Testing Assessment Procedures with a Damaged Real Building

İ.E. Bal
EUCENTRE, Pavia, Italy

M. Kutanis
Sakarya University, Depart. of Civil Engineering

K. Beyen
Kocaeli University, Depart. of Civil Engineering



ABSTRACT
In the current practice of assessment of existing buildings it is common to use the static pushover methods in order to represent the capacity of the structure. There are several procedures in the existing literature some of which are included in the assessment codes in combination with the Nonlinear Static Procedures (NSPs). It is always a question however, whether these static pushover procedures would be able to accurately predict the real damage state of an existing building or not. Almost all, if not all, of the NSPs are continuation of the well known Capacity Spectrum Method (CSM) which means that they are in intensive use not more than for 15 years. The main dilemma in this point is that none of these methods was tested against the real damage states caused by real earthquakes. Existing calibrations have been done by rather limited experimental results and mostly by more complicated nonlinear time-history analyses. It is comprehensible that there are several inherent uncertainties which are not and cannot be considered in these methods; however, the main goal of these methods is still to predict efficiently the expected damage level. Consequently, the aim of this paper is to investigate the assessment procedure that is given in the latest Turkish Earthquake Code of 2007 (TEC'07) and to compare the outcomes both with the demand level predicted by the most updated CSM of ATC 55 and the damage state of the case study real building that was found heavily damaged in Sakarya, Turkey, following the 1999 Earthquakes. Multi-mode adaptive pushover analyses have been run on this structure to come up with strain levels of the RC sections to be compared with the strain limits given in TEC'07. Uncertainties, such as prediction of the ground shaking level, the dominant direction of the earthquake, inabilities in mathematical modelling etc., have also been discussed and the point of view of the authors has been presented for each case. Errors between the prediction and the real damage state have been identified and possible improvements have been listed.

Keywords: static pushover, assessment, damaged buildings, RC buildings, Turkish Earthquake Code

1. INTRODUCTION

Assessment of existing structures is of prime importance in countries where the building stock has been constructed within the decades in which the aseismic design rules were not the main concern of the designers and the authorities. The existing design codes have evolved during the last two decades and assessment of existing structures has finally become an important topic in the most elaborate seismic codes. The complicated nature of the assessment procedure causes a large scatter in the results of these methods and the assumptions used in the assessment codes; however, the main idea remains more or less the same. Most of these assessment codes, if not all, suggest using static pushover analyses, in several different forms depending on the code, and to combine the pushover results with demand that is related to the code-defined hazard level. There are several challenging uncertainties, both in capacity and in demand parts. Namely, apart from the numerical issues *(i.e. the use of concentrated or distributed plasticity, the material or hysteretic models to be used, etc…)* the accuracy of the monotonic pushover curves is the main question in the capacity side. The main issue in the demand side is the representation of the MDOF demand with SDOF tools, and an additional question arises when the multi-hazard spectra are used for representing a certain level of hazard in the given site.

Despite all difficulties and uncertainties exiting in the procedure, the challenging task of finding the

expected response or damage of a given structure in a given site remains. Thus, the role of the engineers is to answer this question as accurately as possible by using the available tools.

This paper investigates the possibility of matching the predictions of the damage state of a real structure with the damage it has experienced after the strong earthquakes of 1999 in Kocaeli and Düzce, Turkey. Displacement-based Adaptive Pushover analyses have been used in combination with the assessment procedure defined in the most recent Turkish Earthquake Code of 2007 (TEC'07). Results have been compared in global and in element bases. The results obtained from TEC'07 have also been compared with that of the Capacity Spectrum Method (ATC 55, 2005).

2. ASSESSMENT PROCEDURE USED IN THIS STUDY

The most recent Turkish Earthquake Code of 2007 (TEC, 2007) suggests using either force-based linear or displacement-based nonlinear analyses approaches for the determination of seismic performance of RC buildings. There defined information levels in order to account for the uncertainties in defining the material properties. The information levels *(IL)* varying from 0.75 to 1.00 are assigned regarding the availability of the required information.

During the analyses, each structural member has to be defined either as brittle or as ductile, depending on the analysis results. Shear and compression failures are defined as brittle whilst the flexural failure is considered to be ductile.

