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Rationalising assessment approaches for masonry arch bridges

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Abstract

Masonry arch bridges, most of which have far exceeded modern design lives, have demonstrated themselves to be sustainable structures with low life-cycle costs. However increased traffic loading and material deterioration over time necessitate periodic reassessment of these structures. There are numerous different analytical methods available for the assessment of masonry arch bridges. The expectation is that for increasing levels of assessment complexity an increase in load capacity converging on the ultimate capacity would be achieved. In this paper it is demonstrated that this is not always the case. This has cost implications for both the bridge assessment itself and for costs associated with load restrictions and strengthening measures. Five different assessment methods were selected to assess a set of 11 single span bridges, ranging in span from 2.4 m – 15.2 m, with the objective of reviewing and rationalising current assessment guidelines for masonry arch bridges. The bridges chosen are a representative sample of the stone arch bridges on the Irish National Roads network. It was found that there is a significant variation in assessed capacity depending on the assessment method used. Limit state analysis methods were found to generally result in higher ratings for segmental bridges while elastic methods resulted in higher ratings for three-centred or semi-circular bridges. Assessment ratings found using the MEXE method were difficult to rationalise across the bridge set considered in this study. Following a review of the origins of the MEXE method and its current form as set out in the assessment guidance, it is recommended that its use as the predominant tool in a simplified assessment procedure is not appropriate and that a more rational approach is required for a more realistic and reliable calculation of bridge capacity. The development of an improved assessment methodology is being considered as part of the current study.

Keywords: Brickwork & masonry, bridges, codes of practice & standards

NOTATION

a	rise at centre span
d	ring thickness
h	depth of fill
f	compressive strength of masonry
L	span
LB	lower bound estimate of ring thickness
M_1	bending moment
M_u	ultimate bending moment capacity
P_1	axial force
P_u	ultimate axial force capacity
PAL	provisional axle load
UB	upper bound estimate of ring thickness
W_1	safe wheel load based on middle half rule
W_2	safe wheel load based on limiting compressive stress
W_A	safe axle load
σ	density of masonry and fill

1. INTRODUCTION

There are important arguments for a coherent strategy and set of guidelines for masonry arch bridges. Firstly, masonry arch bridges account for a significant proportion of the bridge stock both in Ireland and throughout Europe. They form an integral part of European transport infrastructure on the road, canal and rail networks. In Ireland there are an estimated 16,000 masonry arch bridges constituting approximately 80% of the national bridge stock (Molloy, 1988), and in Europe masonry arches are estimated to account for 40% of railway bridges

(Sustainable Bridges, 2007). In a survey carried out by the International Union of Railways (UIC), a total of 200,000 masonry arch bridges accounted for 60% of the bridges on the railway networks of the fourteen participating countries (Orbán, 2007). Outside of Europe, 18% of railway bridges in India are masonry arches (Orbán, 2007) and arch construction of various forms comprises 70% of all bridges in China (Xiang, 1993). In the United States, they are found in smaller numbers: Pennsylvania, for example, has 386 masonry arches (Senker, 2007) which are noted for their cultural and historical significance.

Secondly, there is a multitude of assessment methods available for masonry arch bridges including empirical methods (Department of Transport, 1997; Network Rail, 2006), limit analysis methods based on the formation of plastic hinges (Heyman, 1982; Crisfield and Packham, 1987; Falconer, 1987; Harvey, 1988; Gilbert and Melbourne, 1994; Hughes, 2002; Ng and Fairfield, 2004), methods based on Castigliano's energy theorems (Pippard, 1948; Bridle and Hughes, 1990; Brencich *et al.*, 2001; Wang and Melbourne, 2010), one dimensional finite element methods (Crisfield, 1984; Choo *et al.*, 1991; Brencich and De Francesco, 2004), two dimensional finite element methods (Woolfenden, 1993; Ng *et al.*, 1999; Boothby, 2001; Martín-Caro *et al.*, 2004; Cavicchi and Gambarotta, 2006; Robinson *et al.*, 2010), three dimensional finite element methods (Molins and Roca, 1998a; Fanning and Boothby, 2001; Frunzio, 2001), two dimensional and three dimensional discrete element methods (Ford *et al.*, 2003; Gifford and Partners, 2003; Drosopoulos *et al.*, 2006; Toth *et al.*, 2009; Kamiński, 2010) and discontinuous deformation analysis methods (Thavalingam *et al.*, 2001).

