Annexure

S01: Frame Models of Complete Bridge Structures for Tier 1 Assessments

August 2013

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Amendment Register

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1 Introduction

1.1 Background

Traditional practice for quantifying bridge behaviour is to model the bridge superstructure and substructures separately. The superstructure is modelled with supported nodes at the pier locations. The reactions from the superstructure model are then transferred to separate models of the substructures for analysis.

This is the same as having a rigid substructure in the superstructure model and means there will be essentially no sharing of load between the headstock and the adjacent transverse members in the superstructure. The result is that the loading applied to the headstocks tends to be conservative, which is generally a desirable outcome.

On the other hand the determination of the worst vehicle positions to maximise the loading on individual elements of the substructure is more difficult with separate models, and also there is the risk of error in manually transferring reactions from the superstructure model to the substructure model.

With advances in modelling software and computing power, a viable alternative to the separate substructure and superstructure models is to develop a single model of the complete bridge structure and apply all loads to this model. However, experience has shown that unless great care is taken in the development of this single model it is possible that some behaviours of the model are not compatible with realistic bridge behaviour and hence the results from such models are also not realistic, and in some cases non conservative.

This annexure focuses on complete bridge models and does not specifically address “separate superstructure and substructure models”, although some of the issues raised are equally applicable to separate as well as complete models.

1.2 Purpose

The purpose of this annexure is to:

- provide recommendations for developing frame models of complete bridge structures suitable for a Tier 1 assessment, which will result in suitably conservative and reliable results
- identify and discuss some issues with complete bridge models, which can cause erroneous results.

It is not the purpose of this document to express a general preference for complete bridge models, over separate superstructure and substructure models, for the analysis of bridge structures.

1.3 Scope

In scope:

- Tier 1 Modelling of existing bridges for assessment purposes.
- Deck unit and simply supported girder bridges only.
- Self-weight and vertical traffic loading only.

Out of scope:

- Models for the design of new bridges.
- Models for the assessment of the lateral load effects on bridges.

- More advanced (Tier 2) modelling of existing bridges for assessment purposes.
- Guidelines for modelling the actual superstructure. This includes the stiffness, orientation, and spacing of transverse members.
- The methodology for the application of moving loads to the superstructure.
- Guidelines and parameters for modelling soil structure interaction.

It should be noted that despite the above limitations of scope, the issues raised will in most cases, be relevant to all analysis purposes and all types of structures and loading.

The annexure has been prepared assuming SpaceGass frame models, although the discussion and recommendations are applicable to other frame structural analysis packages.

1.4 Related documents

This guideline annexure is to be read in conjunction with *Annexure S02: Modelling Deck Unit Bridge Superstructures for Tier 1 Assessments*.

1.5 Responsibilities of users

This document is to be applied by structural engineers who use their engineering knowledge and experience to model structures. Engineering organisations and engineers applying this document are to convey any concerns and/or suggested improvements to the Deputy Chief Engineer (Structures Section), in writing, in a timely manner.

There will be bridges where, due to the particular geometry or conditions at hand, these recommendations are not appropriate, although the issues raised in *Appendix A* will still be relevant to the final model developed by the engineer.

Although this document provides some modelling guidelines and recommendations, its primary purpose is to raise the issues and identify solutions. Ultimately, it is the responsibility of the engineer to apply reasonable engineering judgement in the development of the models to ensure that the results are appropriate for a Tier 1 assessment of the structure.

1.6 Separate versus complete bridge models

Both “separate superstructure and substructure models” and “complete bridge models” may be used in the Tier 1 assessment of existing departmental bridges.

1.7 Layout of annexure

This annexure is presented in two sections and an appendix:

- Section 2 – This section provides a list of definitions
- Section 3 –This section provides recommendations for Tier 1 assessments
- Appendix A – This appendix discusses potential modelling issues including:
  - issues relating to the modelling of the various bridge elements in their correct relative elevation
  - substructure modelling
  - modelling of construction staging and creep
  - orientation of bearings on skewed deck unit bridges

- interpretation of frame model analysis results.

2 Definitions

Frame models:	Models consisting of beam elements.
Complete bridge model:	In the context of this document a complete bridge model means that the frame model includes the superstructure and the entire substructure. The boundary conditions at the remote ends of supporting elements such as columns and piles may be simplified.
Separate superstructure and substructure models:	In the context of this document, this means that the bridge is represented by two frame models. The superstructure is modelled separately with supported nodes at the pier locations. The reactions from the super structure model are then transferred to separate models of the substructures for analysis.
Longitudinal direction:	The direction on a bridge generally parallel with the road centre line.
Transverse direction:	The direction on a bridge generally perpendicular to the road centre line.
Complete bridge model:	In the context of this document a complete bridge model means that the model includes sufficient of the superstructure and the substructure to model all actions. The boundary conditions at the remote ends of supporting elements such as columns and piles may be simplified.

3 Recommendations for the department's projects

3.1 Introduction

This section sets out the recommendations for the complete bridge modelling of deck unit and simply supported girder bridges for the purpose of Tier 1 assessments of bridges for self-weight and vertical traffic loading.

3.2 Aim

To produce models which capture the important behaviours of the complete bridge structure subjected to vertical traffic loading and which allows a straightforward extraction of information which will be useful for the assessment of the bridge.

The models should be relatively simple and straightforward to develop, and have a low risk of predicting unrealistic or unreliable behaviours.

3.3 Superstructures

3.3.1 Vertical location

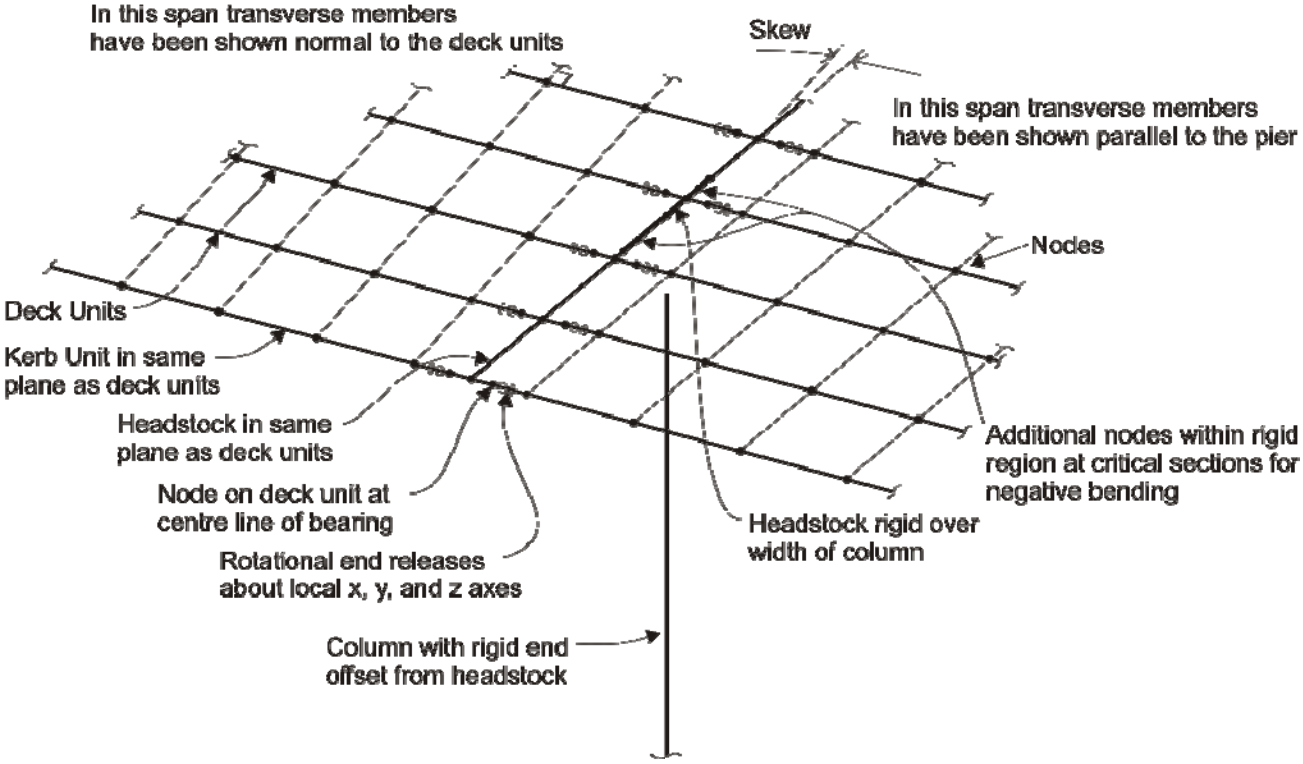
Many of the issues discussed in *Appendix A* relate to problems that arise from modelling the superstructure and substructure in their correct relative elevations. To avoid these issues it is recommended that the frame members representing the girders or deck units are all modelled in the same plane as the headstocks.

If the model will not be used for any lateral loading effects, then the correct substructure geometry can be maintained by lowering the deck to the plane of the headstock. (Models that include lateral load effects are beyond the scope of this document)

3.3.2 Details at piers

Figure 3.1 shows recommended typical details at a pier for a deck unit bridge. Models based on these details are simple to construct and will give reliable results.

Figure 3.1: Modelling details at a pier for a typical skew deck unit bridge



The following points should be noted:

- The kerb units, deck unit, and headstock are all in the same plane.
- The frame members at the end of each deck unit are attached to the node on the headstock. A node is inserted at the location of the centre of bearing of the deck unit. End releases are applied to the deck unit member adjacent to this node.
- The end releases at the end of the deck unit member must include a torsional release, particularly if the bridge is skew. To allow accurate correction of the headstock shear force diagrams (refer Section 3.6) it is recommended that the ends of the deck unit members be torsionally released in all cases.
- Additional end releases for longitudinal movement will be required if a sliding joint exists.
- The stub members attached to the headstock can usually retain the section properties of the longitudinal members. The exception is if these members are longer than usual (for instance, if the longitudinal members are supported on a corbel on the side of the headstock). In this case their stiffness should be increased appropriately.
- The flexural stiffness of the headstock must not be underestimated. The uncracked stiffness should be used and the concrete stiffness should be at the upper end of the likely range. Regions of the headstock which are over columns or walls should be modelled as essentially rigid for bending in a vertical plane.
- The stiffness of the transverse deck members near the piers should not be over estimated so as to ensure the headstock load effects are not underestimated. For example, in a girder

bridge the diaphragm could be modelled as a cracked section or omitted during a Tier 1 Assessment. (Different approaches to the modelling may be appropriate for a Tier 2 Assessment)

- The capacity of the transverse members in the superstructure should be checked. If the capacity is insufficient to carry the predicted loads then their stiffness should be reduced and the structure re-analysed.

The details at a girder bridge headstock would be similar to those described above for a deck unit bridge.

3.4 Substructures

All elements of the substructure which form part of the vertical load carrying path should be included in the model. This includes headstocks, walls, columns, pile caps, piles and footings.

