Forensic Structural Assessment of Madulla Bridge Failure, Sri Lanka

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Abstract –*Sri Lanka has an extensive road and railway network connecting most part of the country since the transportation had been identified as an essential need from the colonial time. With policies of successive governments during last few decades, most part of rural road network of the country has been upgraded as asphalt concrete paved roads with the intension of improving the quality of road network. However, upgradation of bridges in these roads was not carried out in parallel with road upgradation in most rural roads due to high capital expenditure involved for replacement or rehabilitation of a bridge. Quite a few failures or major structural damages of rural bridges have been reported in recent times. Among these failures, the failure of a rural bridge at Madulla in Monaragala district was reported in October 2023. Madulla bridge was a reinforced concrete decked steel composite beam bridge which was constructed about 20 years ago. This study was carried out to find the root cause of this bridge failure. According to the detailed investigation carried out with a site survey and analytical simulations, it releveled that the reinforced concrete deck of the bridge is structurally inadequate to withstand the reported loading at the time of the failure, and local failures of the slab and excessive deflection of the slab has led to the global failure of the bridge causing lateral bulking of one of the supporting stee beams. Nonavailability of proper connection between supporting beams and the deck would be one main reason for the lateral buckling of the failed beam.*

Keywords: Lateral torsional buckling, Structural assessment, Steel concrete composite beam bridges,

1. INTRODUCTION

Colonial government of the Ceylon initiated the construction of modern road network in the country in early part of 19th century and the network was further expanded with development projects initiated after the independence of the country. Sri Lanka has around 11000 road bridges in roads of classes A, B, C and D. Class C and D roads hosts around 6500 road bridges and can be termed as rural road bridges (Munasinghe et al. 2023).

In any road bridge, structural safety during its entire lifetime is utmost important as failure of bridges can cause catastrophes causing losses of human lives. Further, sudden loss of a bridge may cause considerable impact of routine lifestyle of nearby communities. Also, replacement or rehabilitation cost of a bridge in such situation may also be an implication especially for a country like Sri Lanka. However, failures of bridges without prior warnings have been reported in Sri Lanka in certain occasions. Failure of Paragastoata bridge (1999), Ehelakanda bridge (2006), Rakwana bridge (2014) and Kappitigala bridge (2015) are some the major bridge failures recorded in Sri Lanka. Forensic structural investigation had been conducted regarding these failures (Munasinghe et al, 2023, Atapattu 2016, Abeysuriya 2014, Baskaran 2011) and various root causes have been identified as reasons for these failures. In these studies, it has identified that lateral torsional buckling of top chords as the root cause for failures of Ehalakanda and Kappitigala bridges. This emphasizes the need of paying attention to lateral torsional buckling of steel bridges as it is one critical condition relevant to failure of steel bridges.

One common structural system used for short to medium span bridges in rural roads of Sri Lanka is beam bridge type having reinforced concrete deck supported on steel beams. This technique has been used over long periods of time and factors like simple construction process, lesser construction time and reasonable costs would be probable reasons for the popularity of the system. Figure 1 shows a photograph of this type of steel concrete composite bridge. The same structural system had been used for the collapsed Madulla bridge as well.

Fig. 1. An existing composite beam bridge in rural road in Sabaragamuwa province

These bridges can be constructed with or without having direct connection between beams and top slab. Where direct connection is provided, shear studs are used in top flanges of beams to achieve effective connection between top slab and beams. In such a case, full lateral restraint for the top flanges of the beams would be available from the concrete slab. Even in case of non-availability of shear studs, friction between slab bottom and top flanges of beams would provide the lateral restraints for compression flanges (under gravity loading) as per BS5950:2000. However, factors like excessive deflection of top slab, amount of roughness of slab at bottom level (which may be related to deteriorations relevant to ageing) may be severely affected to lateral restraints for beams in cases where shear studs are not available. Certain literature has highlighted this as a concern (Snijde et al, 2007, Al-Hasany & Al-Zaidee, 2015, Rossia, 2020). According to full scale test results of non-composite steel floor beams with concrete slab carried out by Al-Hasany & Al-Zaidee, 2015, the level roughness of the interface between top flange and the concrete slab greatly influences on buckling load of a beam.

