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SEISMIC BEHAVIOR OF RC STRUCTURAL WALLS AND EUROCODE 8 PROVISIONS

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ABSTRACT

Slovenia was the first country to adopt the European standard for the design of the seismic resistant structures Eurocode 8 as the national code. In parallel several trial designs of buildings with structural walls were made. Some observed problems were analyzed in view of the latest research findings obtained by the shake table tests. The selected research topics, presented in the paper include the shear magnification factor, 3D structural interaction through slabs and coupling beams and minimum confining requirements in the boundary areas of the walls. Although predominantly related to the particular case of the Eurocode 8, most findings are generally valid and applicable.

Introduction

RC structural walls are frequently used to provide seismic resistance. Nevertheless, their behavior is still less understood in comparison with some other structural components. The limited knowledge also reflects in the latest European standard for the design of seismic resistant structures Eurocode 8 – EC8 (CEN 2004). Many problems are related to the fact that relatively thin structural walls without boundary columns, which are frequently used in Europe (similar walls are used in several other countries like Chile and China), differ considerably from the relatively heavily reinforced walls typically built in the New Zealand, USA and Japan.

When Slovenia adopted EC8 as the national code in January 2008, the need for more in-depth study arose. Some of these background studies using shake table tests and their main results are briefly overviewed in the next section. Important related research was also done by Wallace and co-authors (i.e. Orakcal 2006 and 2009). Due to the space limitation, only some selected problems and related EC8 requirements are discussed in the following sections. They include shear magnification factor, the influence of the coupling beams and slabs on the overall response of the walls and minimum confinement of the boundary areas of the walls. Special attention has been given to the capacity design for shear and related shear magnification factors.

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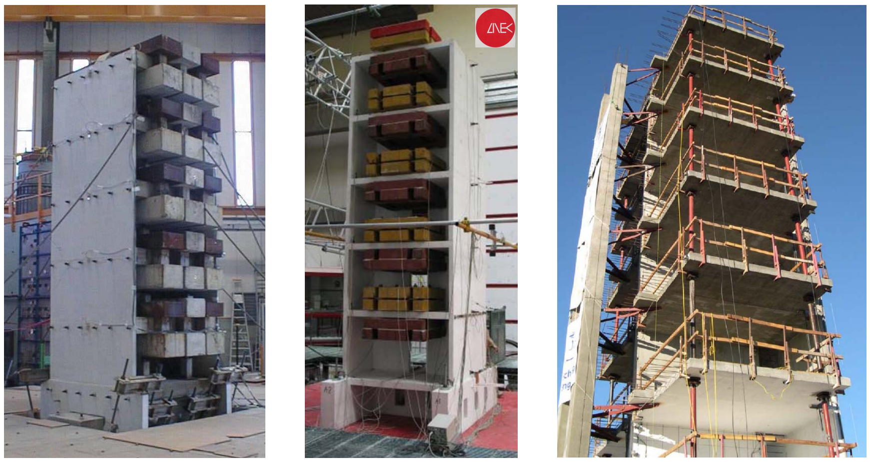
Short description of the background studies

The research presented in this paper is mainly based on the three background studies – the EC8 pre-normative research CAMUS-3 (Combescure and Chaudat 2002) and ECOLEADER-SLO (Fischinger 2006) as well as on the recent shake-table test performed at the UC San Diego (Panagiotou 2007).



(a) (b) (c)

Figure 1. Large-scale models tested on shake-tables (a) CAMUS-3 - Cantilever wall designed according to EC8, (b) ECOLEADER-SLO - Lightly reinforced coupled wall designed according to the Slovenian practice, (c) UC San Diego - Full-scale model of a 7-story building slice.

CAMUS-3 (Shake table tests on a cantilever wall designed according to EC8)

A cantilever RC wall, designed according to the Eurocode 8 provisions (Fig. 1a), was subjected to a sequence of four strong earthquakes on the shaking table at the Centre d'Etudes de Saclay in France (Combescure and Chaudat 2002). Benchmark prediction and post-experiment analyzes were made at the University of Ljubljana (Fischinger 2004). Most important experimental results include: (a) The behavior was flexural and governed by a formation of the plastic hinge at the base as foreseen by EC8; (b) Confinement of the boundary areas according to EC8 was efficient; (c) Significant influence of pre-cracking was observed; (d) Very large deformation of tension reinforcement was measured and some brittle reinforcement bars (of small diameter) fractured already at the intermediate test; (e) Significant axial force variations due to the rocking effect and elongation of the neutral axis appeared during response.