There are three performance levels defined in the code. These performance levels are: PIO: Immediate Occupancy, PLS: Life Safety, and PCP: Collapse Prevention. If the performance level of the structure is lower than PCP, the building is then assumed as collapsed *(PCL)*. Further details about this procedure can be found in TEC'07 (TEC, 2007) and in Kuran et al. (2007).

The displacement-based nonlinear approach aims to obtain the plastic deformation or force demand at each structural member regarding its ductile or brittle behaviour, respectively. These demands are then compared with the internal force capacities and strain limit states of the member. As opposed to many other assessment codes, the TEC'07 assumes a certain level of damage to be reached only if some percentage of the beams or columns has reached the sectional limit state defined *a priori*.

The pushover curve needs to be transformed to a SDOF curve in which the y axis is acceleration and x axis is displacement. This allows user to compare the response with the SDOF demand spectrum. The MDOF to SDOF transformation is done by dividing the x axis to the modal mass coefficient and y axis to the modal participation factor. The procedure is then completed by extending the initial overall stiffness of the structure up to the spectrum and finding the equivalent displacement in case the elastic period of the structure is longer than the corner period of the code-defined spectrum. Equivalent energy rule is applied when the elastic period of the structure is shorter than the corner period of the spectrum. Details of the method can be found in the Code (TEC, 2007). The soil is Z3 (corresponds to Soil Type C in NEHRP classification) with a corner period of 0.60sec for the structure analyzed hereinafter.

The second assessment procedure is the updated Capacity Spectrum Method (ATC55, 2005) that is based on the initial CSM (ATC40, 1996) but several issues, such as effective damping, effective period and spectrum reduction factors, have been revised following nonlinear analyses on SDOF characteristic systems. Details of the CSM in ATC 55 are not given here due to lack of space; however, it is a well known method that is accessible by the earthquake engineering community.

2.1. Sectional Limit States in the Turkish Earthquake Code of 2007

The sectional limit states in the TEC'07 have been defined in terms of strains in cover concrete, core concrete and the steel rebars. "Minimum" damage limit is the state in which the strain of the outer fibre of the cover concrete to reach 0.0035 or the vertical steel bars to reach 0.010 tensile strain. The

maximum strain values for the "Safety" level on the other hand, are 0.0035+0.01 (ρ_s/ρ_{sm}) ≤ 0.0135 for the core concrete and 0.040 for steel. The limits for the "Collapse" level are 0.0040 +0.014 (ρ_s/ρ_{sm}) ≤ 0.018 for the core concrete and 0.06 for the steel. In these limits, ρ_s and ρ_{sm} stand for the volumetric ratio of existing and code-required transversal reinforcements, respectively. It should be noted that the improvement in the strain limits as fraction of the ratio (ρ_s/ρ_{sm}) is possible only if the confinement has been done according to the recent rules of the Code *(i.e. the stirrup ends to be closed 135° instead of 90°)*. A further research is needed to decide whether the code defined limit states are able to be reached simultaneously for the most common cross-sections used in Turkey; however, in this stage of the study, the code-defined strain limits have been used.

2.2. Global Limit States

TEC'07 follows a relatively more realistic *(as compared to the other assessment codes)*, yet conservative approach, by assuming the limit state of the structure is reached if a percentage of beams and/or column has exceeded given strain limits. A short description of the global flexural limit states used in the Code is given below:

- Limit State 1 is exceeded *(moderate damage)* if 10% of the beams or any of the columns in any section reaches the "Minimum" damage strains. Otherwise, the building is below Limit State 1 *(no or slight damage)*.
- Limit State 2 is exceeded *(heavy damage)* if more than 30% of the beams in the direction of loading or 20% of the columns reach the second limit state of "Safety" level. Limit State 2 is also reached if two ends of the columns, which contribute in total to more than 30% of the base shear capacity, reach the first limit state which is "Minimum" damage level. The rest of the columns must remain below the second limit state in this case.
- Limit State 3 is exceeded *(collapse)* if more than 20% of the beams in the direction of loading reach the second limit state *("Safety" level of strains)*. Limit State 3 is also reached if two ends of the columns, which contribute in total to more than 30% of the base shear, reach the first limit state of "Minimum" damage level. The rest of the columns must remain below the third LS.

