ازدیاد فشار منفذی سبب کاهش تنش موثر در خاک شده که منجر به کم و یا حتی صفر شدن مقاومت برشی شود. به این حالت سیال‌گونه خاک اصطلاحا روانگرایی گفته می‌شود. در این مقاله به مطالعه این پدیده در جنوب دشت ارومیه با استفاده از آئین نامه پل‌های بزرگ راهی ژاپن1999خواهیم پرداخت. اساس این روش با مقایسه مقاومت روانگرایی ارزیابی شده از دانه بندی و عدد SPT خاک با نسبت تنش برشی (یعنی نسبت تنش برشی ایجاد شده توسط زلزله بر تنش موثر عمودی) ناشی از زلزله می باشد. از تعداد 6 گمانه مورد مطالعه فقط یکی از گمانه ها با خطر روانگرایی بالا مورد ارزیابی قرار گرفت. پس خطرپذیری روانگرایی در خاک جنوب دشت اورمیه خیلی پائین است. در حالت کلی منطقه جنوب دشت ارومیه به علت تراکم صنایع و راه های موجود خصوصا راه آهن جدیدالاحداث ارومیه- مراغه مستلزم بررسی های بیشتر ژئوتکنیکی و همچنین مطالعات ژئوفیزیکی از جمله استفاده از امواج برشی است.

واژه‌هاي كليدي: روانگرایی، آزمایش مقاومت نفوذ استاندارد، جنوب دشت ارومیه، خاک، ژئوتکنیک لرزه ای، پل های بزرگ راهی ژاپن1999

1. INTRODUCTION

Liquefaction is one of the main topics of seismic geotechnics. The effects of liquefaction on structures and installations during an earthquake can be very destructive (F. Canaslan Comut, 2016). In the two earthquakes Alaska and Niigata Japan in 1964, spectacular examples of earthquake-induced failures, such as rupture of slopes, deformation of foundation in buildings and bridges, and the floating of buried structures a result of the flow of soil bed occurred (M. Motagh et al 2017). soils tend to be denser when non-adherent, saturated and loose soils are exposed to earthquake vibrations. However, in certain granulation ranges, drainage is somewhat slow, as the rapid fluctuations in the soil due to lack of drainage will necessarily increase the pore pressure (Das, 2010). Increasing pore water pressure results in a reduction in effective stress, which results in reduced shear resistance of the soil. Based on the relationship between the geostatic stresses in the soil, the increase in pore pressure may reduce the effective stress in the soil, which results in low or even zero shear strength and, consequently, fluid state of the soils, which is called liquefaction (Alizadeh 1390). Generally, liquefaction occurs in uniformly

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saturated sand deposits with a loose or moderate condensation state (Islami, 2007). Considering the importance of the southern regions of Urmia Plain where construction of major industrial and road projects occurs such as Zobahan, Petrochemical, Power Plant, National Railways and Human Population in this region and the development of civil engineering projects, this research has been performed for determination of the liquefaction ability of the studied area (Alizadeh, 2011).