The following recommendations apply:

- Blade Piers and walls should be represented by a series of columns supporting the headstock. The region of the headstock over the pier or wall should be modelled as essentially rigid for vertical bending. Similarly if the wall or pier is supported on a pile cap, then the region of the pile cap under the wall should be treated as essentially rigid.
- Where walls or columns are supported on spread footings then they can generally be modelled with simple boundary conditions at the end of the column unless engineering judgement for the particular geometry indicates otherwise.
- Lateral pressure, or lateral support on abutment walls need not be modelled unless it is necessary for the overall stability of the structure. However lateral pressure effects should be considered when assessing the capacity of abutment walls.
- Rigid end offsets should be used at the tops of piles and the tops and bottoms of columns. The offset length should be selected so that the ends of the pile/column are at the correct location of the critical section for bending assessment in these members.
- In most cases the full length of piles need not be modelled. The piles may be modelled as fully fixed at some depth into the ground. A fixity depth of 4 times the pile diameter is considered reasonable unless the material is loose or unsupported as in a spill through abutment. The exception is where pile lengths vary significantly within a pier and the resulting variation in pile axial stiffness may adversely affect the structure. In this case some representation of the differing axial stiffnesses needs to be included in the model.
- Pile fixity depths should take into account the likelihood of scour, particularly for slender piles where the axial capacity is influenced by buckling.
- Where the geometry of the substructure is such that the piles may carry additional loads from embankments or similar then these loads need to be included in the assessment.

3.5 Modelling dead load in deck unit bridges

For deck unit bridges it should be assumed that the dead load is applied to the completed structure. A more detailed assessment of the dead load distribution may be investigated if necessary in a Tier 2 analysis.

For further information regarding the analysis of deck unit bridges refer *Annexure S02: Modelling Deck Unit Bridges Superstructures for Tier 1 Assessments*.

3.6 Interpretation of frame model results

This applies to headstocks on deck unit bridges and may apply in other cases where a member is supporting a continuous deck or similar which is represented by a series of beam members.

Where significant, the shear force, torsion, and bending moment diagrams for the supporting member, as output by the frame model, should be adjusted to take account of the actual width of the load transfer between members in the model.

Usually this is only significant for shear and torsion diagrams. In these diagrams the step change, which results from the load transfer all occurring at a node in the frame model, should be replaced by a linear change over the actual width of the connection between the members.

This correction is only accurate if there is no torsion being transferred from the supported member. This will normally be the case for deck units if the recommendations in Section 3.3.2 above are followed.

Shear force and torsion envelopes may be modified using the same procedure. That is, by treating them as if they were diagrams for an individual load case.

3.7 Validation of frame models

Each model and its loading system require validation. Useful checks include:

- ensuring deflected shapes are appropriate (magnitude and shape)
- ensuring reactions are correct and that the restraints are appropriate
- ensuring the bending moment, shear force, torsion and axial forces are appropriate
- ensuring that there is no unintended composite action between members (e.g. a headstock acting compositely with the superstructure is generally inappropriate)
- ensuring the loads, including wheel loads, have been applied appropriately.

Appendix A Background information

While models of the complete bridge structure have the potential to provide more accurate analysis results for the assessment of the bridge structures, these models are generally more complicated and hence are more prone to error. It is important that the interactions of the various components of the bridge are represented accurately or at least in a manner that will prevent load being carried by unreliable or non-existent mechanisms.

Some issues, which have been identified and can result in erroneous results, are discussed in this section.

A.1 Issues relating to the modelling of the various bridge elements in their correct relative elevation

It would be usual to produce a complete model of the bridge so that each beam element is located at the centroid of the bridge member that it represents. This means that the superstructure elements will be displaced vertically from the headstock, and even all of the superstructure elements may not be in the same plane. The nature of the connections between the elements in the different planes will have a large effect on how those elements interact, and the resulting behaviour of the overall system.

A.1.1 Composite action between superstructure and headstock for transverse bending

If the connections between the superstructure and headstocks are stiff, then the transverse elements in the superstructure (i.e. transverse members in a deck unit bridge or diaphragms in a girder bridge) will effectively act compositely with the headstock. This can make the headstock / superstructure system artificially stiff and will result in erroneous load effects in both the headstock and the transverse elements of the superstructure.

Indicators that this behaviour is occurring are large axial forces in the headstock and in the transverse members of the superstructure near the pier. Another indicator is large shear forces in the transverse direction in the members connecting the superstructure to the headstock.

As an example refer to Figure A.1 below. This shows the SpaceGass frame model of a pier on a deck unit bridge. In this model the superstructure and the pier headstock have been modelled in their correct relative locations. The deck units are connected to the headstock by a series of rigid links with end releases to allow rotation and sliding (this is an expansion pier). However there are no end releases allowing any movement in the transverse direction.

An inspection of the deformed shape of that model under dead load, as shown in Figure A.2, shows the expected response and gives no indication of any modelling issues. However a closer inspection of the member forces in the vicinity of the headstock show that composite action between the superstructure and headstock is occurring. Figure A.3 shows the axial forces in the headstock and transverse superstructure members. It can be seen that significant axial tensions have developed in the headstock with balancing compressive forces in the transverse members near the headstock.

Figure A.1: SpaceGass model at a pier on a deck unit bridge with superstructure vertically separated from the headstock

