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An Experimental Study into the Distribution of Earthquake Forces in Steel Plate Girder Bridges

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ABSTRACT: Recent earthquakes have resulted in damage to components of steel plate girder bridge superstructures and have highlighted the need to design these components for seismic loading. A $^{2}/_{5}{}^{th}$ scale model was constructed in order to study the seismic response of a typical concrete slab-on-steel girder bridge superstructure. The initial focus was on the transverse response of the bridge superstructure, with earthquake loading simulated by pseudo static loads applied at the deck level, using twin actuators. The distribution of resulting forces was determined from experiments and supporting finite element analysis. The longitudinal load path was also studied using finite element analysis. Critical elements identified in the load path include the shear connection between the deck and steel girders, as well as the end cross frames, web stiffeners and bearings. Except for bearings, the effect of each of these components is discussed in this paper.

1 INTRODUCTION

In high seismic regions, typically short to medium span bridges are designed to withstand the effects of large earthquake ground motion using ductile design of the substructure. This design methodology is based on the assumption that the superstructure is able to withstand seismic forces and remain elastic. This assumption is generally valid for monolithic concrete superstructures. However, typical concrete slab-on-steel girder bridge superstructures consist of a number of components critical in the seismic load path, which when not specifically designed for earthquake loading, may not remain elastic. Recent earthquakes have caused damage to some of these components, including bearings, cross frames, web stiffeners and localized damage to the steel girders (Astaneh-Asl 1994, Shinozuka 1995). Such damage has emphasized the need to understand and control the load path for lateral inertia loads, from the deck level through the superstructure and into the substructure. Furthermore, this damage has highlighted the potential role these superstructure components may play in reducing the earthquake demand on a bridge superstructure, if they were to be capacity designed.

A single span model was constructed in order to identify and study the critical components in the seismic load path of a typical steel girder bridge superstructure. This paper focuses on identification of the load path and evaluation of the forces in these components when subjected to horizontal transverse and longitudinal earthquake loading. Having identified these components, it was anticipated that they could be designed to withstand, or ideally, mitigate the effects of an earthquake. The superstructure model was first designed to withstand earthquake loads elastically allowing for a ductile substructure. Then lower strength, end cross frames were designed for inelastic behaviour in attempts to limit the forces and necessary ductility demand in the substructure.

2 BRIDGE MODEL

The bridge superstructure model consisted of two 60ft (18.3m) steel I-girders connected with shear studs to a reinforced concrete deck slab as shown in Figure 1. The model was designed according to the AASHTO LRFD design specifications (AASHTO 1998) and represents two girders and a single span of a typical multi-girder, multi-span bridge at $^{2}/_{5}{}^{th}$ scale. Cross frames were located in vertical planes between the two girders and were placed at the ends, i.e. above the supports, and at intermediate locations along the length of the bridge at 10ft (3.05m) intervals, to represent the 25ft (7.62m) maximum spacing requirement in the AASHTO Standard Specifications (AASHTO 1996). The details of the cross frames used in each experiment are given in Section 3.2 below. For these experiments the bearings were laminated rubber pads typical of those used to allow for thermal expansion. The bearings were restrained from deflecting in the transverse direction. Typical cross sectional dimensions at a support are shown in Figure 2.