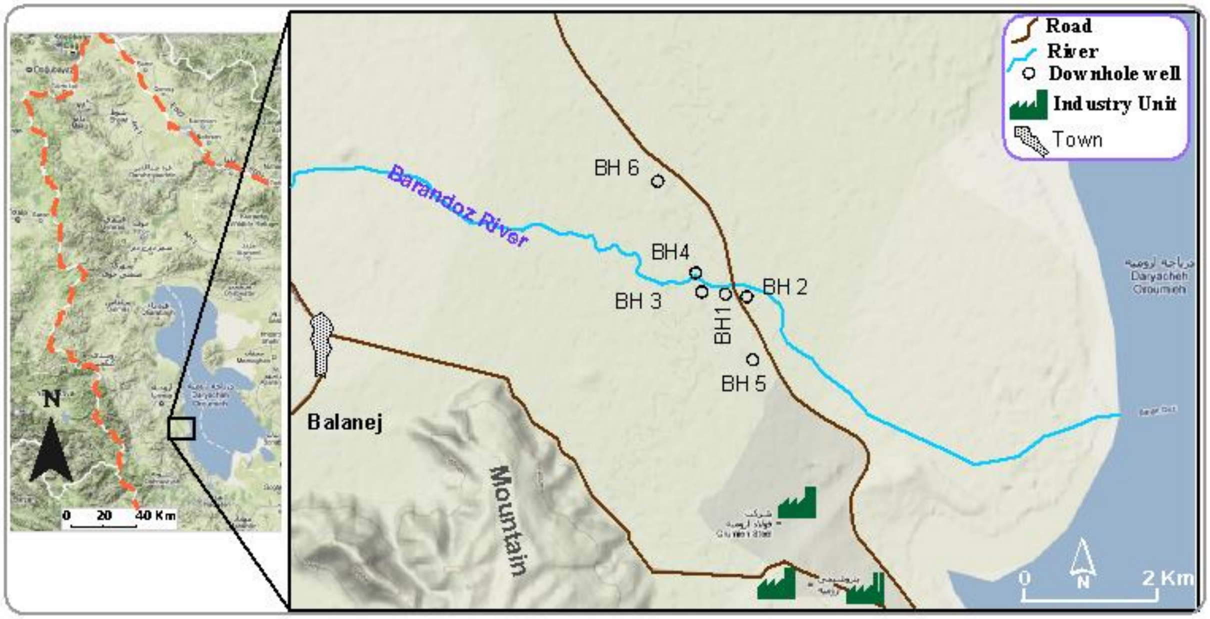

Figure 1. The geographic location of the east of Urmia plain and the location of drilled boreholes

2. MATERIAL AND METHODS

Several methods have been introduced to evaluate the liquefaction potential. In this study, the Japanese specifications for highway bridges 1999 has been used. The basis of this method is the evaluation of the liquefaction potential, by comparing the liquefaction resistance assessed from soil SPT with the shear stress ratio induced by the earthquake to the normal effective stress from the earthquake. (Robert ,2002)

2.1. Properties of soils susceptible to liquefaction

A) the hydrostatic level is greater than 10 meters;

B-) The desired layer has a depth of less than 20 meters.

C) $0.02 < D50 \leq 2$ mm.

D) ($PI \leq 15\%$) and ($FC \leq 35\%$) (Japanese specifications for highway bridges 1999)

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Of course, in Section (c), according to new studies, sandy soils with a D50 of over 2 mm can also be liquefying. (Noorzad, 2010).

2.2. Cyclic Stress Ratio (CSR)

With the CPR 1 ratio and the CSR Cyclic stress ratio, we can obtain the confidence coefficient for the studied soil for an earthquake of 7.5 magnitude. Given the fact that the magnitude of the Investigated plan earthquake may be above or below 7.5, the earthquake correction factor should be defined. (Sonmez et al, 2005)

$$(CSR) = \frac{\sigma_{\max}}{\sigma'_{vo}} = \frac{a_{\max}}{g} \cdot \frac{\sigma_{vo}}{\sigma'_{vo}} .rd \qquad (1)$$

The Cyclic stress ratio is equal to equation 1, in which is the maximum ground acceleration, is total stress, is effective stress, is gravity and is stress reduction coefficient, which is a function of the depth of the desired layer and its value is obtained from equation 2. (Japanese specifications for highway bridges 1999).

$$rd = 1 - 0.015z \qquad (2)$$

2.3. Cyclic Resistance Ratio

From the accumulation of three factors of slag pressure, grain size and fineness, the cyclic shear value is obtained, which its equations are given below. equations 3,4 and 5

$$R_L = CRR = R_1 + R_2 + R_3 \qquad (3)$$

$$R_3 = 0.0882\sqrt{\frac{N_j}{\sigma'_v + 0.7}} \qquad (4)$$

$$R_2 = \begin{cases} 0.19 \quad 0.02mm \le D_{50} < 0.05 \\ 0.225\log\frac{0.35}{D_{50}} \qquad 0.05mm \le D_{50} < 0.6mm \\ -0.05 \quad 0.6mm \le D_{50} < 2.0n \end{cases} \qquad (5)$$

$$R_3 = \begin{cases} 0.0 \dots\dots\dots\dots & 0\% \le Fc < 40\% \\ 0.004FC - 0.16 \dots & 40\% \le Fc < 100\% \end{cases} \tag{6}$$

$$N_j = 0.833(N_1)_{60}) \tag{7}$$

D50: Mean grain size, FC: grain percent, σ'_v :Effective vertical stress at study depth (Kpa), and NJ: SPT number (Japanese specifications for highway bridges 1999).