3. CASE STUDY BUILDING

The case study structure is a 5-storey RC building that was still under construction during the 1999 Earthquakes of Kocaeli and Düzce. The plans of the structure can be seen in
Figure **1**. It consists of a sub-floor that was designed to serve inside the first commercial floor that has the height of 5.75m. The subfloor has the height of 3.25m. The regular storey height is 2.80m. Infill walls were used only in one bay in the ground floor and partly in some perimeter frames in the first upper floor (see
Figure **2**).

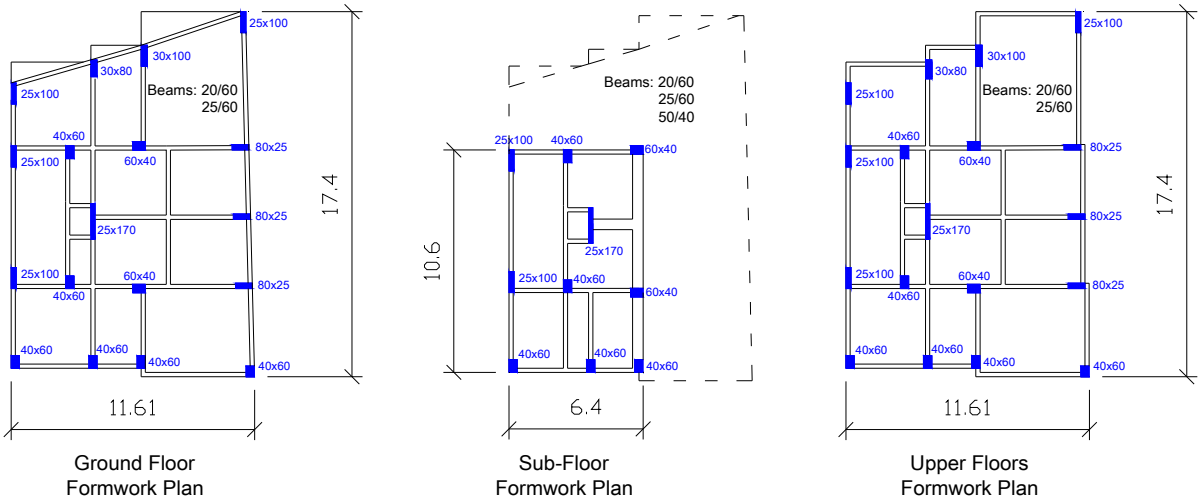

Figure 1. Formwork plans of the case study structure

As part of this research work, a material test campaign has been scheduled for this building. According to this, concrete and steel samples will be taken from the remains of the demolished structure *(the structure has been demolished by the State since it was heavily damaged and could not be used again)* so that the material properties can be obtained more accurately. In this stage of the work; however, the material properties of the structure have been assumed as the values found in the drawings. According to this, the concrete has 20MPa compressive strength and steel has 220MPa yield strength. In the initial design, stirrup spacing has been defined as 9cm in the confinement zones, as part of the design code, but as a general construction practice in Turkey before the 1999 Earthquakes, this design condition was neglected during the construction.



Figure 2. General view of the case study structure

3.1.Observed Damage on the Building

The structure was marked as "heavily damaged" according to the Turkish regulation about the post-event assessments. Heavy damage state occurs if the structure has suffered an irreparable damage. This kind of damage manifests itself either by the structure experiencing more than 1% overall inclination or buckling of the rebars of the columns leading thus loss of elevation of the floors.

The main damage in the structure was observed in the ground floor and in the sub-floor that was constructed inside the ground floor. The corner of the building in the front façade had a residual displacement of around 46cm, translating to 8% drift level for the first floor. The upper floors, on the other hand, remained intact without any significant residual displacement or damage. The 8% drift of the structure could have easily caused the increase of the second order moments on the ground floor columns leading thus to complete collapse of that floor; however, luckily, the upper floors were not completed and they were much lighter than what they would be at the end of the construction. This decreased significantly the second order moments and prevented the building from a total collapse.

There were 3 main types of structural damages observed in the structure. The first type is the pure flexural damage that was observed in column ends *(see damages I and II in*
Figure **3***)*. The second type is the combination of the failure of the outer joints and the flexural hinging of the related column *(see damage IV in*
Figure **3***)*. The last type of damage is the shear damage of the beams *(see damage III in*
Figure **3***)* that occur in the middle of the beam due to existence of less longitudinal bars in that region. There are other types of damages found in the structure as well (i.e. slab damages close to the beam-column joints in the ground floor) but they are not mentioned here due to the lack of space and their negligible effect on the overall damage state.