Figure 1. Bridge model with actuators applying pseudo-static transverse loads

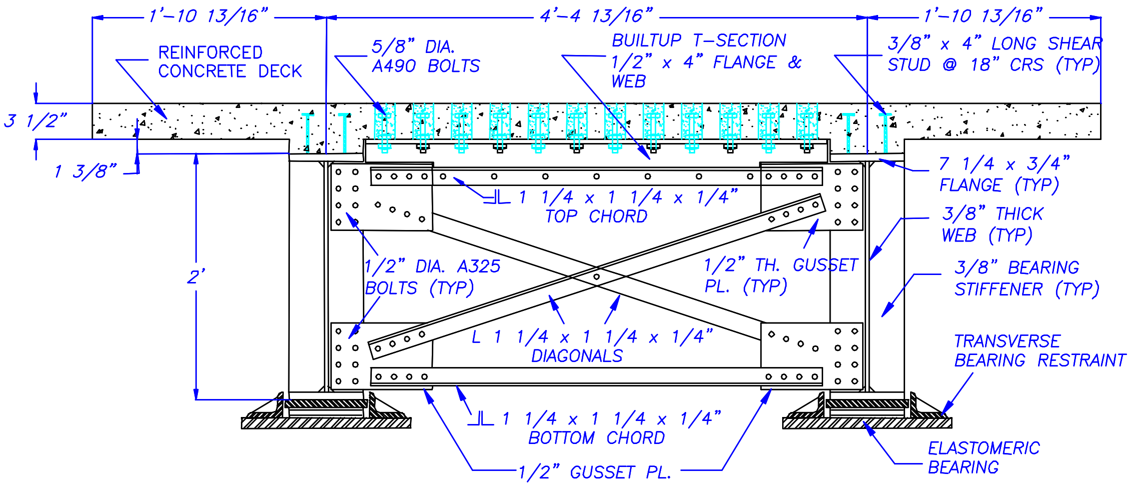


Figure 2. Cross section of bridge model at support

The lateral contribution of the various components in the bridge model was established in a series of experiments, listed in Table 1. Earthquake loading was simulated pseudo-statically in the transverse direction of the bridge model. The pseudo-static loads were simulated using increasing amplitude cyclic forces in two actuators connected at the $^{1}/_{3}$ points along the length of the bridge (Fig. 1). They were connected to the bridge at deck level, where typically 80% of the mass is located and therefore the majority of inertia load is generated.

A finite element model was constructed for the bridge model. Shell elements were used for the girders and deck, and frame elements for other components such as cross frames and shear studs. Elastic shell elements, used to model the deck, were given an equivalent elastic modulus to allow for cracking. This did not fully capture the true strain profile in the deck but, overall, results compared well with measured values from the experiments (Carden, Unpubl.)

Table 1. Bridge model configurations for pseudo-static experiments.

Expt #	End Cross Frames				Intermediate
	Diagonals	Top Chord	Bottom Chord	Composite	Cross Frames
1	Large Bolted	Bolted to Gusset Plates	Bolted to Gusset Plates	End 2 Only	Yes
2	Large Bolted	Bolted to Gusset Plates	Bolted to Gusset Plates	Both Ends	Yes
3	None	None	None	None	Yes
4	None	Bolted to Gusset Plates	None	Both Ends	Yes
5	None	Bolted to Gusset Plates	Bolted to Gusset Plates	Both Ends	Yes
6	None	None	None	None	No
7	None	Pinned to Web Stiffener	Pinned to Web Stiffener	Both Ends	Yes
8	Small Welded	Pinned to Web Stiffener	Pinned to Web Stiffener	Both Ends	Yes