ECOLEADER-SLO (Shake table test of a thin lightly reinforced coupled wall designed according to the Slovenian practice)

Shake table test of a 1:3 scaled specimen (Fig. 1b), representing relatively low, thin and lightly reinforced H-shaped structural wall with openings was performed at LNEC in Lisbon, Portugal. Different types of confinement reinforcement at the free edges were used and compared. Thin, diagonally reinforced coupling beams were studied. A sequence of seismic loadings was applied in two horizontal directions simultaneously. Main results include: (a) Considerable overstrength was observed in the wall with minimum reinforcement; (b) Deformation capacity was limited to less than 1% of the height; (c) Relatively thick slab enhanced the strength of thin coupling beams considerably. Consequently, they did not perform as expected in capacity design and high axial and shear forces which were induced into the wall piers through the coupling beams led to the shear failure of the piers; (d) The EC8 confining reinforcement proved to be efficient; (e) Simpler confining details (i.e. U-shaped stirrups) might be acceptable for low walls (5-storey) and/or in the case of low seismic intensity; (f) Sequence of loading and pre-cracking influenced the response considerably; (g) The influence of bi-axial loading was relatively low.

Test at the UC San Diego (Full-scale test of a 7-story building slice)

A full-scale 7-story building slice with rectangular RC structural wall (Figure 1c) was tested on the shaking table at the UC San Diego in the frame of the NEES project (Panagiotou 2007). A parallel international benchmark was organized. Selected results include: (a) The wall behaved well even though some reinforcement details did not satisfy all present code requirements in the USA; (b) The observed shear force was much higher than foreseen by the present codes; (c) Very thin (slotted) slab had a very important effect on the interaction of the wall with other structural elements (see a separate section in the following text).

Shear magnification factors

It has been long known that during the inelastic response the actual shear forces in structural walls are typically much higher than the forces foreseen by the equivalent elastic procedures (Blakeley 1975). This is due to the flexural overstrength as well as to the amplified effect of the higher modes in the inelastic range. Therefore Eurocode requires to multiply the shear forces obtained by the equivalent elastic analysis V_{Ed}' with the shear magnification factor ε

$$V_{Ed} = \varepsilon \cdot V_{Ed}' \tag{1}$$

In the case of designing the wall to exhibit large plastic deformations (ductility class high – DCH structures) shear magnification factor is calculated by the expression (2) originally proposed by Keintzel (1990)

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0,1 \cdot \left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \begin{cases} \leq q \\ \geq 1,5 \end{cases} \tag{2}$$

q is the behavior (seismic force reduction) factor used in the design;
M_{Ed} is the design bending moment at the base of the wall;
M_{Rd} is the design flexural resistance at the base of the wall;
γ_{Rd} is the factor to account for overstrength due to steel strain-hardening;
T_1 is the fundamental period of vibration of the building in the direction of shear forces;
T_C is the upper limit period of the constant spectral acceleration region of the spectrum;
$S_e(T)$ is the ordinate of the elastic response spectrum.

In the case of the moderate plastic deformations (ductility class medium – DCM design) smaller increase of the shear forces is expected and the shear magnification factor can be simply taken as $\varepsilon = 1.5$.

Background of the expression in EC8

Keintzel (1990) made a parametric study comparing the results obtained with the lateral force method and inelastic response history analyses. Based on the results of this study he assumed that modal combination can be applied also in the inelastic range and that only the contribution of the first two modes is important

$$V_{Ed} = \sqrt{\left(V_{Ed,1}\right)^2 + \left(V_{Ed,2}\right)^2} \tag{3}$$

V_{Ed} is the design seismic shear at the base of the wall;
$V_{Ed,1}$ is the design seismic shear at the base of the wall caused by the building oscillation in the first mode;
$V_{Ed,2}$ is the design seismic shear at the base of the wall caused by the building oscillation in the second mode.