Finally, despite the prevalence of numerous methods there is no widely accepted framework for their application. For example, for highway loading in the UK the current guidance is set out by the Highways Agency in BD 21/01 (Department of Transport, 2001) and BA 16/97 (Department of Transport, 1997). In Ireland, where both of these documents have been adopted by the National Roads Authority (NRA), they are further supplemented by a Stage I Assessment Methodology Report (NRA, 2009).

BD 21/01 expresses a clear preference for the modified MEXE method stating "it shall be used wherever possible before any of the more complex methods ... are tried", with all other methods categorised simply as alternative methods. That said, BD 21/01 requires the use of an alternative method under a number of circumstances:

- where the bridge is outside the scope of the modified MEXE method. Limitations to the modified MEXE method relating to the span length, span to rise ratio and multiple spans are stated in BA 16/97;
- where the depth of fill is greater than the ring thickness. BD 21/01 states that "when the depth of fill at the crown is greater than the thickness of the arch barrel, the results shall again be confirmed using an alternative method. There is a possibility that for such cases the MEXE method may be unconservative". The Stage I Assessment Methodology Report (NRA, 2009) similarly states that under such circumstances the modified MEXE results should be corroborated using an alternative method;
- where the modified MEXE method indicates that the bridge is inadequate. However, it should be noted that the current guidance operates on the assumption that where a modified MEXE assessment is appropriate the results will always be always conservative, except where the depth of fill is greater than the ring thickness as noted above, with the ensuing guidance that even when an alternative assessment gives a lower capacity the higher modified MEXE assessment result shall take precedence: BD21/01 states that "when a bridge is found to have a lower capacity than that given

by the modified MEXE method, the MEXE assessment shall stand unless there is good reason to believe that it is unconservative for the case in question".

The Stage I Assessment Methodology Report (NRA, 2009) specifies that a modified MEXE assessment is to be carried out in accordance with BA 16/97. However the NRA methodology differs from BD21/01 and BA16/97 in that it provides more specific guidance on the type of assessment method to be used where an alternative assessment is required. Where arch geometry or arch conditions do not allow a MEXE analysis to be carried out, a three hinge limit analysis using the software package ARCHIE (Harvey 1988) is required. Additionally, where the depth of fill is greater than the ring thickness the NRA methodology requires that the MEXE result is corroborated using an ARCHIE analysis. This restricts the alternative assessment to a particular method but ensures a consistent assessment approach across the masonry arch bridge stock at the Stage I Assessment level.

The main requirements of BD 21/01 are summarised in the flowchart shown Figure 1. First, a check is carried out to determine whether the bridge parameters are within the limits for the modified MEXE method to be applicable. These include being within the span range of 1.5 m – 18 m, with a total crown thickness of 0.25 m – 1.8 m, a maximum span to rise ratio of 8, and an arch profile which is not appreciably deformed; otherwise an alternative assessment method must be used. If the bridge parameters are within the scope of the modified MEXE method a MEXE assessment is carried out. However if the depth of fill is greater than the ring thickness the MEXE result is said to be potentially unconservative and an alternative assessment is required for the purpose of confirming the modified MEXE result. Otherwise the modified MEXE result is assumed to be conservative. If the modified MEXE result is equal to the highest Gross Vehicle Weight (GVW) category of 40/44 tonnes the assessment procedure is then finished. Otherwise, an alternative assessment can be undertaken to see if a higher assessment rating can be achieved. If this alternative assessment result is higher than the modified MEXE method, the result from the alternative method can be taken for the bridge. However, if the result for the alternative assessment method is lower than the result for the modified MEXE method, the result from the modified MEXE method is deemed to still stand. This implies an inherent assumption in the assessment guidelines that the modified MEXE method, when used within its limits, can be taken as a conservative assessment approach.