3 TRANSVERSE LOAD PATH

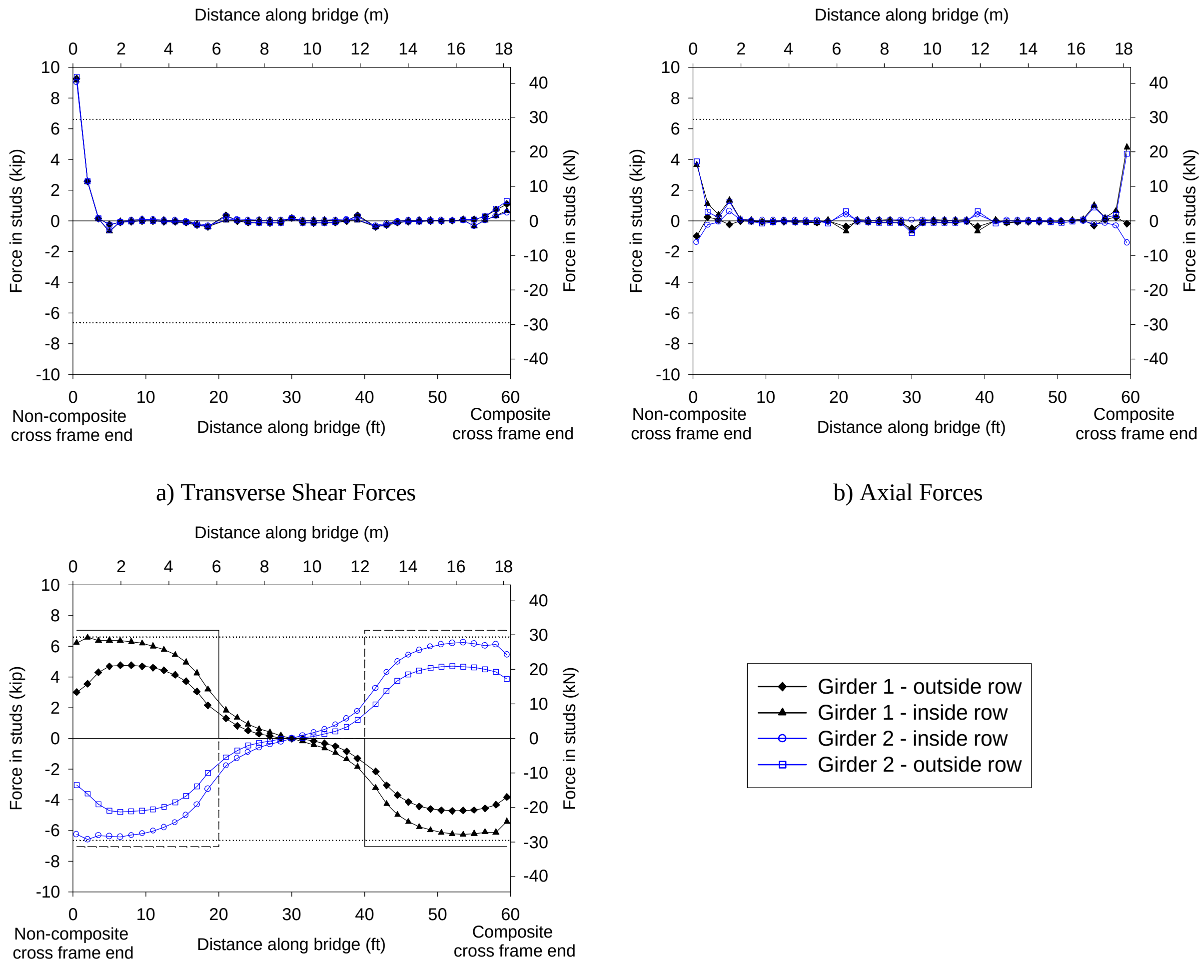
3.1 *Composite action*

One important consideration, generally overlooked in the seismic design of steel bridges, is the connection between the deck and the girders. This is usually provided by the shear studs welded to the top flange. In a steel plate girder bridge, the inertia loads generated during an earthquake are concentrated in the deck slab and must be transferred to the girders through the shear studs. However the shear studs are not currently designed for seismic loads and, if AASHTO procedures are followed, their design is typically governed by fatigue from vehicle loading.

The first experiment on the bridge model resulted in failure of three rows of shear connectors on each girder at one end of the bridge. At the other end there was no evidence of failure, which was attributed to a different end cross frame detail where the top chord was connected to the deck, as shown in Figure 2. This type of detail is sometimes used at abutments to mitigate the effects of wheel impact loading. The composite top chord of the cross frame appeared to help transfer the inertia loads into the end cross frame reducing the transverse shear forces in the studs of the main girders. There was no composite member at the other end where failure occurred.

Finite element analysis supported the above observation. A transverse load, equal to that at the completion of Experiment 1 (94 kips, 418 kN) divided equally between the two actuator attachment points, was applied to the finite element model. Figure 3a shows that the transverse shears were small in all of the shear studs, except those at the end with the non-composite cross frame where the forces sharply exceed their ultimate level as indicated by the dotted line. At the other end, where the cross frame was composite with the deck, the transverse shear forces were small, consistent with experimental observations.

The analysis showed that transverse loading not only subjected the shear studs to transverse shear forces, but also to longitudinal shear and axial forces. Like the transverse shears, relatively large axial forces were concentrated at the ends of the bridge while elsewhere along the span the forces were negligible (Fig. 3b). However, unlike the transverse shears, the axial forces were similar at both ends of the bridge and the forces were only concentrated in two of the four rows of studs. It can be concluded that these forces are the result of overturning of the girders as illustrated in Figure 4, exaggerated for clarity. The edge of the haunch (pap) on the loading side of each girder is considered to be in contact with the edge of the girder flange. Therefore, as the lever arm is short the studs on the contact side were expected to have small forces, while the studs on the other side of each



a) Transverse Shear Forces

b) Axial Forces

c) Longitudinal Shear Forces

Figure 3. Axial and shear forces in the shear studs of the bridge model.

