



# Army and Navy Flyover ECC Bridge No. 1000

**Column Movement Investigation Report** 

23<sup>rd</sup> October, 2018





# Army and Navy Flyover - ECC Bridge No. 1000

Ringway Jacobs - Essex County Council

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B3553S67/1001/01 | Rev 01 23<sup>rd</sup> October 2018 ECC Bridge No. 1000



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Ringway Jacobs Ltd

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# 1. Introduction

Army and Navy Flyover is a 15-span structure located over the Army and Navy Roundabout, Chelmsford. The structure was closed to traffic on 11/09/18 following the discovery of sheared holding down bolts and lateral movement of support base plates at trestles T6 and T7. At the time of writing, Essex Highways have introduced propping to the two affected trestles and are undertaking various investigations on the structure, including intrusive investigation of the holding down bolts, weld testing, and concrete testing.

# 1.1 Client Brief

Essex Highways has commissioned Jacobs to assist in an initial investigation into the possible causes of the column base plate movements noted at trestles T6 and T7 of the Army and Navy Flyover. Relevant information on the situation was transferred to Jacobs on 21/09/18. Jacobs are to investigate and report on load effects that could plausibly contribute to the defects noted in the support fixings. Additionally, the size and grade of bolt required to resist the forces generated by the movements are to be determined. Recommendations on immediate repairs and further investigations are to be included.

# 1.2 Report Aims

The aim of this report is to summarise the recorded defects and to describe the work undertaken by Jacobs to identify plausible causes of the column baseplate movement. The report will discuss the structural analysis undertaken, summarise the findings and present recommendations for interim defect repairs.



# 2. Description of Structure

Army and Navy Flyover carries a single 3.10m wide carriageway over the Army and Navy roundabout at the junction of the A414 and A138. It operates on a tidal basis under traffic signal control with traffic running from east to west in the mornings and from west to east in the afternoons. The speed limit on the bridge is 20mph, previously having been 30mph. Traffic over the bridge is restricted to light traffic and maintenance vehicles. The structure is curved in both horizontal and vertical planes.

The flyover comprises fifteen spans which are simply supported and vary from 11.03m to 20.46m in length. Each span consists of a reinforced concrete deck slab supported by regularly spaced steel cross beams and two longitudinal steel beams. The reinforced concrete deck slab acts compositely with the steel beams which are supported by cast iron bearings with a downstand plate at the beam ends sitting in a groove in the bearing.

The deck is supported by steel portal frame trestles consisting of cross beams and columns. Lateral stability of the structure is provided at each trestle by cross beams with stiffened haunches that ensure portal frame action. Longitudinal stability of the structure is provided by additional framing that links four columns beneath spans D5 and D11.

The flyover was originally designed in 1977 for a loading of 20% HA load or a 9.9 tonne gritting lorry with a 1.5 tonne trailer. The structure was built in 1978 and was originally intended as a temporary solution to improve the traffic situation at this busy junction.

For the purposes of this report, the bridge spans are numbered D1 to D15 from west to east as shown in Appendix A. Trestles are numbered T1 to T14, T1 starting at the west end of the structure.



Photograph extract from BING Maps Birds Eye View

Figure 1: Aerial View of Army and Navy Flyover



# 3. Description of Defect and Available Site Information

### 3.1 Introduction

Essex Highways contacted Jacobs on 18/9/18 with a description of defects noted in the base plate connections of trestles T6 and T7. The defect was described as a recent lateral movement of the base plates at these locations. It was communicated to Jacobs that the structure had been closed to traffic as a result of the reported defects and that urgent works were in progress to prop the affected trestles due to the uncertainty of the cause.

Essex Highways confirmed that, whilst the exact date of the column base movement was unknown, it was considered to have occurred between the period of 21/6/18 to 8/8/18. It was also noted that the flyover had been closed for maintenance work unassociated with the base plate defect over the period of 23/7/18 to 12/8/18.

There is no evidence of recent vehicle impacts on the bridge structure or supports in the vicinity of the reported defects.

Essex Highways confirmed that temperature monitoring is active on the flyover to record the surface temperature. The recorded surface temperature reached up to 47.9 C° during July 2018.

Weather monitoring information from nearby Chelmer Village indicates that ambient summer temperatures reached peak levels of 34.1 C° and 33.9 C° in late July and early August respectively. These were higher than 2017 peak temperatures of 29.1 C° and 28.6 C° recorded during the same months.

On notification of the existence of the defects, Essex Highways commissioned site surveys and measurements of the affected elements and adjoining structure. A detailed description of the reported defects is included below based on the site surveys and measurements undertaken by Topbond during the week ending 21<sup>st</sup> Sept 2018.