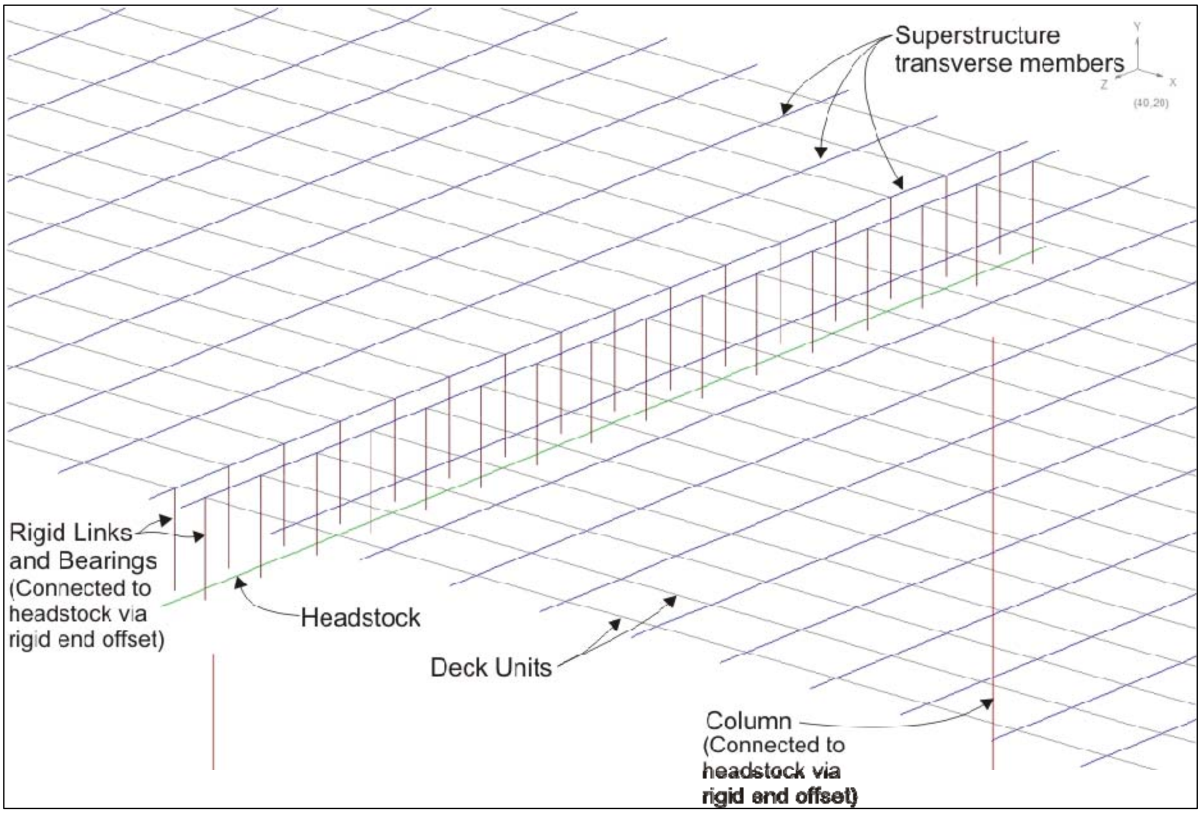


Figure A.2: Deflected shape under dead load

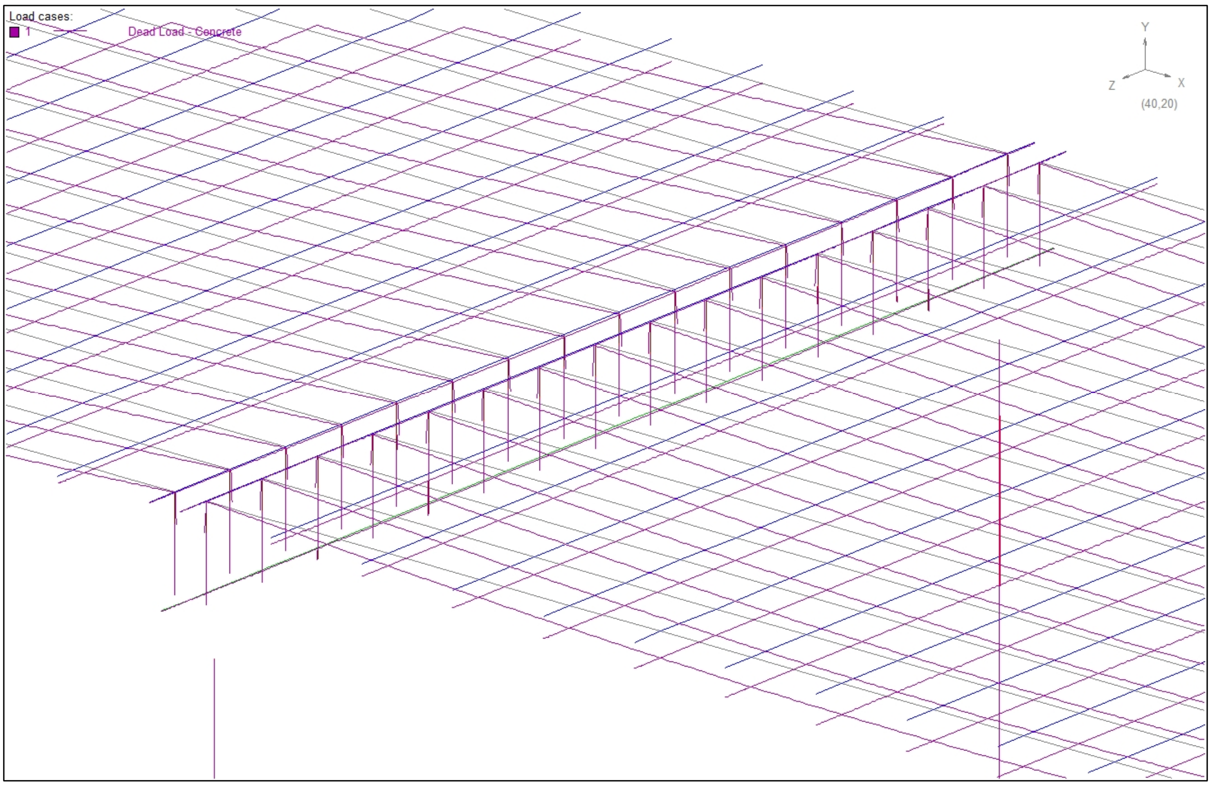


Figure A.3: Axial forces in the headstock and adjacent transverse members

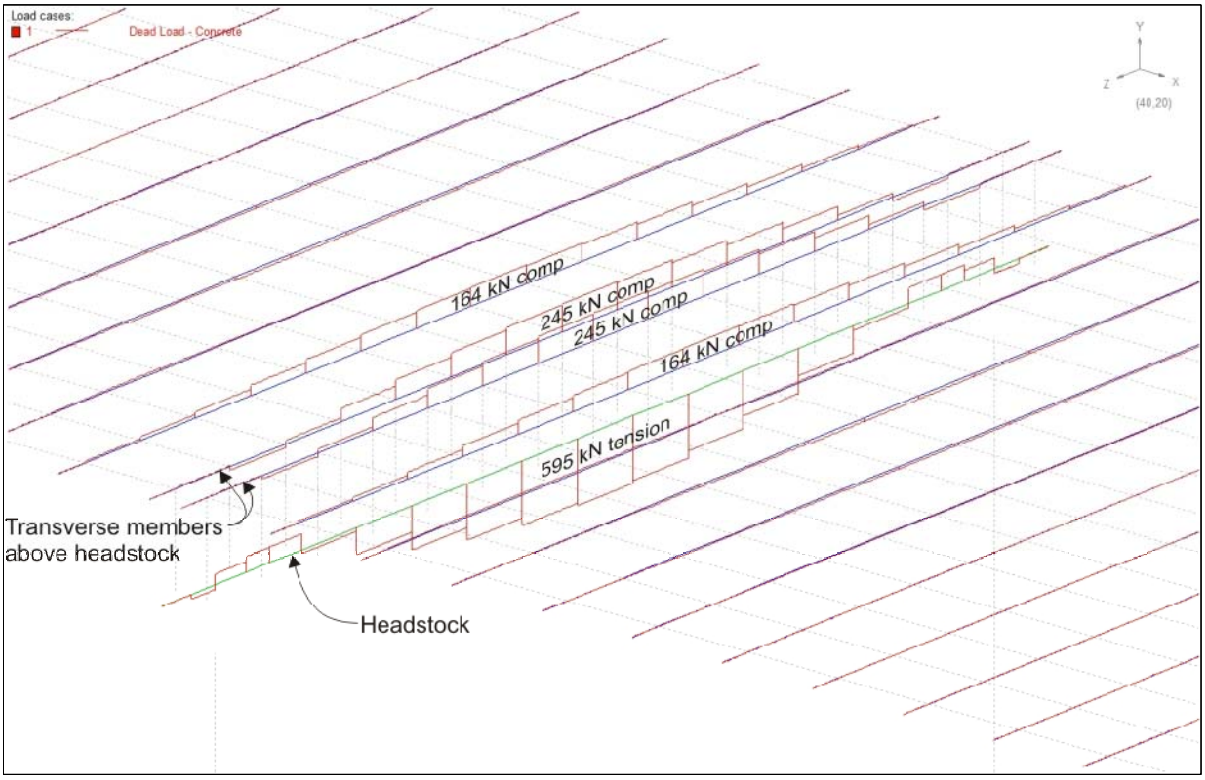


Figure A.4: Transverse shear forces in links between headstock and superstructure

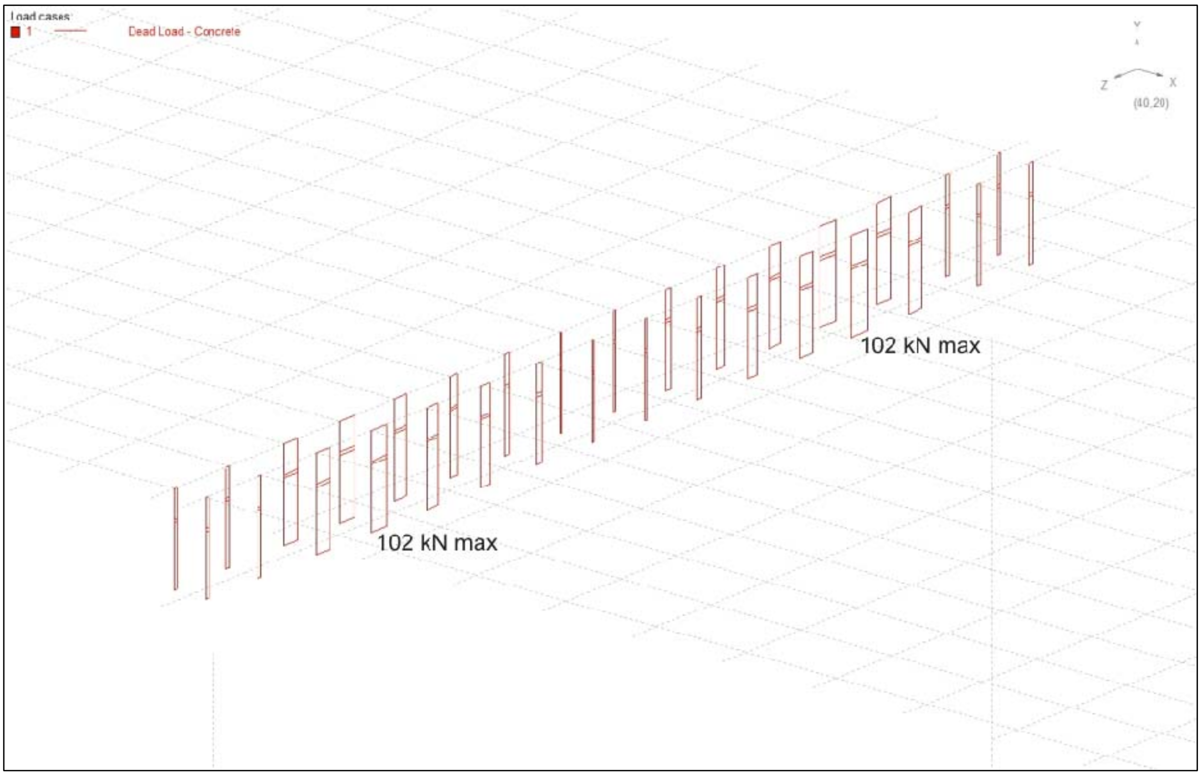
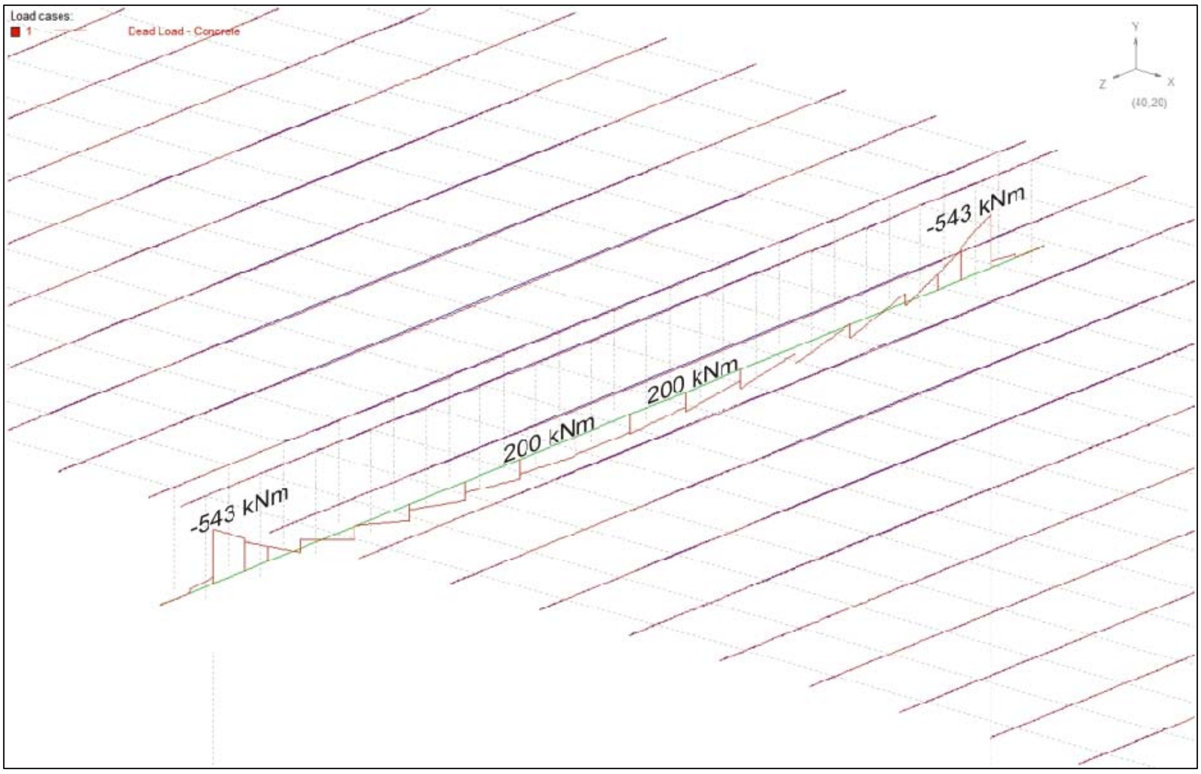


Figure A.4 shows the transverse shear forces in the links between the headstock and the superstructure. This is for unfactored dead load only and under factored dead load plus live load would be much higher. Even if these were mortar pad bearings, their ability to resist these loads with no movement is improbable and certainly not reliable.

Figure A.5 shows the bending moments in the headstock. These moments are low. There are also relatively large steps in the moment diagram due to the shears and loads being transmitted through the links.