Examining these requirements raises a number of issues. Firstly, where the depth of fill is greater than the ring thickness, indicating that the modified MEXE method may be unconservative, there would appear to be no benefit in carrying out a modified MEXE assessment. Secondly, where an alternative assessment method has produced a lower result than the modified MEXE method it is difficult to rationalise why the modified MEXE result should still be assumed to be conservative and take precedence. As the guidance on alternative methods is limited and the applicability of the MEXE method is open to question, the intent of this work was to examine the current assessment approach with a view to developing a more rational and reliable assessment methodology including a review of the origins and applicability of the MEXE method, an evaluation of available alternative analysis methods, their application to typical arch bridges found in Ireland and the development of an improved arch bridge assessment protocol.

In this paper five assessment methods, including the modified MEXE method, were selected to assess a set of eleven bridges characteristic of the Irish bridge stock with the objective of reviewing and rationalising current assessment guidelines for masonry arch bridges by examining the range of assessment ratings achieved from a variety of assessment approaches and the implications of these varying ratings for current assessment guidelines. The assessment methods that were chosen are simplified methods considered appropriate at preliminary stages of assessment and excluded the more complex and computationally demanding methods, e.g. nonlinear finite element methods, discrete element methods and discontinuous deformation analysis, which might be considered at later stages in an assessment hierarchy. Three dimensional finite element methods incorporating nonlinear material models can be used to model progressive cracking of the masonry, the interaction between the fill, arch barrel, spandrel walls and surrounding soil medium, accounting for the transverse capacity of the arch barrel, providing an accurate representation of the axle load distribution, providing diagnostic information in the presence of defects, capturing the abutment response and predicting the evolution of damage leading to failure of the complete structural system. However, these methods are technically challenging, require extensive material data and as such are not practical for the assessment of a large body of bridges. Nonlinear one dimensional finite element approaches (Crisfield, 1984; Choo et al., 1991) allow for the consideration of a cracked or crushed zones in the masonry by updating the geometry in a stepwise iterative procedure and can also control the ductility of the masonry by limiting the inelastic strains (Brencish and De Francesco, 2004). These methods do not consider the interaction between the arch barrel and fill, however they are much less computationally demanding than three dimensional finite element methods. Despite their relative simplicity even these one dimensional nonlinear models are not widely used.

2. ASSESSMENT METHODS

The assessment methods chosen for this study were the modified MEXE method, a three hinge limit analysis method, a rigid block limit analysis method, a two dimensional elastic analysis and a three dimensional elastic analysis. Each of the methods is described briefly below and further details can be found in the relevant references.

2.1. The modified MEXE method

The Military Engineering Experimental Establishment (MEXE) developed its empirical assessment method for masonry arch bridges (MEXE, 1963) based on the work of Pippard (1948). It is now found in its current format in BA 16/97 (Department of Transport, 1997) and is referred to as the modified MEXE method. Pippard (1948) is based on a two pinned parabolic arch, with a span to rise ratio of 4, loaded at mid-span. Pippard acknowledged that this was not the most onerous load position for an arch, but rather that it was the location for the least amount of load distribution and therefore the greatest concentration of load. The load was assumed to be distributed at a ratio of 1:1 and, as a simplification, the density of fill was assumed to be equal to the density of the masonry.

On the basis of these assumptions two equations for the safe wheel load, W_1 and W_2 were derived. The equation for W_1 , Equation 1 below, (where L is the span, a is the rise, h is the depth of fill, d is the ring thickness and σ is the density of the masonry and fill) was based on keeping the line of thrust within the middle half of the bridge. The middle half as opposed to the traditional middle third rule was chosen on the basis that the even a weak lime mortar would be capable of carrying some tension, a conclusion that was drawn following experiments carried out on 3.048 m (10 foot) span model arches jointed using lime mortar, cement mortar and also without mortar (Pippard and Ashby, 1939).

