Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 13 (1988)

Artikel: Dynamic response and subsequent retrofit of a tied-arch bridge

Autor: Kulicki, John M. / Mertz, Dennis R. / Murphy, Richard E.

DOI: https://doi.org/10.5169/seals-13017

Dynamic Response and Subsequent Retrofit of a Tied-Arch Bridge

Dynamisches Verhalten und Verstärkung einer Bogenhängebrücke

Comportement dynamique et réhabilitation d'un pont arc à tirants

John M. KULICKI
Partner
Modjeski and Masters
Harrisburg, PA, USA

John M. Kulicki received his bachelor's degree from Lafayette College, Easton, PA and his masters and doctorate from Lehigh University, Bethlehem, PA. He joined Modjeski and Masters in 1974 and was named a Partner in 1980. Dr. Kulicki is the Partner-in-Charge and Principal Investigator on the I-24 Tied Arches Study.

Dennis R. MERTZ
Engineer
Modjeski and Masters
Harrisburg, PA, USA

Dennis R. Mertz was awarded his doctorate from Lehigh University, Bethlehem, PA in 1984. While at Lehigh, he was involved in the study of fatigue and fracture of steel bridges. Dr. Mertz is currently Project Manager of Modjeski and Masters' study of the I-24 Tied Arches.

Richard E. MURPHY
Engineer
Modjeski and Masters
Harrisburg, PA, USA

Richard E. Murphy received his engineering degree from Drexel University, Philadelphia, PA. In six years with Modjeski and Masters, he has been involved in design, development of in-house software and field inspections. Mr. Murphy's responsibilities on the current I-24 project include field investigation, finite element modelling and analysis.

SUMMARY
Two tied-arch spans across the Ohio River are experiencing excessive vertical dynamic displacement under normal traffic. The distortion, in addition to being intolerable due to public reaction, has resulted in fatigue cracking of the welded-floorbeam webs. The problem was studied through a combination of field studies and finite element modelling. These investigations suggest that the distortion problem can be solved through the addition of a longitudinal stiffening truss near the roadway level, and bolted connections spanning existing web gaps.

RÉSUMÉ
Deux ponts arcs à tirants sur l'Ohio présentent des mouvements verticaux excessifs sous des charges normales de trafic. Cette situation anormale n'est pas acceptée par les utilisateurs et résulte aussi en l'apparition de fissures de fatigue dans les poutres soudées du tablier. Le problème a été étudié sur place et au moyen d'un modèle par éléments finis. Ces recherches montrent que le problème de distorsion peut être résolu par l'addition d'une poutre de rigidité longitudinale près du tablier et par des assemblages boulonnés entre les poutres existantes.

ZUSAMMENFASSUNG
Zwei über den Ohio führende Bogenhängebrücken weisen unter normalen Verkehrslasten übermässige vertikale dynamische Verformungen auf. Diese Verformungen haben schon zu öffentlichen Reaktionen und in den geschweissten Fahrbahnträgern zu Rissen geführt. Das Problem wurde mit Feldmessungen und Finite-Elemente-Modellen untersucht. Die Abklärungen führen zum Schluss, dass das Verformungsproblem mit einem versteifenden Längsträger nahe der Fahrbahn und mit geschraubten Verbindungen über die vorhandenen Oeffnungen im Flanschbereich gelöst werden kann.

1. BACKGROUND

Two tied-arch spans carrying I-24 across the Ohio River near its confluence with the Mississippi River are experiencing severe distortional problems. These spans are 192 m and 223 m in length and of the bowstring-type tied arch in which the tie girder contributes insignificant bending stiffness to the span. The longitudinal bending stiffness of the spans is concentrated in the very significant arch rib. The spans are connected together and to the riverbanks by continuous plate-girder spans forming a river crossing totalling over 1,700 m in length. A photograph of one of these spans is shown in Figure 1. The resultant problems are twofold. The global vertical displacement field of the tied-arch spans is excessive and, due to the public's loss of confidence in the bridge, intolerable. Secondly, out-of-plane distortions of the floorbeams, which are related to the global displacement, have resulted in ever-increasing fatigue cracking in the webs of the welded floorbeams at locations shown in Figure 2.