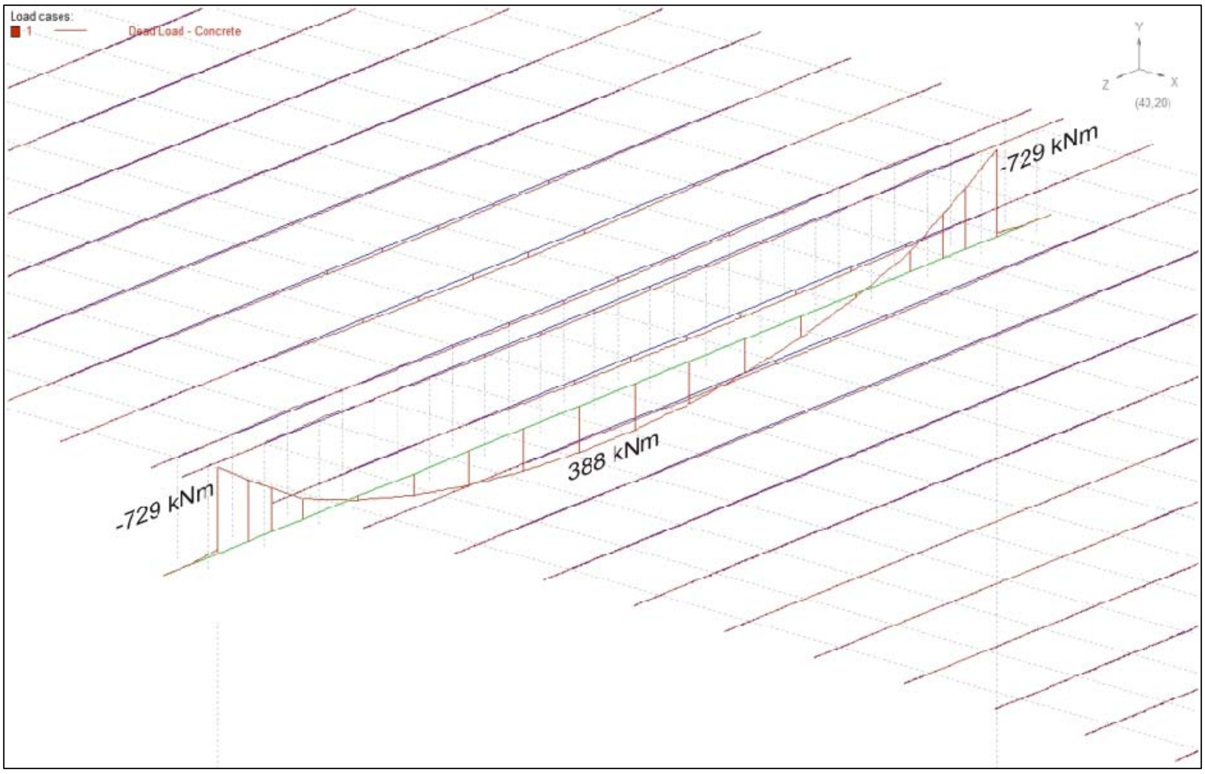
Figure A.5: Bending moments in headstock



The model was corrected by introducing releases into the links to allow rotation about the longitudinal direction, and displacement in the transverse direction. To avoid model instability one of the links was left without a translation release. The bending moments in the headstock for the corrected model are shown in Figure A.6. It can be seen that the negative moments at the columns are 34% higher and the positive moments at midspan are 94% higher than for the incorrect model where the substructure and superstructure acted compositely.

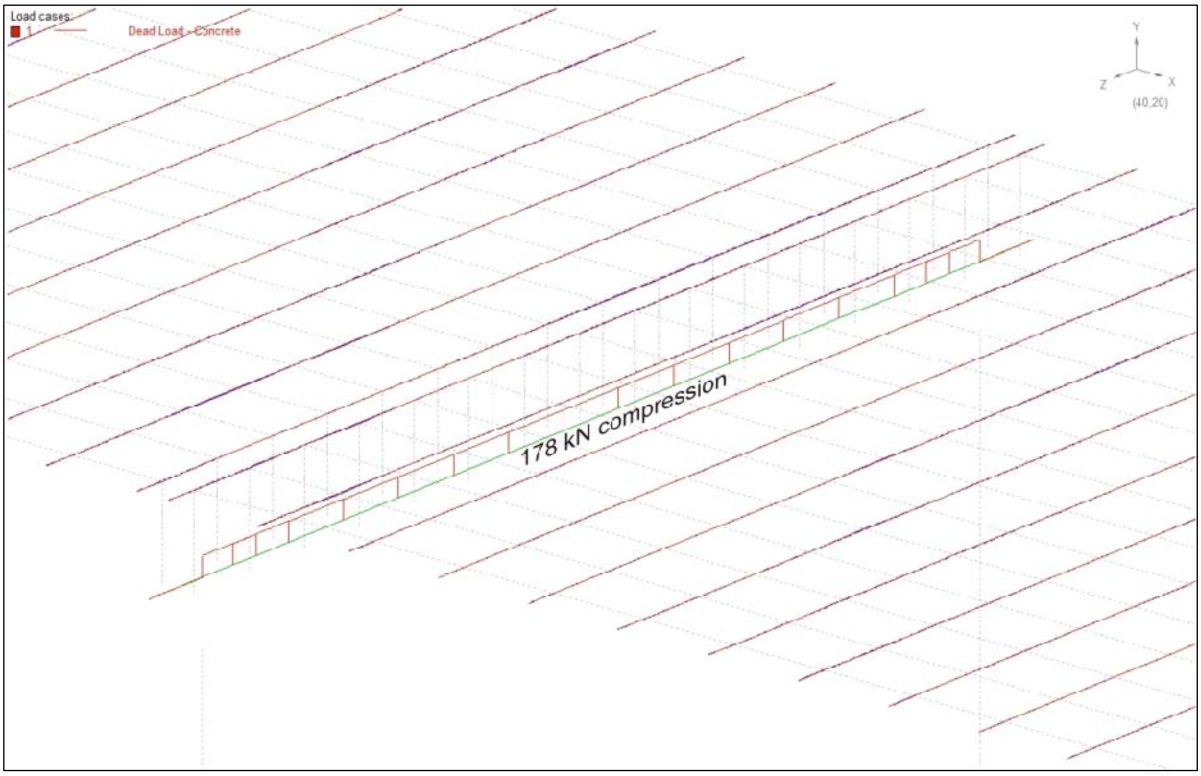
It should be noted there are still small bending moments in the transverse members of the superstructure which indicates a sharing of the transverse load carrying between the superstructure and the substructure. This is because the superstructure is being forced to conform to the same deflected shape as the headstock. The discussion of this behaviour is covered in A.2.1.

Figure A.6: Bending moments in headstock for corrected model



The axial forces in the headstock and adjacent transverse members for the corrected model are shown in Figure A.7. It can be seen that although the axial forces in the superstructure transverse members is negligible there is still a significant axial compression of 178 kN in the headstock. This force is a result of the portal action in the headstock and columns and is not related to the composite action between the headstock and superstructure. Its presence explains the imbalance between the tension and compression axial forces due to composite action in Figure A.3.

Figure A.7: Axial force in headstock in corrected model



It can be seen from this example that if the bridge is to be modelled with the superstructure and the headstocks in their correct relative elevation, then it is important to ensure the properties of the links joining the two are such that composite action cannot be developed. Results should be checked to ensure this has been achieved.

An alternative approach would be to model the headstocks and superstructure in the same plane. This could lead to inaccuracies where lateral loads are involved, however for Tier 1 modelling of simple bridges subject to traffic loading this would not be an issue. It would generally be best to keep the substructure geometry accurate, and lower the deck so the deck and headstock centroids were in the same horizontal plane. A modelling strategy based on this approach is discussed in Section A.5.

A.1.2 Restrained longitudinal movement at bearings due to flexure

Rotations at the ends of longitudinal members due to flexure result in longitudinal movements at the bearing level. This is due to the vertical offset between the centroid of the girder/beam member and the bearing. Hence if a girder or deck unit is pinned at both ends, then flexure in the beam will result in an overall lengthening of the span at the bearing level. This behaviour is real and if this movement is restrained then very large forces can develop.

In girder bridges there is generally an expansion joint, however it is not uncommon to have a deck unit bridge of several spans with no expansion joint. Further, in these bridges the gap between the ends of the deck units is often grouted. In such bridges the actual restraint forces that develop depend on the actual stiffness of the abutments against longitudinal movement, and also on the actual stiffness of the bearings.

If in the complete bridge frame model, the abutment representation is excessively rigid, then the results will show very high restraint forces. This can happen, for instance, if an abutment headstock supported on piles is modelled with restrained nodes at the pile locations rather than modelling at least part of the piles.

Figure A.8 shows a two span deck unit structure with no expansion joints. In this model, the abutment headstock has restrained nodes at the pile locations. As a result of this inflexible abutment, very high restraint forces result from bending in the deck units. In this example a live load roughly equivalent to a factored standard tri-axle was applied at mid-span.

Figure A.8: Model of bridge with bearings offset from deck unit members and artificially restrained abutments

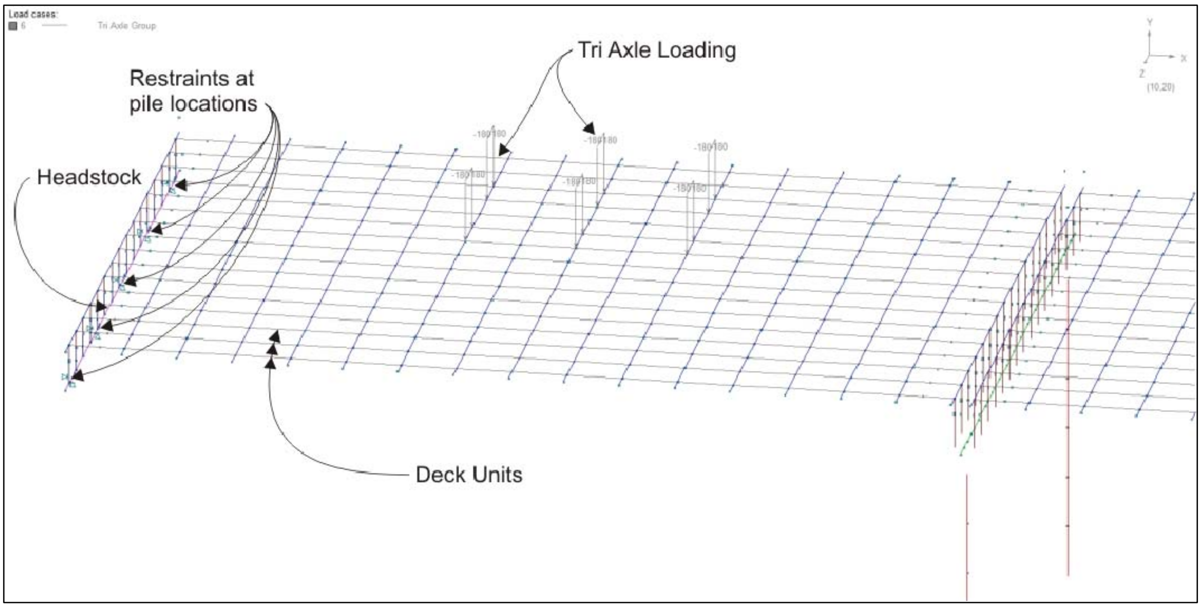


Figure A.9 shows the horizontal shears in the abutment as a result of the restrained flexure in the deck units. It can be seen that very high local effects result. High torsions also exist due to the eccentricity of the shear relative to the headstock centroid. Figure A.10 shows the bending moments in the deck units. It can be seen that the restraint has resulted in a significant negative moment existing at the ends of the units and consequently there is a reduction in the mid-span positive moment. These moments are unrealistic because the abutments and the bearings will not actually be as rigid in the longitudinal direction as they are in the model.

This problem can be rectified by introducing a small amount of flexibility in the abutments. In this example, the model was modified to include the piles to a depth of about four pile diameters below the abutment headstock. Figure A.11 shows the horizontal shears from this revised model and it can be seen that they are significantly reduced. Similarly the deck unit moments (Figure A.12) are as they should be with negligible negative moment at the ends and an increased mid-span positive moment. It can be seen that only a small amount of flexibility is required to relieve the restraint. This can be expected to be present even if the deck unit bridge is propping the top of an abutment wall.