Equation 1

$$W_1 = \frac{32\sigma L h\left\{2a^2 + 4ad + 21d(h+d)\right\}}{21(28a - 25d)}$$

The second equation for the safe wheel load W_2 (Equation 2 below where f is the compressive strength of the masonry) was based on the compressive stress that would arise at the crown extrados. Initially Pippard (1948) then argued that the governing equation would be the lesser of the two values for W_1 and W_2, which would be dependent on the value chosen for the compressive strength of the masonry.

Equation 2

$$W_2 = \frac{\dfrac{256fhd}{L} - 128\sigma Lh\left(\dfrac{1}{21} + \dfrac{h+d}{4a} - \dfrac{a}{28d}\right)}{\dfrac{25}{a} + \dfrac{42}{d}}$$

However on choosing a compressive strength of 1.4 MPa (13 tons per square foot), on the basis of full-scale load tests which had been carried out by the Building Research Station during and prior to WWII, and which were later reported by Davey (1953), it was concluded that the equation based on the limiting compressive stress, W_2, should be taken as the governing equation, despite W_1 often yielding a lower value. This was justified by arguing that when taking W_2 as the governing case the maximum tensile stress produced was acceptable, which for the bridges considered was less than 0.7 MPa (100 pounds per square inch). In final versions of his work Pippard used a value of 2243kg/m^3 (0.0625 tons per cubic foot) for the density of the masonry and the load was assumed to be applied over a 0.305 m (1 foot) wide wheel base. Finally, to arrive at a safe axle load the safe wheel load was simply doubled, a valid assumption for the depths of fill considered in his paper. The safe axle load, W_A, given in Equation 3, is expressed in imperial units and is equivalent to that presented in

Pippard (1948); with the safe axle load in units of tons, and the span, ring thickness and depth of fill in units of feet.

Equation 3

$$W_A = 2\left[\frac{3330\frac{(h+0.5)d}{L} - 8L(h+0.5)\left(\frac{1}{21} + \frac{(h+0.5)+d}{L} - \frac{L}{112d}\right)}{\frac{100}{L} + \frac{42}{d}}\right]$$

Converting this to metric units gives Equation 4, with the safe axle load in units of tonnes, and the span, ring thickness and depth of fill in units of metres.

Equation 4

$$W_A = \frac{2}{9.81\mathrm{e}^3}\left[\frac{358.4\mathrm{e}^6\frac{(h+0.152)d}{L} - 2816\mathrm{e}^3 L(h+0.152)\left(\frac{1}{21} + \frac{(h+0.152)+d}{L} - \frac{L}{112d}\right)}{\frac{100}{L} + \frac{42}{d}}\right]$$

The modified MEXE method in its present format found in BA 16/97 (Department of Transport, 1997) centres around a nomogram relating the arch span and the total crown thickness to a provisional axle load (*PAL*) for a double axle bogie (tonnes). Alternatively, Equation 5 is provided with an upper limit of 70 tonnes, where the span, ring thickness and depth of fill are measured in metres.

Equation 5

$$PAL = \frac{740(d+h)^2}{L^{1.3}}$$

This is then modified by a number of factors intended to account for variations in bridge geometry and materials and also for defects and deterioration. There are also axle factors to allow for conversion from allowable double axle loads to single and triple axle loads.

Pippard's work in itself is coherent with appropriate arguments and justifications advanced for the assumptions made. However, and irrespective of the validity of Pippard's assumptions, there is a lack of full traceability between Pippard's referenced work and the modified MEXE method in its current format – this does not promote confidence in its results and makes it difficult to understand the various limits of applicability for MEXE set out in the guidance. Furthermore, provisional axle loads calculated from the modified MEXE method are significantly higher than those calculated from Pippard's equations - single axle loads determined from Pippard's equations and the modified MEXE method are plotted for varying depths of fill for a 12.192 m (40 foot) span bridge with a ring thickness of 0.457 m (1.5 feet) in Figure 2. The comparisons are equally inconsistent for all of the other dimensions considered in Pippard (1948).