$$F_L = \left(\frac{CRR_{7.5}}{CSR}\right).MSF.K_\sigma.K\alpha \tag{8}$$

MSF is the correction factor for earthquake magnitudes, K_σ the correction coefficient for the overhead stresses on the soil and $K\alpha$ the correction coefficient of the initial shear stresses on the soil in the static state.(Zhang,2015)

$$K_\sigma = \left(\frac{\sigma'\partial}{P_a}\right)f - 1 \tag{9}$$

σ' Effective overhead stress on the soil, pa atmospheric pressure, and f power a function of relative density, tensile history, sediment age and pre-consolidation ratio. f Relative density function D_r for relative density of 40 to 60 percent of f power varies between 0.8 and 0.7, and for relative density between 60 and 80 percent of f power values, 0 lies between 0.7 and 0.6 σ'_{v0}

$$\alpha = \frac{\tau st}{\sigma' vo} \tag{10}$$

τ_{st} Static shear stress on the soil layers (the weight of the upper heavy structure or the soil slope, if any), σ'_{v0} effective overhead stress

$$W(Z).d(Z)P_L = \int_0^Z F(Z) \tag{11}$$

Z depth from the earth surface in meters, F (Z) is a function of the fluid resistance parameter F_L, obtained from equation 8. (Maurer,2015)

$$F(Z) = 1 - F_L \tag{12}$$

F_L The confidence coefficient of liquefaction occurrence, if $F_{L>1}$, then $F(Z)$ is equal to zero.

The PL value is between zero and 100. (Sonmez et al, 2005)

$$W(Z) = 10 - 0.5Z \quad (13)$$

3. RESULTS AND DISCUSSION

Six boreholes have been excavated to study the soil properties and field experiments in the area, which are mostly focused on the Barandouzchay River Bridge (Alizadeh, 2011). To determine soil liquefaction potential, we first study the soil properties according to 1. In case of soil liquefaction conditions, we will investigate the CSR according to 2 and the cyclic resistance ratio (RL) using clause 3. Given the above values, in the case of obtaining values less than 1 for FL, the desired layer bears the liquefaction potential.

The results of field experiments and soil characteristics for the BH1 borehole are presented in Table 1.

Table 1. Results of field and laboratory experiments in BH1 borehole

Depth of experiment(m)	SPT (N)	Kind of materials	Group of soil	Sieve passed 200%	Sieve passed 4%	PI	LL %	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	Descried of materials
3	11	Clay with low Softness	CL	90	100	24	45	<0.001	0.0062	12	1.7	average
5	17	Clay with low Softness	CL	92	100	24	43	<0.001	0.007	17	1.7	Stiff
7	30	sand with low gravel	GP-GM	9	48	NP	-	0.1	6.0	30	1.7	dense
10	31	Clay with low Softness	CL	91	100	21	35	<0.001	0.023	27	1.7	Very stiff
12	24	Clay with low gravel	CL	89	100	18	35	<0.001	0.016	20	1.7	Very stiff
15	11	Clay with	CL	77	100	12	29	<0.001	0.04	8	1.7	Stiff

		low gravel										
17	14	Clay with low Softness and gravel	CL	80	100	23	41	<0.001	0.01	10	1.7	Stiff
18	17	Clay with high Softness	CH	91	100	35	56	<0.001	0.0026	12	1.7	Very stiff
20	26	Clay with high Softness	CH	90	100	34	55	<0.001	0.0025	18	1.7	Very stiff
24	19	Clay with low Softness and gravel	CH	64	95	32	51	<0.001	0.007	12	1.7	Very stiff
25	18	Clay with low Softness and gravel	CL	67	93	11	29	<0.001	0.05	11	1.7	Very stiff

For BH1 boreholes, soil conditions are only prone to a depth of 15 meters. The proportions of stress ratios and RL ratios, as well as the relationships listed in Table 2 for BH1 boreholes, are presented.

Table 2. Calculation of FL and PL coefficient of BH1 borehole

Z(m)	N SPT	σ'v0 (Kpa)	σv0 (Kpa)	CSR	CRR7.5	MSF	Kσ	Kα	FL	PL %
15	11	177.64	310.64	0.298	0.355	1.32	1.33	1.0	2.1	0
										PL BH1=0

According to Table 2, the BH1 borehole is not susceptible to liquefaction at a depth of 15 meters.

The results of field experiments and soil characteristics for the BH2 borehole are given in Table 3.