As already mentioned, Madulla bridge was also a similar type of bridge where no shear studs are provided for top flanges of supporting steel beams. The failure of the bridge had been taken place while an open truck full of timber logs was crossing this bridge. Therefore, carrying out a detailed forensic structural engineering investigation on this incident would be beneficial to identify real root cause of this failure. Further, findings of the study can be used to effective management of similar bridges in the road network of the country for the avoidance of this type failure in future. Accordingly, this study was focused on forensic structural investigation of this bridge failure and identification of necessary improvements for similar bridges based on the analytical study of this bridge.

This type of bridge could fail due to various reasons. Some of the main probable reasons are;

1. Overloading of the bridge
2. Lack of structural capacity of the deck (due to less original design load and/or due to deterioration of the bridge with the aging)
3. Lack of structural capacity of the beams (due to less original design load and/or due to deterioration of the bridge with the aging)
4. Foundation/abutment related issues of the bridge
5. A combining effect of all or few aspects listed above

Since this bridge has already failed and debris were immediately removed by the relevant authorities, investigation of certain aspects of bridge became almost impossible and this study was focused on identification of probable reasons for the sudden failures based on an analytical approach developed considering observations and measurements taken at the site immediately after the failure with rational assumptions.

2. CHARACTRISTICS OF THE FAILED BRIDGE

The failed bridge was located in Meegahapitiya- Kottagala Class C road in Madulla area of Monaragala district, Sri Lanka. The maintenance, repair and rehabilitation of this bridge comes under the purview of Provincial Road Development Authority of Uva province of Sri Lanka. The span of the bridge is about 12 m and the bridge width is about 4.2 m. 150 mm thick concrete slab supported on three steel beams had been used as the bridge deck . A view of the bridge before failure is shown in Figure 2. Typical structural details of the bridge are presented as Fig.3.

Fig. 2. A view of the Madulla bridge before failure

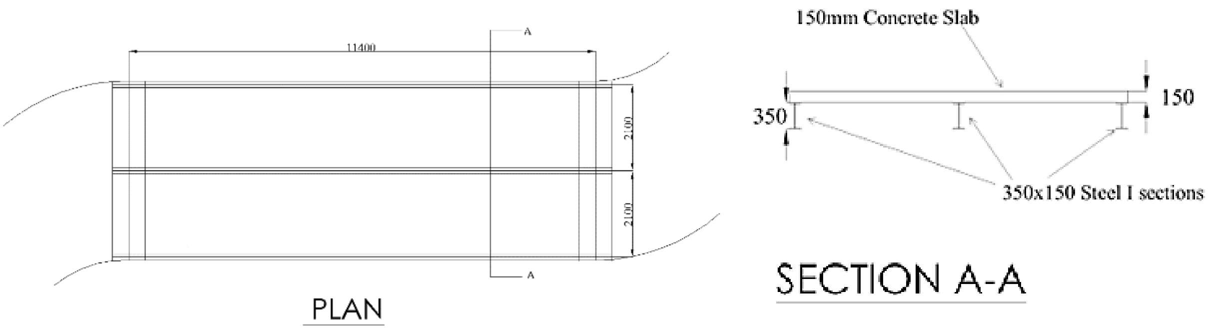

Fig. 3. Layout of the bridge

Part of the bridge (one supporting beam and portion of the top slab) had collapsed when an open truck full of timber logs was travelling on this bridge. Figure 4 shows a photograph of the truck travelling on the bridge at the time of failure and two views of the failed bridge. It was rationally assumed that the truck had approximate weight of 25 MT at the time of the failure (As per the manufacturer's specifications for the truck, the maximum load capacity of truck- Lanka Asoka Leyland Model No. 2516 including self-weight of the vehicle is 25 MT. when considers the load available on it at the time of failure, it could expect that truck was having load close to its load carrying capacity of 25 MT).

Fig. 4. Toppled truck and failed bridge

With objective of executing extensive root cause analysis for this failure, a careful visual inspection of remaining part of the bridge while obtaining possible measurements were performed by the authors. According to the investigation, extensive lateral deformation of failed steel beam was observed (see Fig. 5). It was further noted that portion of the deck slab collapsed along the edge of the mid beam as it can observe in Fig.4.

