SEISMIC EVALUATION OF 300 DAMAGED BUILDINGS IN DARB-E-ASTANEH, IRAN EARTHQUAKE OF MARCH 2006 FOR THEIR REPAIR DESIGN

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ABSTRACT :

In 31st March of 2006 an earthquake of 6.1 magnitude hit the Silakhor plain in Lorestan Province, Iran, which affected cities of Boroojerd and Dorood, and created damage in hundreds of building, including houses in these two cites and their neighboring villages. 'Behsazeh-Andishan Aria Consulting Engineers' has been chosen as one the consulting firms for seismic evaluation and retrofit or repair design of damaged buildings, and has been responsible for around 300 buildings in Boroojerd. Most of these buildings were of non-reinforced masonry (almost 70% of residential buildings in Iran are to masonry type, mostly non-reinforced), and semi-skeleton type. At first, a visual screening was done to select the buildings which were worth to repair. The selection criterion was the estimated damage of less than 70%. The visual inspections were focused on the visible structural and nonstructural deficiencies of the building, including the lack of structural system, particularly the lateral load bearing mechanism, the disruption in the seismic load path, cracks in walls, and other major damages. In cases of buildings which were realized as repairable the required measurements were made, so that the architectural and structural maps can be prepared. In the next step, the seismic capacity of each building was calculated based on FEMA 356 Guidelines, and then some analyses were performed by reduced strength and stiffness based on FEMA 306 Guidelines. The results of these calculations are given and discussed in detail in this paper.

KEYWORDS: Boroojerd City, Non-reinforced Masonry, Semi-skeleton Buildings, Visual Screening

1. INTRODUCTION

In 31st March of 2006 an earthquake of 6.1 magnitude hit the Silakhor region (Epicenter close to Darb-e Astaneh), Lorestan Province, Iran, which affected cities of Boroojerd and Dorood, killed 66, injured 1289, and damaged hundreds of building, including houses in these two cites and their neighboring villages (Figure 1).

Figure 1. Samples of damages to city buildings (left) and village buildings (right) (IIEES, 2006)

Behsazeh-Andishan Aria Consulting Engineers was chosen as one of the consulting firms for seismic evaluation and retrofit or repair design of damaged buildings, and was responsible for around 300 buildings in Boroojerd. Most of these buildings were of non-reinforced masonry, as shown in Table 1.

Table 1. The general information about the evaluated buildings

Building Type	No. of Stories			Sub-total
	1	2	3	
Masonry	118	142	38	298
Steel	0	2	3	5
Total				303

It is worth mentioning that almost 70% of residential buildings in Iran are of masonry type, mostly non-reinforced, and a small percent are of semi-skeleton type. Therefore, the results of this study can be very useful for whole country particularly those cities which have building architecture and texture similar to Boroojerd. To evaluate the considered damaged buildings for retrofit, at first a visual screening was done to select the buildings which were worth to be evaluated in detail to realize if it is worthy enough to retrofit them. The selected buildings then were evaluated in detail and the ones which were realized repairable by a reasonable cost were finally introduced. Regarding the of the 303 damaged buildings at hand, 298 were brick masonry buildings, evaluation of the 5 steel buildings is not discussed here.

2. QUICK (QUALITATIVE) EVALUATION OF BUILDINGS BY VISUAL INSPECTION

The criterion for selection of buildings for further evaluation was the estimated damage of less than 70%. The visual inspections, which were done based on ATC-21 forms, were focused on the visible structural and nonstructural deficiencies of the building, including the lack of structural system, particularly the lateral load bearing mechanism, the disruption in the seismic load path, cracks in walls, and other major damages. The issues, which were checked qualitative evaluation, include:

- Continuity in the vertical and lateral load path
- Integrity of the building
- Diaphragm(s) integrity
- Components (if any) which can be helpful for carrying the lateral loads
- The vertical separation joints between masonry units
- Connections of crossing walls
- Distance of openings from walls edges
- Vertical and plan irregularities

3. QUANTITATIVE EVALUATION OF BUILDINGS BY NUMERICAL ANALYSES

In cases of buildings which were realized as repairable the required measurements were made, so that the architectural and structural maps can be drawn. Then, the seismic capacity of the building was calculated based on FEMA 356 Guidelines, and then some analyses were performed by reduced strength and stiffness based on FEMA 306 Guidelines, as explained briefly in the following sections.

