SEISMIC PERFORMANCE OF MULTI-SPAN SIMPLY SUPPORTED HIGHWAY BRIDGES HAVING STEEL COLUMNS

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ABSTRACT

Seismic response of existing multi-span simply supported steel highway bridges, never designed to resist earthquakes, is studied. Dynamic analyses are conducted for bridges with different bearing stiffness, span length and number of spans. It is found that the response of two span simply supported bridges is highly dependent on the stiffness of fixed-bearings on the abutments, but this effect vanishes as the number of spans increases. The transverse direction seismic capacity of bridges having more than two spans is not a function of the number of spans. These bridges may be damaged by earthquakes having peak accelerations less than 0.20g. However, bridges with identical end-to-end length but subdivided into a smaller number of spans are found to be more vulnerable to seismic excitations than those with larger number of spans. Increasing span length is also found to have a negative impact on the seismic capacity of these bridges. Additionally, analytical expressions to calculate the minimum required seat width are developed.

Keywords : Steel, Multi-span simply supported bridges, seismic excitation, bearing stiffness, seat width, seismic capacity

INTRODUCTION

Extensive research has been done on multi-span simply supported (MSSS) reinforced concrete bridges to identify the parameters which affect their seismic response. To count a few, the importance of restrainer ties and shear keys at the expansion joints (Tseng and Penzien, 1973, Penzien and Chen, 1975, Douglas, 1979, Imbsen and Penzien, 1986), the effect of impacting between the adjacent bridge decks on the superstructure-substructure connections (Tseng and Penzien, 1973), the effect of skewness on the reinforced concrete columns (Ghobarah and Tso, 1974), adequacy of lateral reinforcement, end anchorage and splice length in reinforced concrete columns (Degenkolb, 1978), nonuniform distribution of columns stiffness along the bridge and disproportionate flexural and shear strengths of the columns (Priestley, 1988), and the shear-failure vulnerability of bridge piers (Priestley, 1985;1988, Priestley and Park, 1987, Ghobarah and Ali, 1988, Saiidi et al., 1988) have received considerable attention. However, information on the seismic behavior of MSSS steel bridges remains scarce in the literature.

The literature review on the cyclic behavior of steel members showed that such research has mostly concentrated on building members and only a few researchers addressed the weak column strong beam behavior more typical of steel bridges (Popov et al., 1975, Takanashi and Ohi, 1984, Uchida et al., 1992, Schneider and Roader, 1992). Bridges generally have much longer columns than those commonly used in buildings. The slenderness ratios of bridge columns are also much higher. Furthermore, most columns in old steel bridges were not designed with intent to absorb energy through cyclic inelastic deformations and therefore are often non-compact sections. Considering the above factors and the lack of information on the behavior of steel bridge columns, in this study, they are conservatively assumed to fail as soon as the capacity delimited by statically derived interaction curve is reached. The stability interaction equations proposed by Duan and Chen (1989) are used for this purpose.

Most steel bridges in North America have never been designed to resist earthquakes and located in what is now considered seismic regions. In this paper, seismic performance of existing MSSS steel highway bridges, designed without the consideration of seismic forces, are studied. Dynamic analyses are conducted to find the bearings seismically-induced forces and column moments. The following four Western USA earthquakes are used in the analyses of the bridges; (i) El Centro, 1940, E00S, (ii) Taft, 1952, S69E, (iii) San Fernando (Pacoima Dam), 1971, S16E and (iv) Parkfield (Chaloma Shand. 2), 1966, N65E. The average of the results from these earthquakes are presented in this paper. Additionally, equations are derived to check the adequacy of seat width in existing MSSS steel highway bridges. In all cases abutments and foundations deformation and/or damage as well as soil-structure interaction are beyond the scope of this study.

DEFINITION OF THE BRIDGES STUDIED

Most existing steel highway bridges were constructed in the 1960's. Accordingly, to study the seismic performance these bridges, 2 and 3 lane bridges with two spans of each ranging between 20 and 60 metres are designed in compliance with the 1961 American Association of State Highways Officials (AASHO) code (American..., 1961). A typical two span bridge is shown in Figure 1. The left span is attached at one end to the abutment by fixed bearings (abutment-fixed deck) and at the other end supported on the column by expansion bearings, and the right span is attached at one end to the column by fixed bearings (column-fixed deck) and at the other end supported on the right abutment by expansion bearings.

