Queen's University Belfast

CSHM-6
Belfast - May 2016

Analysis of Load Test on Composite I-Girder Bridge

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Abstract

This paper showcases the importance of field testing in efforts to deal with the deteriorating infrastructure. It demonstrates a load test performed on a healthy but aging composite reinforced concrete bridges in Exeter, UK. The bridge girders were instrumented with strain transducers and static strains were recorded while a four-axle, 32 tonne lorry remained stationary in a single lane. The results obtained from the field test were used to calculate transverse load distribution factors (DFs) of the deck structure for each loading case. Additionally, a 3-D finite element model of the bridge was developed and calibrated based on field test data. Similar loading cases were simulated on the analytical model and behaviour of the structure under static loading was studied. It was concluded that the bridge support conditions had changed throughout its service life, which affected the superstructure load distribution characteristics. Finally, DFs obtained from analysis were compared with factors provided in Design Manual for Roads and Bridges Standard Specification for similar type of bridges

Keywords: Bridge Field Testing, Strain Measurements, Load Distribution Factors

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Introduction

Bridges are expensive and critical structures that connect communities and serve as regional lifelines. Over time, they are exposed to many degradation processes due to environmental factors and changing loading conditions. It is found in recent studies that more than half of the Europe's 1 million bridges were built before 1965 and so they are nearing the end of their 50-year design lives [1]. Their replacement cost is equal to 30% of gross domestic product so it is not feasible to replace them. Thus, bridge owners are particularly interested in accurate and inexpensive methods for verifying remaining service life and safety of such aging structures.
Current bridge assessment techniques are mainly based on qualitative assessment and fail to estimate the hidden strength reserve of aging bridge assets in a vast majority of cases. Based on such inspections, more than 20% of 155,000 bridges in the UK are reported as structurally deficient in some form [2]. However, actual load-carrying capacity of structures is often much higher than predicted by analysis [3]. For example, load test was performed on a decommissioned skewed I-girder steel bridge where test load of 17 times higher than the design load was applied to the bridge and results showed that it had been decommissioned despite a significant remaining load capacity [4]. In another study, a 50 year old Swedish reinforced concrete railway bridge was tested to failure [5]. The results indicated that the bridge could sustain almost five times the design load. Thus, field testing is an important topic in an effort to deal with the deteriorating infrastructure, since it can reveal hidden reserves of structural strength at the same time verifying safety.
In this research study a load test was performed on a composite reinforced concrete bridge in Exeter, UK. The bridge girders were instrumented and static strains were recorded while a 32 tonne lorry passed several times along each lane. The results obtained from the field test were used to calculate transverse load distribution factors (DFs) of the deck structure for each loading case. In parallel, a 3-D FE model of the bridge was developed and calibrated based on the measurements. Similar loading cases were then simulated on the analytical model and behaviour of the structure under static loading was studied. It was concluded that the bridge support conditions had changed throughout its service life, affecting the structure load distribution characteristics. Further, DFs obtained from analysis were compared with factors provided in Design Manual for Roads and Bridges Standard Specification for similar type of bridges.

Test Structure

Two almost identical adjacent bridges known as Exe North and South Bridges form a large roundabout spanning the River Exe in Exeter, UK. Exe North Bridge was chosen as a test structure. It is 59.35m long and consists of two 19.85m outer spans and a 19.61m centre span, resting on two wall type pier structures in the river and abutments at the ends. It was constructed in 1969 to replace the previous three hinged steel arch bridge.
The superstructure is 18.9m wide, carrying four lanes of traffic and connecting Okehampton Street (South) with Bonhay Road (North). It consists of 12 composite precast girders placed at 1.53m apart and 0.23m deep cast in situ reinforced concrete deck. The total depth of superstructure is 1m. The girder elements were designed as composite type, where steel beams were embedded in reinforced concrete I-girders. The steel beams are 762x267x197 mm universal beams with additional plates welded to the top and bottom flanges. Full composite action between steel and concrete girders is provided through double shear connectors, closely placed (125mm) at supports and gradually increasing towards the mid-span (500mm).