Fig. 5. Extensive later deformation of edge beam

Also, a considerable amount of corrosion was noted in all three steel beams and area losses of steel beams due corrosion at certain locations were also noted. Deformed steel beam and badly corroded locations of that beam are shown in Fig. 6.

Fig. 6. Visible Corrosion and the area loss due corrosion of the deformed beam

The sofit of the remaining part of the slab was also inspected and photograph of the sofit slab is presented in Fig. 7. Corrosion of reinforcement was clearly visible and insufficient cover for reinforcement seems to be the main cause for this corrosion. It was noted that the cover for reinforcements was less than 10mm in certain places.

Fig. 7. Exposed reinforcements of the deck slab

As per the visible reinforcements, it had provided main reinforcements parallel to the directions of beams at an approximate spacing of 100 mm. The spacing of distribution reinforcements, provided perpendicular to main reinforcements, was about 300mm. However, when it considers about load transferring pattern of the slab, this slab should have been designed as a one-way spanning slab spanning between steel beams even though reinforcements were provided otherwise as per observations. No top reinforcements were found in slab during the observation.

According to the visual observation of the failure site, it was hinted that the ultimate failure of the bridge would have been caused by the lateral buckling of the supporting beam. If the ultimate failure was caused by the failure of deck slab, it could not expect such lateral deformation of the supporting beam. Also, the probability of occurring flexural failure of the beam would be rare as per the deformation pattern of the failed beam. However, it was decided to perform detailed structural assessment of the deck and the beam considering the probable loading on the slab to verify this.

It is very difficult to simulate the exact load condition of the bridge at the time of failure. However, considering the position of the toppled vehicle after the incident, road geometry of the location and geometric details of the vehicle and the bridge, probable layout of the vehicle on the bridge at the time of failure was developed and presented in Fig.8.

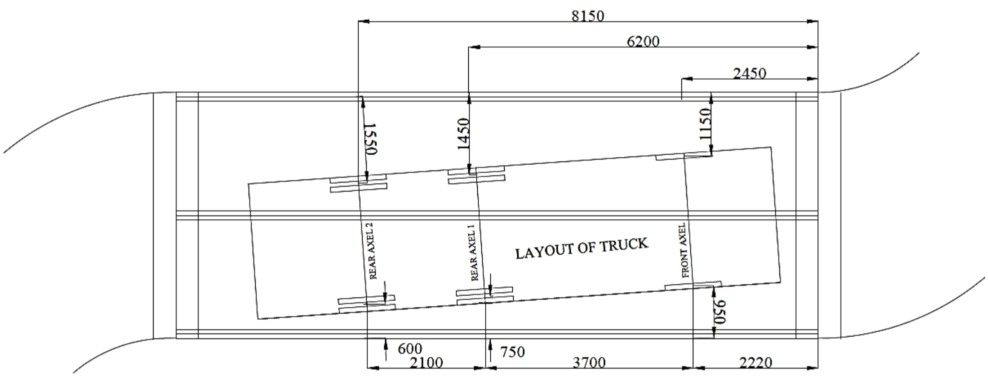

Fig. 8. The probable layout of the truck at time of the bridge failure

As per vehicle specifications of this truck model, the front and rear axle loads are 9000kg and 16,000kg respectively under its full load capacity. Since this vehicle is having two rear axles, it can assume that each rear axle is bearing load of 8000kg. As already mentioned, it can rationally assume that truck was at its full load capacity at the time of the incident. Hence, it was decided to perform a static load analysis of the bridge considering these assumed loads and the developed layout as a forensic engineering analysis.

3. NUMERICAL MODELLING OF THE BRIDGE

Based on this layout presented in Fig. 8, a static load analysis of the bridge with the vehicle loading at time of the failure was performed by developing a SAP2000 v.14 numerical model of the entire bridge. Wheel loads were applied to the model as point loads considering assumed full load condition of the vehicle in addition to the self-weight of slab and steel beams. Analysis was performed under un-factored loading as this analysis was focused on simulating actual failure scenario.

As considerable lateral deformation was observed in the beam that was collapsed during the incident, a separate buckling analysis of that beam was performed using the buckling analysis tool provided in the SAP 2000 v.14. In this regard, an analytical model was developed for the beam using thin two dimensional (2D) four node shell elements to perform the buckling analysis of the beam. The actual thickness of web area and the flanges that were noted at the site were assigned to respective shell elements. The SAP2000 numerical model developed to simulate buckling of the beam is shown in Fig.9.