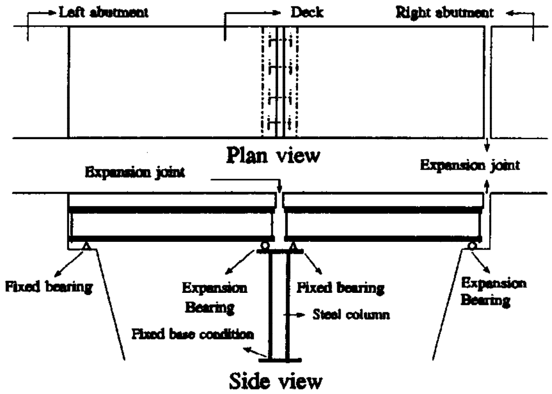

Figure 1 Typical two span simply supported bridge

The bridges are assumed to have fixed and expansion sliding-bearings of the type often found in short to medium span old steel bridges. Other than these bearing types, the theoretical cases of bearings with zero and infinite longitudinal and rotational stiffness are also considered. In all the cases transverse stiffness is assumed to be infinitely rigid.

ANALYSES RESULTS - TRANSVERSE DIRECTION EXCITATION

Bearing Forces

Dynamic analyses of 2 and 3-lane simply supported bridges of spans ranging between 20 and 60 metres are conducted in the transverse direction for different bearing stiffnesses. The transverse bearing coefficient (TBFC) for various bearing types is plotted as a function of span length in Figure 2. TBFC is obtained by dividing the maximum of the resultant bearing forces due to seismic loading in transverse direction, by the bridge mass and the peak ground acceleration (PGA). As seen in that figure, in the case of bearings-set with infinite rotational stiffness, the TBFC and bearing forces increase with span length for spans up to 40 metres, beyond this range the TBFC decreases, but the absolute bearing forces in fact slightly increases. However, the forces in bearings-set with no rotational stiffness are nearly zero. This is largely attributable to the bridge's behavior, since the decks can rotate freely about their supports at the abutments and columns are flexible, the bridge acts as a mechanism.

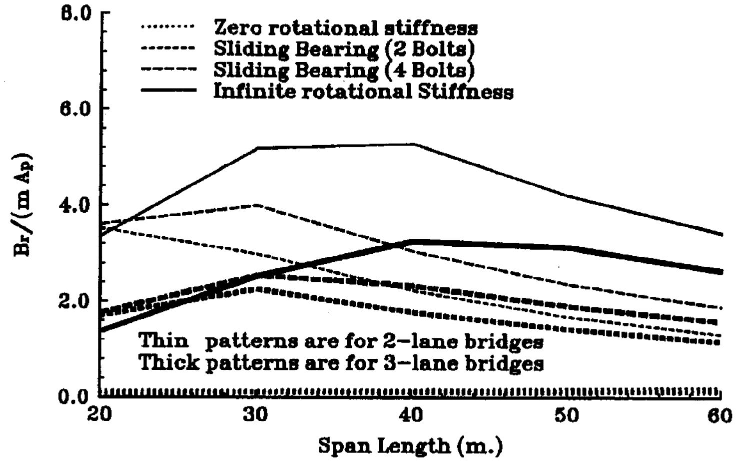

Figure 2 TBFC for two-span simply supported bridges

In Figure 3 the ratio of bearing forces of 2 to 3 lane bridges is plotted as a function of span length. It is observed that for bearings-set with zero rotational stiffness this ratio is almost constant and approximately equal to 1. However, for other types of bearings, the ratio is larger than 1 for short span bridges and gradually decreases with increasing span length.