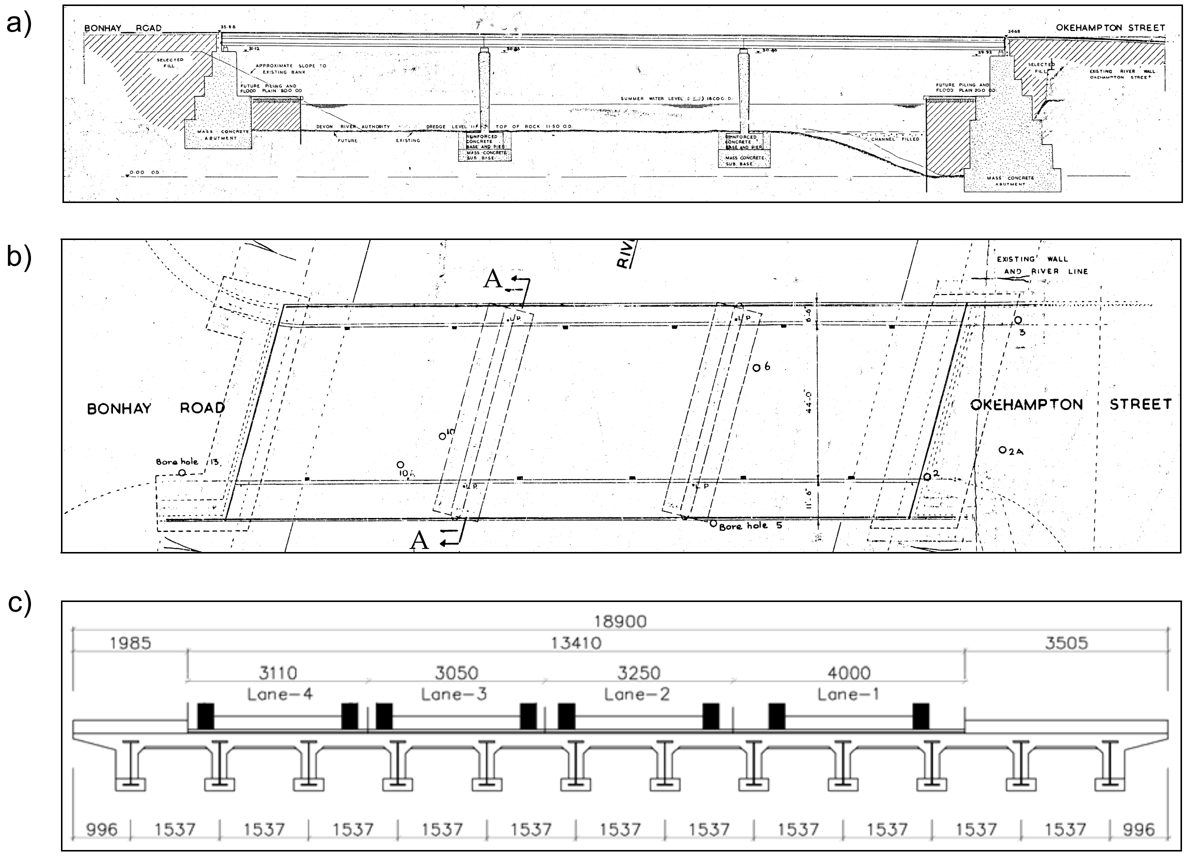

Figure 1: (a) Bridge elevation. (b) Plan view of Exe North Bridge (c) Cross section of superstructure.

The substructure consists of two wall type pier structures and two cantilever type abutments at 15 degree skew with respect to spans and parallel to the river bank. The connection between superstructure and substructure is provided with laminated elastomeric bearings designed to allow free span movement in the bridge longitudinal direction. Continuity between spans is cut off by 10 – 25mm wide gaps filled with bituminous rubber so that each span is simply supported.