3.1. Evaluation of Seismic Forces

The minimum values of shear force, V, along the main axes of the building plan are calculated by Eqn (1).

$$V = 0.33\,AIW \tag{1}$$

Where A, I, and W are respectively design base acceleration, importance factor, and the dead load plus part of the live load of the building based on the seismic design code. Then the lateral force at level i of the building,

F_i, is calculated by Eqn (2).

$$F_i = \frac{W_i h_i}{\sum_{j=1}^{n} W_j h_j}.V \tag{2}$$

in which h_i and W_i are respectively the height of level I above the foundation level and the weight of i^{th} floor. With regard to distribution of lateral forces at each story level between the walls of that story, as experience has shown that the jack arc floors do not act as rigid diaphragm, the tributary areas of walls have been used as the forces distribution factors. Of course, after retrofit, in which the floor diaphragm are strengthened to act as rigid the lateral forces at story levels should be distributed based on the walls' stiffness values.

3.2. Determining the Buildings' Capacities

The building capacity consists of shear and bending capacities of walls. In the case of building at hand because of little heights of walls their shear capacities is dominant, which can be calculated by Eqn (3).

$$v_a = 0.1\, v_t + 0.15\, \sigma_c \tag{3}$$

where v_a is the allowable shear stress of wall and v_t is that of mortar, assumed here to be equal to 1.0 kgf/cm^2 since no material test has been done, and σ_c is the normal stress due to vertical loads. By using Eqn (3) the shear stress values for non-bearing wall at ground and first floors have been obtained respectively as 0.30 and 0.90 kgf/cm^2, and for bearing wall at ground and first floors respectively as 0.80 and 1.9 kgf/cm^2. It should be noted that these values are for intact building, and should be decreased for damaged buildings, as suggested by FEMA-274 guideline, as explained here later. The lateral stiffness of cantilever walls is calculated by:

$$K = \frac{1}{\dfrac{h_{eff}^3}{3E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \tag{4}$$

and for the walls between windows by:

$$K = \frac{1}{\dfrac{h_{eff}^3}{12E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \tag{5}$$

In these equations h_{eff}, A_v, and I_g are respectively the height, the shear area, and the moment of inertia of the wall, and E_m and G_m are the modulus of elasticity and shear modulus of wall material respectively. The values obtained by Eqns (4) and (5) should be decreased based on FEMA-306 guidelines by using the following graph, and its related table, in which λ_k and λ_Q are the reduction factor for stiffness and strength respectively.

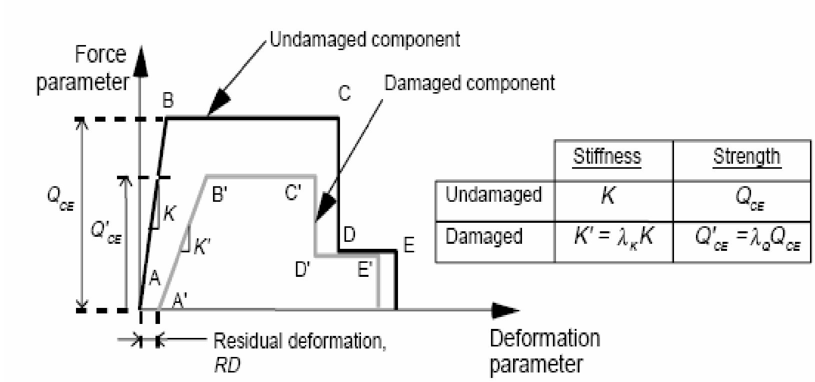

Figure 2. Force-Deformation relationship of masonry walls before and after damage (FEMA-306)

Furthermore, because of cracks at the top and bottom of the walls similar to those shown in Figure 3, some other reduction factors should be used.