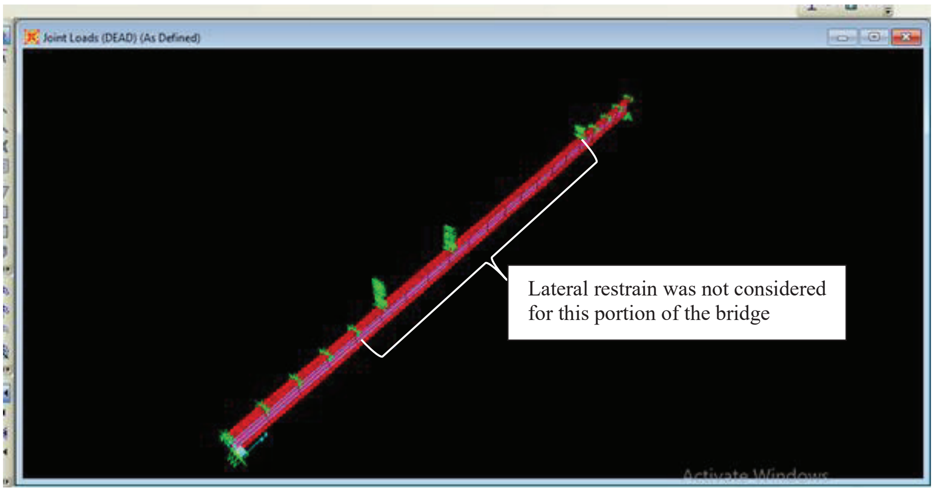

Fig. 9. Prepared numerical model of failed beam

The wheel loads that would have been transferred to the beam were assigned as point loads considering the wheel arrangement presented in Fig. 8. Accordingly, in case of front wheel load, no load was assigned on the beam considering possible local failure of the slab. In case of rear wheel loads, load of 20kN (50% of first rear wheel load) and load of 30kN (75% of first rear wheel load) were applied for the first and second rear wheel locations respectively considering wheel positions represented in Figure 8. Buckling analysis of the beam was carried out considering loss of lateral restraint in a part of the beam length (from 2.5m to 9.5m) due excessive deformation/ local failure of slab. This would be the probable situation that would have existed relevant to this beam at the time of the ultimate failure with excessive deformation/local failure of slab (presence of excessive deformation and local failure assessment of slab has been discussed under Results and Discussion with bending moments and deformation results).

4. RESULT AND DISCUSSION

Fig. 10 shows the resulting bending moments of the bridge deck extracted from the 3D model developed for the bridge in graphical form. Figure 11 shows the deflection contours reported in the bridge model.