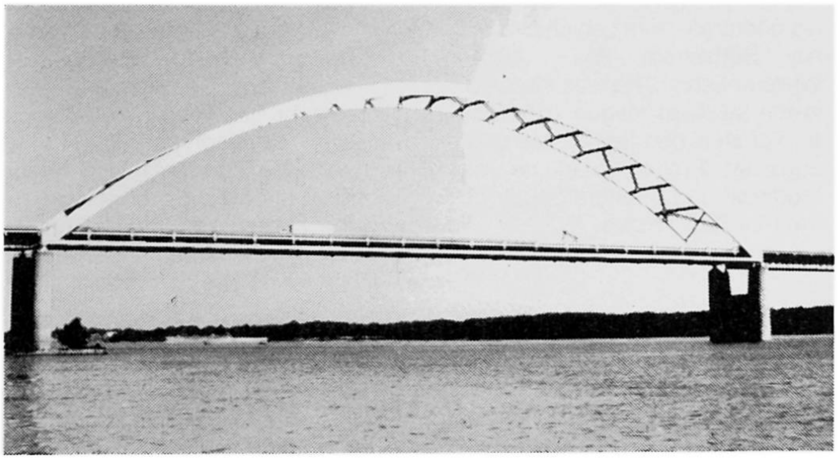


Figure 1 - View of 223 m tied-arch span.

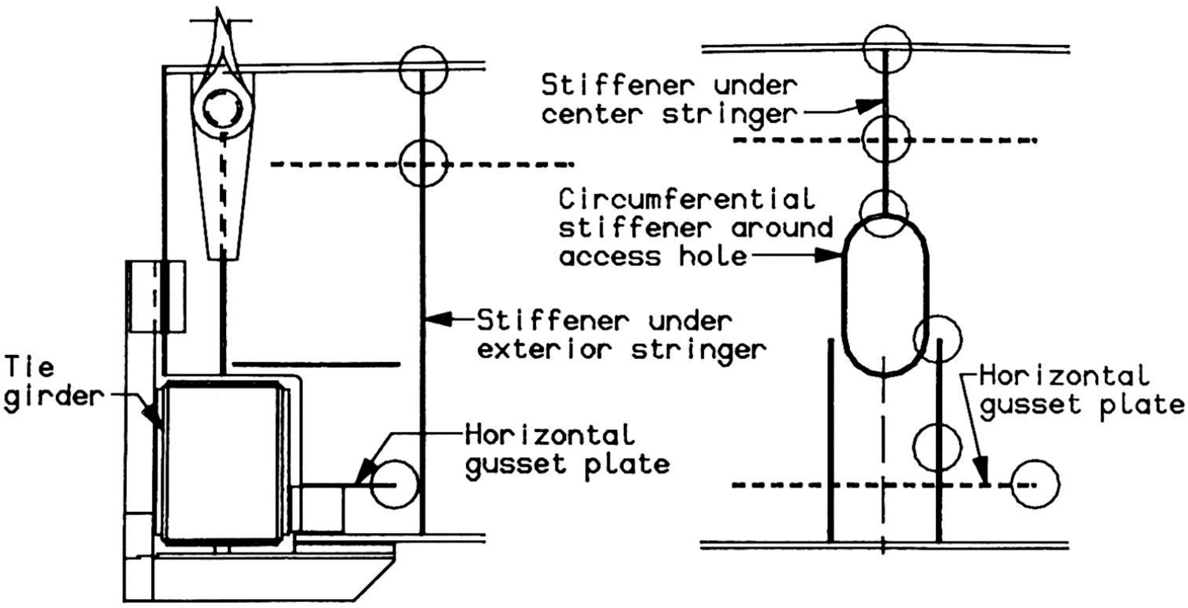


Figure 2 - Locations of typical fatigue cracks on welded floorbeams.

The distorted shape of the tied arches under vehicular traffic is dominated by the classic first bending mode for an arch, i.e., a 360 degree sine wave. The maximum vertical displacement at the quarter points of the longer span have been observed by the owner to be approximately +0.25 m and -0.25 m, resulting in a total excursion of one-half of a meter. The most significant distortions are produced by single trucks (or the chance occurrence of trucks traversing the tied-arch span side-by-side for the entire span), since trucks on other portions of the span tend to counteract the displacement and dampen the motion.

