Analysis and design of link slabs in jointless ramp viaduct

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Abstract. Due to the internal and external movement of the bridge deck, it is necessary to provide expansion gaps between multi span bridge decks. However the water leaking and debris trapped in the expansion joints will cause high maintenance cost. Therefore an alternative to eliminate expansion joints is vital. This paper contains comparison between the 3D, 2D model and a manual analysis of a link slab in a curved ramp bridge. This link slab is modelled as Finite Element Model using SAP 2000 software as a global model with the superstructure model for the calculation of the global effect from temperature, shrinkage, and creep with live HA, HB loads and superimposed dead loads and the most unfavourable conditions are identified.

1. Introduction
Infrastructure plays a key role in the rapid development. Highway construction projects are a main aspect of this development. As a result, the existing highways were expanded. Most of the parts of expressways lay on multi span bridges. Traditional practice was to provide expansion gaps between the bridge deck and piers or abutments which provide enough space for the girder movement between piers or abutments. As expansion joints won’t provide a durable and economical solution to the long term water leakage which will cause to the corrosion of the superstructure, replacement of expansion joints will be costly and it will reduce the smoothness of the road [1]. Because of that, a new design concept which replaced the expansion joints with link slabs was introduced [2]. Link slabs allow the rotation of the girders through their flexural flexibly and relatively low stiffness. This will simplify the construction detail and will give an effective solution to most of the problems associated with expansion joints.

2. Deboned Link slab system
Link slabs are provided in a pier where the girders in either side of the pier are simply supported 20m “T” girders [3]. Based on the serviceability and the cost, the deboning length of the link slab is taken as 7.5% of the girder length in each side [4]. 20mm thick debonding material is used to debond the girder and link slab, the end of link slab is monolithically cast into girder and deck slab which will transfer rotation of the girder and axial force to the link slab [5].
3. Super structure finite element model

Global model was prepared considering 2 spans of 20m simply supported girders with a curvature of 50m radius and the deck was kept continuous by the link slab over the pier. The composite behavior between the bridge deck and the girder is enabled by restraining the translational and rotational degrees of freedom using link elements in sap 2000 between the deck and the girder beam. The support conditions were modelled using link elements which have been used to model bearings. As shown in figure 2 and 3 both 2D and 3D analysis were done considering the horizontal curvature of the ramp viaduct. Same loads were considered in both cases and the bending moments were identified from each model separately for the reinforcement design. The basic difference between the 2D and the 3D model is that the link slab is modeled as a frame element in the 2D model where as in 3D model the link slab is modeled as a shell element which will facilitate to obtain results in transverse direction.

As the self-weight is taken automatically by the software, super imposed dead load is applied for the 100mm thick deck slab. Traction force and line loads calculated as per BS 5400 with respect to notional lane width and applied to the frame element model by considering an effective width for a girder. Modeling was carried out considering the effect of the live loads in different locations. Dissipation of the loads through the asphalt layer is also considered.

Temperature loads are calculated according to the Sri Lankan Road Development Authority guide lines, creep and shrinkage loads are applied by considering the strain following the guidelines of BS 5400 according to the actual conditions in Sri Lanka. Shrinkage strain is calculated using long term and short term shrinkage of the link slab and the deck.

Settlement of 5mm has been modelled introducing a link element with a stiffness of “k”.

4. Analysis and results

Debonding length of the link slab is taken as 7.5% of the girder length and the gap between the girders is 80mm, total length of the link slab in 20m girder is 3080mm. Thickness of the slab is 200mm. Three notional lanes were considered for the total carriage way length of 8.5m. The material properties are shown in table 1.
Table 1-Material properties considered

| Description                                      | Value   |
|-------------------------------------------------|---------|
| Characteristic strength of concrete for girders  | 50 Nmm$^2$ |
| Characteristic of strength of concrete for link slab | 40 Nmm$^2$ |
| Characteristic strength of reinforcement, fy      | 500 Nmm$^2$ |
| Elastic modulus of reinforcement                  | 200 Nmm$^2$ |
| Concrete density                                 | 25 kN/m$^3$ |
| Poison ratio                                     | 0.2     |

4.1. Bending in main direction

Since simply supported girders will achieve continuity through link slabs, longitudinal axial forces arise in deck due to internal and external actions. Creep and shrinkage can be named under internal actions. Traction, wind and temperature cause external axial forces in link slab. Apart from above axial forces, link slab should sustain moments due to differential settlements, live loads and girder deflections. Slab will be designed for moments with estimated axial forces. The longitudinal axial forces under different load cases are shown in table 2.