Table 3. Results of Field and Laboratory Experiments in BH2 Borehole

Depth (m) of experime	SPT (N)	Kind of materials	Group of soil	Sieve passed 200%	Sieve passed 4%	PI	LL %	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	Descried of materials
2	12	Silty gravel with sand	SM	31	71	NP	-	0.02	0.35	15	1.7	average
4	39	Silty gravel	SM	24	63	NP	-	0.02	1.3	38	1.7	dense
6	24	Silty gravel	SM	8	60	NP	-	0.18	2.8	23	1.7	dense
8	5	Clay with silt and gravel	CL-ML	58	99	5	22	<0.006	0.062	5	1.7	loose
9	14	Clay with gravel	CL	74	99	13	33	<0.001	0.024	13	1.7	Stiff
10	23	Clay with gravel	CL	82	99	21	40	<0.001	0.032	20	1.7	very Stiff
11	26	Clay with gravel	CL	76	9	14	31	<0.001	0.018	22	1.7	very Stiff
13	13	Silty gravel	SM	46	100	NP	-	<0.003	0.14	10	1.7	average
15	13	Clay with high Softness	CH	86	100	33	51	<0.001	0.004	10	1.7	Stiff
16	15	Clay with gravel	CL	74	100	11	26	<0.001	0.048	11	1.7	Stiff
18	17	Clay with high Softness	CH	77	94	37	60	<0.001	0.002	12	1.7	very Stiff
20	19	Gravely clay	CL	62	92	22	39	<0.001	0.035	13	1.7	very Stiff
22	26	Gravely clay	CL	68	86	29	48	<0.001	0.008	16	1.7	very Stiff
24	30	Clay with gravel	CL	73	93	19	34	<0.001	0.028	18	1.7	very Stiff

For BH2 boreholes, soil conditions are prone to 13,11,8,4,2 and 16 m in depth. The coefficients of cyclic stress ratio and cyclic resistance ratio (RL) as well as FL are given using the equations listed in Table 4 for the BH2 borehole.

According to Table 4 at depths of 4.2 and 13 meters, BH2 is prone to liquefaction.

Table 4. Calculation of FL and PL coefficient of BH2 borehole

Z(m)	N SPT	σ‘v0 (Kpa)	σv0 (Kpa)	CSR	CRR7.5	MSF	Kσ	Kα	FL	PL %
2	12	37.24	40.24	0.231	0.051	1.32	1.0	1.0	0.3	13
4	39	58.84	81.84	0.288	0.015	1.32	1.0	1.0	0.1	29
8	5	102.04	165.04	0.313	0.258	1.32	1.01	1.0	1.1	0
9	14	112.84	185.84	0.313	0.353	1.32	1.06	1.0	1.6	0
11	26	134.44	227.44	0.311	0.366	1.32	1.16	1.0	1.8	0
13	13	156.04	269.04	0.305	0.134	1.32	1.31	1.0	0.8	9
16	15	188.44	331.44	0.294	0.345	1.32	1.37	1.0	2.1	0
										PL BH2=7

The results of field experiments and soil characteristics for the BH3 borehole are given in Table 5.

A depth of 2 meters of groundwater level was reported for BH3 and BH4 boreholes during drilling.

For BH3 samples, soil conditions are not susceptible to liquefaction, so they are ignored.

Table 5. Results of Field and Laboratory Experiments in Boreholes

Depth (m) of experiment	SPT (N)	Kind of materials	Group of soil	Sieve passed 200%	Sieve passed 4%	PI	LL %	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	Descried of materials
2	12	Clay with low softness	CL	82	99	17	37	<0.001	0.0042	15	1.5	average
4	11	Clay with low softness	CL	50	96	17	34	<0.001	0.013	11	1.5	average
7	16	Clay with low softness and gravel	CL	82	97	20	39	<0.001	0.0037	16	1.5	Stiff
9	17	Clay with low softness and gravel	CL	83	100	19	39	<0.001	0.005	15	1.5	very Stiff
11	30	Clay with low softness	CL	72	100	20	40	<0.001	0.0065	26	1.5	very Stiff
14	26	Clay with low softness	CL	77	98	19	39	<0.001	0.005	20	1.5	very Stiff
16	23	Clay with low softness and gravel	CL	77	98	24	44	<0.001	0.0042	17	1.5	very Stiff
18	58	Clay with low softness	CL	82	100	25	47	<0.001	0.0022	40	1.5	strong

The results of field experiments and soil characteristics for the BH4 borehole are presented in Table 6. For BH4 boreholes, soil conditions are only prone to a depth of 13 meters. The coefficients of cyclic stress ratio and cyclic resistance ratio (RL) as well as FL are given using the equations listed in Table 7 for the BH4 borehole. According to Table 7, BH4 borehole is not susceptible to liquefaction at a depth of 13 meters.

