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ASSESSMENT OF EC8 PROVISIONS FOR REINFORCED CONCRETE FRAMES

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ABSTRACT

Out of the nine Eurocodes currently being published as prestandards in Europe, the Eurocode 8 is devoted to seismic design and is considered to represent the latest word in codified earthquake resistant design. The implications of its provisions for reinforced concrete structures are presented and discussed as the result of a large prenormative research study which included the trial design of a set of typical reinforced concrete buildings followed by the analytical evaluation of its seismic response well into the nonlinear range. In general terms the study confirmed the soundness of EC8 provisions but it was possible to identify the matters on which improvements could be made in its future revision to become a definitive European Standard.

KEYWORDS

Eurocodes, seismic design, reinforced concrete, ductility classes, reinforcing steel.

INTRODUCTION

In the last few years the Comité Européen de Normalisation (CEN) has been publishing a set of design codes known as the Eurocodes. CEN encompasses the countries belonging to the European Union and to the EFTA and the set of Eurocodes, nine in total and each one with various Parts, is intended to cover in a unified and internally consistent way the establishment of the common basis of design principles, the definition of the design actions and the presentation of design and construction rules for various structural materials and also for foundations and geotechnical aspects and for earthquake resistant design. For the moment these documents have been published as Prestandards (ENVs) intended to be subjected to testing through trial application in the various countries and to ultimately be revised and published as European Standards (ENs). It is expected that by the year 2000 the set of Eurocodes needed for the design of buildings shall be available in such form.

BASIC FEATURES OF EUROCODE 8

The design provisions for earthquake resistance of structures are presented in Eurocode 8 (EC8) (CEN, 1994; Carvalho, 1994) which is a performance oriented code which states that its purpose is to ensure that in the event of earthquakes: a) human lives are protected; b) damage is limited and c) structures important for civil protection remain operational. These design objectives are translated into two basic requirements to be met by structures, one dealing with the no-collapse requirement of the structure under the design earthquake and the other with the requirement of damage limitation under a seismic action having a larger probability of occurrence (i. e. having a smaller return period).

The establishment of the seismic zonation is left to the National Authorities except to the fact that the seismic hazard has necessarily to be described in terms of the effective peak ground acceleration in rock or firm soil so that it is compatible with the basic representation of the earthquake motion which, in EC 8, is made by the elastic ground response spectrum. Merely as an indication the Code suggests that the design acceleration is established as the value with 10% exceedance probability in 50 years (or equivalently as the value with 475 return period).

In connection to the two basic requirements, the performance of the structures is judged with reference to the usual Ultimate (ULS) and Serviceability Limit States (SLS) whose definition may vary with the type of structure under consideration (for instance a building or a bridge). The ULS is related to the full exploitation of the ductility available in the structure leading to the attainment of a situation very close to collapse, whereas the SLS is related to the attainment of a certain amount of damage associated to some limitations of use. Recognising the need to rely on the ductile nonlinear response of the structures for its seismic protection, EC8 relies on a design spectrum suitable for linear analysis methods, derived from the basic representation of the earthquake motion through the concept of a (global) behaviour factor q.

The regularity classification for buildings is subdivided into regularity in plan and regularity in elevation, the former being linked mostly with the requirement related to the refinement of the structural model and analysis method (plane vs. spatial and static vs. dynamic) and the latter having incidence also in the values of the behaviour factor which has to be decreased for more irregular structures. It should be recognised that this attempt of EC8 to cover a wide variety of situations in a rational and complete way leads, for the more general cases, to computationally demanding Code provisions as those inherent to spatial dynamic analysis including the effects of accidental eccentricities and the simultaneous action of the earthquake components.

The concept of Ductility Classes is adopted and corresponds to the recognition that, although earthquake resisting structures must have simultaneously resistance and ductility, a certain trade-off between these two characteristics is possible. In fact, the resistance and ductility to be assigned to the structure are related to the extent to which its non-linear response is to be exploited and therefore the Code allows the designer to choose among different Ductility Classes, enabling an adjusted solution to each design case.