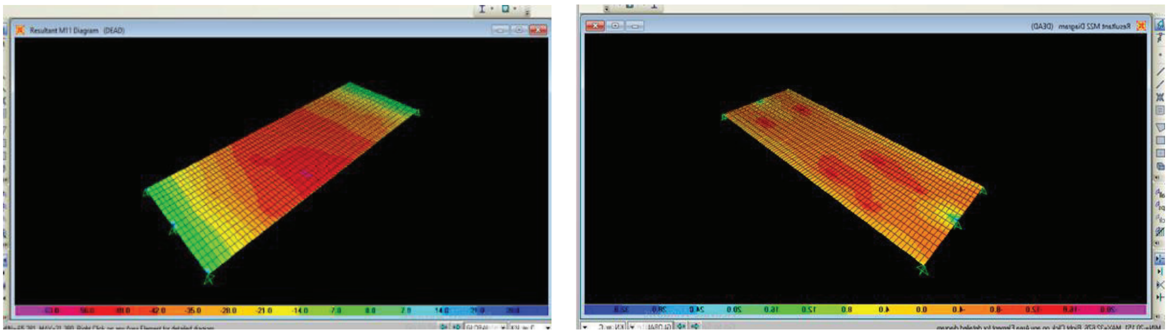

Fig. 10. Prepared numerical model of the bridge deck

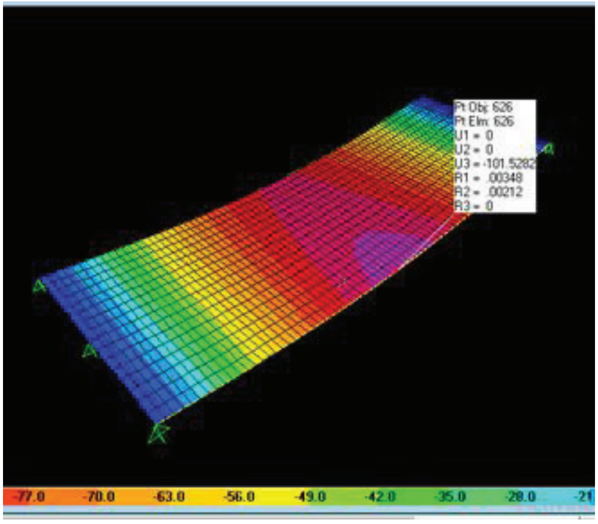

Fig. 11. Deformation of the deck

As per deflection contours of the analysis presented in Fig. 11, the maximum deflection of 101 mm was reported in the slab. This is an unacceptable service deflection as it violates specified allowable deflection limit of L/200 (60 mm) by a fair margin. The deflection contours have widened close to the edge where partial collapse of the bridge was observed at the site compared to the other edge. This would have led to uplifting of the slab from edges where minimum deflections were reported as there was no direct connection between concrete deck and beams in the actual structure.

Reported bending moments in the slab is presented in Table 1. As it can observe in the Table 1, the largest bending moment of 60 kNm/m has been reported in the direction parallel to the steel beams and the reported maximum bending moment in the perpendicular direction (where slab panel should be subjected to predominant bending as an one way spanning slab) is 15 kNm/m. Excessive deflections of beams would be the probable reason for this observation.

Considering reinforcement spacing obtained in the visual inspections, design verifications of slab under flexure in both directions with these reported bending moments were carried out as per BS8110:1997, while considering characteristic yield strength and bar diameter of reinforcements as 460 N/mm^2 and 12 mm respectively. Accordingly, Table 1 also shows calculated bending moment capacities in the slab along main orthogonal directions.

Table 1 Comparison of reported bending moment and moment capacities of slab

Direction	Reported maximum Bending moment (kNm/m)	Moment capacity as per provided amount of r/f (kNm/m)
Parallel to the direction of steel beams	60	45
Perpendicular to the direction of steel beam	15	12

According to the design verification, reported bending moments have exceeded the flexural capacity of the slab by considerable margins in both directions under this loading condition (This would have been further worsened if the corrosion effects of reinforcements were considered for the design verification). Accordingly, probable local failure would have taken place in the edge slab panel where above maximum bending moments were reported. Such failure would have also contributed for losing of lateral restraint to the edge beam.

When it is considered that bending moments (BM) reported in the failed beam (149 kNm), it is below the design bending moment capacity of the beam (185 kNm) if the fully restrained condition for compression flange was assumed as per the provision of BS5950:2000 considering friction between deck and beam tops. (However, due to the corrosion reported in beams, design bending moment capacity of beams would have slightly reduced from above calculated value, but still it would have been a higher value than the reported bending moment in the analysis as ratio of reported BM/BM capacity is 0.80 as per this calculation).

Hence, as already described, a buckling analysis of the edge beam was also performed developing a local model of the beam in SAP2000 v14. as presented in Figure 9. According to results of buckling analysis, Factor of Safety (FOS) of 0.48 was reported against buckling in the beam. The graphical results showing deformation of the beam under buckling analysis is shown in Figure 11(a) and the actual deformation observed in the beam at the site is shown in Figure 11(b). A reasonable match between the deformation pattern of analytical model and the actual beam can be observed. Further, a manual design check of Lateral Torsional Buckling (LTB) of the beam under this scenario was performed as per the approach specified in the BS5950:2000 and it also proved the structural inadequacy of the beam under LTB justifying the results of this modelling(see Table 2). Therefore, it provides a strong suggestion that the ultimate failure of the bridge would have been caused by the LTB of the considered beam once lateral restraint from the beam for the slab was lost by the excessive deformation and the possible local failure of the slab. Summary of results relevant to flexural capacity assessment of the beam is presented in Table 2.