Figure A.9: Horizontal shear in headstock resulting from restrained flexure in deck units

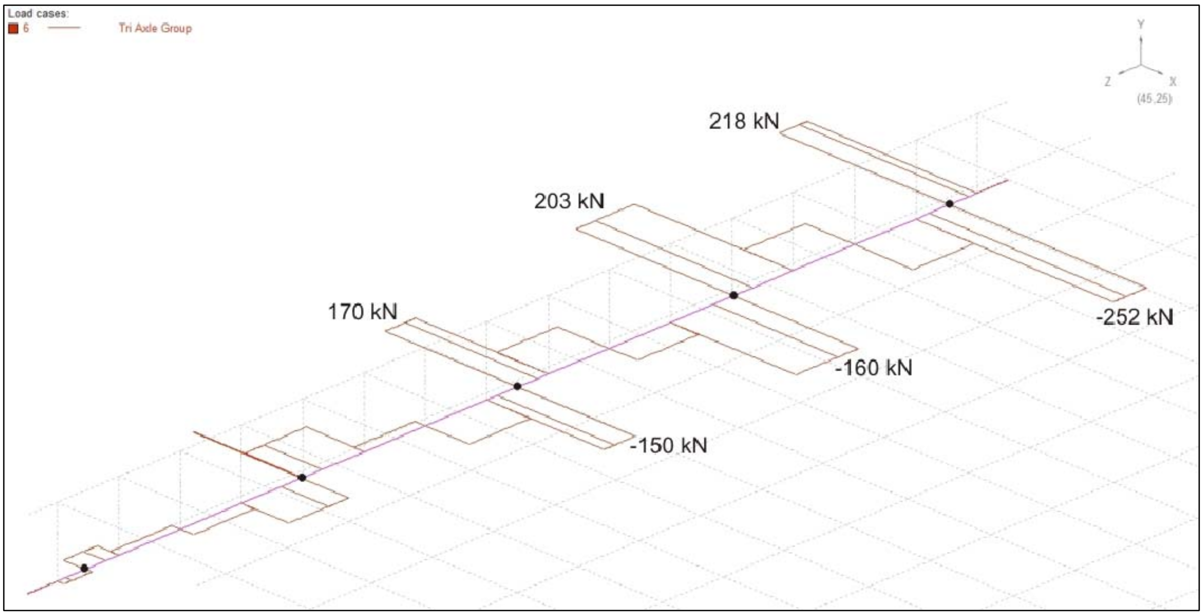
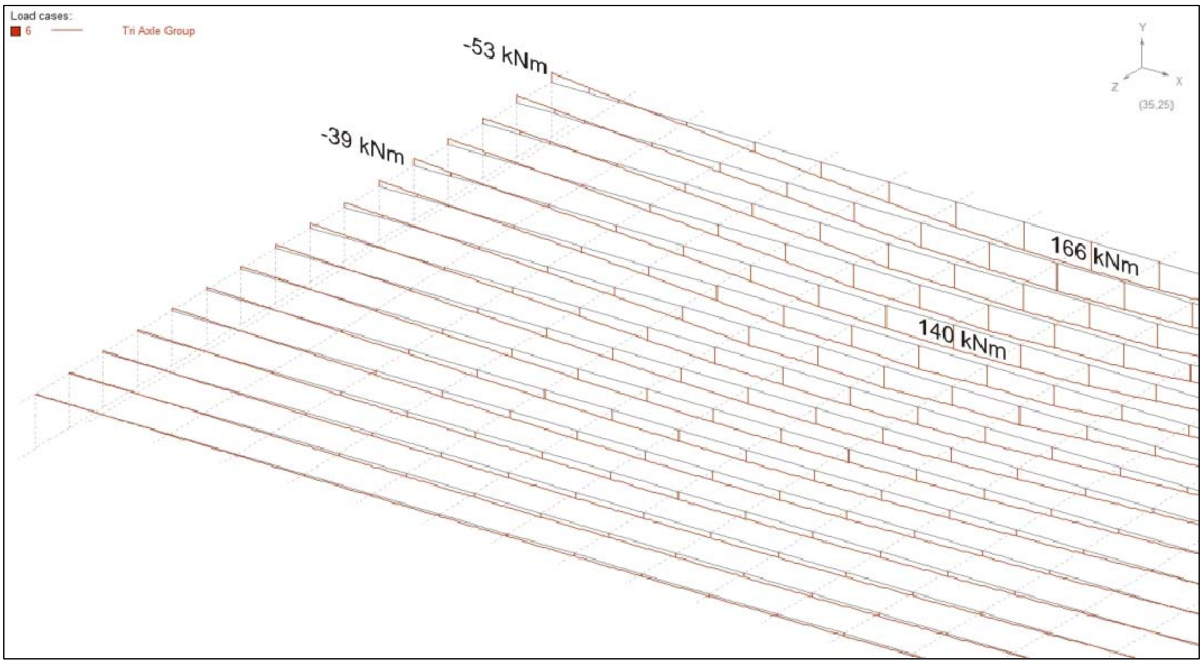


Figure A.10: Bending moments in deck units showing negative end moments due to longitudinal restraint



Another approach to avoid these types of problems is to introduce a small amount of flexibility into the longitudinal restraint at nominally pinned bearings.

Alternatively, these types of issues can be eliminated from the analysis by ignoring the offset between the longitudinal member centroid and the bearing in the model. This means simply applying the pinned or sliding pin end release directly to the end of the frame members representing the deck unit. If this is done then there is no lengthening of the distance between bearings due to flexure and the problem does not arise.

Figure A.11: Horizontal shear in headstock with headstock supported on short piles

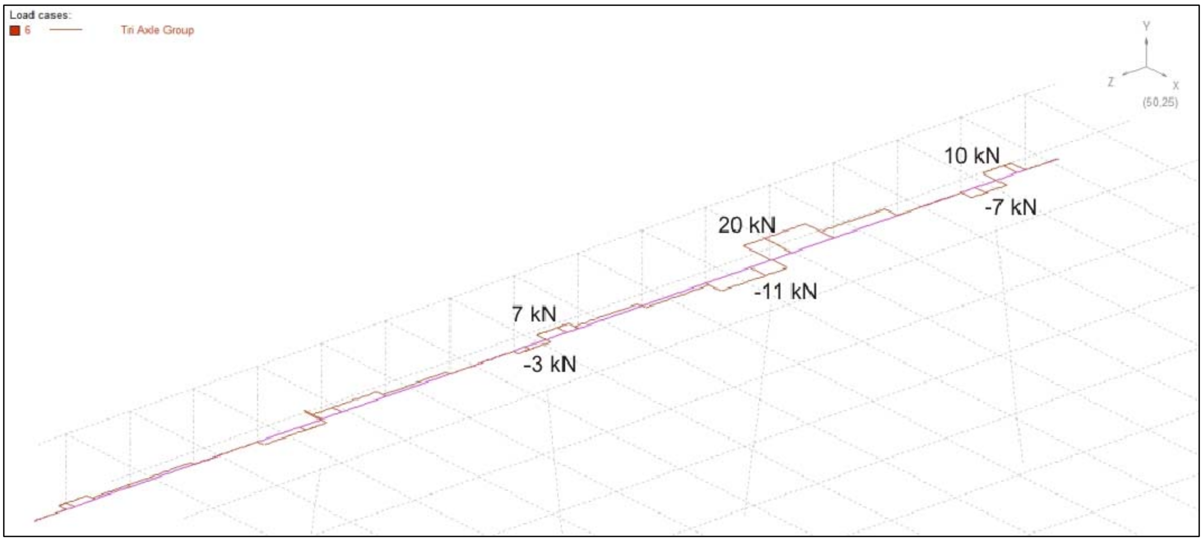
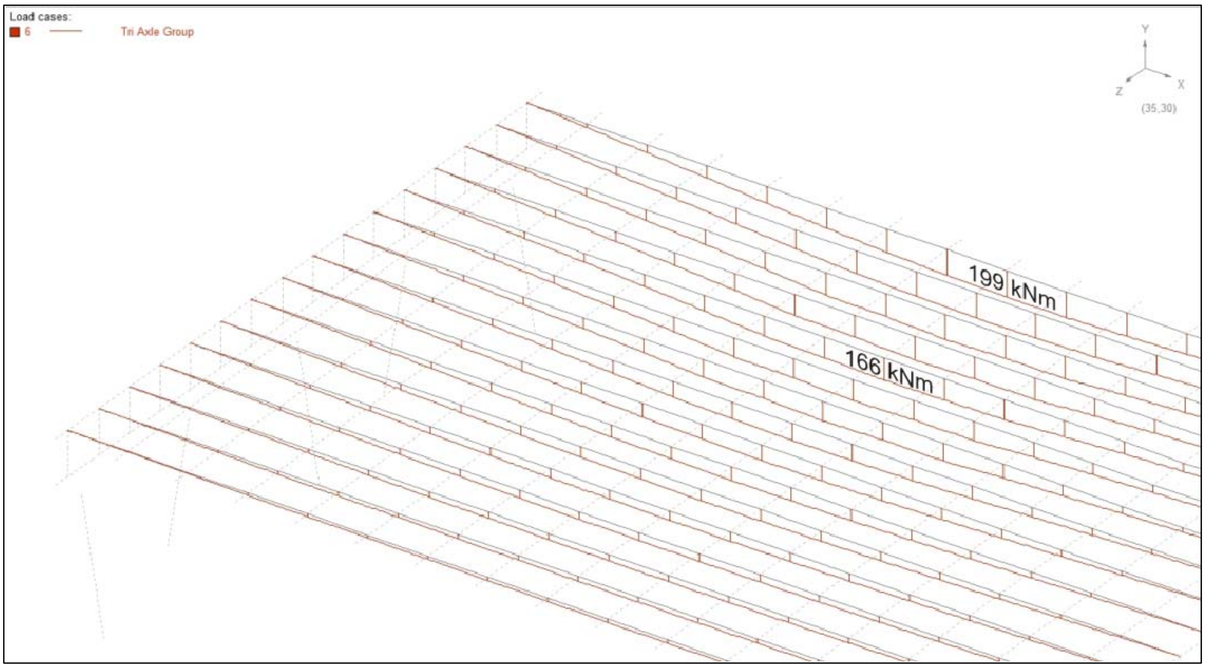


Figure A.12: Deck unit moments in model with headstock supported on short piles



Note that this discussion only relates to the issue of horizontal forces generated at the bearings due to flexure in the deck units. The effects of restrained thermal expansion or the transfer of propping forces through the abutment must still be dealt with in normal bridge design. (Although they may not be relevant to Tier 1 assessments for traffic loading).

As noted above, this behaviour is real, but normally either one end of the girder is free to slide, or there is enough flexibility in the system that the forces are small. If the movements are rigidly restrained then the forces are large, and damage is likely. If such damage is evident then this aspect of the bridge will need closer investigation as part of a Tier 2 assessment.