In Figure 1, bridge longitudinal and plan views and deck cross-sections are shown. Figure 2 illustrates a picture of the bridge with structural characteristics of the bridge summarized in Table 1. Since the spans are not continuous, only the north span was chosen for testing purposes.

Figure 2: Exe North Bridge Spanning River Exe.

Table 1: Summary of the bridge structural characteristics

Total bridge length	59.35m
Number of spans	3
Span lengths	19.85m, 19.61m, 19.85m
Continuity	Simple supported
Skew angle	15 degrees
Deck type	composite I girders in situ RC deck
Deck width	18.9m
Number of lanes	4
Deck depth	1m
Substructure type	Cantilever type abutment and wall type pier
Bearing type	Laminated elastomeric bearing

Measurements

ST350 model strain transducers provided by a company named Bridge Diagnostics, Inc. (BDI) were used to measure the strains during the load. These are reusable Wheatstone full bridge resistive sensors encased in rugged transducer packages that are mounted on the structure with bolted tabs. The strain sensor itself is 76mm long. But the gage length of sensors is 0.6 m because aluminium extension rods are used, this is to account for local micro cracks associated in RC structures, and average strain values were recorded. Figure 3 shows the sensor installed on a girder soffit. These sensors were wired into three 4-channel nodes wirelessly linked to a host data acquisition system. The data were recorded with a sampling rate of 250 Hz. Based on gain, excitation and full-scale range of the sensors and software settings, 0.3 micro-strain resolution was determined for the measurement readings.

Figure 3: Strain Transducer attached on a girder soffit with aluminium extension rod.

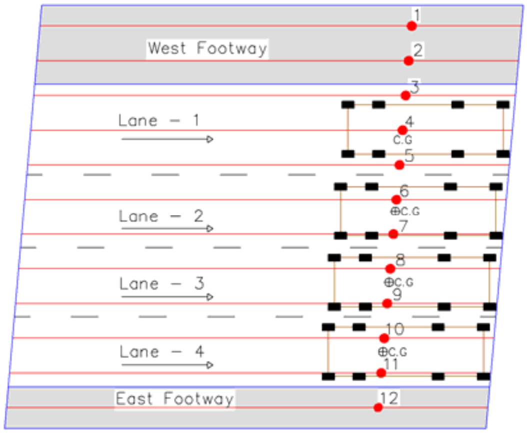

Figure 4: Sensor and vehicle location layout

The bridge spans over water, which made installation a difficult task. The only access to the deck soffit, avoiding working in water, was at quarter span, through the 5 m wide footpath along the river bank. Hence, the strain transducers were installed on each girder soffit at quarter span close to the North abutment. Gauges installed on each girder enable DFs to be calculated for each lane to indicate the load – shedding paths. The plan view of the sensor layout is provided in Figure 4. The beams are indicated as red lines and the sensors are labelled 1-12.

As a test vehicle, a four-axle, 32 tonne lorry was used to obtain quasi-static strain response. Figure 5(a) shows the truck during the load test. It has axle spacing of 1.35 m, 3.56 m and 1.94 m from rear to front. Figure 5(b) depicts the axle configurations and Table 1 tabulates weight for each axle. The truck made several passes in each of the four lanes (Lane 1, Lane 2, Lane 3, and Lane 4), stopping every time for 30-45 seconds to record the static strain. The front axle of the truck, while it was stationary, aligned approximately with the supports at the north abutment, with vehicle centre of gravity in line with sensor locations. Figure 4, illustrates the positioning of the vehicle in each of the four lanes. 16 passes were made in total and the test was performed overnight to avoid traffic on the bridge. In the load cases corresponding to Lane 1 and Lane 4, the exterior most wheel line is approximately 0.8 m and 0.3 m from the kerb, respectively.