In damages number I and II in
Figure **3**, the left column has experienced the plastic hinge at the bottom whilst the right column has developed the hinge at the top section. This is simply due to the number of beams that connect to these columns. The moments of the left column have been balanced with a single beam section causing smaller moments at the top section than that of the bottom section. In the right column; however, the moments of the column have been balanced with two beams that caused the top section to develop higher moments than the bottom section.



Figure 3. Damage examples from the structure

4. RESULTS OF THE ANALYSES

Seismostruct software (Seismosoft, 2010) has been used for analyses presented in this paper. Seismostruct is a free software that can conduct nonlinear analyses on distributed-plasticity based elements. The elements may be force- or displacement-based. Force-based elements have been used in the models due to their capability of obtaining the sectional responses more accurately.

The structure has been analyzed by using the Displacement-based Adaptive Pushover (DAP) (Antoniou and Pinho, 2004). Both the TEC'07 and ATC 55 suggest the use of conventional pushover methods in which the load pattern follows the first-mode shape along the height of the building. ATC 55 suggest using more than one loading profile so that the real response can be enveloped. The TEC'07, on the other hand, allows the use of the conventional pushover when the mass participation of the first mode, in the direction of calculation, exceeds 70%.

The case study structure exhibits torsion behaviour in which the first two mode shapes represent heavy torsion behaviour. This, obviously, causes the mass participation of the fundamental modes in two orthogonal directions to be less than 40% for x direction and less then 50% for y direction *(x is along the shorter axis of the structure)*. What TEC'07 suggests in this case is an advanced pushover method, IRSA, that accounts for the contribution of each significant mode into the response in each step individually (Aydınoğlu, 2003 and 2004). The authors, however, preferred to run a displacement-based adaptive multi-mode pushover analysis in IRSA's stead due to the difficulties in implementing the IRSA method without writing an in-house software, which is beyond the scope of this study. The structure may be analyzed once more with IRSA as soon as a computer code becomes available.

The DAP procedure has been applied in x direction of the structure in order to mimic the earthquake effect that seems to be present with residual displacements in that direction. A further study on finding the real dominant direction of the shaking that affected the building may be needed.

The DAP procedure has provided the pushover curve in the form of top displacement versus base shear.

This curve has been transformed to SDOF response, as described in Section 2 of this paper. The design spectrum of the TEC'07 has been transformed into the ADRS format of CSM. The displacement demand has been calculated once by using the equivalent displacement rule given in the TEC'07 and once by using the graphical iteration scheme given in ATC 55. The displacement demand found by using the TEC'07 is 0.097m *(top displacement)* while the top displacement demand is found as 0.187m by using the CSM. The difference between the two methods is almost double that creates serious doubts about the results. The CSM, on the other hand, as being an older and sounder method, seems to be more reliable; nevertheless, this issue needs to be investigated in more detail in the future. The calculation of the displacement demand by using the TEC'07 and ATC 55 methodologies have been graphically shown in
Figure **4**. The method that is used in the TEC'07 is the calculation of an equivalent displacement while the ATC 55 methodology has been applied by using a locus of possible responses.