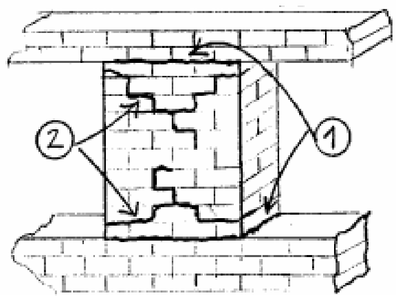

Figure 3. The crack type considered for the walls based on FEMA-306 (λ_k=0.8, λ_Q=0.8)

Also because of adjacent buildings another reduction factor of 0.9 should be used. Another check is with regard to the resistance of wall against out-of-plane forces. For this purpose the force acting on the wall is calculated as:

$$F_P = 0.7\ A\ \ I\ \ W\ = 0.294\ \mathrm{W} \tag{6}$$

And then the stress value due to axial force and bending moment is at the wall section is obtained by:

$$\sigma = \frac{P}{A} \pm \frac{MY}{I} \tag{7}$$

The bending at wall section can be calculated as:

$$M = \frac{\omega H^2}{8} = \frac{F_p \times H}{8} \qquad I = \frac{L \times t^3}{12} \qquad y = \frac{t}{2} \tag{8}$$

The allowable compressional and tensional stresses are supposed respectively as 40.0 and 4.0 kgf/cm^2 for checking the bending resistance of walls.

4. A BUILDING EVALUATION SAMPLE

The evaluation of a 2-story building, with residential use for around 10 people, is presented here as a sample. This building does not have horizontal and vertical ties, and just its walls provide the load paths. The calculation of seismic forces for this building is given in Table 2. The visual inspection of the building, as presented in Table 3, showed that the building needs further investigations, and therefore quantitative evaluation was also performed for it.

Table 2. Calculation of shear and story forces of the sample building

Story	Base shear	Wi (tonf)	Hi (m)	Wi x Hi	(Wi x Hi) / ∑ WiHi	Fi (tonf)	Story shear
Ground	58.55	158.55	3.5	554.93	0.2	11.61	58.55
1st		150.93	7	1056.52	0.38	22.11	46.94
2nd		112.96	10.5	1186.09	0.42	24.82	24.82
∑		422.44		2797.54	1	58.55	

Table 3. Sample of qualitative evaluation of buildings

OCCUPANCY		STRUCTURAL SCORES AND MODIFIERS												
Residential	No. of Persons	Building Type	W	S1	S2	S3	S4	C1	C2	C3/S5	PC1	PC2	RM	URM
Commercial	0-10			(MRF)	(BR)	(LM)	(RC SW)	(MRF)	(SW)	(URM INF)	(TU)			
Office	11-100	Basic Score	4.5	4.5	3	5.5	3.5	2	3	1.5	2	1.5	3	1
Industrial	100+	High Rise	N/A	-2	-1	N/A	-1	-1	-1	-0.5	N/A	-0.5	-1	-0.5
Public		Poor Condition	-0.5	-0.5	-0.5	-0.5	-5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
School		Vertical Irregularity	-0.5	-0.5	-0.5	-0.5	-5	-1	-0.5	-0.5	-1	-1	-0.5	-0.5
Government Building		Soft Story	-1	-2.5	-2	-1	-2	-2	-2	-1	-1	-2	-2	-1
Emergency Service		Torsion	-1	-2	-1	-1	-1	-1	-1	-1	-1	-1	-1	-1
Historical Building		Plan Irregularity	-1	-0.5	-5	-0.5	-0.5	-0.5	-0.5	-0.5	-1	-1	-1	-1
Non-structural		Pounding	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
Falling Hazard		Large Heavy HEAVY Cladding	N/A	-2	N/A	N/A	N/A	-1	N/A	N/A	N/A	-1	N/A	N/A
DATA CONFIDENCE		Short Column	N/A	N/A	N/A	N/A	N/A	-1	-1	-1	N/A	-1	N/A	N/A
		Post Benchmark Year	+2	+2	+2	+2	+2	+2	+2	N/A	+2	+2	+2	N/A
* Estimated subjective		SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
Unreliable data		SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
Do Not Now		SL3 & 8 to 20 stories	N/A	0.8	0.8	N/A	0.8	0.8	0.8	0.8	N/A	0.8	0.8	0.8
		FINAL SCORE											0.9	
COMMENTS			Building property : 2-story masonry building											
			Area : 280 m^2											
Building is vulnerable			Date of inspection: 02/02/2005											
			Date of construction : 1979											

Because of the existence of some cracks in the building, as shown in Figures 4 and 5, the reduction factors were applied to its stiffness and strength as well.