Figure 6 illustrates a typical time series strain measurement recording during the test. The black plots in Figure 7 (a)-(d) show the average strain in each girder for truck positions in Lane 1-4 respectively.

a) b)

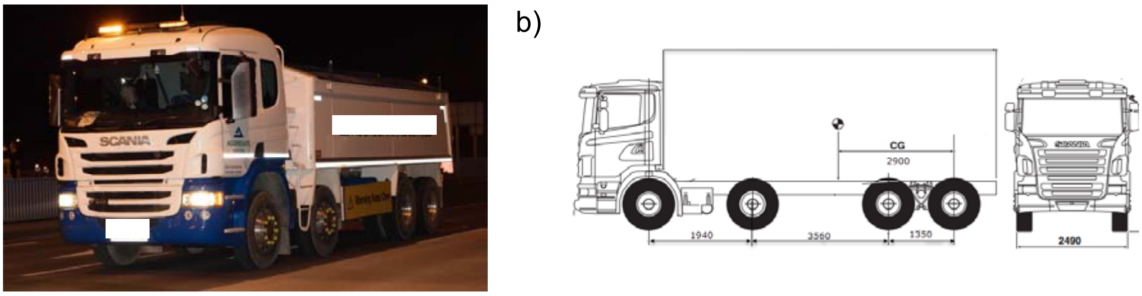

Figure 5: (a) Test vehicle in the first lane during load testing. (b) Axle configuration of test vehicle.

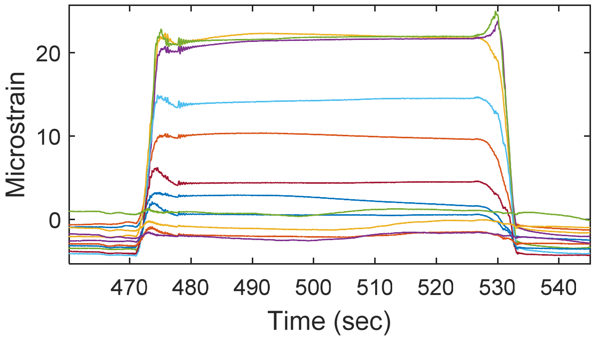

Figure 6: Typical strain recording during the test

Table 2: Axle weight configuration of test vehicle.

Truck Axles	Axle Weight (kN)
Axle – 1 (Rear)	89.8
Axle – 2	89.8
Axle – 3	67.2
Axle – 4 (Front)	67.2

a) b) c) d)