For reinforced concrete buildings three Ductility Classes are established: a) Ductility Class L (DCL) corresponding to structures designed and dimensioned according to Eurocode 2 (Code for concrete structures) and only lightly supplemented by a few additional detailing rules for the enhancement of ductility; b) Ductility Class M (DCM) corresponding to structures designed, dimensioned and detailed in order to enable the structure to enter within the inelastic range without brittle failures and c) Ductility Class H (DCH) corresponding to structures for which the design, dimensioning and detailing provisions ensure the development of chosen stable mechanisms associated with large hysteretic energy dissipation. The values of the behaviour factor q decrease as the Ductility Class decreases. For reinforced concrete buildings, the q-factor varies proportionally to 1,00, 0,75 and 0,50 respectively for DCH, DCM and DCL. For all the higher ductility classes therein foreseen, EC 8 resorts to Capacity Design procedures which aim at forcing a certain behaviour into the structure considered to be more suitable for the dissipation of energy under the seismic excitation. These procedures oblige to design the individual elements in a certain sequence in order to account to the actual resistance of the adjacent members.

Although the available version of EC8 has reached sufficient maturity for practical application as a Prestandard a prenormative research program (PREC8) was devised to check the present Code formulations and to contribute to its improvement (Carvalho, 1995). The project was supported by the European Union and coordinated by the University of Pavia. One of its objectives was the to study of the interrelation between a number of design parameters used in EC8, which in a combined form, influence the nonlinear behaviour of structures subjected to earthquake motion. The parameters studied are the regularity classification, the methods of analysis, the behaviour factor values and the capacity design procedures in reinforced concrete buildings and ultimately the aim is to assess whether the Code provisions lead to safe and economic solutions within the extended field of its applicability with particular attention to the equivalence of structures designed at different Ductility Classes. Also included in this topic is the clarification of the requirements specified in EC8 for reinforcing steel in the light of the new steel production technologies in Europe.

REINFORCING STEEL CHARACTERISTICS

The mechanical properties of the reinforcing steel bars used in reinforced concrete structures play an important role in the overall ductility properties of the structures and as such EC8 requires that the reinforcing steel fulfils a number of requirements which vary quantitatively as a function of the intended structural Ductility Class. The requirements set forth in EC8 (see Table I) are expressed in relation to the uniform elongation at maximum load ε_{su}, the hardening ratio f_t/f_y (where f_y is the yield strength and f_t is the tensile strength) and $f_{y,act}/f_{y,nom}$, the latter ratio referring to the limitation of the overstrength of the yield stress (subscripts act refers to "actual" and nom refers to "nominal").

Within PREC8, information was collected from nine countries (Belgium, France, Germany, Italy, Luxembourg, Netherlands, Portugal, Spain and United Kingdom) regarding the characteristics of heat treated (Tempcore) steels with grades (yield strengths) from 400 MPa to 500 MPa which nowadays constitutes the bulk of the European production for medium and large diameter bars. All the information gathered was analysed (Plumier and Vangelatou, 1995) and it indicates that: a) the results from the different origins are essentially consistent among each other; b) for Tempcore bars the mean values of the hardening ratio f_t/f_y range from 1,15 to 1,22 depending on the steel grade. The ratio values are strongly correlated with the yield strength, decreasing as this parameter increases; c) also for Tempcore bars, the mean values of the uniform elongation at maximum load ε_{su} range from 9,4% to 12% also depending on the steel grade. As before, ε_{su} decreases as the yield strength increases. The scatter of values is larger than for f_t/f_y but part of this may be attributed to the bigger difficulties in measuring this parameter.

The values presented indicate that the Tempcore reinforcing bars presently produced in Europe are not essentially incompatible with the requirements of EC 8. In spite of this general conclusion, it could be advantageous to modify the way in which the hardening ratio is specified in EC8 adopting the concept of upper and lower characteristic values for f_t/f_y instead of the mean value presently used. This modification would align EC8 with the corresponding choice in the ENV 10080 (the general European pre-standard for reinforcing steel, not covering the specific needs for seismic areas) and should in principle assure a better production control of this characteristic.