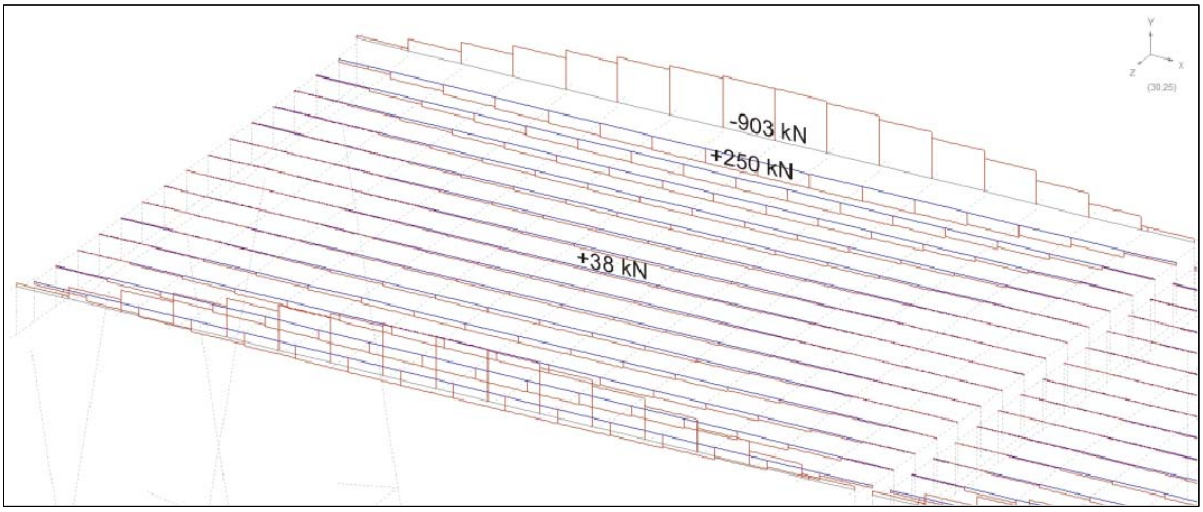
In some cases local horizontal shears can arise in the headstock even if the span is only pinned at one end and free to slide at the other. This occurs when the girders or deck units within the same span are of different depths (e.g. a Kerb unit in a deck unit bridge). The different heights from the centroid to the bearings result in different amounts of longitudinal movement at the bearing which induces local horizontal shears and torsions in the headstock and local bending restraint to the ends of the girders. This can occur even though the other ends of the girders are free to slide.

A.1.3 Girders or deck units of different depths

Apart from the issues described above with artificial restraint at the ends of girders of different depths there is also a general issue when the centroids of the main longitudinal members are not in the same plane. Again, this commonly occurs in deck unit bridges when the kerb unit is deeper than the deck unit.

If the analysis model is created with the kerb unit centroid higher than the adjacent deck units, and the transverse members have sufficient stiffness and are closely enough spaced to ensure the transfer of longitudinal shear between the deck and kerb units, then composite action will result and axial forces will be generated in the deck and kerb units. Figure A.13 shows typical axial forces in a deck unit bridge if modelled with the unit centroids in their correct relative elevation.

Figure A.13: Typical axial forces in a deck unit span with composite action



As can be seen this results in a significant compressive force in the kerb unit, and tensile forces in the deck units. These forces are large in the deck unit adjacent to the kerb unit and then reduce towards the middle of the bridge. As a result of this composite action, the bending moments in the deck units are lower.

Table A-1 below summarises the factored load effects at midspan for a 13.8m span deck unit bridge with the deck unit members modelled in their correct relative elevation, both with and without composite action between adjacent units. It can be seen that the axial forces developed are significant, although the effect on the maximum bending moment (at least in this case) is not very large (about a 6% reduction for the kerb unit).

Table A-1: Comparison of load effects in deck units with and without composite action

Location	Load Effect	Without Composite Action	With Composite Action
Kerb Unit	Fx (kN)	0	-1576
	Mz (kNm)	935	882
Deck Unit adjacent to KU	Fx (kN)	0	+473
	Mz (kNm)	435	412
Central Deck Unit	Fx (kN)	0	+55
	Mz (kNm)	415	400

There is no doubt that this behaviour is real to some extent, although it is questionable if the friction between the kerb unit and adjacent deck unit will be sufficient to fully develop the required axial forces at the Ultimate Limit State (ULS) loading.

For the above example, the maximum longitudinal shear between the kerb unit and the adjacent deck unit was 385 kN/m. Based on a transverse stressing force of 350 kN per bar (29 diameter bar at half capacity, bars at 2.05m) the prestressing force would be 171 kN/m. Thus a coefficient of friction in excess of 2 would be required to resist the ULS longitudinal shear. Clearly this cannot be reliably achieved. This behaviour cannot be relied upon at the ultimate limit state and hence should be eliminated from the modelling.

This can be done by constructing the frame model so that all of the longitudinal elements are in the same plane, or by modifying the properties of the transverse members so that longitudinal shear cannot be transferred between the adjacent longitudinal members. If the latter approach is taken then this can be achieved by reducing the bending stiffness (about an axis perpendicular to the deck) of the transverse members to close to zero, or by introducing an end release (in the direction of the longitudinal member) to one end of the transverse members.

A.2 Substructure modelling

A complete bridge model for the purpose of assessment for traffic loading needs to include all of the elements of the substructure that participate in the load path. This includes headstocks, columns, walls, pile caps or footings and piles.

In some cases the stiffness of the substructure can affect the load distribution within the superstructure, and in turn the distribution of the load on the substructure. One case is discussed in Section A.1.2 above. Another is the stiffness of the headstock and its effect on the sharing of transverse bending loads between the superstructure and the substructure. This is discussed below.

Other issues discussed in this section are the modelling of piles and modelling of walls.

It is beyond the scope of this document to discuss geotechnical issues.

A.2.1 Stiffness of headstocks

Even if composite action between the headstock and superstructure is eliminated as discussed in A.1.1 above, there will still usually be some sharing of transverse bending between the headstock and the superstructure in a complete bridge model. This is due to the fact that the headstock and the superstructure must conform to the same transverse deflected shape at the pier.

This behaviour is real; however careful consideration must be given in the modelling to ensure the representation is realistic, and the results are not unconservative for either the headstock or the transverse members in the bridge.

The distribution of bending moments between the headstock and the transverse superstructure members depends on their relative flexural stiffness. Hence to ensure that the load effects in the headstock are not underestimated, it is important to ensure the stiffness of the headstock is correctly modelled and not underestimated.

To correctly model the stiffness of the headstock the following should be considered:

- The headstock flexural stiffness should be made essentially rigid over the full width of the supporting columns (or piles). It should be noted that the critical section for negative bending in the headstock is not at the face of the column but some distance within the column. This distance is code defined and is usually 0.15 times the column width, except for wide columns. It is usually helpful from the results processing point of view, to place a node at this location.
- For headstocks with blade piers, the pier should be split into several columns and the previous point also followed. The spacing of the columns should be not more than about 1.2 times the depth of the headstock. For vertical loading only, no linking of the columns is required. However, for more general loading some linking of the columns or other method to get the correct transverse stiffness would be necessary.
- For tapered blade piers the above also applies, but the column spacing at the bottom will be different from that at the top and the section properties should also vary.
- If the headstock has a significantly varying cross section, then the conservative approach would be to use the maximum stiffness. Alternatively, consideration could be given to varying the stiffness in the model according to the average stiffness of a frame member.
- For concrete headstocks use the uncracked stiffness and use a concrete Young’s Modulus at the upper end of the likely range.

If the headstock stiffness is correctly modelled, then over-estimating the stiffness of the transverse members in the superstructure will give non-conservative results in the headstock and visa versa.

For the purposes of a Tier 1 assessment for traffic loading the preferred approach is to underestimate the transverse stiffness in the superstructure (at least for the transverse members near the piers). This will tend to give conservative results for both the headstock and for the superstructure girders or deck units.

In all cases it must be checked that the capacity of the transverse members in the superstructure is sufficient to carry the loads determined by the analysis. If this is not the case, then the stiffness of the transverse members should be further reduced until this is satisfied.

A.2.2 Abutment walls

Abutment walls can usually be modelled as a series of columns, each with a cross section appropriate to the portion of the wall it is representing. As noted in A.2.1 the headstock over the width of the wall should be essentially rigid for bending about the horizontal axis. In some cases a more complex model may be required to properly represent the load carry mechanisms for lateral load.

The lateral soil pressure on the abutment walls need not be included in the assessment model. However they must be calculated separately and included when assessing the capacity of the abutment wall. Usually, the increase in axial load in the wall from heavier vehicles will be beneficial to the wall's bending capacity.

In some cases the soil reaction on an abutment headstock or wall may be necessary to maintain equilibrium under self-weight and vertical loads (for example on a bridge on a steep grade). In this case it may be necessary to include soil springs or similar to the substructure model.

A.2.3 Spread footings

Again, for the case of assessment of bridges due to traffic loading, and with generally inadequate geotechnical information, the inclusion of the flexibility of the spread footing and the sub-grade spring stiffness in the model is not warranted. It would normally be adequate to apply appropriate simple boundary conditions to the bottom of the column. However, there might be some situations where engineering judgement indicates this is not appropriate and more detailed modelling is required.

The capacity of the footing to distribute the reactions must still be assessed.

A.2.4 Piles

When assessing old bridges there is usually insufficient geotechnical information to allow detailed modelling of soil spring stiffnesses. Further, for traffic load assessments the bending moments in the piles are usually low and the effect of interest is the pile axial load. Considering the above, it would normally be reasonable to model the piles as extending to some depth into reasonable ground and then being fully fixed at this depth.

A reasonable depth to fixity would be 4 pile diameters into reasonable ground, although this could be varied in accordance with engineering judgement to account for some soil conditions. When considering the depth to fixity, the possibility of scour should be considered. This is particularly the case if buckling will influence the pile's axial capacity.

In a Tier 1 assessment, the buckling capacity of the pile would be determined in accordance with the code based on a first order analysis. Thus the requirement for the frame model is that the pile length is appropriate so the analysis gives reasonably accurate results for the pile axial loads and end moments. If a pile is so slender that a second order analysis is required, then this would be outside the scope of this document.

If the piles on a particular pier or abutment vary significantly in length then the resulting difference in vertical stiffness may adversely affect the load distribution in the structure. In this case the difference in axial stiffness should be included in the model. This can be achieved by introducing a spring in the pile axial direction at the pile tip.

Piles should be connected to nodes on the pile cap or headstock with horizontal and vertical rigid end offsets to correctly locate the top of the pile. The vertical length of the offset should be chosen so that the end of the pile is located at the critical section for bending on the pile. This will generally be 0.15 times the headstock/pile cap depth above the underside of the headstock/pile cap.

If, due to the particular geometry of the substructure, a pile may carry significant loads that are not in the analysis model, such as embankment loads or down drag loads due to embankment settlement, then these loads should be calculated separately and their effects included in the pile assessment.

A.3 Modelling of construction staging and creep

The distribution of load effects from dead load in both the superstructure and substructure can vary depending on the assumptions made in modelling the construction staging. This is particularly the case in deck unit bridges where the kerb units are much stiffer than the deck units.

When a deck unit span is first erected each unit carries its own dead load. After grouting, creep will occur resulting in a redistribution of the dead load. Since the kerb unit is stiffer and deflects less, it will pick up some of the load of the adjacent deck units as creep progresses.