### 3.2 Trestle T6, North Column

15mm overall southward movement of the base plate. Approximately 30° inclination of eastern holding down bolt which has not fractured. 24mm southward movement of the western holding down bolt, which has fractured.



Figure 2: Trestle T6 – north column baseplate, facing south-east

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Figure 3: Trestle T6 – north column, east holding down bolt (inclined)



Figure 4: Trestle T6 – north column, west holding down bolt (fractured)

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### 3.3 Trestle T7, North Column

32mm southward movement of base plate and both holding down bolts (both fractured).



Figure 5: Trestle T7 – north column baseplate, facing south-east



Figure 6: Trestle T7 – north column, east & west holding down bolts (fractured). Plinth concrete broken out during site investigation and propping works



# 3.4 Temporary Deck Propping

As discussed, on notification of the defective footings, Essex Highways instructed that the affected trestles were to be supported on temporary propping. This was undertaken on the 19/9/18 and 20/9/18. In addition, it was decided to jack the props up beneath the trestle heads at T6 and T7 in order to remove load from the legs of those trestles. When this was undertaken it was noted that the dislodged trestle legs and base plates did partially return to their assumed original position.



# 4. Desk Study Investigation of Defect Cause

# 4.1 Structural Form and Behaviour of Flyover

In order to investigate potential root causes for the defects recorded on site an initial review of the structural form and expected structural behaviour under various applied loads was undertaken.

#### 4.1.1 Superstructure

The superstructure of the flyover is approximately 240.5m total length. Each span comprises a reinforced concrete deck slab supported by two longitudinal steel beams and regularly spaced transverse steel beams. Each deck can therefore generally be considered to be a separate steel-concrete composite span which is structurally independent of adjacent decks under typical self-weight and vertical traffic loading. The concrete deck slab provides torsional restraint to the beams within the span. Each deck span is supported on grooved cast iron bearings. The ends of the main beams are bolted to these bearings with spring washers between the bolt heads and the bearing surface. This has the effect of restraining the beams against lateral movement but allowing freedom to rotate at the base of their end plates ie to act as a simply supported span.

Thrust pads, consisting of sandwiched steel and rubber plates, are bolted to the top halves of the main beam end plates, on the side of the adjacent beam. At the abutments, the beam end details are similar but with additional laminated packs at the thrust pads. It is considered that this joint detail was originally designed to eliminate the transfer of longitudinal sagging or tension forces between adjacent spans throughout the length of the bridge deck. The detail could, however, be expected to allow the transfer of axial compression associated with longitudinal forces arising from expansive or braking/skidding loads between adjacent decks. To what extent the transfer of axial loads occurs is unknown due to the unknown stiffness of the thrust pads in their current condition.

Significant expansive corrosion product, such as that observed at the cast iron bearings, end plates, bolts and thrust pads, could affect the joint assembly, reducing the capability of the thrust pads to absorb thermal expansion forces between adjacent decks.



Figure 7: View of typical deck and joint (facing southwest)



#### 4.1.2 Substructure

The bridge substructure consists of 14 No. haunched steel portal frames (trestles) supported on shallow spread footings with simple bolted baseplate connections. Four of these trestles, under spans D5 and D11, are stiffened with the addition of bolted longitudinal beams and additional holding down bolts.

At each end of the flyover there is an approach ramp between the abutment and the end of flyover. Record drawings indicate that the approach ramps are formed from precast concrete wall elements (wingwalls), which are tied together with 30mm diameter ties. The void between the walls is filled with lean mix concrete.

As discussed above, longitudinal forces on the deck are transferred directly onto the supporting trestles via the bearing detail. Typically, the trestles offer little resistance to longitudinal movements or forces except at spans D5 and D11 where additional framing is provided. Other than at these locations, longitudinal restraint is provided only at the abutments.

Without detailed knowledge of the material properties of the thrust pads and a full analytical investigation of the structure, it is unknown whether the longitudinal framing at spans D5 and D11 contribute significantly to overall longitudinal restraint of the deck or whether this is provided mainly at the abutments, which are considered to be rigid end supports.

The trestle and bearing detail provides lateral restraint to the deck at each deck end. Typically, a lateral load applied at the top of a portal frame will cause the frame to sway in the direction of force application and will induce simultaneous uplift and shearing reactions at the support behind the direction of sway. These forces would be transferred into the foundation by the holding down bolts.

As the recorded failure indicates a movement of the base plates to the south, it is considered that loading mechanisms tending to sway trestles T6 and T7 in a southerly direction should be investigated to determine whether they are likely to result in sufficiently high support forces to result in bolt failure.