Another anomaly is the sensitivity of the provisional axle load to the ring thickness. Pippard's method treats the ring thickness and the depth of fill as separate parameters while these are combined as a single parameter, the total crown thickness, in the modified MEXE method. To illustrate the effect of this, the resulting single axle loads for a constant total crown thickness of 0.914 m (3 feet) but with the ring thickness representing differing percentages of this total are plotted, for both approaches, in Figure 3. The modified MEXE result, being a function of the total crown thickness, is constant (irrespective of whether the ring constitutes 10% or 80% of this total crown thickness). On the other hand Pippard's original work is sensitive to the percentage of total thickness made up of the ring - as the percentage of the total crown thickness made up of ring thickness increases from 25% of the total crown thickness to 75% the axle load capacity increases, beyond this point, as the ring thickness increases further, the resulting decrease in the depth of fill limits the load distribution and increases the concentration of load, thereby reducing the load capacity. This is in contrast to the modified MEXE result which is constant, being a function of the total crown thickness.

2.2. Limit analysis methods

The two different limit analysis assessment methods used in this study are both based on the work of Heyman (1982). Heyman's method identifies the minimum thickness of arch required to contain a line of thrust associated with the formation of four plastic hinges, at which point the arch would be unstable. This thickness is then used to define a geometrical factor of safety relative to the actual arch thickness.

2.2.1. Three hinge limit analysis method

The three hinge limit analysis method developed by Harvey (1988) is currently available as the commercial software package ARCHIE-M. Harvey's method is based on Heyman (1982) but determines the line of thrust associated with the formation of only three plastic hinges. At this point a mechanism has not formed and the arch is still stable. As such, it can be considered a lower bound limit analysis method. If such a line of thrust, for a particular load, can be found within the boundaries of the arch ring a fourth hinge will not form and the available ring thickness, and hence the arch barrel, is deemed to be sufficient to resist the load being considered. If this line of thrust cannot be contained within the thickness of the arch ring the load is deemed to be unsafe.

2.2.2. Rigid block method

The rigid block method developed by Gilbert and Melbourne (1994) is currently available as the commercial software package RING 3.0. Similar to Heyman (1982) this method models the arch ring as a series of rigid blocks with frictional interfaces and therefore can also allow for sliding failure between the blocks. This rigid block method determines the load required for the formation of the four plastic hinges. A failure load factor, the ratio of load required for the formation of four hinges to the assessment load, is then used to determine whether or not a bridge is safe. Being based on the formation of four plastic hinges this method can be considered as an upper bound limit analysis method.

2.3. Elastic methods

2.3.1. Two dimensional elastic analysis

Boothby (2001) and Boothby and Fanning (2004) describe a 2D elastic analysis method based on a unit width plane frame model of the arch barrel. This method evolved from a series of in-service load tests and numerical models reported in Fanning and Boothby (2001). The arch is divided into a number of segments running along the centreline of the arch ring and modelled as straight beam elements. A single modulus of elasticity is assumed for the masonry mortar continuum. At the arch springing points the arch is taken to be fixed in the vertical direction and also against rotation. Spring supports are provided in the horizontal direction. This allows the model to account for movement where the abutments are not well founded. Live loads are distributed over a 3m width and a corresponding unit width load is determined. In the longitudinal direction, the live loads are applied over a 300mm wheel contact length and distributed longitudinally at a ratio of 1:2, horizontal to vertical. The fill is included in the model as a dead load only and hence does not contribute strength or stiffness to the arch beyond generating compressive stresses in the system.

This method is based on a defined ultimate limit state stress distribution at a cross-section. For a given masonry compressive strength a cross-sectional force-moment strength envelope can be derived (Boothby and Fanning, 2004). Any combination of force and moment at a cross-section within the strength envelope is deemed to be safe. The method also allows for the inclusion of a limited tensile capacity in the masonry mortar continuum. Further discussion on the inclusion of tensile strength for the masonry mortar continuum and validation of the modelling assumptions against service load testing can be found in Fanning *et al.* (2005) and Fanning and Boothby (2003). The effect of increasing the tensile capacity is to increase the range of moment and force combinations permissible, as shown in the strength envelopes determined from Boothby and Fanning (2004) in Figure 4 for a masonry arch with a compressive strength of 7.0 MPa, a ring thickness of 0.460 m and tensile capacities of 0 MPa and 5% of the compressive strength, i.e. 0.35MPa. The margin of safety is determined by the ratio of the distances from the origin of the strength envelope to (M_1, P_1) and (M_u, P_u) respectively, Figure 4, where M_1 and P_1 are the bending moment and axial force at a given cross section and M_u and P_u are the ultimate bending moment capacity and axial force capacity as defined by the boundary of the strength envelope.