Table I - EC8 requirements for reinforcing steel

Mechanical Property	Ductility Class in EC8		
	DCL	DCM	DCH
ε_{suk} (1)	$\geq 5\%$	$\geq 6\%$	$\geq 9\%$
$(f_t/f_y)_{mean}$	$\geq 1{,}08$ (2)	$\geq 1{,}15$ $\leq 1{,}35$	$\geq 1{,}20$ $\leq 1{,}35$
$f_{y,act}/f_{y,nom}$	-	$\leq 1{,}25$	$\leq 1{,}20$

(1) 10% characteristic value

(2) 10% characteristic value as defined for Type A steel in Eurocode 2

Table II - Proposed revision of the EC8 requirements for reinforcing steel

Mechanical Property	Ductility Class in EC8	
	DCL	DCM and DCH
$\varepsilon_{suk\ 0,10}$	ENV 10080	$\geq 8\%$
$(f_t/f_y)_{k\ 0,10}$		$\geq 1{,}15$
$(f_t/f_y)_{k\ 0,90}$		$\leq 1{,}35$
$f_{y,act}/f_{y,nom}$		$\leq 1{,}20$

Together with this modification it is also apparent, from some associated parametric analysis which were performed, that the presently separated requirements for the characteristics of steel to be used in DCH and DCM structures could possibly be merged, simplifying the matter without detrimental consequences for the ductility of those structures. The proposed revision of the characteristics is presented in Table II.

In this proposal, the requirement for the elongation at maximum load ε_{su} for DCH is slightly relaxed (8% instead of 9%) but this does not seem to harm the overall response of real structures for which global drift limitations (as imposed in the SLS verifications in EC8) tend to limit the exploitation of the steel elongation to values quite below the one proposed in Table II.

In concluding it seems that the Tempcore steels currently being produced in Europe are able to satisfy the requirements of EC8 (or in the revised proposal), particularly for grades 400 MPa. For steels of grade 500 such fulfilment may not be as easy as for grade 400 but it is believed that this is still feasible. As a final remark it should be noticed that experimental evidence on the response of structural elements reinforced with steels of grade 500 is still limited (Pipa et al, 1994) and the little that has been done so far calls the attention to some possible detrimental effects that the higher strength of steels may have in the response of structures subjected to large amplitude reversals of deformation due to the increased difficulties of ensuring proper anchorage and the increased tendency to the buckling of bars subjected to higher compressive stresses (particularly if these higher strengths are associated with lower hardening ratios).

BEHAVIOUR FACTORS AND DUCTILITY CLASSES

In order to assess the behaviour factors and the ductility classes presented in EC8 a large parametric study was included in PREC8, based on the design, according to EC2 and EC8, of a set of buildings followed by the analytical computation of its response to earthquakes of increasing intensity. Besides the variation of ductility class, the variation of other parameters in the study envisages to check whether in a wide spectrum of situations the EC8 provisions (namely the quantified link between q-factors and capacity design rules) ensure a uniformly satisfactory response of the structures.

Structures designed in accordance with EC8

Twenty six different reinforced concrete buildings were considered covering: a) buildings with three different uses to verify the influence of the relative importance of vertical versus horizontal loads which may affect the outcome of the capacity design provisions; b) buildings with four different heights (3, 4, 8 and 12 storeys) to verify the influence of the natural period and the effect of minimum design provisions (dimensions and reinforcement quantities) which may apply at the upper storeys of taller buildings; c) buildings located at two different seismic zones with design accelerations of 0,15g and 0,30g, to verify the goodness of the EC8 provisions in regions of higher or lower seismicity; d) buildings with framed structures and with a central core. It is obvious that the variation of the chosen parameters only very partially covers the universe of possible cases but already represents an extremely large amount of calculations and information to be treated.

Except for a so-called industrial building, with three storeys and large spans, the whole set of buildings is based on a similar rectangular plan configuration with 15 m (3 bays of 5 m) by 20 m (5 bays of 4 m or 3 bays of 8/4/8 m). The height of the storeys is 3 m and in one configuration an irregularity situation was created by the omission of 4 columns at the ground level (out of a total of 20 in the upper storeys). The seismic analysis was performed in general using dynamic analysis methods but in two particular cases static methods were used to verify its influence.

The design of this set of buildings was made at the University of Patras (Fardis, 1995). The design was made in a computerised way and allowed an assessment of the usability of EC8 and the identification of the matters for which improvements are recommendable. It is clear that the application of EC8 requires the use of a computer with appropriate software. In particular, at the top of the most computationally demanding code provisions are those regarding the combination of the seismic action (namely in what concerns the eccentricities in plan to be considered with several different combination of signs) and the capacity design rules for Reinforced Concrete buildings which introduce considerable coupling between various phases of design of one same element and between the design of adjacent elements in the structure. This entails the need to store large amounts of information which has to be accessed at different stages of the design.