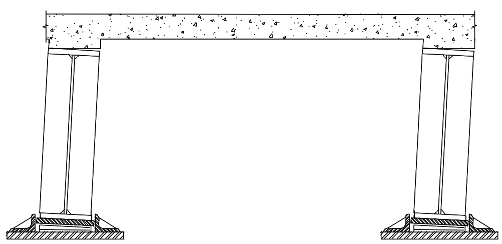


Figure 4. Deformed shape of end region in bridge model

girder have larger tension forces. The magnitude of the axial forces in the shear studs is dependent on the lateral displacement at the ends of the bridge. Therefore while the axial forces were below ultimate levels in Figure 3b, this was not the case for more flexible cross frames, discussed in Section 3.4 below.

The longitudinal shear forces in the studs were considerable, as shown in Figure 3c, with maximum forces close to the capacity of the studs and occurring over the end-thirds of the span. These forces were due to the transfer of stresses between the deck, subjected to transverse bending, and the girders. It can be observed in Figure 3c that the distribution of longitudinal forces in the shear connectors is similar to the pattern of the shear force in the bridge. Therefore in an attempt to develop a simple model to explain these forces, a section through the bridge was considered. The section was transformed to an equivalent steel section and a transverse shear force was applied for convenience through the shear centre of the section. The shear flow between the deck and the girders was then calculated using conventional theory. This model does not account for torsional effects due to loading eccentric from the shear centre and true end restraints which make the analysis much more complex, beyond simplified methods. However, the resulting theoretical distribution of longitudinal forces in the shear studs, as shown by the dashed lines in Figure 3c, can be seen to give a relatively good, yet conservative estimate of the forces calculated in the finite element model. Further parametric studies are required before definitive statements can be made for different superstructure configurations.

A shear stud can be considered to have two possible failure modes; failure of the studs, or failure of the concrete surrounding the studs. The ultimate theoretical shear capacity for the 3/8 in (10mm) diameter studs was calculated to be 6.64 kip (29.5kN) and was governed by the failure of the stud itself. However, theory suggests that the limiting force for concrete failure around the stud is only 10-20% greater than the stud capacity (AASHTO LRFD (6.10.7.4.4c), Ollarrd et al (1971)). Therefore, at the ultimate limit state of the stud, the concrete is also close to its ultimate limit state. Past research on cyclically loaded shear studs (McMullin 1994) show that typical studs have a very brittle failure mode. Cracks were observed in the haunch of the bridge model at each shear stud location along $^{1}/_{3}$ of the length at both ends of the bridge model, indicating relatively large forces at these locations. This is consistent with the longitudinal shear forces calculated in the finite element model. Combining the shears and axial loads, given in Figure 3, it is inferred that the ultimate force would be exceeded in the first 2-3 rows, which is consistent with the observed failure in the bridge model.

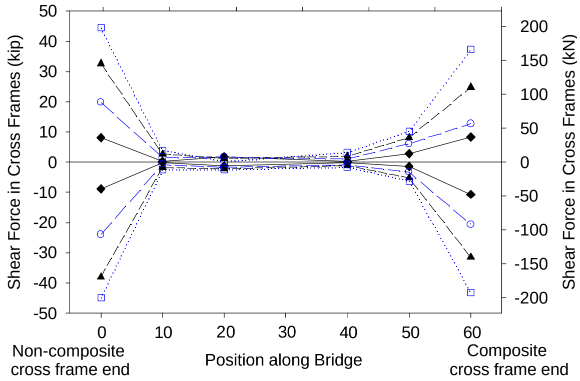
It is important to note that the maximum transverse load applied to the bridge model was considerably greater than that used in design, and stud failure in the prototype bridge is unlikely to occur except in the most severe earthquake. However the onset of damage to the studs occurs at a much lower level of load and capacity checks should be introduced into the design procedure for these superstructures. It is also noted that the number and spacing of the studs in this model were less than that used in current design and are more representative of details found in older bridges. For both of these reasons, the probability of stud failure is less in a bridge designed to modern standards and subject to moderate sized earthquake. But the need to consider earthquake loads in the design of shear studs is evident, with special consideration necessary for bridges designed to be non-composite in negative moment regions. Further parametric studies will investigate the forces in a range of typical multi-span, multi-girder bridges in order to develop design procedures for the shear studs.