2.3.2. Three dimensional elastic analysis

Two-dimensional assessment methods cannot fully account for transverse load dispersion, a significant feature of the response of arch bridges to loading. Equally they cannot assess the transverse capacity. Insufficient transverse capacity frequently manifests itself as longitudinal cracking, a common defect identified in visual inspections of many masonry arch bridges. With the aim of proposing a relatively simple assessment tool which can account for the transverse capacity of the arch without recourse to a full 3D model including the arch, spandrel walls, and soil-structure interaction with the fill and surrounding soil, the 2D elastic method in Boothby and Fanning (2004) was extended into a 3D elastic method with the arch barrel being modelled using shell elements. Consistent with the 2D approach the fill is again only included as a dead load and the spandrel walls are not included. The exclusion of the spandrel walls is considered to be prudent as it has been shown that spandrel wall separation may occur without any visual evidence at the barrel spandrel wall interface (Fanning *et al.*, 2005). Again, a single modulus of elasticity is taken for the masonry mortar continuum, the live loads are assumed to be distributed laterally over a 3m width and in the longitudinal direction at a ratio of 1:2, and the masonry arch is assigned a compressive strength and, if appropriate, a limited tensile capacity.

The advantages of this method are that by modelling the arch as a three dimensional shell the transverse dispersion of load is accounted for and the transverse capacity can also be assessed. In the presence of longitudinal cracking a 3D elastic approach should not be considered but otherwise an enhanced rating relative to the 2D elastic approach is expected.

3. BRIDGES ASSESSED

The National Roads Needs Study (NRA, 1998) identified and surveyed 646 masonry arch bridges on the National Primary and Secondary Routes in Ireland, the vast majority of which serve as river crossings. Analysis of this bridge set, Figure 5 and Figure 6, identified these bridges as predominantly short single span structures with 92% of spans being less than 12.5m and 63% of bridges having one span only. All of the masonry bridges documented on the national routes were of stone construction. Brick arches in Ireland are rare although some can be found on the rail network.

The eleven bridges selected for assessment were chosen so as to be representative of the largest proportion of bridges on the Irish road network. They are all single span structures of stone construction ranging in span from 2.4m to 15.2m.

Whilst the bridges assessed are representative of a large proportion of bridges on the Irish road network with a diverse range of spans, bridge shapes, ring thicknesses, depths of fill, span to rise ratios and masonry strengths, there are a number of arch bridge types that are not part of this subset of masonry arches, for example brickwork arches which are commonly encountered throughout Europe particularly on the railway network (Sustainable Bridges, 2007) and require consideration of debonding and ring separation between multiple rings (Gong et al., 1993; Melbourne and Gilbert, 1995; Melbourne et al., 2004; Sustainable Bridges, 2007). Additionally, all of the bridges selected are single span and as such the results do not address the complex issue of pier behaviour and the differing mechanism formation that occurs in multiple spans. The assessment of multiple spans as a series of single span assessments has been shown to overestimate the capacity of multiple span bridges and is not recommended (Hughes, 1995; Melbourne et al., 1997; Molins and Roca, 1998b; Ponniah and Prentice, 1998). Furthermore, shapes such as pointed segmental arches or obscure shapes such as the four-centred Tudor arch are not specifically considered.

3.1. Bridge profiles and geometry

The profile of these bridges varied between segmental, semi-circular and three-centred or semi-elliptical. The condition and state of repair also varied considerably. The bridge profiles and geometries are listed in Table 1. Four of the bridges selected for assessment are shown in Figures 7-10.