2. CORRELATIONS BETWEEN FIELD AND ANALYTIC STUDIES

Investigations began with field studies using accelerometers and strain gages. Analytic studies utilized extensive finite element dynamic modelling. The finite element models used to study the global dynamics were three-dimensional assemblages of plate bending, beam and truss elements as shown in Figure 3. A comparison of field-measured and calculated frequencies and mode shapes are shown in Figure 4. In this figure, the closed circles represent measured normalized amplitudes at the discrete accelerometer locations, while the continuous solid lines represent calculated mode shapes. Note that the mode numbers indicate that not all of the theoretical modes are significantly excited by the controlled truck traffic used in the field studies. During the course of the study of the tied-arch spans in their original configuration, it became evident that only the lower frequencies were significant. This corroborates the field observation of displacement-induced crack locations that suggests that the first bending mode is dominant. The comparison of field and analytic results indicates that the computer modelling accurately represents the physical situation. This step provided a confident base onto which various retrofits could be superimposed.

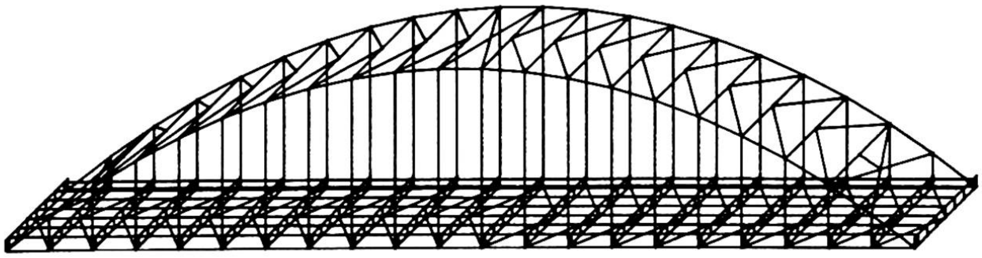
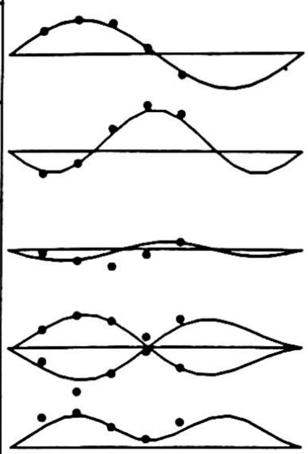


Figure 3 - Isometric view of three-dimensional finite element model.

192 m SPAN

CALCULATED		MEASURED	
MODE NO.	FREQUENCY	MODE NO.	FREQUENCY
1	0.317 HZ	1	0.316 HZ
2	0.654 HZ	2	0.679 HZ
3	0.684 HZ	3	0.953 HZ
4	0.894 HZ	4	1.050 HZ
8	1.276 HZ	5	1.165 HZ



223 m SPAN

CALCULATED		MEASURED	
MODE NO.	FREQUENCY	MODE NO.	FREQUENCY
1	0.271 HZ	1	0.273 HZ
2	0.558 HZ	2	0.566 HZ
6	1.012 HZ	3	1.023 HZ
8	1.165 HZ	4	1.250 HZ
9	1.488 HZ	5	1.550 HZ

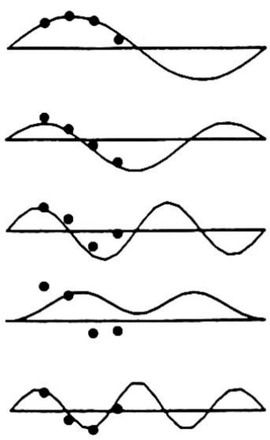


Figure 4 - Comparison of field-measured vs. calculated frequencies and mode shapes.

3. PARAMETRIC STUDIES OF POTENTIAL GLOBAL RETROFITS

3.1 Structural Retrofits

The effects of varying certain bridge stiffness parameters were considered in an attempt to determine an appropriate corrective course of action. The parameters singled out as potential sources of decreasing the magnitudes of the displacement field of the tied arches were the addition of several inclined hangers; and increases in tie girder axial stiffness, tie girder flexural stiffness, hanger axial stiffness, hanger flexural stiffness and arch rib flexural stiffness.