He further assumed that the level of the reduction of seismic forces belonging to each mode was proportional to the level of the seismic moment at the base of the wall contributed by the excitation of that mode. In the case of the seismic forces related to the first mode of excitation, the reduction is high, equaling seismic reduction factor q, as the first mode related moment contributes the majority of the overall seismic moment (as well as related energy dissipation) at the base. On the other hand, seismic forces due to the higher mode act on the structure with the unreduced elastic value ($q \cdot V_{Ed,2}$)

$$V_{Ed} = \sqrt{\left(V_{Ed,1}\right)^2 + \left(q \cdot V_{Ed,2}\right)^2} \tag{4}$$

Considering also the flexural overstrength of the wall and that in the response spectrum analysis the contribution of the second mode is about 30% ($\sqrt{0.1}$) of the contribution of the first mode, expression (2) can be finally derived.

Keintzel also determined that ε is limited by the upper value of q. The same assumption was adopted in EC8. While it is true that the upper bound for V_{Ed} is its elastic value V_E, the assumption that V_E equals $V_{Ed,1} \cdot q$, neglecting the contribution of higher modes, is not valid. This will be further discussed in the continuation of the paper.

Verification of the shear magnification factor in the Eurocode

While Keintzel's research was certainly up-to-date in its time, the parametric study was rather limited in the view of the modern earthquake engineering: (i) Keintzel's expression was originally tested with a limited number of wall parameters (just 2 and 3 storey walls were analyzed) and an unsuitable analytical model for RC elements was used from the today's point of view; (ii) Although Keintzel's equation was developed to be applied on the seismic shear forces obtained by considering just the first mode of excitation, this is not specified in Eurocode. In the design practice multi-modal analysis is typically performed and this can lead to quite conservative results; (iii) The limitation $\varepsilon \leq q$ is not adequate. Consequently additional extensive research was done recently (i.e. Rutenberg 2006 and Kappos 2007). The common conclusion was that Eurocode procedure needs some revisions in order to estimate the shear magnification factors better.

Since the use of the RC structural walls is very popular in Slovenia as well as to overcome some problems related to the application of the EC8 requirements in the everyday design, additional studies have been also done at the University of Ljubljana. 45 different cantilever walls were analyzed and designed for the EC8 DCH requirements. Number of stories (n) varied from 4 to 20. Within each group of walls having the same number of stories the following parameters were varied depending on the design requirements and the feasibility of the construction:

- the length of the wall l_w (between 2 and 8 m);
- the longitudinal reinforcement resulting in different overstrength ratios ($\omega_{Rd}=M_{Rd}/M_{Ed}$ between 1.1 to 3.6);
- the wall-to-floor area ratio A_w/A (1.5%, 2.0% and 2.5%).

14 artificial accelerograms with spectra matching the EC8 elastic spectrum were used in the response history analyses. The values of the EC8 seismic design shear forces at the base of the walls (denoted as V_{Ed}) were compared with the ones obtained by rigorous inelastic response history analysis (denoted as V_{IA}). V_{Ed}' in equation (1) was determined by the modal response spectrum analysis considering all important modes. Results are presented in Figures 2 and 3. The variation of the basic input parameters l_w , $\omega_{Rd}=M_{Rd}/M_{Ed}$, and A_w/A is additionally illustrated at the bottom of the figures.

Figure 2 illustrates large shear magnification factors, in particular for the structures with longer fundamental periods (due to the higher effect of the higher modes) resulting into the ε values up to 4.5. It is also important to notice substantial difference between the base shear $V_{Ed,1}'$ obtained by considering the fundamental mode only and the base shear considering all important higher modes V_{Ed}'.

The EC8 values are then compared with the mean results of the inelastic response history analysis V_{IA} in Figure 3. Although the formula in EC8 has been based on the limited parametric study and several simplifications (see discussion in the previous sections) it yielded very good results in the case of the analyzed walls. Nevertheless some modifications have been proposed by the authors to further improve the results (see the curve for $V_{Ed,mod}$ in Figure 3). The limitation of $\varepsilon \leq q$ was not considered and ε was applied to the shear force calculated based on the first mode only (as originally assumed by Keintzel).