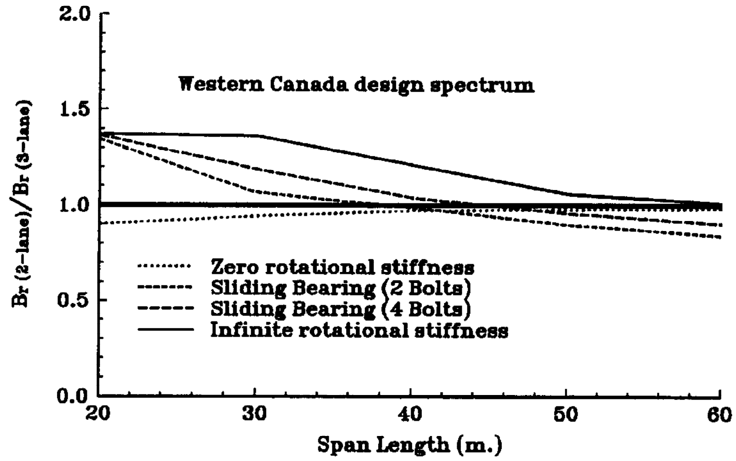

Figure 3 Ratio of bearing forces of two-lane and three-lane bridges

Columns Response

Dynamic analyses for the two span bridges are conducted to obtain the maximum resistible peak ground accelerations (MRPGA) that can be reached prior to column failure. The results are plotted as a function of span length in Figure 4. As seen, the MRPGA's for 3 lane bridges are larger than those for 2 lane bridges when bearings that develop rotational resistance at the supports are used. The contribution of the deck stiffness to the overall transverse stiffness of 2 span simply supported bridges is more effective when bearings having larger stiffness are used at the abutments, resulting in less lateral displacements of the columns. Thus, the difference between the MRPGA of 2 and 3 lane bridges increases as the rotational stiffness of the bearings-set increases. Note that when the bearings-set have zero rotational stiffness, the stiffness of the abutment-fixed deck does not contribute to the lateral stiffness of the structure. Therefore, the MRPGA's are identical for 2 and 3 lane bridges and very low. As observed in Figure 4, increasing span length has a negative impact on the seismic capacity. Although longer bridges may attract smaller spectral accelerations due to their long periods, their displacement at the columns location is very large. Therefore, the column forces become larger in longer bridges, resulting in smaller resistance to seismic forces.

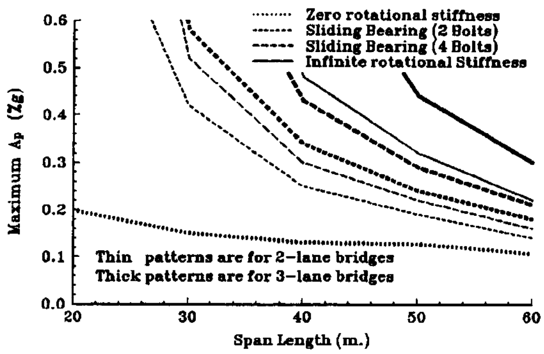

Figure 4 Maximum resistable peak ground acceleration as a function of span length

Additionally, simply supported bridges with 2, 3, 4, and 5 spans are analyzed to obtain the MRPGA's. As seen in Figure 5, the 2 span bridge has a relatively high capacity when sliding-bearings are used. However, the capacity rapidly drops to more than half when bearings-set with zero rotational stiffness are used. The effect of bearing stiffness on the seismic capacity practically disappears for bridges having more than 2 spans. Seismic capacity of such bridges is almost identical. Additionally, the response of a 3-span, simply supported bridge with 120 metres end-to-end length is compared to that of a 2-span simply supported bridge of identical end-to-end length. It is found that the seismic capacity of 3 span bridge is 35 percent higher than that of 2 span bridge of identical end-to-end length.

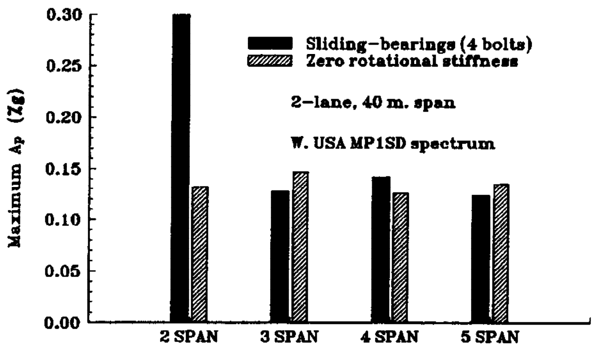

Figure 5 Effect of number of spans on seismic capacity

It is noteworthy that the MRPGA's obtained from the analyses are very small regardless of the number of spans. These results are actually not surprising. Imbsen and Penzien (1986) studied the seismic response of the Fields Landing Overhead which suffered major damage during the Trinidad-Offshore, California Earthquake of 1980. They estimated that a PGA of about 0.1 to 0.2g occurred at the bridge site during the earthquake. Furthermore, Longinow et al. (1979) categorized MSSS bridges as unsound i.e, certain to fail during earthquakes, if not retrofitted. Results of the current study agree with these findings, and provide a quantitative measurement of the vulnerability for typical steel highway bridges.