BD 21/01 (Department of Transport, 2001) requires axle loading to be applied within 2.5m transverse lane widths. For each bridge the minimum width of arch barrel available to support a single 2.5m lane was determined and is referred to as the "width for assessment" presented in Table 1. This value was dependant on the number and location of lanes, verges, parapets, longitudinal cracks etc. for each bridge. Backing material was not included in any of the assessments.

3.2. Material properties

The material properties were determined on the basis of visual inspections and are presented in Table 2. The values for the density of masonry were based on BS 648:1964 Schedule of Weights of Building Materials. The compressive strengths for the masonry were based on BD 21/01 Figure 4.3 Characteristic Strength of Normal Stone Masonry (Department of Transport, 2001). The values for tensile strength, Young's modulus and Poisson's ratio were based on the guidance provided in Boothby and Fanning (2004). For all of the bridges the tensile strength was limited to 5% of the compressive strength.

For the limit analysis methods the density of the masonry, the compressive strength of the masonry and the density of the fill are required. For the two elastic methods all of the material properties listed in Table 2 are required. The modified MEXE method is an empirical method and while modifying factors based on the bridge materials are used, specific material properties are not required. For the 2D and 3D elastic methods the abutments were fixed in the horizontal direction for consistency with the two limit state analysis approaches which assume fixed abutments.

4. ASSESSMENT PROCEDURE

The assessments were carried out in accordance with the current relevant guidance provided in BA 16/97 (Department of Transport, 1997) and BD 21/01 (Department of Transport, 2001). In addition the Stage I Assessment Methodology Report (NRA, 2009), which complements these guidance documents and is used in Ireland, was considered. It generally follows similar requirements to BD 21/01 and BA 16/97 but there are additional requirements, some of which are discussed below.

4.1. Inspection and site investigations

The Stage I Assessment Methodology Report (NRA, 2009) states that for a Preliminary Assessment of masonry arch bridges, unless site investigations are being carried out on an adjoining concrete extension, the inspection is generally limited to a visual inspection recording the relevant dimensions, the type and condition of the materials, the presence of defects and the general condition of the structure. Frequently for stone arch bridges, the voussoir stones at the fascia have a greater thickness than those across the rest of the arch. Therefore, where only a visual inspection has been carried out, the Stage I Assessment Methodology Report (NRA, 2009) requires that lower bound and upper bound estimates of the ring thickness are made. The lower bound (LB) estimate is taken as 0.6 of the thickness of the voussoirs at the fascia and the upper bound (UB) estimate is taken as the full fascia stone thickness. The resulting difference in ring thickness can be substantial; for example in the case of Glennagevlagh Bridge the resulting range is 0.276 m to 0.460 m. BA 16/97 provides no guidance in relation to variations in ring thickness across the width of the bridge and indicates that the thickness of the arch barrel should be measured adjacent to the keystone, adjusting this value only for defects such as mortar loss, loss of material due to the provision of services, ring separation and displaced voussoirs.

4.2. Loading and load factors

When masonry arch bridges are short in span they are assessed using axle load configurations. For masonry arch bridges greater than 20 m in span assessment is required for both HA loading and for axle loading. The axle configurations that are applied are for road legal vehicles as set out in The Road Vehicles (Authorised Weight) Regulations 1998 (House of Commons, 1998) for gross vehicle weights up to 40/44 tonnes. These are the common European legal vehicle weights as set out in Directive 96/53/EC.

For all of the assessment methods bar the modified MEXE method, the limit state approach and the use of partial factors is adopted. The modified MEXE method determines the allowable axle loads directly without the use of safety factors, and the factor of safety is deemed to be inherent in the process. For the assessments presented in this paper a double axle bogie (1.8m axle spacing) was considered for each bridge. The axle loads were factored in accordance with BD 21/01 which requires that a load factor of 1.9 is applied to all of the axles and a further impact factor of 1.8 is applied to the critical axle. The vertical road alignment for one of the bridges, Glanbehy Bridge, was humpbacked and therefore additional conversion factors were applied for axle lift-off for the double axle bogie. BD 21/01 requires that a conversion factor of 1.28 is applied to the critical axle and a conversion factor of 0.5 is applied to the other axle. The various partial factors that were applied are summarised in Table 3. For the modified MEXE method different axle lift-off factors which are dependent on the span length, specified in BA 16/97, were included as appropriate.