It should be emphasized that just simple cantilever walls were considered in the study. Further studies are required to determine the suitability of the EC8 procedure for more general systems containing structural walls.

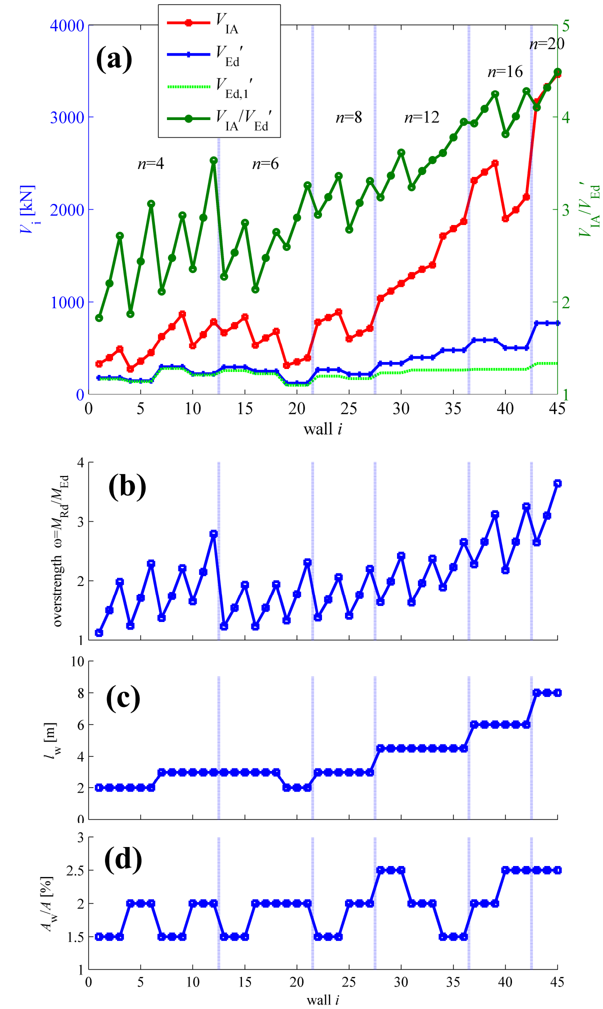


Figure 2: (a) Base shear for the analyzed walls obtained by different methods. The variation of the basic input parameters is shown in Figures (b), (c) and (d).

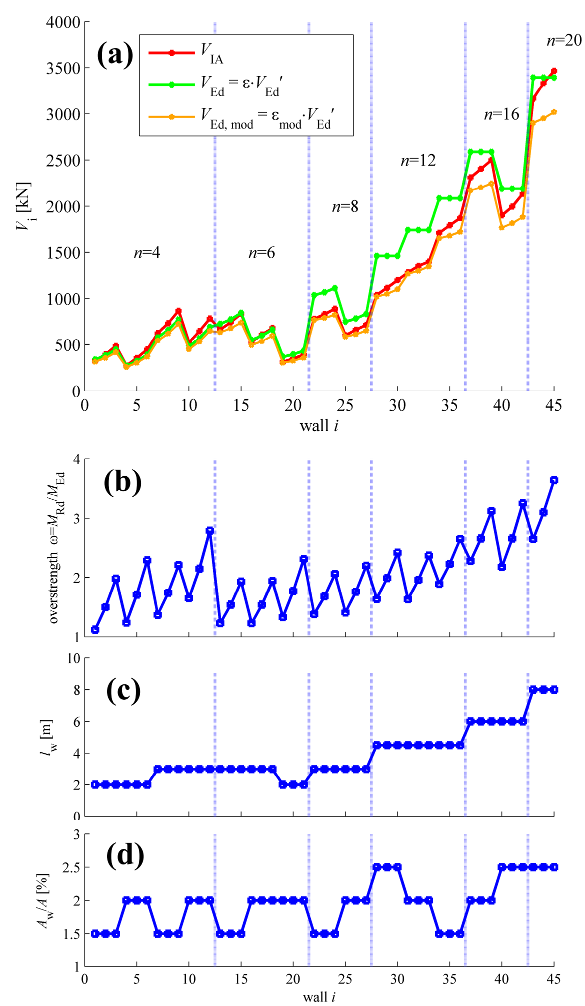


Figure 3: (a) Comparison between the base shear obtained by the EC procedure, modified EC procedure and inelastic response history analysis.