In what concerns the needs for improvement of the design provisions, the experience with this trial design shows that, in contrast to the rules for beams and columns for which no special limitations of applicability were found, the current rules of EC8 for reinforced concrete walls seem to have some limitations and in some cases result in what appears to be excessively conservative designs. Those rules may be considered too complicated in view of the still imperfect knowledge of the seismic behaviour of walls and its application beyond the very simple case of single walls of rectangular cross section, as coupled walls or channel or L-shaped walls, is not obvious. In particular, the application of the capacity design rules for the shear design of the individual walls in a coupled shear wall system should be revised since the present ones may lead to unreasonably high magnification factors.

The results of the trial design cases provide very interesting information regarding the characteristics of structures designed in accordance with EC8. The material quantities needed in each case were evaluated and show that in frames the three ductility classes lead to about the same total quantities of steel and concrete, with a shift in the ratio of column-to-beam total steel from about 55%-45% for DCL, to 60%-40% for DCM and to 65%-35% for DCH, and with a change in the ratio of longitudinal-to-transverse steel from about 80%-20% for DCL, to 75%-25% for DCM and to 60%-40% for DCH. For the wall structures the ratio between longitudinal-to-transversal steel came out relatively independent of the ductility class. These tendencies observed in the relative quantities of steel are very easily understandable in view of the capacity design provisions of EC8.

Based on the results of the trial design of the framed structures, Ponzo (1994) performed a number of analysis in order to interpret the outcome of the designs at the different ductility classes. These analysis started from the overstrength factors as proposed and computed by Fardis (1995) in his post-processing of the trial designs. These factors k_{of} essentially reflect the "margin" of flexural resistance that the beams and the columns, at the end of the design process, present in comparison to the original internal forces coming out directly from the structural analysis for the earthquake design spectrum. Naturally k_{of} is always larger than one and, considering the capacity design rules included in EC8, it tends to be larger in columns than in beams and to increase as the ductility class increases.

From these factors it was possible to define, at the structural element level, another interesting variable denoted as the "effective behaviour factor" $q_{eff} = q/k_{of}$ where q is the (global) behaviour factor used in design (namely 5,00; 3,75 and 2,50 for frames of DCH, DCM and DCL). This effective behaviour factor thus represents the value by which the internal forces resulting from a design spectrum with $q = 1$ should be divided to obtain those that result from the whole design process (i.e. allowing a direct and simpler computation of internal forces still ensuring the purposes of the capacity design rules).

As a general trend, the effective behaviour factors increase with the ductility class but are generally much smaller (in many cases smaller than 1) than the global behaviour factor used in each case. Values are higher at the 1st storey than at the top and larger in beams than in columns. Larger values of the effective behaviour factor indicate a larger likelihood that nonlinear response shall occur and thus such trend is in line with the general design philosophy of "strong columns-weak beams". On the other hand the effect of the number of storeys is not very marked.

In order to assess the relative overstrength of columns with regard to beams, the ratio of the effective behaviour factors in beams and in columns was computed and denoted by β. Values found in the parametric study range from 1,74 to 3,64 which illustrate not only the effects of capacity design rules but also of other design and detailing rules which tend to make columns stronger than beams. A word of caution must however be placed here since the values of β are somewhat inflated by the fact that columns were designed under bidirectional bending whereas the resistances on which these values are based consider separately the response in each direction (in other words, part of the margin of resistance in columns inferred in the β values is to be absorbed by the bidirectional response expected in reality).

Table III - Overall average values of β as a function of the ductility class and the design acceleration

		Ductility class	
a_g (g)	DCL	DCM	DCH
0,15	2,55	3,45	(3,87)
0,30	(1,76)	2,67	3.09

Note: The values in parentheses correspond to an extrapolation (in a best fit surface) of the values for the four combinations actually available.

To allow an overall view of the range of β for all the cases included in the trial designs, the global mean values of β as a function of the ductility class and the design acceleration are presented in Table III. These values should be judged carefully, taking into account the limited number of situations that they represent, but it is clear that the relative flexural strength of columns as compared to beams increases with the ductility class due to the larger overstrength factors used in design and that it also increases as the design acceleration decreases.