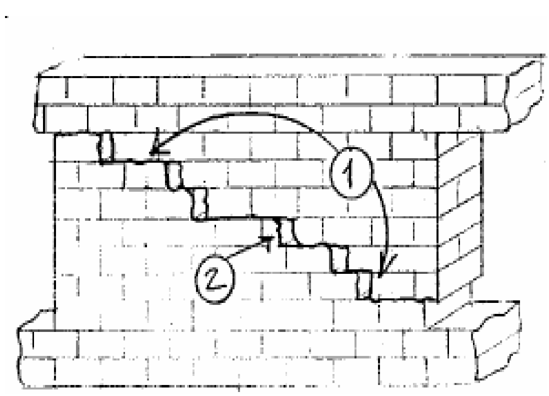	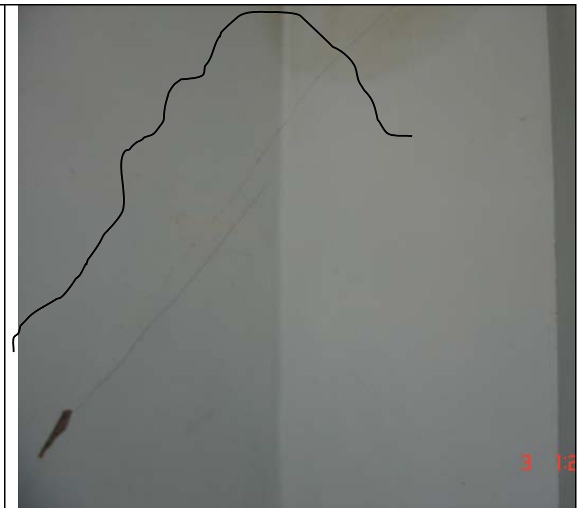
$T_C \sim 5$ mm(1/4”) $\lambda_k = 0.6$ $\lambda_Q = 0.6$	Track code: A Story: 1st Wall direction: N-S Wall No.: 7

Figure 4. A sample of cracks of type A (FEMA-306), observed in the building, and its related reduction factors

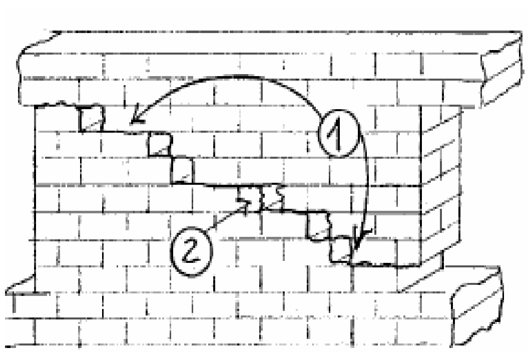	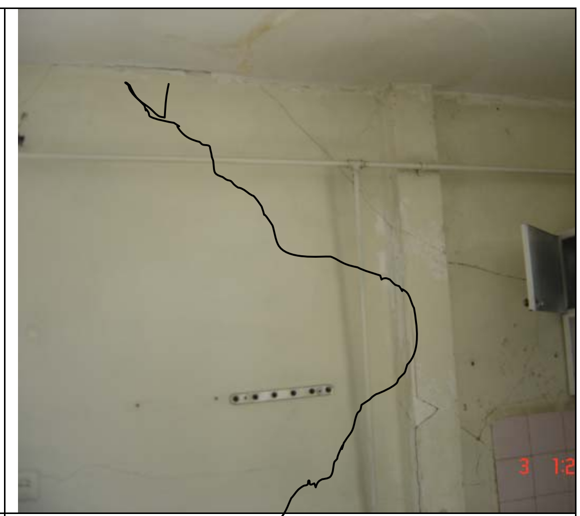
$T_C \sim 10$ mm (1/2”) $\lambda_k = 0.4$ $\lambda_Q = 0.4$	Track code: B Story: 2nd Wall direction: N-S Wall No.: 1

Figure 5. A sample of cracks of type B (FEMA-306), observed in the building, and its related reduction factors

The calculations of building stiffness, both before and after damage, have been performed as presented in Table 4, in which TC is the crack width in mm, λ_k is the stiffness reduction factor, K1 is the stiffness of the building before damage, and K2 its value after damage. The fine cracks in the building have been indicated in Table 4 as HAIR, and a reduction factor of 0.8 have been applied because of them. The K2/K1 ratio for the building, shown by R in Table 4, has been obtained as 0.71 for the N-S walls in the ground floor.