This process is further complicated because the units are usually prestressed and have a hog that tends to increase as creep progresses. If there was no hog, or even if the hog of the deck and kerb units was the same, then there would be no redistribution of load. However, since the eccentricity of the prestress is usually higher in the kerb units, these tend to have a higher hog and hence pick up more load as creep progresses.

As a result of this behaviour the final distribution of dead load is difficult to predict. It is likely to lie somewhere between the case of all the dead load being applied to the completed structure, which would maximise the load in the kerb units, and the case of each deck unit carrying its own dead load.

To attempt to understand the possible magnitude of the variation, a 3 span deck unit bridge was analysed with a number of different assumptions regarding construction staging and creep. The bridge analysed was an actual structure, although some changes were made to provide more useful results.

The substructures at the two piers are different with one being controlled by hogging moments at a cantilever, and the other by sagging moments at mid-span of the headstock. Two cases were considered, one with no skew, and the second with a skew angle of 30°. A description of the loading cases considered and the results are given in Figure A.14.

Figure A.14: Selected results for different modelling strategies for staged construction and creep

Effect of Different Modelling Strategies for Construction Staging and Creep

Based on a 3 span deck unit bridge.
Pier 1 is a "Cantilever Headstock" in which the two columns supporting the headstock are at the quarter points. Hence the controlling effects are negative moment and shear in the cantilever.
Pier 2 is a "Portal headstock" in which the two columns supporting the headstock are at the ends. Hence the controlling effects are shear and moment at the columns and midspan moment.

All results are factored ULS loads

Construction cases considered:

Case	Description
1	No staged construction, no creep. i.e Dead load applied to completed structure
2	Staged construction, no creep. i.e Each deck unit carries it own dead load
3	Staged construction, with creep. i.e As for 2, then some transfer of load back to KU due to creep.
4	Staged construction, with creep, including prestressing. i.e As for 3, but higher transfer of load to KU due to KU having higher hog.

Skew=0

Load Effect		PE Only				LL only (RV1-TR1)	PE Range as % of Avg PE	PE Range as % of Avg (PE+LL)	Total as % of Case 4 Total (i.e assuming Case 4 to be correct)		
		Case 1	Case 2	Case 3	Case 4	All Cases			Case 1	Case 2	Case 3
Cantilever Headstock	Mz at Col (kNm)	-286	-265	-277	-294	-429	10.3%	4.1%	98.9%	96.0%	97.6%
	Vy at Col (kN)	-258	-237	-250	-261	-400	9.5%	3.7%	99.5%	96.4%	98.3%
Portal Headstock	Mz at Col (kNm)	-214	-248	-227	-218	-363.3	15.0%	5.8%	99.3%	105.2%	101.5%
	Vy at Col (kN)	368	398	379	370	662	7.9%	2.9%	99.8%	102.7%	100.9%
	Mz at Midspan (kNm)	435	520	463	449	770	18.2%	6.9%	98.9%	105.8%	101.1%
Deck Unit - Central	Mz at Midspan (kNm)	57	74	64	50	121	39.2%	13.2%	104.1%	114.0%	108.2%
Deck Unit - next to KU	Mz at Midspan (kNm)	53	46	50	20	106	78.1%	22.3%	126.2%	120.6%	123.8%
Kerb Unit	Mz at Midspan (kNm)	259	189	229	334	494	57.4%	19.4%	90.9%	82.5%	87.3%

Skew=30

Load Effect		PE Only				LL only (RV1-TR1)	PE Range as % of Avg PE	PE Range as % of Avg (PE+LL)	Total as % of Case 4 Total (i.e assuming Case 4 to be correct)		
		Case 1	Case 2	Case 3	Case 4	All Cases			Case 1	Case 2	Case 3
Cantilever Headstock	Mz at Col (kNm)	-435	-356	-401	-356	-637	20.4%	7.7%	108.0%	100.0%	104.5%
	Vy at Col (kN)	-319	-285	-303	-311	-503	11.2%	4.2%	101.0%	96.8%	99.0%
Portal Headstock	Mz at Col (kNm)	-248	-314	-272	-265	-422	24.0%	9.5%	97.5%	107.1%	101.0%
	Vy at Col (kN)	381	448	411	500	707	27.4%	10.4%	90.1%	95.7%	92.6%
	Mz at Midspan (kNm)	450	587	499	510	825	26.8%	10.3%	95.5%	105.8%	99.2%
Deck Unit - Central	Mz at Midspan (kNm)	41	65	51	48	97	46.8%	16.2%	95.2%	111.7%	102.1%
Deck Unit - next to KU	Mz at Midspan (kNm)	43	43	43	20	88	61.7%	18.4%	121.3%	121.3%	121.3%
Kerb Unit	Mz at Midspan (kNm)	221	173	200	336	418	70.1%	25.1%	84.7%	78.4%	82.0%

It should be noted these results are for a particular bridge only and can't necessarily be applied to other bridges. Further, modelling these complex behaviours involves numerous assumptions and there are limitations to the accuracy of the results.

The key point from these results is the variation in the calculated load effects in the substructure and the deck units is significant.

- For the square bridge the variation of the load effects in the substructure is up to 7% and in the deck units is up to 22%. These are percentages of the total ULS design load effects.
- For the skew bridge the variation of the load effects in the substructure is up to 10% and in the deck units is up to 25%.

It is beyond the scope of a Tier 1 analysis to try and predict these effects. The important point is to note that there is significant uncertainty as to the actual distribution of dead load in a deck unit bridge. If the kerb unit is carrying more load than assumed, then the moments and shears in cantilever headstocks may be non-conservative. Similarly, if the kerb units are carrying less load than assumed, then the mid-span moments of portal headstocks may be non-conservative.

Against this background, it can be seen that there is good reason to take a conservative view point when assessing the substructures of deck unit bridges.

For the purposes of a Tier 1 assessment of deck unit bridges, it is recommended to treat the dead load as if it is applied to the completed structure (i.e. Case 1 in Figure A.14 above). This is appropriate because:

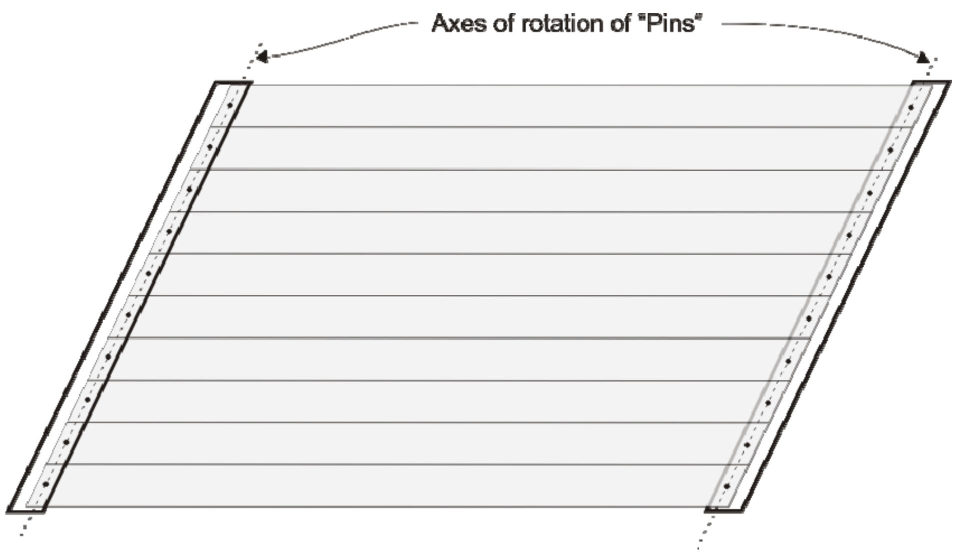
- for low skew bridges the error is small
- this method tends to give higher load effects in the cantilevers of piers. Since there is no redundancy in a cantilever it is important not to underestimate loads here
- this method may underestimate load effects in the central span of portal frame headstocks. However there is considerable redundancy here and more opportunity for moment redistribution
- the method is straight forward from a modelling point of view.

The implications for deck unit assessment are less severe, despite the large variations noted above. This is because the practice of redistributing load from the kerb unit to the adjacent deck units at the ultimate limit state is accepted.

A.4 Orientation of bearings on skewed deck unit bridges

In skew deck unit bridges in which the deck units are sitting on a bed of mortar, the axis of rotation of the “pinned” support is actually parallel to the pier, rather than perpendicular to the deck units. (See Figure A.15). Correct modelling of such bridges should take account of this.

Figure A.15: Orientation of axes of rotation for the "pins" in a skew deck unit bridge



Skewed supports can complicate the modelling of such structures, as it means the pin cannot be simply modelled (in SpaceGass) as an end release on the member representing the deck unit. Rather, a more complicated model with stub members perpendicular to the pier containing the end release is required.

To help understand the significance of this issue, the structure described in A.3 above, with a skew of 30°, was analysed with the bearing pin axis correctly oriented parallel to the pier, and again with the pin axis perpendicular to the deck units. The loading used was the 45.5 t semi-trailer with no traffic restrictions.

A comparison of the results is shown in Table A-2. It was found that when the orientation of the pin axis was changed without any other changes to the end release conditions, the critical load effects in the deck units and at some locations in the headstocks of the bridge were under predicted by 10% to 15%. However, when the ends of the deck units were also torsionally released there was little difference in the results from those with the correct pin orientation.

Table A-2: Comparison of load effects on a skew deck unit bridge as affected by pin orientation

Load Effect		**Pin Axis Parallel to Pier**	**Pin Axis Normal to Deck Units**	
			No Torsional Release	**With Torsional Release**
Cantilever Headstock	Mz at Col (kNm)	-820	-780	-818
	Vy at Col (kN)	-732	-812	-726
Portal Headstock	Mz at Col (kNm)	-811	-788	-812
	Vy at Col (kN)	1240	1083	1261
	Mz at Midspan (kNm)	1578	1574	1576
Deck Unit - Central	Mz at Midspan (kNm)	196	177	197
DU next to KU	Mz at Midspan (kNm)	143	126	146
Kerb Unit	Mz at Midspan (kNm)	605	523	601

The above results demonstrate that if the pins at the ends of the deck units are modelled by simply using an end release at the ends of the frame members representing the deck units, then it is important to also release the ends to allow twisting about the member axis.

A.5 Interpretation of frame model analysis results

Frame models represent members that have a finite width by beam elements with zero width. As a result the connections between members occur at a single point (a node) and the load transfer between members occurs as a concentrated load. However, in reality the concentrated load is distributed over the width of the connection. This results in steps in shear force diagrams, torsion diagrams and moment diagrams which are not real. At some locations the use of these diagrams without modification can result in significant errors in the load effects used to assess a section.

A.5.1 Shear force diagrams

One example where use of the unmodified shear force diagram can lead to significant errors is in the headstock of a deck unit bridge. This is because in most cases the deck units are continuously supported meaning that the loading on the headstock is essentially a distributed load, whereas in the frame analysis it is represented as a series of point loads.