# 4.2 Loading Types Potentially Resulting in Southerly Sway of Trestles T6 & T7

Loading types that could potentially act on the structure to produce a southerly lateral load onto the top of the trestles resulting in are:

- Impact directly onto the deck or legs in a southerly direction
- Wind perpendicular to the northern deck face
- Braking or skidding load applied in a southerly direction on the deck surface
- Centrifugal loads from traffic moving at speed around the curve of the structure
- Thermal expansion of the deck superstructure

#### 4.2.1 Impact Loading

There is no evidence of vehicle impact on the northern edge of the bridge deck structure at span D7 or directly on the northern legs of trestles T6 or T7 that might have caused a shear failure of the holding down bolts. This is therefore, discounted as a plausible cause of the dislodged base plates.

#### 4.2.2 Wind Loading

The northern elevation of span D7 is significantly shielded by dense, mature vegetation within the roundabout. Also, there were no reported exceptional wind events in the period over which the defect was reported to have occurred. For these reasons wind loading is discounted as a plausible cause of the dislodged base plates.

#### 4.2.3 Braking or Skidding Load on Superstructure

The assessment of highway bridges and structures in the UK is typically undertaken in accordance with Standard BD 21/01 from the Design Manual for Roads and Bridges. However, braking and skidding loads do not form part of a typical UK bridge assessment and BD 21/01 does not provide load models to represent forces resulting from vehicles braking or skidding on the deck surface. The application of skidding and braking loads are, however,



considered in relevant design Standards and are considered to be a plausible cause of lateral sway to be investigated further.

#### 4.2.4 Centrifugal Loading on the Superstructure

The assessment of highway bridges and structures in accordance with Standard BD 21/01 does consider centrifugal loading associated with traffic travelling at speed around a curved alignment. However, the assessment approach does not directly apply a lateral centrifugal load, as per the typical design load model. Instead it considers an enhancement to vertically applied wheel loads resulting in an asymmetric vertical loading on the trestle. The effect of unbalanced vertical loading on the deck does not produce significant shear or tension in the footing, especially when the effect is factored down to account for the significantly reduced traffic speed and vehicle weight on this structure compared to the full 40t vehicle assessment load model. Therefore, centrifugal loading is not considered to be a plausible cause of the noted defect.

#### 4.2.5 Thermal Expansion of the Deck Structure

Under increasing temperature the structure will tend to expand in length and, depending on the nature of how this expansion is restrained, stresses and deformations will occur in the structure. As discussed above, the interaction between adjacent decks will be affected by the performance of the thrust pads and also by the amount of restraint provided by the stiffened trestle frames under spans D5 and D11. However, in general the curvature of the structure on plan could lead to a southward loading on trestles T6 and T7 during temperature-related expansion events and this is considered to be a plausible cause of lateral sway to be investigated further.

### 4.3 Analysis of Thermal Expansion Effect

A simplified, conservative 2D line beam model of the Army & Navy Flyover deck was created to conduct an initial approximate analysis of the effects of thermal expansion and lateral loads from heavy traffic.

Member section properties of the composite deck units were determined in accordance with relevant assessment Standards BD21/01 and BD44/15. Individual span member ends were released in bending only, in order to reflect the simply supported nature of each span. A conservative, worst-case assumption was made that there were no axial releases specified between adjacent spans ie in order to model the behaviour of the structure as if the intermediate joints between decks were 'locked up', either due to corrosion product and/or degradation of the thrust pad materials. This assumption effectively meant that the bridge was expanding over it's full length between the fixed abutments. This would tend to maximise the effects of thermal expansion in the structure. The true behaviour of the structure is likely to be significantly more complex and less conservative than this assumption in terms of modelling the effects of thermal expansion.

Intermediate supports were given varying spring stiffnesses to reflect the sway stiffness of each unique trestle. Trestles T4, T5, T10, and T11 were given additional spring stiffness for the contribution of the additional longitudinal elements. End supports representing the abutments were modelled as fixed points.

A more advanced 3D modelling analysis was not feasible due to the time constraints imposed by the urgency of the initial investigation and by the need for materials testing to ascertain the nature of the thrust pad materials. From this conservative model it was determined that significant sway could potentially be experienced at a number of trestles including T6 and T7. It also confirmed, as expected, that under thermal expansion the global sway behaviour is southwards and is at its greatest at trestles T6 and T7.





Figure 8: Line beam model of Army and Navy flyover.

### 4.4 Analysis of Trestle Frames & Support Reactions

In order to investigate the loads imparted on the trestle frames associated with the baseplate movements recorded on site, 2D portal frame models were created for trestles T6 and T7 based on the specific steel frame dimensions and member sizes at those locations.