Similar study, not discussed in this paper showed that using constant value of $\varepsilon = 1.5$ for the ductility class medium walls is typically too small (similar conclusions were made before by Rutenberg 2006). The authors suggest that the procedure required for DCH walls is also used for DCM walls.

Coupling effect due to the slabs

Similar to many other codes, EC8 provide only the requirements for the in-plane stiffness of the slabs. The coupling of the structural walls due to the out of plane resistance of the slabs is not explicitly considered. However, recent experiments showed that this effect is much more important that it was believed before and that the whole concept of the relevant models for the inelastic analysis should be revised.

The effect of the wall to gravity column interaction in the San Diego experiment

The slabs in San Diego experiment were supported by “gravity columns” (Fig. 4). There was a small gap permitting limited unrestricted vertical movement of the slab. However, when the edge of the tested wall yielded, the edge of the wall and adjacent slab moved upward for several centimeters. The upward movement of the slab was restricted by the gravity columns. This caused the 3D interaction between the tested wall and the gravity columns trough the deformed slab. Similar effect was observed many years before during the “Tsukuba experiments” (Kabeyasawa 1983). Being aware of this the first and the third author of the paper approximately considered this effect in the benchmark study. This has been believed to contribute decisively to the best prediction made.

Figure 4. San Diego test setup with gravity columns. Benchmark documentation provided by the UC San Diego; courtesy of Prof. Restrepo.

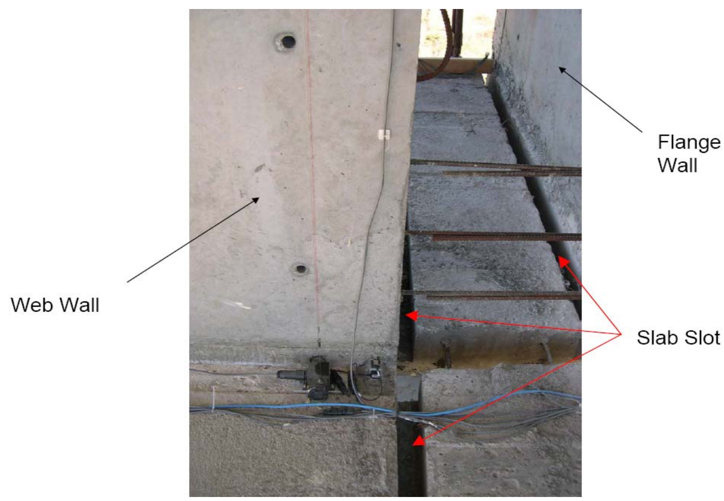


Figure 5. Slotted slab. Benchmark documentation provided by the UC San Diego; courtesy of Prof. Restrepo.

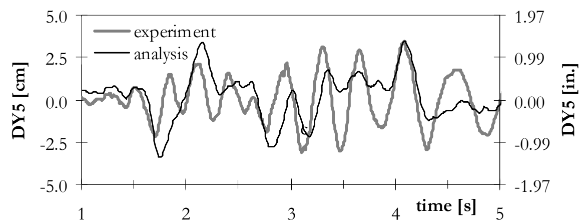
The effect of the slotted slab in the San Diego experiment

Although the benchmark prediction of the displacements and even accelerations made by the research group (Fischinger 2008) was very good, the match in shear forces was poor. This was not understood until the explanation was given by the benchmark organizers (e.g. Panagiotou et. al. 2007). The analyzed wall was connected by the perpendicular stabilizing wall by slabs only. To further decrease the coupling effect, slab was slotted (Fig. 5). It was only 5 cm (2 inches) thick at the slot. To all benchmark participants it seemed natural that this coupling was negligible and they all assumed hinged connection between the analyzed and the perpendicular wall. In reality a considerable shear force was generated along the whole length of the slot resulting in the considerable increase of the axial force in the tested wall associated with the increase of the flexural capacity of the wall and the related shear.