Figure A.16: Shear force diagrams for the headstock of a deck unit bridge

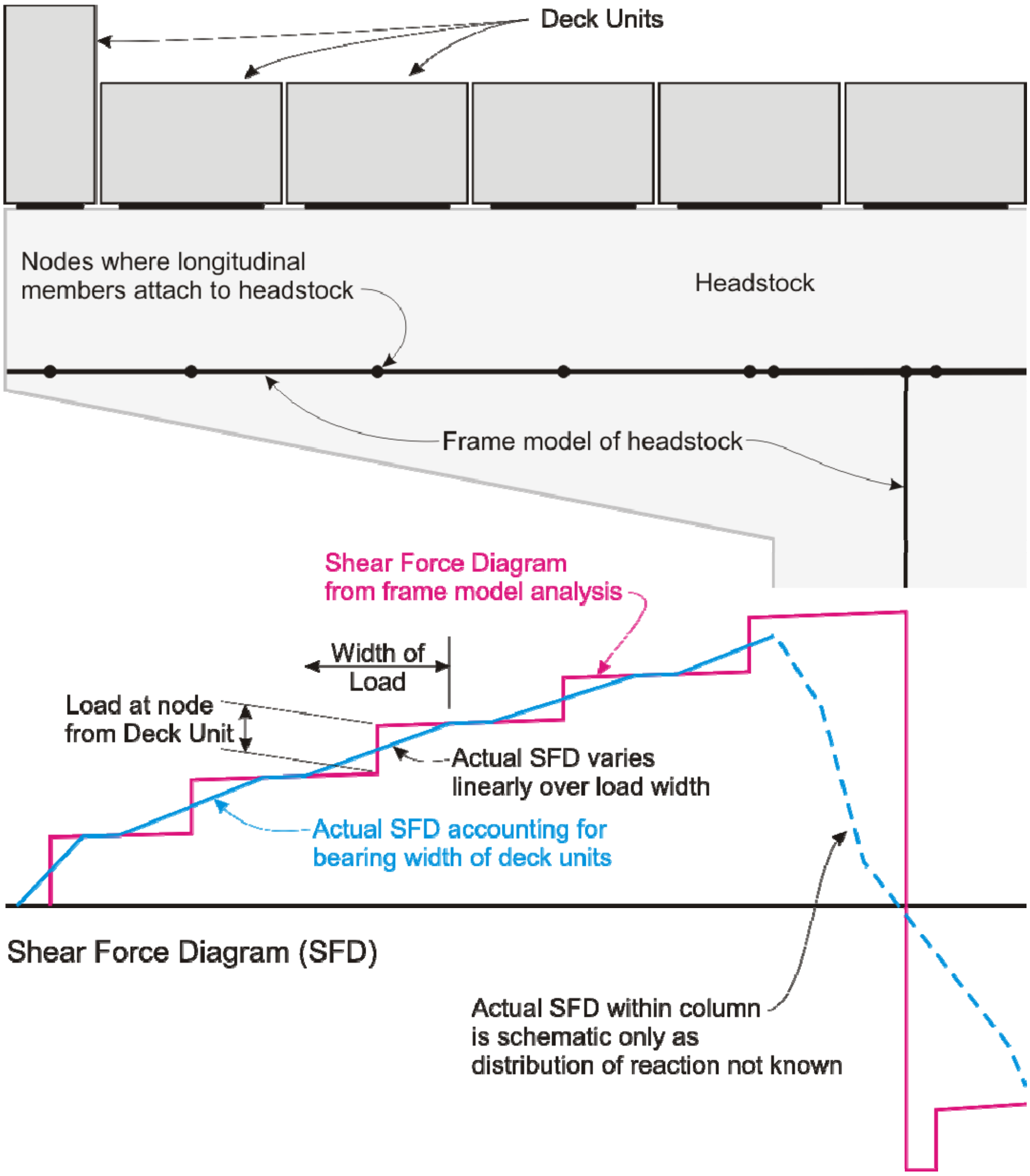


Figure A.16 shows the shear force diagrams for a cantilever headstock on a deck unit bridge. In this case the deck units are supported on bearings so the support is not quite continuous. In this example, it is assumed the ends of the deck units are torsionally released as recommended in Section A.4, so that there is no torsion to be transferred from the deck units into the headstock and the reaction from each deck unit is uniformly distributed over the width of the support.

It can be seen that the shear force diagram expected from the frame model has a step in it at each node where the frame member representing the deck unit connects to the member representing the headstock. This is not real, because the deck unit support has a finite width, and so in reality the reaction from the deck unit is applied to the headstock over the width of the bearing. Hence the increase in the shear force due to the reaction from the deck unit occurs as a lineal increase over the width of the applied load from the deck unit, and not a step.

The actual shear force diagram determined from these considerations is also shown in Figure A.16. It can be seen that if a cross section being assessed is near a node, then the error in the shear force diagram can be a significant portion of the total shear, with the outcome being either conservative or non-conservative depending on which side of the node the section is located.

The above discussion relates to a shear force diagram for a particular load case; however in the assessment of multiple load cases it would be usual to consider a shear force envelope to determine the worst effects at a section. It can be shown that if the shear force envelope output from a frame model is modified in the same method as described above, then the resulting diagram will be, if anything, slightly more conservative than the actual envelope. (The actual envelope would be determined by modifying the shear force diagrams for each individual load case before enveloping them).

The above principles apply in all cases for frame model shear force diagrams but are most relevant where the members applying the load (to another member) are relatively wide compared with the spacing of the members. Hence another case where the effect might be significant is where transverse deck members are transferring significant shears between longitudinal members. Hence steps in shear force diagrams on longitudinal members due to reactions from transverse deck members, should be smoothed in the manner described above.

The above principles can also be applied to torsion diagrams. In this example such torsions might result from the deck unit reaction not being on the headstock centreline.

A.5.2 Bending moment diagrams

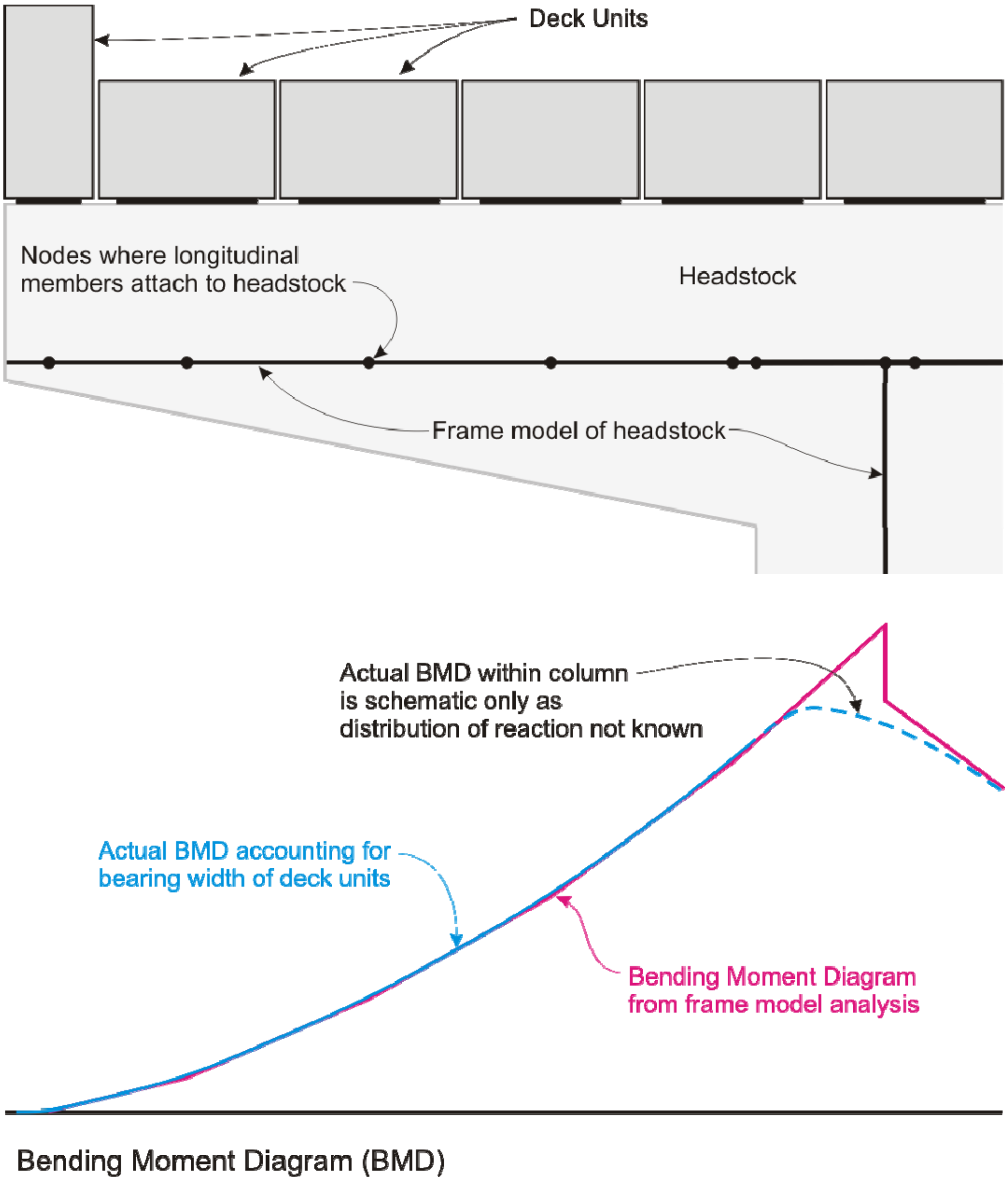
The bending moment diagrams from a frame model in which a distributed load is represented by a point load are also incorrect, although the error is usually small if the actual load width represented by the point load is small.

Figure A.17 below shows the frame model and actual bending moment diagrams for the headstock shown in the previous example. It can be seen that the difference is small, and in this case can be neglected.

For a downward load, the actual bending moment is more negative than the moment at the node by a small amount. For a uniformly distributed reaction the error is a maximum of $PL_a/8$ at the node where the members are connected reducing to zero at the ends of the actual load width. In this expression P is the point load applied at the node and L_a is the actual load width. As long as L_a is small compared with the total span or cantilever length, this error will be small.

One location where the difference can be significant is over a support as shown in Figure A.17. However, at the location of the critical section for bending, which is near the face of the column, the error is usually small and conservative.

Figure A.17: Bending moment diagrams for the headstock of a deck unit bridge



A.5.3 Limitations

The correction to the shear, torsion, and bending moment diagrams described in the previous sections is limited to the case where the reaction from the supported member consists of a vertical load only with no torsion. This means that the actual load applied to the supporting member is a uniformly distributed load. This will normally be the case if the ends of the supported members are torsionally released as recommended in Section A.4 for deck unit bridges.

If there is a torsion load from the supported member being transferred into the supporting member, then the situation is more complicated. The change in the shear force over the actual width of the member will not be linear as shown in Figure A.16 but will be nonlinear with the shape depending on the sign and magnitude of the torsion. Such a modification is considered to be out-of-scope of this document.

If torsion is being transferred, but the linear variation is still assumed and the shear force diagram modified as above, then non-conservative errors may still exist. However, the maximum error will generally be less than if the unmodified shear force diagram is used.