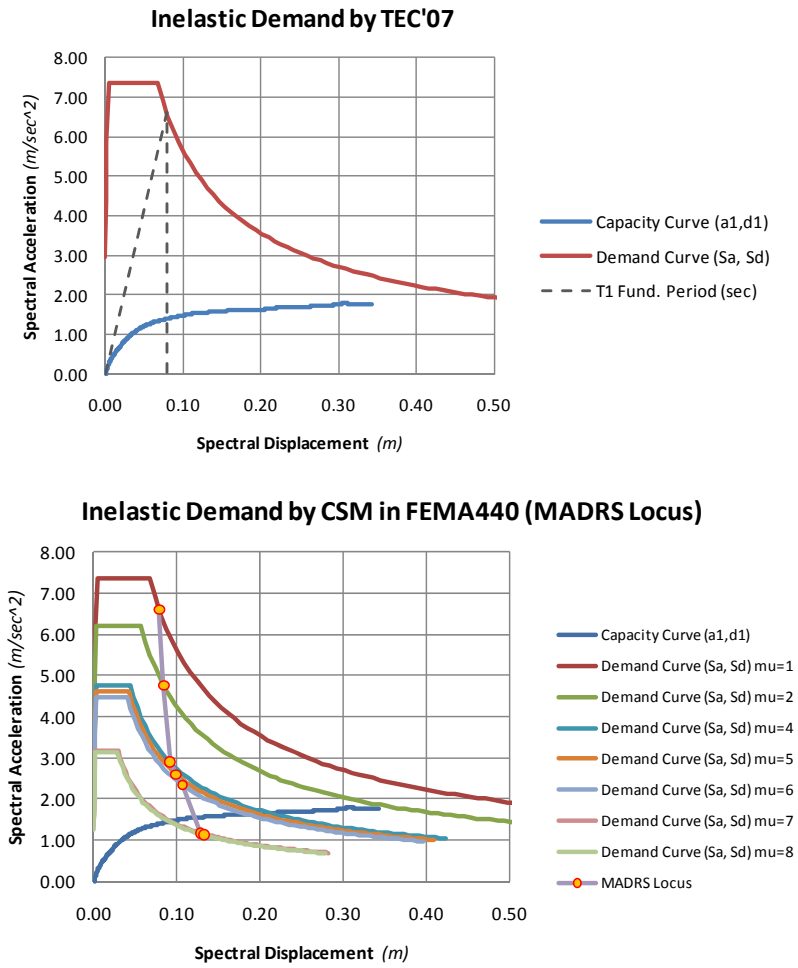

Figure 4. Demand calculation according to the TEC'07 (top) and CSM in ATC 55 (bottom)

Damage level of each member has also been calculated by using the limit states that are defined by the TEC'07 and explained in Section 2.1 of this paper. A graphical representation of each element and the limit state it has reached has been shown in
Figure **5**. It can be seen in that figure that the demand level predicted by the TEC'07 *(0.097m top displacement)* does not cause significant damage in the structure. Most of the members in the ground floor remain below the Limit State 1. The overall damage of the structure, according to the Global Limit States of TEC'07 that are defined in Section 2.2, is found as "Moderate" damage. When similar sectional and global limit states are applied, the damage prediction by using the top displacement demand level of CSM *(0.187m)* is "Heavy" damage. The decision of heavy damage is governed by the damage on the ground floor columns.

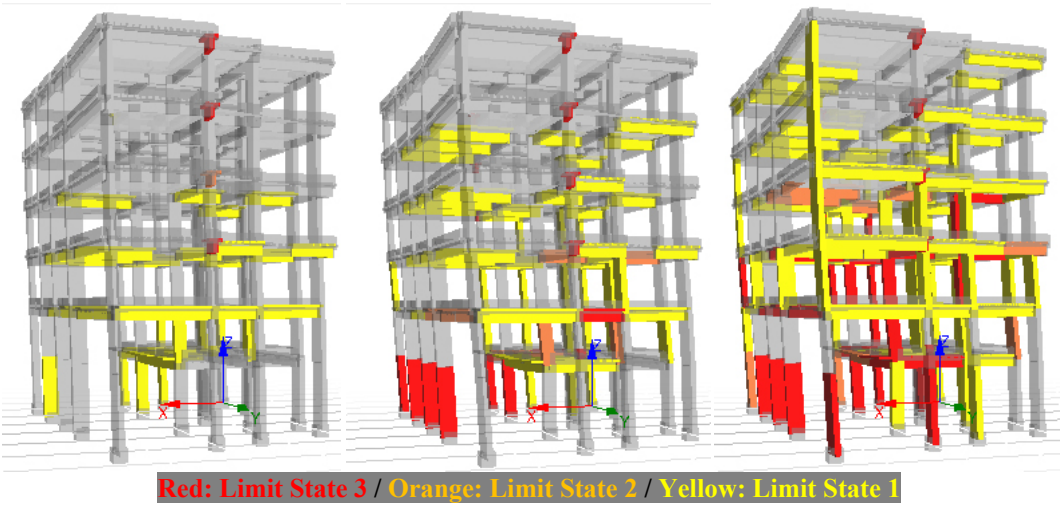

Figure 5. DAP results up to the displacement demand found by TEC'07 (left), up to the displacement demand found by ATC 55 (middle) and up to the displacement level that remained as residual displacement after the earthquake (right)