The dynamic properties of the original tied-arch spans and their potential retrofitted versions were studied through more efficient two-dimensional finite element modelling. Fundamental frequencies and their accompanying mode shapes were determined through modal analysis utilizing the SAPIV finite element library. The effects of increasing the original span parameters, indicated above, are illustrated in Figure 5.

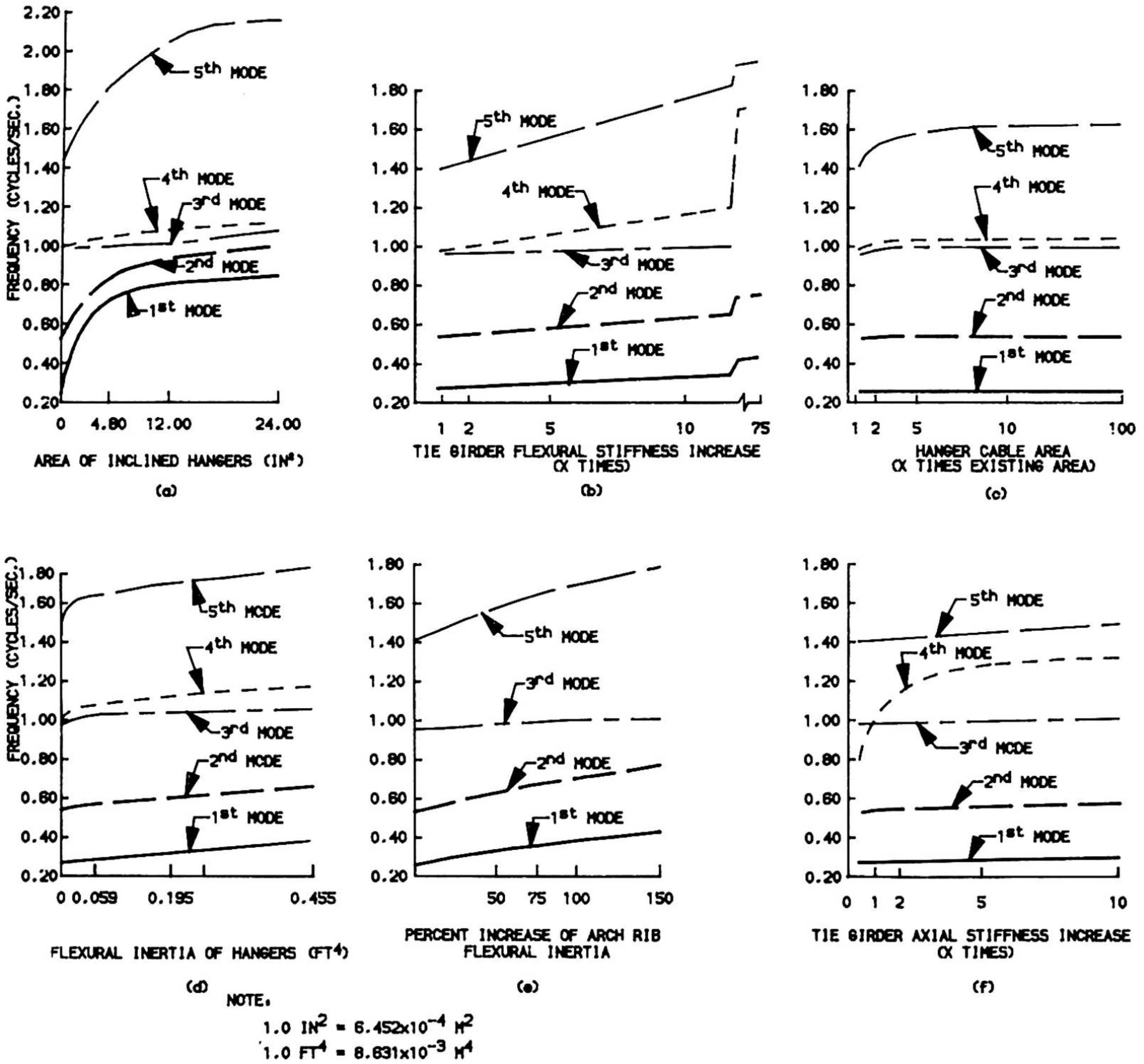


Figure 5 - Structural parameters vs. frequency.