Nonlinear response

The nonlinear calculations on the 26 buildings included in the trial design were carried out by 6 different partners of the PREC8 network (LNEC, University of Patras, University of Basilicata, Imperial College, the French group GRECO and the JRC in Ispra). Each case was analysed by two different partners in order to ensure a certain control of the results and a number of basic assumptions (mean values of resistances and the accelerograms) were taken commonly by all the partners. The calculations were performed for the two horizontal directions with four different artificial accelerograms matching the EC8 spectrum for soil type B and for peak accelerations of 0,15 or 0,30g as appropriate considering the design acceleration of each case. Besides the analysis for the nominal intensity, nonlinear analysis were also performed for accelerograms with increased intensity with scaling factors of 1,5 and 2,0 in order to assess the vulnerability function of the structures. Besides the geometry, some general assumptions regarding the mechanical properties of materials (steel B500 with a stress-strain diagram typical of Tempcore steel, concrete C25/30 with mean compressive unconfined strength of 33 Mpa), and the quantification of damping were fixed for all the partners.

University of Patras (UP) The nonlinear calculations performed at UP (Fardis and Panagiotakos, 1995) deal with 12 out of the 26 buildings, with regular shaped structures, framed in two orthogonal directions and with 3, 4 and 12 storeys. The 12 structures studied resulted from the consideration of 3 basic configurations (each one corresponding to a combination height - use) designed considering 4 different combinations of the design acceleration (0,15g; 0,30g) with the ductility class (DCL; DCM; DCH). The results obtained were considered by UP to reflect that the application of EC8 leads to safe structures, possessing significant overstrength. Softening of the structures due to concrete cracking and yielding of the reinforcement significantly reduces seismic force demands and contributes to the good performance of the structures. However it increases very much displacements and deformations, possibly giving rise to significant second-order (P-δ) effects. Designing for higher ductility class does not have a strong systematic effect on the overall response in terms of displacements and energy dissipation, but reduces overall member damage and shifts energy absorption to the beams.

National Laboratory for Civil Engineering (LNEC) The analyses performed at LNEC (Coelho et al, 1995) deal with the same 12 structures. The results obtained agree reasonably well with the results from the UP, leading to the same type of conclusions on the structural response, namely in what regards the relatively low values of damage indices found and the effect of the ductility class. The response of the structures designed for different ductility classes appears to be not significantly different, although the structures designed for DCH presented a slightly better behaviour. In this respect, a significant effect was attributed to the relative importance of the seismic loading in controlling the design. For low and medium seismicity the vertical loading has an important contribution thus not allowing to exploit differently the ductility.

Imperial College (IC) The nonlinear calculations performed at IC (Salvitti and Elnashai, 1995) deal with the analysis of 4 buildings, having the same rectangular plan shape, with a central core associated to the basic framed structure and with 8 storeys. The structures studied resulted from 4 different combinations of the design acceleration (0,15g; 0,30g) with the ductility class (DCL; DCM; DCH). The results obtained by IC showed that the behaviour of the 4 structures at the design and at twice the design intensity was satisfactory, with the weak-beam-strong-column capacity design principle confirmed in all the analyses. Plastic hinges were observed at horizontal member ends before vertical elements entered the post-elastic region. Greater demands were experienced by the short and deep coupling beams, particularly at the higher levels. With increasing intensity, greater overstrength was displayed by the structures detailed for lower ductility. The evaluation of the actual behaviour factors of the structures analysed based on implemented yield and collapse criteria showed that the behaviour factors in EC8 appear to be low for the structural configuration considered, at least for the given modelling assumptions, set of input motions and chosen limit state criteria. It was stated by IC that the current values of code q-factors could be slightly increased. In fact the values of the actual behaviour factors evaluated on the basis of the limit state of collapse are in excess of twice the design q-factor. On the other hand the structures did present a satisfactory behaviour under twice the design acceleration in terms of global response, and also the values of local ductility demands were well below the available ductility inherent to the code provisions.