#### 4.4.1 Determination of Support Reactions and Bolt Capacities

As neither the actual loading condition or degree of movement in the trestle head at time of bolt failure were known, the analysis was based on an assumption that the recorded dislodged position at the foot of the column would be approximately equivalent to the deflected position of the column head immediately prior to the holding down bolt failure. Therefore, the analysis calculated the support reaction forces and frame member effects produced in the undamaged trestle frame when it was deflected by 32mm southwards displacement at T7. Similarly, a 24mm southwards displacement was imposed on T6.

The resulting support reactions at the defective column footing were then compared with the calculated capacity of the original holding down bolts in combined shear and tension. This analysis found that the resulting support reactions were below the calculated capacity of the original M24 grade 8.8 bolts. This is indicative that either the loads or imposed deflections applied to the trestle frame and subsequently to the bolts was greater than assumed or that the bolt capacity was less than calculated for the original bolt size.

The analysis progressed to determine a reduced bolt capacity equivalent to the assumed applied loading generated by the 32mm head movement. This indicated that a reduction in the bolt diameter to approximately 20mm would have reduced the capacity of the bolts below the assumed applied loading.

The investigation also calculated the degree of sway movement that would have been required to generate support reactions equivalent to the calculated capacity of the original M24 holding down bolts. This was found to be 42mm at Trestle T7 and 50mm at Trestle T6. It is noted that these values are both greater than the recorded position of the baseplates.



#### 4.4.2 Comparison with Braking/Skidding Loading Effects

Having established the assumed applied support reactions associated with the recorded column deflection and having determined the capacity of the holding down bolts, a comparison was made with the support reactions potentially generated by braking and skidding loads on the deck. As discussed above, the proposed traffic usage restrictions resulted in the original design traffic loading being limited to 20% of the standard UK design traffic load model in use at that time. Considering this, a similar reduction was applied to the design load model for skidding presented in BD37/01. Applying this load effect at the head of the trestle produced support reactions that were significantly below the capacity of an M24 Gr 8.8 holding down bolt. The utilisation factor under this loading was calculated to be only 10-15% of the available capacity.

#### 4.4.3 Comparison with Thermal Expansion Model

Although it is discussed above that the 2D line beam model was recognized to be a simplified conservative approximation of the true behavior of the structure under thermal expansion, an estimate was made of the temperature changes required to generate the above trestle sway movements.

The temperature increase required to generate a 42mm head sway movement in the model at Trestle T7 was estimated to be only  $+15^{\circ}$ C. Based on the recorded high deck surface temperatures over the period in which the failure is claimed to have occurred, it is considered likely that the positive temperature change in the deck would have been in excess of  $+15^{\circ}$ C.

Whilst the 2D model is not sophisticated enough to accurately model the magnitude of deformations and load effects in the structure arising from thermal expansion, it is sufficient to demonstrate patterns of movement consistent with the conjectured reason for the base movements and the observed bolt failures.



# 5. Conclusion and Recommendations

The limited investigation described above has concluded that the recorded bolt fractures and baseplate movements are most likely to be attributed to high ambient temperatures linked with corrosion and/or deterioration of bridge components, including the holding down bolts at the affected columns. The expansion of the deck structure due to high ambient temperature is considered to have generated a southerly deflection of the trestle heads at T6 and T7 which led to a combined shear and tension failure of the corroded holding down bolts.

Other potential causes, such as forces due to vehicle braking/skidding or centrifugal effects from normal traffic, have been investigated but would not on their own have imposed loads of the necessary magnitude to cause the holding down bolt failures.

Preliminary analysis indicates that replacement of the M24 grade 8.8 bolts is a suitable interim measure until a more rigorous assessment of the structure is undertaken, particularly into the effect of thermal expansion under high temperature.

Replacement bolts should be corrosion resistant, such as galvanised or stainless steel, with suitable isolation to prevent bi-metallic corrosion of baseplates. It is recommended that the condition of the holding down bolts at the other trestles are investigated within the next 6 months and replaced if there are signs of significant corrosion or section loss.

At the time of writing, it is understood that M30 grade 10.9 bolts are being installed at Trestles T6 and T7 which will provide a significantly increased capacity.

It is recommended that further investigation be undertaken to examine the effects from continuing thermal expansion cycles on the structure so that an adequate long-term solution can be designed and implemented. This should include investigation of the effectiveness of the thrust pads at the abutments and at the ends of each deck span.

It should also be considered that the structure was implemented as a temporary measure. Further structural issues may arise due to its age exceeding its original design life.



# Appendix A. Original Drawings of Army and Navy Flyover





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DETAIL OFCHANNEL !! PARAPCT FIXING-S

TYPICAL DETAIL OF GULLIES

For gully location see drawing N° B1000/2

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