Table 2. Longitudinal axial forces

| Load Pattern | P1(kN) |
|--------------|--------|
| SLS kN/m     |        |
| Creep        | 9.18   |
| Shrinkage    | 7.77   |
| Traction     | 31.31  |
| Temperature  | 6.3    |
| 5m settlement| 24.97  |
| Combination 1| 48.22  |
| Combination 4| 79.54  |
| ULS kN/m     |        |
| Combination 1| 57.864 |
| Combination 4| 97.01  |

As shown in table 2, the most critical longitudinal axial force is from the combination 4 of 97 kN. Since axial force less than 0.1 f$_{cu}$ Ac section can be designed by ignoring axial force.

When considering the traction force, link slab is designed for the buckling moment as shown in figure 4. Full traction force is assumed to act on the link slab and the moment is calculated assuming the link slab as a column. Both ends of the column assumed to be fully restrained and the effective length is taken as 0.7 times of the total length. External moment for the column section is taken from the calculated moment of the creep and shrinkage. Bending moment due to the axial force is calculated using the following equation.

$$M_{tx} = M_{tx} + \frac{N_{hy} \left( \frac{le}{h} \right)^2}{1750} + \left( 1 - \frac{0.0035le}{hx} \right)$$  \hspace{1cm} (1)

Figure 4. Girder arrangement in piers
When designing the main reinforcement, both hogging and sagging bending moments in the longitudinal direction of the link slab are considered. Calculations are done without considering the angle of girders with link slab. Since the angle of girders with link slab is small, it can be neglected. Moment due to the unintentional settlement in a single bearing line caused by jacking, single girder upward and downward deflection and single wheel load is considered.

Unintentional settlement or jacking will create moments in the bearing line. Unintentional settlement is considered as 5mm. Effect from 5mm settlement is incorporated into link element and stiffness is adjusted according to maximum bearing force. This link is assigned only for one side of the pier. Sagging and hogging moments due to the single wheel load, dead load and superimposed dead load are calculated assuming that the restrained ends of the link slab are fixed.

Moments due to the girder rotation are calculated for each load case individually as due to some load cases give a hogging moment at the link slab middle while others give a sagging moment. In this calculation both sagging and hogging moment are considered in deciding the reinforcement. Because of that, individual effect should be calculated. Loads in the longitudinal direction are ignored as the effects from the axial forces are ignored. For consideration of the main and transverse reinforcement, loads from the combination 3 from BS 5400 are used. Results from the 2D model and 3D model are compared with the manual calculation. When generating the 2D model, in order to get the maximum girder rotation from the live loads, influence line method was followed to decide the location for the point loads. Maximum sagging bending moment was taken placing the point load on the middle of the link slab. Depending on the situation, location of the live loads is changed to get conservative design.

Moment due to the girder rotation is calculated using the following equation.

$$M = \frac{2AEI}{L^2}$$  \hspace{1cm} (2)

Sagging and hogging moments due to the effects of the load are directly taken from the 3D model and compared with the bending moment calculated using the girder rotation from the 2D model as shown in figure 6.

Bending moments calculated from the girder rotation are shown in table 3.

| Load type         | Moment type | Rotation | Moment (kNm/m) |
|-------------------|-------------|----------|----------------|
| Asphalt           | hogging     | 0.00004  | 0.6            |
| Temperature positive | hogging     | 0.0003   | 14.7           |
| Temperature negative | sagging    | 0.001    | 14.7           |
| HA                | hogging     | 0.0019   | 27.8           |
| HB                | hogging     | 0.0021   | 30.9           |
| Creep             | sagging     | 0.00144  | 2.8            |
| Shrinkage         | sagging     | 0.00122  | 2.2            |
Depending on the moment type and load combinations total ULS and SLS bending moments are calculated and reinforcement is provided for those bending moments.

Bending moments in the main direction were generated by 2D frame model as illustrated below from figure 7 to figure 10.

Figure 7. Bending moment for combination 1 ULS form 2D frame model
Figure 8. Bending moment for combination 3 ULS form 2D frame model
Figure 9. Bending moment for 5mm settlement combination 1 form 2D frame model
Figure 10. Bending moment for 5mm settlement combination 3 form 2D frame model

Bending moments from the 3D model were taken in the M11 direction for the ULS combination 1 and combination 3. Results are shown from figure 11 to figure 14. Combination 4 was ignored as effects from the longitudinal axial forces to the link slab are minimum when compared to other load combinations.