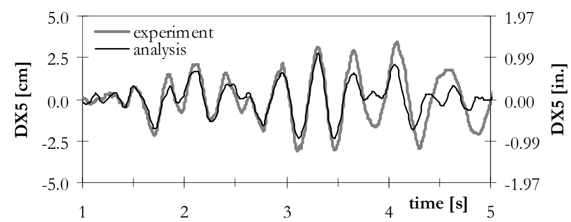
The effect of the shear overstrength of the coupling beams in the ECOLEADER-SLO experiment

EC8 does require considering distributed slab reinforcement in the calculations of the flexural (over)strength. Overstrength in shear is not directly addressed. However shear strength of the coupling beams in the ECOLEADER-SLO wall was much higher than predicted by any inelastic model. Consequently, the individual wall piers were heavily loaded and they failed in shear (Fig. 6), this was neither predicted, nor modeled in the initial study.

Figure 6. Damage to the ECOLEADER-SLO wall at the end of the test.



(a) Initial model



(b) Coupling beam/slab interaction and inelastic shear behavior of wall piers considered

Figure 7. Top displacement response history in the direction of the ECOLEADER-SLO flange wall with openings.

After the test the full 3D model, taking into account the interaction of the slab and coupling beam was analyzed by ABAQUS program. Practically elastic shear behavior of the coupling beam, observed in the test, was confirmed. In addition the inelastic shear behavior of the wall piers was modeled by the modified compression field theory. Values valid for monotonic loading were used in the study. Results (Fig. 7b) improved considerably in comparison with the results obtained with the initial model

Minimum confinement of the boundary elements of RC walls

EC8 requires a special confining reinforcement in the boundary regions of the wall cross-sections in the potential plastic hinge zones. The minimum required length of the confined boundary regions should be at least:

$$l_c = 0.15 \cdot l_w \text{ or } 1.50 \cdot b_w \tag{5}$$

where l_w and b_w are the height and width of the wall cross-section.

The maximum distance of the longitudinal bars, that are supported by the confining reinforcement is 15 cm (5.9") and 20 cm (7.9") for the walls designed to have high (DCH) and medium ductility (DCM), respectively. Maximum distance of the confining reinforcement along the height of the plastic hinge must not exceed the $b_o/3$ and $b_o/2$ for DCH and DCM design, respectively (b_o is the width of the confined zone).

In the relatively thin walls, which are typically built in many European countries (the width is usually no more than 20 cm – 7.9"). Consequently these minimum requirements yield quite heavy confinement. It was demonstrated by the ECOLEADER-SLO experiment that for structures built in Central Europe such confinement might not be needed if the building was low (up to 5 stories) and the wall-to-floor ratio was higher than 1.5.

Conclusions

The complex behavior of RC structural walls entering far into the inelastic range as well as the complex interaction of all structural elements in buildings with structural walls is still far for being adequately understood.

The limited knowledge reflects even in the most up-to-date seismic codes like the latest version of the Eurocode 8. When Slovenia as the first country in EU adopted Eurocode standard as the national code several problems related to the seismic design of RC structural walls emerged. A limited number of these problems have been discussed in the paper based on the results of some recent major research projects.

Large shear magnification factors ε (up to 4.5) have been re-confirmed, although questioned by many design practitioners. The proposed expression for the shear magnification factor given in Eurocode worked fine for the Ductility Class High, although it had been based on very limited parametric study and some crude assumptions. Nevertheless some minor modifications were proposed by the authors to further improve the results. Similar study (not shown in this paper) re-confirms that that the constant value ε = 1.5 allowed by Eurocode for Ductility Class Medium walls is too low.

It has been repeatedly demonstrated that the coupling effect due to the out of plane resistance of the slabs is typically underestimated in most inelastic models used in the research as well as in most codes, including the Eurocode. Neglecting this effect can lead to gross errors and inefficient capacity design. The underestimation of the shear strength of the coupling beams in the ECOLEADER-SLO coupled wall similarly led to inefficient capacity design and shear failure of the wall piers.

The Eurocode requirements for the confinement of the boundary areas of structural walls in the potential plastic hinges proved efficient in all studies. Moreover, it has been indicated, that these requirements could be released in the case of walls in the low-rise buildings (up to 5 stories) having wall-to-floor ratio more than 1.5 and being built in the areas of small to moderate seismicity ($a_{g,max}$ = 0.1 to 0.25 g).

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