Groupement d'Études Coordonées (GRECO) The nonlinear computations carried out by the GRECO French network included contributions from the INSA-Lyon and the LMT-Cachan (Ile et al, 1995). The analyses deal with the same 4 buildings as the IC. It was considered by GRECO that the structures behaved correctly under the design intensity. The application of EC8 rules was considered to lead to safe structures possessing sufficient overstrength. The good performance of core systems with regard to drift control was stated both for the design and twice the design intensity. For the different input motions within the same intensity, it appeared that damage remains nearly constant for concrete as well as for steel yielding. The behaviour of structures designed for 0,30g was not significantly affected by the different ductility class considered. For design acceleration 0,15g, reduced ductility demands were detected, which may indicate that the ductility class DCL could be the more appropriate in that case. In what regards the structures designed with the same ductility, designing for higher acceleration was found to lead to a better nonlinear behaviour, thus seeming to be more suitable to adopt medium and high ductility structures in high seismicity regions.

Joint Research Centre (JRC-ISPRA) The structures analysed at the JRC (Arede et al, 1995) consist of 2 basic configurations of 8 storey framed buildings, regular in plan and both regular and irregular in elevation. The structures were designed for different ductility classes (DCL, DCM and DCH) and for two design accelerations (0,15g and 0,30g). The aspects related to the design intensity, ductility class, irregularity and design analysis methods were investigated on the basis of the results of the analyses. For the evaluation of the safety of the structures studied, a quantitative assessment was used by JRC based on the computation of their probability of failure, associated to the plastic hinge failure and computed on the basis of the damage vulnerability functions resulting from the nonlinear analysis, and of the probabilistic quantification of the structural member capacity and of the seismic action. In spite of the differences on the design options related to the ductility class, irregularity and structural layout, the results obtained confirmed that EC8 leads to quite uniform seismic performance of the structures, although a certain improvement was found when passing from lower to higher ductility classes. However the values computed for the probability of failure of the structures analysed seemed quite high compared to the ones adopted for other loading cases. A detailed analysis of both the methods used and the assumptions made has been found by JRC to be required, in order to clarify the causes of such high values.

CONCLUDING REMARKS AND ASSESSMENT OF EC8

The PREC8 project (1993-1996) has put together the efforts of 19 European research institutions for the development of studies intended to calibrate the pre-standard version (ENV) of EC8. Its topic on reinforced concrete buildings is still underway but it is already possible to draw some conclusions which may serve in the drafting of the definitive version of EC8 expected to become an European Standard (EN) by the year 2000.

Generally speaking the study has shown the operationality of EC8 although, as recognised above, its generality and completeness is paid by the need of computerised tools for its application. The computed response of the structures which were included in the large parametric study carried out presented a satisfactory performance at the design intensity and, for most of the calculations performed, even at twice the intensity the structures still behaved satisfactorily with little damage (the exception in the JRC calculations may be attributed to the way in which ultimate rotations were computed).

These results indicate that, overall, the present provisions of EC8 are satisfactory and not requiring fundamental modifications. Another important finding, in spite of the limited number of cases studied, is the fact that the structures of all three ductility classes performed well with no clear indication of one class performing much better than other (although DCH structures might perform marginally better in the strongly nonlinear range). This indicates the equivalence of the ductility classes in terms of performance and reliability. On the other hand, in view of the minor and non-systematic effect of the ductility class on the total quantities of steel and concrete, these results may raise the question of the real need to have three ductility classes in the Code. Further to this aspect it should be referred that, regarding the reinforcing steel characteristics prescribed in the Code, the study has resulted in a new slightly modified proposal which differs essentially from the current provisions in merging together the requirements for the two higher ductility classes (DCM and DCH). The results have also shown that in low seismicity regions the effect of the other actions considered in the design, in conjunction with the minimum steel requirements, tend to incorporate additional overstrengths into the structure which means that, in principle, in such regions the behaviour factors could be slightly increased or alternatively the requirement of designing for the seismic action might be relaxed. It is however considered too soon to derive quantitative conclusions in this respect.

In what concerns the design provisions for reinforced concrete structures the most important remark is that it appears to exist a certain unbalance between the rules established for beams and columns and those established for walls. In fact, whereas the former seem to be sufficiently general and resulting in reasonable designs, the latter appear relatively complicated in spite of covering practically only the case of walls with rectangular cross-section. Moreover, there may be a need for special rules for short columns, foundation elements and other types of members.

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