Figure 11. Resultant M11 minimum ULS envelope-hogging form 3D frame model
Figure 12. Resultant M11 maximum ULS envelope-sagging form 3D frame model
Figure 13. Resultant M11 ULS combination 3 hogging form 3D frame model
Figure 14. Resultant M11 ULS combination 3 sagging form 3D frame model

Summary of the bending moment results are shown in table 4 for every analysis with the corresponding types of bending moments.

| Bending in main direction | Bending Moment-2D | Bending Moment-3D | Manual analysis |
|---------------------------|-------------------|-------------------|-----------------|
|                           | S-Main | H-Main | S-Main | H-Main | S-Main | H-Main |
| Combination 1             |        |        |        |        |        |        |
| ULS                       | 44     | 158    | 31     | 65     | 35.7   | 97.7   |
| SLS                       | 35     | 127    | 22     | 50     | 30.1   | 82.1   |
| Combination 3             |        |        |        |        |        |        |
| ULS                       | 50     | 165    | 32     | 67     | 32.2   | 86.5   |
| 5mm displacement ULS      | 72     | 161    | 32     | 63     | 84.3   | 106    |
| Envelope ULS              | 72     | 165    | 32     | 67     | 84.3   | 106    |
| Envelope SLS1             | 35     | 127    | 22     | 50     | 30.1   | 82.1   |
4.2. Bending in transverse direction

When link slab is at rest state, (i.e. support on web of the girders) vehicular load on deck will be transferred to girders in transverse direction. In this case link slab is assumed to behave similar to deck slab.

The width of the road section is 9100mm and the girder spacing is 2486mm. Loads considered for transverse bending are HA and HB single wheel load, dead load and super imposed dead load. Loads are calculated according to BS5400. Effective width of the link slab for the single wheel load is \(a_x+0.6l\), where “\(a_x\)” is the square root of the area of the single wheel load and “\(l\)” is the length between girders.

Sagging moment is calculated from the UDL of the super imposed dead load and the self-weight by considering that the link slab is simply supported at the girders. Hogging moment is calculated assuming that the support conditions of the slab at the girders are fixed. Bending moments in the transverse directions are measured using the diagrams as shown in figure 15 and figure 16.

![Figure 15. Resultant M22 ULS combination 1 sagging](image1)

![Figure 16. Resultant M22 ULS combination 3 sagging](image2)

Bending moments in the transverse direction are shown in table 5.

| Bending moment 3D Manual analysis |
|----------------------------------|
| S-transverse | H- transverse | S-transverse | H- transverse |
| Combination 3 ULS | 17 | 50 | 50.9 | 36.4 |
| 5mm displacement ULS | 17 | 52 | | |
| Envelope ULS | 17 | 50 | 50.9 | 36.4 |
| Envelope SLS1 SLS | 36.1 | 11.5 | | |
| Envelope SLS Permanent SLS | 7 | 27 | 4.6 | 3 |

4.3. Serviceability condition check

Maximum design crack width is 0.25mm according to the severe environment conditions. Crack width is calculated from the loads of combination 1 in BS 5400 by considering 25 HB loading units following the design guide lines of BS 5400.

Sufficient reinforcement should be provided to control crack spacing, so that the reinforcement will not yield before the tensile strength of immature concrete is exceeded. In addition reinforcement should be adequate to ensure that the crack width is at the permissible limit.

The figure 17 and 18 show the link slab in the 50m radius ramp viaduct before concreting and after concreting.

![Figure 17. Link slab before concreting](image3)

![Figure 18. Link slab after concreting](image4)
5. Conclusion

When considering hogging bending moment in the main direction, 2D analysis gives the highest bending moment when compared to the other methods of analysis and 2D frame analysis proves an over estimation of the bending moment. In designing reinforcement for a more conservative method, bending moment from the 2D analysis was considered.

The most critical load case in designing the link slab is the HB live load.

The resultant girder rotation from the creep and shrinkage is higher. As we consider only the half of the link slab, the second moment of area of the slab is less, hence the bending moment is lesser.

Drastic differences in sagging bending moments were not found between the 3 analyses. The highest sagging bending moment is given by the manual analysis while the minimum sagging bending moment is given by the 3D analysis. Effects from the loads combinations are somewhat same in these 3 cases and the only difference between the load cases is which the settlement considered.

In the transverse direction, 3D analysis provides the higher hogging bending moment than manual analysis, however manual analysis gives higher sagging bending moment than 3D analysis.

When considering the 3D analysis and the 2D analysis, bending moment from the 2D analysis is higher than the bending moment calculated from the 3D analysis due to the load distribution in the shell element. Therefore, the most conservative method is the 2D analysis method in link slab design.

In this research only consideration was the major service loads and its behavior and effect to the link slab. Fatigue analysis was not considered in this paper and it can be developed in further researches.

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