ANALYSES RESULTS - LONGITUDINAL DIRECTION EXCITATION

In the longitudinal direction, the displacements of the column-fixed decks can be very high but the decks may collide before the displacements reach the point at which the columns would fail. To investigate this, the maximum longitudinal displacement, Δ_{cL}, that the columns can accommodate before failure is calculated and compared with the expansion joint widths (EJW) for the bridges considered in this study. It is observed that the columns can easily sustain longitudinal displacements larger than the EJW prior to failure, and that collusion of the decks is a more important problem for longitudinal seismic excitation. However, as the number of spans increases, the sum of the EJW's also increases. This sum may be larger than the maximum displacement that the steel columns can accommodate before failure. In this case the safety of the columns in the longitudinal direction cannot be ensured.

The minimum PGA required to produce collision is calculated for the bridges considered in this study. It is found that the PGA's needed to produce collision increase with span length but are all less than 0.1g. In reinforced concrete bridges, impacting between two adjacent sections of the superstructure causes high shear forces and possibly failure in the bearings located above the columns (Imbsen and Penzien, 1986, Zimmerman and Brittain, 1981). Such failure could make the structural system unstable since the superstructure becomes disconnected from the columns and the simply supported decks may fall off their supports if the seat width is not adequate (Zimmerman and Brittain, 1981). Although it is unclear whether bridges having steel columns of lower reactive mass would suffer similarly high impact-induced bearing shear forces, it would be safe to conservatively assume they would while awaiting for further evidence.

MINIMUM REQUIRED SEAT WIDTH AT THE EXPANSION JOINTS

The minimum required seat width at the expansion joint is obtained considering the most critical of; (i) maximum possible expansion joint opening, (ii) the maximum displacement of the columns prior to failure due to longitudinal and transverse direction seismic excitations respectively. These will be assessed individually in the following subsections.

Maximum Possible Expansion Joint Openings

Maximum possible EJO's of a two span and a four span simply supported bridge in the longitudinal direction are illustrated in Figure 6. Assuming that all the joints have the same width and neglecting the deformation of the abutments, the two and four span bridges can respectively have maximum EJO's equal to one and three times the EJW. Similarly, in the transverse direction, due to the rotation of each span, the corners of the decks at the expansion joints displace longitudinally, and eventually, get in contact with each other and with the abutment walls. Thus, neglecting the deformation of the abutments, the maximum possible opening in the expansion joints is also the same in this direction. The EJO's obtained from case studies on MSSS bridges conducted by Tseng and Penzien (1973), Zimmerman and Brittain (1981) and Imbsen and Penzien (1986) confirms that seismically-induced EJO's in MSSS bridges do reach close to their maximum physically possible value.

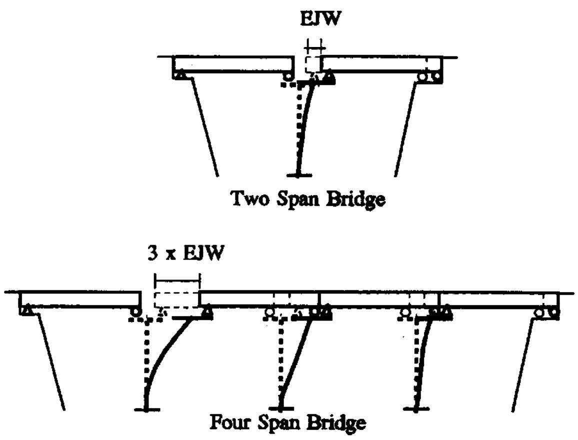

Figure 6 Maximum possible EJO due to longitudinal direction displacement of structure