When the structure is pushed up to the displacement that has remained as residual displacement *(see the snap-shot on the right in Figure* **5***)* the damage of the structure can be identified as "Collapse". The collapse occurs due to the limit that bounds the number of damaged beams; however, the heavy damage *(and the level of damage that could possibly evolve to collapse)* of the structure seems to be caused by the damage on the ground floor columns. This type of analysis *(pushing the structure up to the residual displacement level observed on the field)* does not seem successful either. The main reason behind this is the formation of a beam-sway mechanism in the structure during the analysis while in reality a severe column-sway mechanism is observed. In other words, the damage was concentrated in the ground floor in reality while the damage is found to have onset to the upper floors in the mathematical model. This reflects the two basic problems with the model: lack of representation of the joint failure, and inability to model the shear displacements especially in the below columns of the subfloor *(see the damage types in Figure 3)*.

One of the most important issues that cause differences between the observed damage and the predicted damage is the level of the earthquake shaking. The possible level of the shaking that was experienced by the case study structure during the 1999 earthquakes is a challenging issue to be investigated. A research program by the authors *(project director is the third author)* has been initiated and one of the tasks of the project is to come up with the level of shaking that was experienced by several case study structures in the Sakarya region. A more accurate analysis of this case study structure will be conducted at the end of this research project when a more realistic set of ground shaking is found.

5. CONCLUSIONS

This paper investigates the possibility of matching the predictions of the damage state of a real structure with the damage it has experienced after the strong earthquakes of 1999 in Kocaeli and Düzce, Turkey. Displacement-based Adaptive Pushover analyses have been used in combination with the assessment procedure defined in the most recent Turkish Earthquake Code of 2007 (TEC'07). Results have been compared in global and in element bases. The results obtained from TEC'07 have also been compared with that of the Capacity Spectrum Method (ATC 55, 2005).

The structure has experienced a heavy damage. The first floor displaced 0.46m in average, leading thus to 8% drift level. The upper floors have been found almost intact. The reason why the structure has not reached collapse after such a large level of drift may be the light upper floors. The upper floors were not fully loaded since the structure was under construction during the earthquake. Structural elements have

been damaged either by flexural or shear failures. Some column-beam joints have also been observed to fail before the element ends reach plastic limits. Other types of damages, though not significant in terms of the overall damage, have also been observed. These damages are the cracks on the slab around the beam-column joints, or shear cracks along the beam-to-beam connections.

It was found that the methodology that is suggested by the TEC'07 *(equivalent displacement rule to find the displacement demand)* significantly underestimated the displacement demand. The CSM methodology has also underestimated the displacement demand but the top displacement demand found in CSM is almost double of the one found by TEC'07. One of the reasons of both methods to underestimate the displacement demand may be the use of the design spectrum *(with 0.40g PGA and with 2.5 as the maximum spectral amplification factor between 0.15 and 0.60sec)* while the real demand, possibly, was higher than the one defined by the Code. A further research, tough difficult, is needed in the calculation of the level of the ground shaking that might had affected the structure in 1999.

The damage prediction by using the demand found by TEC'07 and the limit states defined by TEC'07 show that the structure is expected to experience a moderate damage. The damage prediction by CSM points out a heavy level of damage, something which is in agreement with reality. On the other hand, though the overall damage level is predicted correctly be CSM, the spatial distribution of the damage per element has not been captured accurately. Uncertainties and inabilities in mathematical modelling are also expected to play role in this result.

The structure has been pushed up to the displacement level that is found as residual displacement after the strong earthquakes of 1999. The same mechanism (i.e. column-sway mechanism) could not be obtained possibly due to inability of the mathematical model in representing the joint failures and shear damages. This is an issue that needs more improvement in the future.

Finally, the design spectrum has been used to represent the demand; however, a more accurate representation of the shaking that occurred during the 1999 shakings is still essential.

AKCNOWLEDGEMENT

This paper has been produced as part of the research work, *"Improvement of the Performance Based Design and Assessment Methods by Comparisons with the Structural Performances Observed After Earthquakes in Turkey"* that is being funded by *"The Scientific and Technological Research Council of Turkey (TÜBİTAK)"* under the Grant No: 108M303. Their generous contribution is appreciated.

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