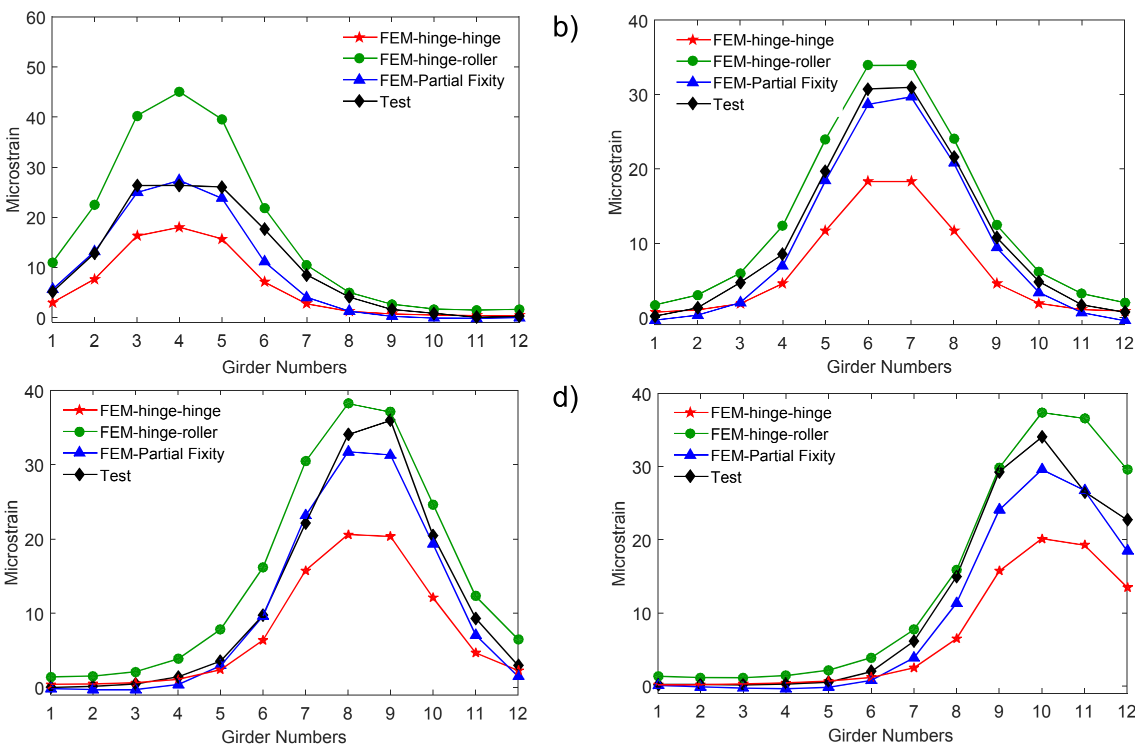

Figure 7: Strains obtained at the girder soffits during field test and from FE model (a) Lane 1 (b) Lane 2 (c) Lane 3 (d) Lane 4

Analytical Model of the Bridge

A 3-D FE model of the bridge was developed in ANSYS V16.0 software [6] according to available structural design drawings. The model includes all the necessary geometric details with composite structural configurations. The model was developed using SOLID185 elements to obtain reliable strains and accurate representation of steel concrete composite behaviour. Since the spans are non-continuous and independent, only the tested (North) span was considered during the modeling. Figure 8 illustrates developed 3-D FE model of the Exe North Bridge.
The bridge model represents of concrete I-girders, steel stringers with stiffening plates at top and bottom flanges and concrete kerbs. Each part of the model was developed separately in ANSYS native scripting language, with pats merged using the NUMMRG command to form the final Exe North Bridge FE model. The final model had to be sufficiently reliable to reproduce behaviour similar to that observed during the load test while being sufficiently versatile to be able to simulate different truck loading conditions over the bridge. This requirement dictated the need for a fine element mesh, so the proposed truck wheel locations would match with relevant nodes. Mesh size of 250 mm was eventually chosen. Mesh size verification analysis also confirmed the chosen mesh size to be reliable.

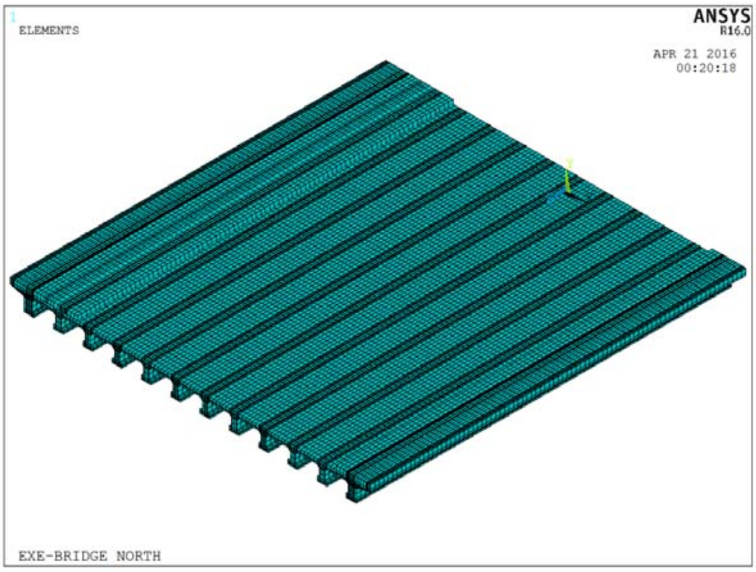

Figure 8: FE model of the North span.