If the fixed bearings at the left abutment seen in Figure 6 are damaged, the abutment-fixed deck is then also free to slide in the longitudinal direction, and can potentially have a maximum displacement equal to the sum of the EJW's. The sum of the EJW's is calculated simply by multiplying the total end-to-end length, L_T, of the bridge by the coefficient of thermal expansion and a maximum or design differential temperature of 70 C° assumed for North America. Also note that when the temperature is at its minimum design value, the expansion bearing is closer to the seat edge. Therefore, an allowance for temperature changes is conservatively calculated by multiplying the length, L_i, (in metre) of the span supported by the expansion bearings at expansion joint i by coefficient of thermal expansion and design differential temperature. Additionally, a 50 mm distance between the edge of the support and the centerline of the bearings is provided to prevent failure of the structure due to local damage at the support edge. Using the above information, the minimum required seat width SW_i (in mm) at expansion joint i is expressed by the following equation;

$$SW_i = 50 + 0.84 \; (L_T + L_i) \tag{1}$$

The above equation gives an estimate of the minimum distance needed between the centerline of the bearing and the edge of the support to prevent bridge decks from falling off their seats.

<u>Expansion Joint Openings Due To Longitudinal Direction Displacements of Columns</u>

The maximum EJO's are controlled by the maximum deformation capacity of the columns prior to failure. The maximum possible EJO's due to relative displacement of columns are illustrated in Figure 7 for two MSSS bridges.

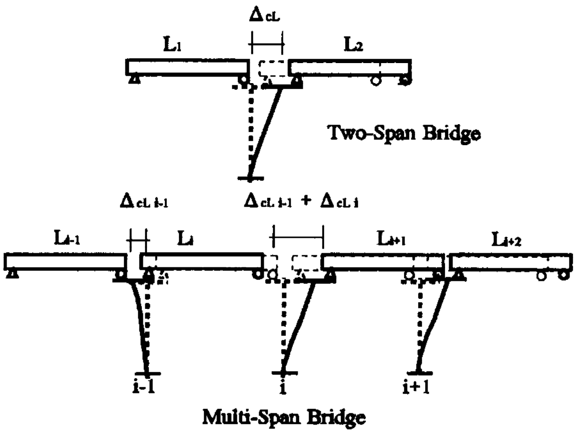

Figure 7 Maximum expansion joint opening before the failure of columns

In such bridges, the opening of the ith expansion joint is equal to the sum of the maximum displacements of columns i-1, and i before failure. As a general rule, when considering the opening of an expansion joint, the column at the expansion joint of interest and the neighbouring column connected to the deck by fixed bearing should always be considered. Accordingly, the minimum required seat width, SW_i at expansion joint i is expressed as;

$$SW_i = 50 + 0.84 \; L_i + \Delta_{cL_{i-1}} + \Delta_{cL_i} \tag{2}$$

Using the maximum possible column displacements before failure obtained using twenty existing steel bridges with different span lengths and column heights, an equation for the maximum possible displacements of the columns is obtained as a function of span length and column height. This equation is then substituted in the above equation to obtain the minimum required seat width as;

$$SW_i = 50 + 0.84 \; L_i + \left(30 - \frac{L_{i-1} + L_i}{10} \right) h_{c_{i-1}} + \left(30 - \frac{L_i + L_{i+1}}{10} \right) h_{c_i} \tag{3}$$

where h_{ci} is the height of column i.

Expansion Joint Openings Due To Transverse Direction Displacements of Columns

The maximum possible EJO's due to seismic excitation in the transverse direction are now considered. The EJO's of a MSSS bridge are shown in Figure 8. The assumed deformed geometry in this figure produces the maximum rotation of the decks hence a maximum EJO.

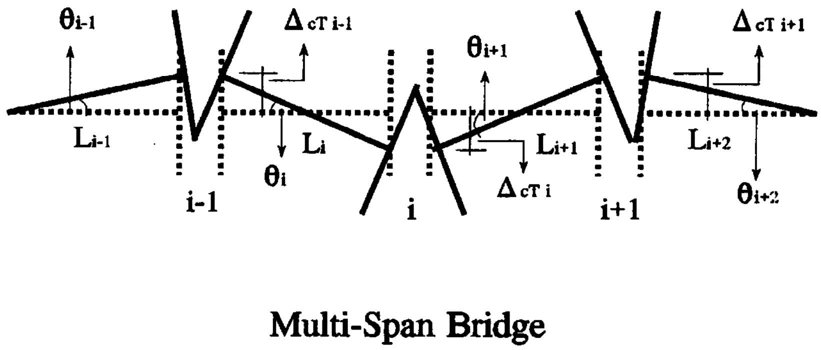

Figure 8 Maximum expansion joint opening before the failure of columns due to transverse direction seismic excitation