Other checks with regard to minimum relative wall and the allowable shear stress have been performed as well, of which the results can not be presented here because of lack of space, and can be found in the main report of the project (Behsazeh Andishan Aria Consulting Engineers, 2007).

Table 4. A sample of calculations of wall stiffness before and after cracking

Floor	WALL DIRECTION	No.	Crack code	T_c (mm)	λ_k	λ_Q	T	L	h	K_1 (tonf/cm)	K_2 (tonf/cm)	k1/(∑k1)	k2/(∑k2)	R
Ground Floor	S-N	1	HAIR		0.80	0.80	0.35	1.05	3.00	5.74	4.60	0.01	0.01	0.80
		2	HAIR		0.80	0.80	0.35	1.38	3.00	11.14	8.91	0.02	0.02	0.80
		3	B	Tc<10	0.80	0.80	0.35	5.22	3.00	107.59	86.07	0.22	0.22	0.80
		4	HAIR		0.80	0.80	0.35	1.49	3.00	13.26	10.61	0.03	0.03	0.80
		5	HAIR		0.80	0.80	0.20	8.80	3.00	112.12	89.70	0.23	0.23	0.80
		6	HAIR		0.80	0.80	0.20	3.98	3.00	43.24	34.59	0.09	0.09	0.80
		7	HAIR		0.80	0.80	0.35	2.00	3.00	24.56	19.65	0.05	0.05	0.80
		8	HAIR		0.80	0.80	0.35	7.61	3.00	167.17	133.74	0.34	0.34	0.80
		Sum						**31.53**		**484.83**	**387.86**	**1.00**	**1.00**	**0.80**
	W-E	1	HAIR		0.80	0.80	0.35	4.10	3.00	78.79	63.03	0.35	0.35	0.80
		2	HAIR		0.80	0.80	0.35	1.80	3.00	19.89	15.92	0.09	0.09	0.80
		3	HAIR		0.80	0.80	0.35	3.05	3.00	51.31	41.05	0.23	0.23	0.80
		4	HAIR		0.80	0.80	0.20	3.11	3.00	30.22	24.18	0.13	0.13	0.80
		5	HAIR		0.80	0.80	0.20	3.25	3.00	32.32	25.85	0.14	0.14	0.80
		6	HAIR		0.80	0.80	0.20	2.07	3.00	15.00	12.00	0.07	0.07	0.80
		Sum						**17.38**		**227.53**	**182.03**	**1.00**	**1.00**	**0.80**
1st Floor	N-S	1	HAIR		0.80	0.80	0.35	1.05	3.00	5.74	4.60	0.01	0.01	0.80
		2	HAIR		0.80	0.80	0.35	1.38	3.00	11.14	8.91	0.02	0.02	0.80
		3	HAIR		0.80	0.80	0.35	5.22	3.00	107.59	86.07	0.22	0.22	0.80
		4	HAIR		0.80	0.80	0.35	1.49	3.00	13.26	10.61	0.03	0.03	0.80
		5	HAIR		0.80	0.80	0.20	8.80	3.00	112.12	89.70	0.23	0.23	0.80
		6	HAIR		0.80	0.80	0.20	3.98	3.00	43.24	34.59	0.09	0.09	0.80
		7	A	Tc<5	0.60	0.60	0.35	2.00	3.00	24.56	14.74	0.05	0.04	0.60
		8	HAIR		0.80	0.80	0.35	7.61	3.00	167.17	133.74	0.34	0.35	0.80
		Sum						**31.53**		**484.83**	**382.95**	**1.00**	**1.00**	**0.79**
	W-E	1	HAIR		0.80	0.80	0.35	4.10	3.00	78.79	63.03	0.35	0.35	0.80
		2	HAIR		0.80	0.80	0.35	1.80	3.00	19.89	15.92	0.09	0.09	0.80
		3	HAIR		0.80	0.80	0.35	3.05	3.00	51.31	41.05	0.23	0.23	0.80
		4	HAIR		0.80	0.80	0.20	3.11	3.00	30.22	24.18	0.13	0.13	0.80
		5	HAIR		0.80	0.80	0.20	3.25	3.00	32.32	25.85	0.14	0.14	0.80
		6	HAIR		0.80	0.80	0.20	2.07	3.00	15.00	12.00	0.07	0.07	0.80
		Sum						**17.38**		**227.53**	**182.03**	**1.00**	**1.00**	**0.80**
2nd Floor	N-S	1	B	Tc<10	0.40	0.40	0.35	1.83	3.00	20.58	8.23	0.07	0.04	0.40
		2	HAIR		0.80	0.80	0.35	1.33	3.00	10.22	8.18	0.03	0.04	0.80
		3	HAIR		0.80	0.80	0.35	1.20	3.00	8.00	6.40	0.03	0.03	0.80
		4	HAIR		0.80	0.80	0.20	4.41	3.00	49.67	39.74	0.16	0.17	0.80
		5	HAIR		0.80	0.80	0.20	4.41	3.00	49.62	39.69	0.16	0.17	0.80
		6	HAIR		0.80	0.80	0.35	7.46	3.00	163.49	130.79	0.54	0.56	0.80
		Sum						**20.64**		**301.58**	**233.04**	**1.00**	**1.00**	**0.77**
	W-E	1	HAIR		0.80	0.80	0.35	10.10	3.00	227.63	182.11	0.60	0.60	0.80
		2	HAIR		0.80	0.80	0.35	3.55	3.00	64.43	51.54	0.17	0.17	0.80
		3	HAIR		0.80	0.80	0.35	4.35	3.00	85.28	68.22	0.23	0.23	0.80
		Sum						**18.00**		**377.34**	**301.87**	**1.00**	**1.00**	**0.80**
		∑								**484.83**	**387.86**			**0.80**