Table 2 Comparison of Bending moments in failed beam

	Analytical Results
Design BM capacity of beam with fully laterally restrained condition	185 kNm
Reported max. BM in the beam from analysis	149 kNm
Ratio of reported BM/BM capacity	0.80
FOS of reported in SAP model under buckling analysis of beam	0.48*
Calculated BM capacity considering buckling **as per SAP model**	71.5 kNm
Calculated BM capacity considering **LTB based on manual analysis**	66.8 kNm

* *Hence, Ratio of reported BM/ BM capacity considering LTB = 1/0.48= 2.08 >> 1*

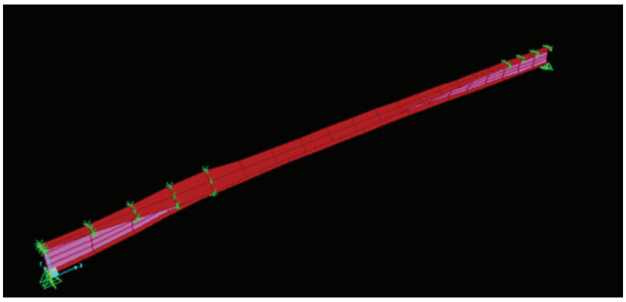

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Fig. 11. Deformed shape of beam at end of the buckling analysis in SAP2000

In summary, according to the results of this study, multiple factors have contributed for the failure of this bridge. Heavy load of the truck is one main reason for this failure. As per results of the analyses, even without considering any deterioration of structural elements of the bridge, probable load of the truck with the timber logs would have been sufficient to initiate the failure of the bridge causing excessive deflection and local failure of the deck.

It can suspect that this bridge would not have been designed by considering this type of heavy vehicular load in the initial design (probably about 20 years back) may be due to lower traffic load expected for this road in the context of low traffic volume presence in the road at that time. Also, the design approach adopted for the initial design is questionable as per the reinforcement pattern available in the deck. This had also contributed to the lack of flexural capacity of the slab.

Further, totally relying on friction between slab deck and steel beams for the prevention of LTB of beams would be another main factor that contributed to this failure. If a composite action between the slab deck and steel beams by means of shear studs or any other appropriate technique was available, the risk of overall failure would be considerably less. However, in recently constructed rural bridges with steel beams in Sri Lanka, shear studs were provided in top flanges in beams to ensure composite action. Fig. 12 shows photograph taken during a recent rural bridge construction and shear studs provided in top flange to connect deck slab is clearly visible here.

Fig. 12: Bridge deck of having shear studs welded to top flanges of supporting beams

Poor workmanship like insufficient cover for reinforcements (causing early corrosion of reinforcement and deterioration of strength) and reduction of cross-sectional areas of steel structural members with the corrosion were also observed in this bridge even though those were not considered in detailed manner for this analysis.

4. CONCLUSION

According to the detailed forensic investigation carried out on the failure of this bridge with the physical investigation of failure site and detailed analytical modeling of critical elements revealed that truck load (of about 25MT) at the time of the failure initially caused the overstressing of the reinforced concrete deck and the excessive deflection with the probable local failure of the deck. It had ultimately led to the global failure of the bridge by causing lateral buckling of one of the supporting edge beams due to the loss of lateral restraint for the compression flange of the beam as results of poor connection between beams and the deck with excessive deflection and local failure of slab.

Hence, it is evident that this type of bridges would be vulnerable for this type of heavy loads. Lack of ultimate loads considered for original designs and non-provision of proper connection of the deck and the supporting beams would be the main probable reason for this vulnerability. Therefore, it emphasizes the need for the adaptation of composite bridge decks with the provision of shear studs as in present construction practice.

The workmanship effects and effects relevant to aging like corrosions were also noted in this bridge. It would have also contributed to this failure even though it was not studied in detail in this analysis due to structural insufficiencies of critical elements found even without such effects. Accordingly, it is clear tha t this bridge was not in a condition to withstand under vehicular load of about 25MT at the time of failure.

Hence, this highlights the need of assessing this type of all rural bridges (especially in rural roads of Class C and D) with probable heavy loads and imposing necessary weight restrictions for required bridges as the probability of travelling this type of heavy vehicles over these bridges is high with the development of rural road network in the country in recent times. That type of assessment would ensure the non-occurrence of similar failures in future.

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