Pier and abutment structures were excluded from the FE model as they are assumed to be infinitely rigid in axial directions. At each support location, the elastomeric bearings were represented in the analytical model by releasing the horizontal displacements. During the model calibration, different boundary conditions considered and these are described in detail in the following section. Many previous studies investigated the effect of skew angle in obtaining DF values and it has been demonstrated that skew has little effect (<1%) for an angle smaller than 20 degrees for this type of bridge [7, 8]. Therefore, the bridge was modeled without a skew angle.

Results and Discussions

Transverse load distribution functions (DFs) are a measure of the load transfer through the structure. Bridges are typically designed in such a way that traffic load is distributed between girders as “fairly” as possible so as not to overstress any particular load carrying member. Any change in bridge condition during its service life might significantly affect its load distribution characteristics. Therefore, obtaining DFs is of vital importance for any bridge assessment activity. Based on the results obtained from the load test and analytical model, stress based DFs were computed from the following equation.

$$DF_i = \frac{\sigma_i}{\sum \sigma_i} = \frac{E_i \varepsilon_i}{\sum E_i \varepsilon_i} = \frac{\varepsilon_i}{\sum \varepsilon_i},$$

Where:

σ_i = stress at soffit of girder i

E_i = Modulus of Elasticity of concrete

ε_i = strain measured at soffit of girder i

Modulus of elasticity values is assumed to be constant for all girders. Figure 9 illustrates the DFs obtained both from the test and the analytical model for each loading cases.