Parameters used in Table 4 are T, L, h, and A, which are respectively thickness, length, height, all in meter, cross section of the wall in square meter. V_a, the shear strength of the wall (in kgf/cm^2), is calculated as:

$$V_a = 0.1\ V_t + 0.15\ \sigma_c = 0.1 + 0.015\ \sigma_c \tag{9}$$

And R_w, its shear resistance (in tonf), is calculated as:

$$R_w = V_a \times A \times \lambda_Q \tag{10}$$

Then the shear force applied to each wall is calculated by:

$$V_W = V \frac{k_i}{\sum_i ki} \tag{11}$$

Finally, the minimum relative wall in each story, A_{min}, is calculated to be compared with the provided wall cross sectional area by the existing walls to realize if the walls are sufficient. In the case of this building they were not. Of the total 298 evaluated masonry buildings the highest vulnerability belonged to the 3-story buildings, 38 ones, with reduction factor of mostly between 0.5 and 0.6, and the lowest vulnerability belonged to 1-story buildings, 118 ones, with reduction factor of mostly between 0.8 and 0.9, as shown in Table 5.

Table 5. Number of 1-, 2-, and 3-story buildings in various levels of stiffness and strength reduction factor

Stiffness and strength reduction factor	No. of stories		
	1	2	3
0.9-1	15	0	0
0.8-0.9	75	25	0
0.7-0.8	20	93	13
0.6-0.5	7	23	20
0.5-0.4	1	1	4
0.4-0.3	0	0	1
Total	**118**	**142**	**38**

5. CONCLUSIONS

The seismic evaluation of 298 masonry buildings, damaged in Darb-e-Astaneh (Silakhor) earthquake of March 2006, showed that around 115 ones are slightly vulnerable, around 176 ones are moderately vulnerable, and 7 ones are extensively vulnerable. Therefore, all of them need to be retrofitted. The non-rigid diaphragms (jack arc system), and insufficient foundations of the major deficiencies in these buildings which makes difficult their retrofit design.

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