3.2 *Distribution of Forces in the Cross Frames*

Previous research (Itani 1996, Zahrai 1998) has identified the end cross frames, i.e. those located directly above the abutments and column bents, as critical elements in the transverse load path. Meanwhile, intermediate cross frames, located at intermediate positions along the span, have minimal effect in resisting seismic loads. AASHTO has limited guidelines for the design of cross frames. Some of the factors that should be considered in design are listed, but little guidance is given as to how they should be designed. Mertz (2001) identified the deficiency in the codes and proposed design recommendations for the design of intermediate cross frames, considering such factors as wind, oversize vehicle impacts and compression flange stability requirements. However, no attempt was made to consider seismic effects and therefore these recommendations are not suitable for end cross frames as the load path for seismic loads is different. Recognising the importance of end cross frames in the seismic load path, the end cross frames in the first experiment, referred to as the 'large' cross frames, were designed to exceed the maximum estimated force induced in developing the full capacity of the substructure. Gusset plates were used to ensure the connections were stronger than the members, as shown in Figure 2. Top and bottom chords were used to evenly distribute forces between the girders and bearings in the event of inelastic behaviour in the diagonal members. Intermediate cross frames were designed with the same size members but simpler connections, as inelastic behaviour was not expected in these members.

The shear forces in the end and intermediate cross frames, measured in the bridge model are shown in

Figure 5. These forces were measured using a series of strain gauges on the tension and compression diagonal members and converted to a shear force in each cross frame. Figure 5 illustrates that the intermediate cross frames have minimal effect in the transverse load path for this bridge. More closely spaced intermediate cross frames at the ends, and inelastic behaviour in the end cross frames, are likely to result in larger forces in the intermediate cross frames, although these forces are unlikely to exceed elastic levels.



Note: Instruments on cross frame at midspan were faulty therefore these data points have been omitted from the figure.

Figure 5. Shear forces in the cross frames of the bridge model at peaks of increasing amplitude cycles.

3.3 *Influence of Various End Cross Frame Components*

The first set of end cross frames exhibited minimal inelastic behaviour in Experiments 1 and 2 (Table 1) before other limit states such as damage to shear studs became critical. In the design of these members the minimum specified steel strength and strength reduction factors were used in order to ensure that the estimated substructure capacity was exceeded, which resulted in considerable over-strength. The design also assumed that the transverse shear was carried entirely by the diagonal members of the cross frames. However it is apparent that other components added to the transverse resistance of the end region. These components include the intermediate cross frames, although the forces in these were shown to be small, and frame action between the bearing stiffeners, top and bottom chords and the deck.

Assuming an elastic response, the transverse force in each of the above components depends on its stiffness. Experiments 3 to 6 (Table 1) investigated their relative contribution by removing each of these components. The measured relative contribution is summarised in Table 2. Values for stiffness were taken from the end (End 2) that was least affected by damage and repair during the first experiment. Stiffness values were calculated using each cycle up to a total applied load of 30 kips (133 kN).

Table 2 shows that the diagonal members represent only 80% of the transverse stiffness at the end of the bridge. This contribution would be expected to decrease once the members exhibit inelastic buckling and yielding. The top and bottom chords contributed 15% to the lateral stiffness, indicating that frame action between these and the web stiffeners is significant. The influence of the intermediate cross frames is shown to be minimal.