Considering EJO openings due to this deformed geometry the minimum required seat width, SW_i at expansion joint i is expressed as;

$$SW_i = 50 + 0.84\, L_i + \frac{b}{2}\left(\frac{\Delta_{cT_{i-1}} + \Delta_{cT_i}}{L_i} + \frac{\Delta_{cT_i} + \Delta_{cT_{i+1}}}{L_{i+1}} \right) \tag{4}$$

where b is the width of the bridge. A procedure similar to the one described in the above section is followed to obtain the minimum required seat width expressed below, as a function of span length and column height;

$$\begin{aligned} SW_i = 50 + 0.84\, L_i &+ \frac{b}{2}\left(\frac{50}{L_i} - 0.15\left(1 + \frac{L_{i-1}}{L_i}\right)\right) h_{c_{i-1}} + \\ &\frac{b}{2}\left(\frac{50}{L_i} + \frac{50}{L_{i+1}} - 0.15\left(2 + \frac{L_{i+1}}{L_i} + \frac{L_i}{L_{i+1}}\right)\right) h_{c_i} + \\ &\frac{b}{2}\left(\frac{50}{L_{i+1}} - 0.15\left(1 + \frac{L_{i+2}}{L_{i+1}}\right)\right) h_{c_{i+1}} \end{aligned} \tag{5}$$

Interpretation of Derived Equations for Minimum Seat Width

Three equations are derived to define the minimum required seat width. Eqs. 3 and 5 are derived considering the maximum displacement of the columns prior to failure due to longitudinal and transverse direction seismic excitations respectively. The purpose of these equations is to determine if a bridge is likely to fall off its supports before the columns reach their capacity. Due to the nature of earthquake excitations, it is found conservative and logical to use the larger of the results obtained from these equations to calculate the minimum required seat width. However, neglecting the deformation of the abutments, the EJO defined by the maximum displacement of the columns cannot be larger than the sum of the EJW's. Therefore, the larger of the results obtained from Eqs. 3 or 5 should be compared with the result obtained from Eq. 1 and the smaller of these defines the minimum required seat width. However, calculation of seat width for 16 bridges with different number of spans and span length showed that Eq. 5 provides results smaller than those of Eq. 3 for all the cases considered. Eq. 5 may yield larger seat widths only for wide and short span bridges where the deck rotations, hence the EJO's, are larger. However, for short span bridges, Eq. 1 yields results smaller than those of Eq. 5 and therefore it governs. Accordingly, Eq. 5 need not be used in most of the cases.

CONCLUSIONS

Bearing stiffness is found to considerably affect the seismic response of two span simply supported bridges. However, for bridges with more than two spans, the effect of bearing stiffness is localized and it vanishes with increasing number of spans. Generally, the bearing forces increases with span length and they are larger for narrower bridges for the range of spans considered.

The seismic capacity of multi-span simply supported bridges considering column instability is studied. It is found that for two span simply supported bridges, the MRPGA's for wider bridges are larger when bearings that develop rotational resistance at the supports are used. The difference becomes more pronounced as the rotational stiffness of the bearings-set increases. However, this effect vanishes for bridges with larger number of spans. For bridges with bearings-set of zero rotational stiffness, the MRPGA's are identical for 2 and 3 lane bridges. It is found that bridges with this type of bearings may be damaged by earthquakes of PGA's less than 0.20g. Increasing span length is also found to have a negative impact on the seismic capacity due to high moments exerted on the columns.

Seismic response of bridges with more than two span is also studied. It is found that, the transverse seismic capacities of multi-span simply supported bridges are almost identical. Bridges with smaller number of spans are found to be more vulnerable to seismic excitations than those with larger number of spans of identical end-to-end length.

For the seismic response in the longitudinal direction, it is found that the PGA's required for collision increases with span length, but they are all less than 0.1g. Therefore, collision of the decks in the longitudinal direction is inevitable. It is noteworthy that impacting between the two adjacent sections of a bridge superstructure upon collision may cause high shear forces in the bearings and therefore these components may fail before the columns. This would produce highly unstable systems for which the superstructure is disconnected from the columns, with high risk of having simply supported decks falling off their support. Therefore, impacting should certainly be prevented to avoid damage to bearings hence to the structure.

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