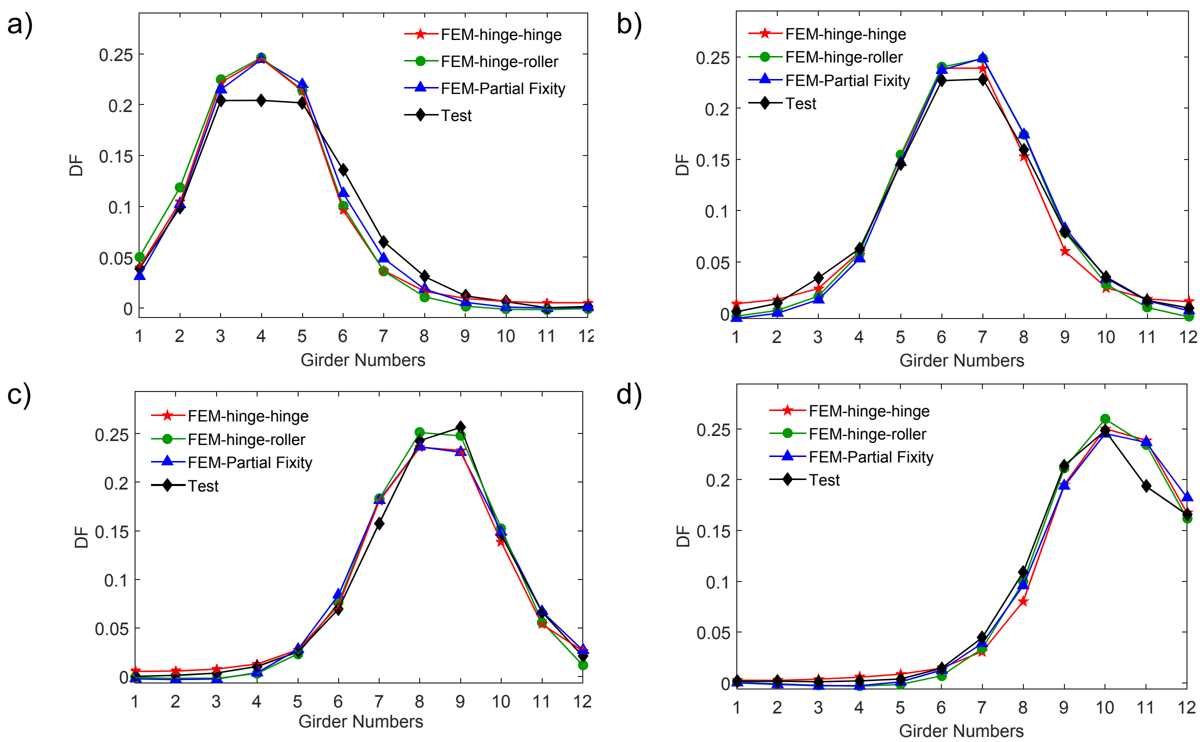

Figure 9: DFs obtained from field test and FE model
(a) Lane 1 (b) Lane 2 (c) Lane 3 (d) Lane 4

Once the strains and DFs for each girder were obtained based on the field test, similar loading cases were simulated using the FE model. Fine meshing made it possible to accurately locate the truck axle configuration at each lane. Several analyses were performed to study the current condition of the bridge and its behaviour under applied load.

Three different boundary conditions were studied to investigate the behaviour of the bridge deck. In the first case, the bridge support conditions were assigned as hinge at one end and roller at the other end (hinge-roller case), which is similar to initial assumption. In the second case, longitudinal movement of supports at both ends was restrained (hinge-hinge case). Figure 7 illustrates the strains obtained for the hinge-roller (green plot) and the hinge-hinge (red plot) cases. Results show that measured (test) strains lay between two limits of boundary conditions, hinge-hinge and hinge-roller, which implies that bridge boundary conditions are partially restrained. The third boundary condition case tried to simulate this partial restraint by taking the hinge-roller model and adding horizontal springs to the top and bottom flanges at the ends of the girder. The springs were modeled with springs using ANSYS COMBIN14 elements attached to top and bottom flanges of girders. Springs at bottom flanges represent elastomeric bearings whereas at top flanges they represent expansion joints. In reality the degree of partial fixity can vary from girder to girder and it is a difficult task to identify such differences accurately and apply the corresponding spring coefficient. In this study girders were grouped and by trial and error, suitable spring coefficients (K) were chosen. It was concluded that movement of elastomeric bearings in horizontal direction are partially fixed. It was found that bearings under girders 3-8 are more restrained than the others. This is not surprising as girders 3-5 corresponds with the bus lane which is more heavily loaded than the others. Also girders 6-8 are located at centreline of the roadway which are exposed to more loads due to typical load-shedding path between girders, which is described later in Figure 10. Figure 7 depicts both measured strain values from the field test and those calculated from the analytical model with different boundary conditions. Results obtained from the FE model with partially restrained boundary conditions are in good agreement with field test data. However, their accuracy could be enhanced further by applying more sophisticated automatic calibration techniques. The foregoing study clearly proves that changes in bearing conditions significantly reduce stress in bridge load carrying members hence increase its load carrying capacity. It simply demonstrates that field testing is an important topic in an effort to dealing with evaluation of aging bridge assets and it could reveal hidden strength reserves which current bridge inspection techniques fail to identify in usual cases.