3.4 *Modification of End Region for Ductile End Cross Frames*

As explained in the introduction, one reason for studying the seismic load path is to evaluate the possibility of placing ductile components in the superstructure and reduce seismic forces in the substructure. Following from this, it was proposed to make the end cross frames ductile, thereby limiting transverse seismic forces in a bridge (Zahrai 1999). In order for these to be effective they need to be able to allow relatively large displacements without attracting large forces. Even if the main members of the cross frames are ductile the bearing stiffeners and deck will still tend to attract forces. Zahrai proposed to trim the bearing stiffeners such that their moment of inertia was minimal allowing relative shear between the top and bottom flanges. However, the bearing stiffeners require a minimum width in order to attach the cross frames and ensure adequate bearing strength, therefore this approach is limited. Following from the observed deformation in the bridge model when there were no end cross frames, it was proposed to promote the mechanism, as shown in Figure 4, where the girders are allowed to effectively “rock” under the deck slab. Pinned top and bottom chords, as shown in Figure 6, were designed to provide minimal lateral stiffness and promote this mechanism. ‘Small’ diagonal X-braces (Fig. 6) were also placed in the model, designed to be ductile at a shear force smaller than the capacity of the substructure. Table 2 shows the relative stiffness of each component using this detail. The top and bottom chords with pinned connections had a reduced lateral stiffness of 6 kip/in [1050 kN/m] compared to 59 kip/in [10300 kN/m] for the old detail. As the lateral forces due to wind and other live loads are small compared to those from a large earthquake, and in fact the cross frames are largely redundant under normal bridge conditions, it is not expected that lowering the lateral stiffness will have a significant effect on the service performance of the bridge.

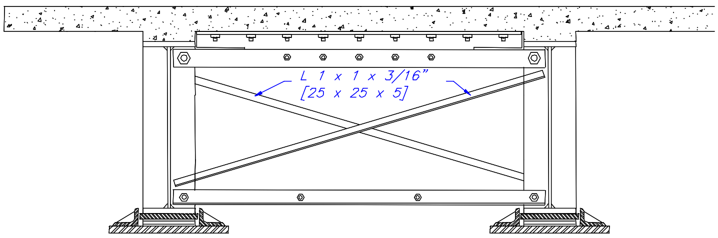


Figure 6. Pinned top and bottom chord with ‘small’ end cross frames.

Table 2. Relative contribution of components in the transverse load path for ‘large’ and ‘small’ single angle end cross frames.

Component	'Large' Cross Frames			'Small' Cross Frames		
	Stiffness (kip/in)	**Stiffness (N/mm)**	**Contribution (%)**	**Stiffness (kip/in)**	**Stiffness (N/mm)**	**Contribution (%)**
Diagonal Members	320	56.1	80%	172	30.0	87%
Top and Bottom Chords	59	10.3	15%	6	1.0	3%
Intermediate Cross Frames	2	0.4	1%	2	0.4	1%
Web Stiffeners and Other	17	3.0	4%	17	3.0	9%
Total	399	70	100%	197	35	100%

4 LONGITUDINAL LOAD PATH

The longitudinal seismic load path in a typical steel plate girder bridge is much more clearly defined than the transverse load path. Again typically 80% of the inertia loads are generated in the deck slab. These are transferred through the shear connectors into the girders and into the bearings. Longitudinal loads were not studied experimentally using the bridge model, however the longitudinal load path was examined analytically using the finite element model. Analysis shows that forces in the shear studs, due to longitudinal loading at deck level, are evenly distributed along the length of the girder. The forces were calculated to be much lower than the capacity of the studs, hence there is no apparent need to check their design for longitudinal earthquake loading. Stresses in the girders and deck due to longitudinal earthquake loading were also minor.

5 CONCLUSIONS

Based on the pseudo-static experiments and analytical studies on both the transverse and longitudinal response of the bridge model, the following conclusions are made:

- The connection between the deck and steel girders is important in the both the longitudinal and transverse seismic load path. The forces in the shear studs due to transverse loading are potentially damaging. Shear forces, both transverse and longitudinal, and axial forces were observed in the shear studs resulting from transverse earthquake loading. Transverse shear forces were largest and concentrated directly above the girder supports. A composite top chord connecting the cross frames to the deck is effective in reducing these forces. Axial forces in the studs are caused by the relative rotation between the girders and deck and are dependent on the lateral stiffness of the end region. Longitudinal forces result from shear flow due to transverse bending of the bridge superstructure. These were conservatively predicted for the bridge model using conventional theory.
- The forces in the shear studs due to longitudinal seismic loading are evenly distributed among the shear studs along the length and result in small forces in individual studs.
- The end cross frames, i.e. those located directly above the supports, transfer the majority of the transverse earthquake forces into the substructure. In typical configurations, earthquake induced forces in the intermediate cross frames are negligibly small.
- Ductile end cross frames appear feasible based on the transverse load path observed in the bridge model. By pinning the top and bottom chords of the end cross frames and minimising the number of shear studs on the girders in the end regions, it was possible to create a mechanism with a low lateral stiffness, increasing the effectiveness of this type of system.

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