Of the structural changes considered the increase in the flexural stiffness of the tie girder (Figure 5b) and the addition of inclined hangers (Figure 5a) held the most promise. Increasing the axial stiffness of the tie girder or one order of magnitude or more (Figure 5a) is not practical. The increase of hanger axial stiffness by two orders of magnitude (Figure 5c) is totally ineffective. Conversion of the hangers into flexural members (Figure 5d) would be very costly and only marginally effective. Finally, adding flexural stiffness to the already significant arch rib (Figure 5e) is less than practical, both in terms of material used and required height of construction, and is only marginally effective.

The required increase in tie girder flexural stiffness of almost two orders of magnitude indicated in Figure 5 can be achieved relatively easily by converting the tie into the bottom chord of a full-length longitudinal stiffening truss. Inherent in this proposed retrofit is a redundant load path for the originally fracture-critical tie girder. The hypothesized addition of several inclined hangers to supplement the existing hangers has significant effects on the dynamic response of the tied-arch spans. This retrofit basically ties the point of zero vertical deflection at mid-span to the points of maximum vertical deflection at the quarter points. Practically speaking, this alternative converts the tied-arch into a hybrid structure of unknown experience, which is troublesome.

The effect of these two most promising retrofit concepts on the dynamic response under vehicular traffic were studied using the three-dimensional finite element models. A typical comparison of the two concepts with the original response is shown in Figure 6. This comparison shows the time history of quarter point displacements under truck passage calculated for the original configuration and hypothetical retrofits based on (1) adding inclined hangers and (2) conversion of the tie into a stiffening truss. The results of these studies indicate that while both are quite effective in reducing the unacceptable vertical displacement, the stiffening truss retrofit concept is more effective in reducing the crack-inducing distortion of the welded floorbeams. Specifically, the stiffening truss would result in live load dynamic displacements of about 30% of the original movements while the inclined hangers would reduce it to only 15% of original. Stress ranges at details near the center of the floorbeam would be reduced to 40% of original by the stiffening truss, or to 70% of original by the inclined hangers.

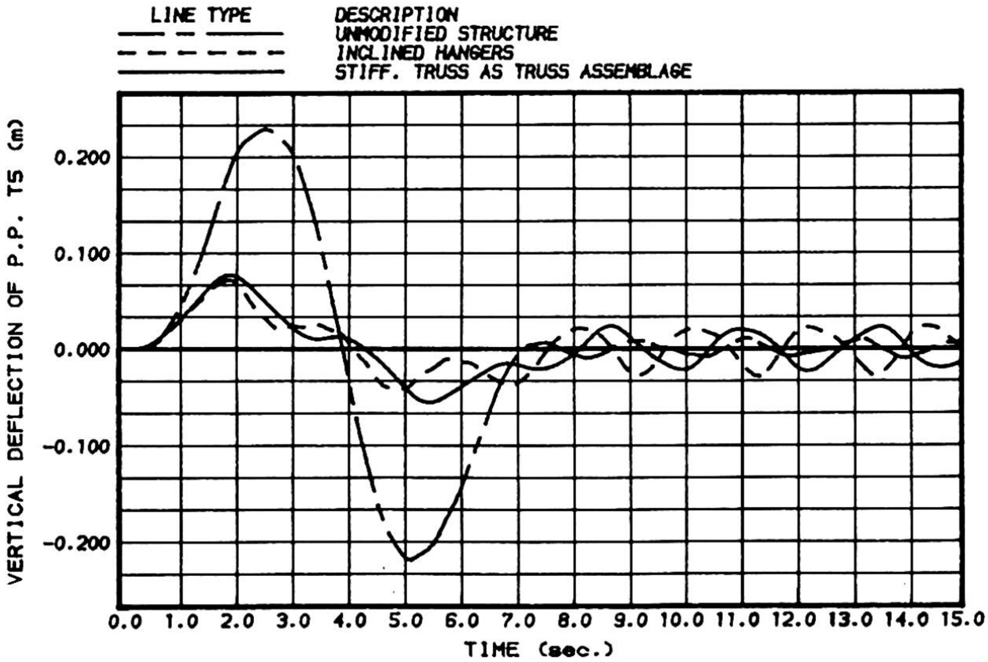


Figure 6 - Quarter point displacements for unmodified and retrofitted structure (223 m span), two trucks at 100 km/h.