The results of field experiments and soil characteristics for the BH5 borehole are given in Table 8.

Table 6. Results of Field and Laboratory Experiments in BH4 Borehole

Depth of experime (m)	SPT (N)	Kind of materials	Group of soil	Sieve passed 200%	Sieve passed 4%	PI	LL %	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	Descried of materials
2	9	Clay with low softness	CL	83	100	22	43	<0.001	0.0035	11	1.5	average
5	8	Clay with low softness	CL	87	100	17	36	<0.001	0.0055	8	1.5	average
8	31	Clay with low softness and gravel	CL	84	99	22	42	<0.001	0.0035	12	1.5	Stiff
11	21	Clay with low softness	CL	80	97	17	35	<0.001	0.005	10	1.5	average
13	30	Clay with low softness	CL	68	100	12	28	<0.001	0.018	10	1.5	very Stiff
15	5	Clay with low softness	CL	84	98	22	41	<0.001	0.005	4	1.5	weak
17	33	Clay with low softness	CL	79	100	23	42	<0.001	0.0038	24	1.5	strong
19	28	Clay with low softness	CL	80	100	20	39	<0.001	0.0048	19	1.5	very Stiff

Table 7: Calculation of FL and PL coefficient of BH4 Borehole

Z(m)	N SPT	‘v0σ (Kpa)	v0σ (Kpa)	CSR	CRR7.5	MSF	Kσ	Kα	FL	PL %
13	12	154.2	269.2	0.309	0.322	1.32	1.24	1.0	1.7	0
										PL BH4=0

The depth of 1.5 meters of groundwater level for BH5 boreholes was reported at drilling time. For BH5, the soil conditions are not susceptible to liquefaction; therefore, they are neglected.

The results of field experiments and soil characteristics for the BH6 borehole are presented in Table 8. The depth of 1.5 meters of groundwater level for BH6 boreholes was reported at drilling time.

For the BH6 borehole, soil conditions are only prone to a depth of 6 meters. The coefficients of cyclic stress ratio and cyclic resistance ratio (RL) as well as FL are given using the equations listed in Table 9 for the BH6 borehole.

Table 8. Results of Field and Laboratory Experiments in BH6 Borehole

(m)	SPT (N)	Kind of materials	Group of soil	Sieve passed 200%	Sieve passed 4%	PI	LL %	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	Described of material
2	6	Clay with low softness	CL	89	100	23	46	<0.001	0.0038	7	2.0	soft
4	8	Clay with low softness	CL	91	100	18	37	<0.001	0.0082	8	2.0	average
6	6	Clay with low softness	CL	88	100	11	29	<0.0014	0.003	6	2.0	average
8	4	Clay with low softness	CL	90	100	16	36	<0.001	0.013	4	2.0	soft
10	17	Clay with low softness	CL	88	100	20	42	<0.001	0.0063	14	2.0	Very stiff
13	18	Clay with low softness	CL	88	100	17	36	<0.001	0.01	14	2.0	Very stiff
15	22	Clay with low softness and gravel	CL	85	100	13	30	<0.001	0.033	16	2.0	Very stiff

Table 9. Calculation of FL and PL coefficient of BH6 Borehole

Z(m)	N SPT	σ‘v0 (Kpa)	σv0 (Kpa)	CSR	CRR7.5	MSF	Kσ	Kα	FL	PL %
6	6	83.2	123.2	0.296	0.404	1.32	1.0	1.0	1.8	0
										PL BH6=0

According to Table 9, the BH6 borehole is not prone to liquefaction at a depth of 6 meters.

4. CONCLUSIONS

Of the 6 existing boreholes, the liquefaction potential obtained for the BH1 borehole is zero. Therefore, the risk of liquefaction is very low. The liquefaction potential for the BH2 borehole is equal to 7. The liquefaction risk is high, therefore, this range requires further research. The BH5 and BH3 boreholes are not prone to liquefaction. For BH6 borehole it is equal to zero. Therefore, the risk of liquefaction is very low. In general, due to the density of industries and existing roads, the southern part of Urmia plain requires further geotechnical studies including boreholes, as well as geophysical studies, including the use of shear waves.

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