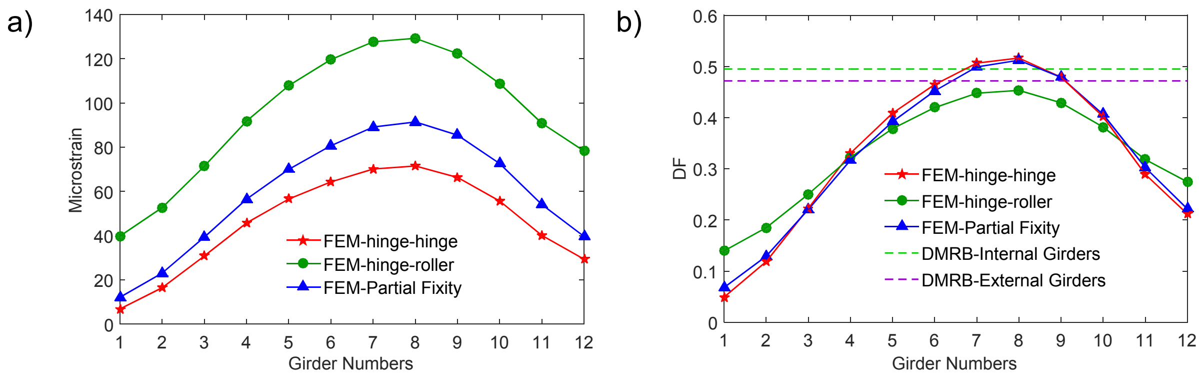

Figure 10: (a) Strains at midspan under full lane loading (b) DFs at midspan

Further analyses were carried out where the bridge FE model was loaded with an equivalent truck load at all lanes at midspan to obtain the relevant load distribution factor of the deck structure. Truck axle positions were located so that centre of gravity was in line with midspan location. Later results obtained from the FE model were compared with distribution factors provided in the Design Manual for Roads and Bridges (DMRB) standard specification [9]. Transverse load distribution factors (DFs) for bridge construction with concrete deck on precast I-girders are derived from relevant graphs provided in DMRB as 0.495 and 0.472 for internal and external girders, respectively. The distribution factors calculated from the DMRB depend on the spacing of the girders and the number of lanes on the bridge. In effect they represent what portion of the load (in a given lane) the critical girder carries, assuming all lanes are loaded equally. Figure 10 illustrates strains obtained for different boundary conditions and DFs calculated from both analytical models and calculations using DMRB for a heavy truck loaded in each lane. Results show that changes in bridge boundary conditions increase DFs but reduce the stress in load carrying members. The load is more uniformly distributed when the support condition is free to move in longitudinal direction (hinge-roller case). In addition, DFs provided in DMRB Standard Specification accurately represent the load shedding path for this particular bridge by only being 10% conservative.

Summary and Conclusions

A load test was conducted on the North Span of the Exe North Bridge in Exeter, UK where 12 strain transducers were attached to the soffit of the girders at quarter span location to record static strains due to a four-axle, 32 tonne truck. The results obtained from the test were used to calculate transverse load distribution factors. Numerical (finite element, FE) models of the bridge were developed and calibrated based on the test results. Three different boundary conditions were studied using calibrated FE models: (1) hinge-roller supports, (2) hinge-hinge supports and (3) partially fixed supports. Once the analytical model was verified to be accurate, loading equivalent to the codified loading condition was simulated in the FE model to calculate transverse load distribution factors (DFs) at midspan. Finally, obtained DFs were compared with DFs provided in Design Manual for Roads and Bridges Standard Specification. The following conclusions result from this study:

- FE modeling techniques applied in this study are reliable for modeling complex structures such as Exe North Bridge and can reliably represent real behaviour of the bridge under quasi static loading.
- Change in boundary condition (movement of bearings in longitudinal direction being partially or fully restrained) increases transverse load distribution factors (DFs) between girders. However, the stress in girder elements are reduced thus load carrying capacity of deck structure is increased.
- Load is more uniformly distributed when the support condition is free to move in longitudinal direction (as a hinge-roller).
- Finally, field testing is an important topic in an effort to dealing with evaluation of aging bridge assets, with capability to reveal its hidden strength reserves.

Acknowledgments

This research project has received funding from the European Union's Horizon 2020 research and innovation programme under the Marie Skłodowska– Curie grant agreement No. 642453. The authors would like to acknowledge the Devon County Council, in particular, Mark Colville for facilitating the field test. Additionally, the authors also wish to thank undergraduate student Nick Trump for his assistance during the field test and data analysis.

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