3.2 Damping Retrofits

Hydraulic and friction dampers were also considered to reduce the global response of the structure. Since the damping force in hydraulic dampers is proportional to the square of the velocity of the mass, they are primarily used to remove high frequency, low amplitude vibration or flutter and, consequently, are not applicable to this situation. In contrast, friction dampers work independent of velocity and can be used in low frequency applications. However, a threshold displacement is necessary to actuate a friction damper. Although the vertical displacement of the structure is large, the relative displacement between adjacent floorbeams where dampers could be placed did not exceed this threshold.

Tuned mass dampers were also investigated through dynamic analysis. Various numbers and masses of dampers were considered and found to be ineffective. The reason for this observation probably lies in the nature of the excitation, which is basically impulsive, rather than resonant.

4. RECOMMENDATIONS

4.1 Global Retrofit

At present, the owner is considering the recommendation to convert the existing tie girder into the bottom chord of a stiffening truss as an attempt to solve the dynamic response problems of these tied-arch spans. The stiffening truss, while reducing the stress ranges at the details which are cracking, was not considered sufficient to prevent continued fatigue crack development.

4.2 Local Retrofits

The underlying causes of the cracking at the locations shown in Figure 2 were also isolated by three-dimensional finite element substructure analyses. Cracking at the connection of the floorbeam to the tie was found to be caused by the difference in the period of the extension of the tie girder and the out-of-plane rotation of the floorbeam. The period of the extension of the tie girder is equal to the time required for a vehicle to cross the span. The period of the floorbeam out-of-plane movement is approximately one-third of the period of tie girder extension for a vehicle traveling at 100 km/hr. This difference in response results in relative distortions of web gaps in the connection and stiffening details. At the centerline of the floorbeam, the lateral (wind) bracing is connected to the bottom flange of the floorbeam and locally interferes with the out-of-plane movement of the bottom flange resulting from floorbeam rotation caused by the vertical movement of the bridge. The addition of bolted connecting angles to these fatigue-prone details have been suggested to further reduce the small relative displacements occurring at locations of large stiffness changes.

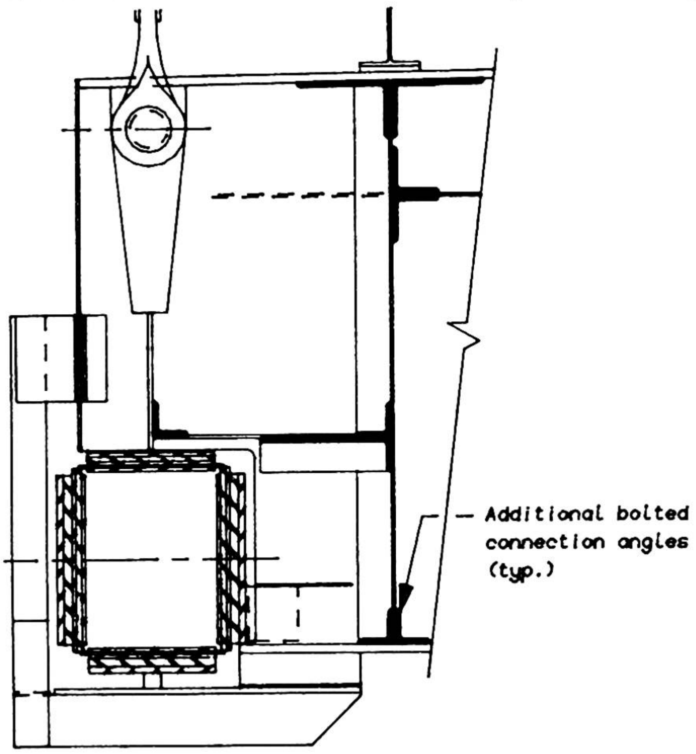


Figure 7 - Local retrofits at end of floorbeam.