5. COMPARISON OF ASSESSMENT METHOD RESULTS

For the purposes of comparing the results from the various assessment methods the allowable axle loads per axle for a double axle bogey (1.8m axle spacing) are presented in Table 4 and also graphically in Figure 11 where they have been normalised with respect to the results from the 3D elastic method. There is significant variation in assessed capacity depending on the assessment method used and no one assessment method consistently produces either the highest or lowest axle load rating. While some level of variation is to be expected, given the differing engineering principles, analysis theories and inputs, on which the various methods are based, the degree of variation is often significant; the resulting maximum to minimum allowable axle load ratios vary between 1.54 and 4.52 (Table 4). The most striking feature is the number of bridges for which the modified MEXE method results in the highest rating – these are discussed in some detail below.

It is noted in BD 21/01 that there is a possibility that axle loads resulting from the modified MEXE method may be unconservative "when the depth of fill at the crown is greater than the thickness of the arch barrel". The four bridges in this category are marked with an asterisk in Table 4 and Figures 11 and 12 – for three of these bridges the highest rating is achieved with the modified MEXE method and relative to the next highest rating the modified MEXE assessment is between +20% and +72% higher. In cases where the depth of fill at the crown is greater than the thickness of the arch barrel the BD 21/01 requirement is that the modified MEXE assessment should be "confirmed using an alternative method". In Ireland the Stage I Assessment Methodology Report (NRA, 2009) also requires corroboration but specifies that the alternative method to be used shall be the three hinge limit analysis method using the software package ARCHIE. However, for the four bridge profiles where this was the case, Whistle Bridge LB, Whistle Bridge UB, Oghermong LB and Owenmore LB, the three hinge plastic method gave a lower result than the modified MEXE method and is therefore unable to confirm or support the modified MEXE result. Equally, with the exception of Whistle Bridge UB, the modified MEXE result could not be corroborated or confirmed with any of the other assessment methods either.

There are five bridge profiles for which the modified MEXE method gave higher results than all other assessment approaches – Glennanevlagh Bridge LB, Glennanevlagh Bridge UB, Whistle LB, Oghermong Bridge LB and Owenmore Bridge LB. Of these the latter three are cases where the modified MEXE result is acknowledged to be potentially unconservative due to the height of fill being greater than the ring thickness. However, the case of Glennanevlagh Bridge, which falls within the scope of modified MEXE method, is noteworthy. The allowable axle load varies between 23.5 tonnes (modified MEXE) and 5 tonnes (three hinge limit analysis) for a lower bound estimate of the ring thickness. In practice an assessing engineer, without knowledge of the result of an alternative assessment, which in this instance is not required, would have no basis to query the result of a modified MEXE assessment. Glennanevlagh Bridge, span 3.1 m, is a short span bridge and the conservatism of MEXE for short span bridges has been questioned previously in the literature (Harvey, 1988; Melbourne and Gilbert, 1995; McKibbins *et al.*, 2006; Sustainable Bridges, 2007; Melbourne *et al.*, 2009; Wang and Melbourne, 2010), but these reservations have yet to establish themselves in the assessment guidelines as currently specified.

The requirement in Irish practice to consider upper and lower bound ring thickness values raises the issue of sensitivity of assessment results to ring thickness. Four of the eleven bridges have been assessed with upper (UB) and lower (LB) bound ring thicknesses. The allowable axle loads for a double axle (1.8 m axle spacing), normalised with respect to the upper bound value of ring thickness for the 3D elastic method, for these bridges only, are plotted in Figure 12. For each method bar the modified MEXE method, there is a significant reduction in allowable axle load due to a reduction in ring thickness. The modified MEXE method is largely insensitive to arch ring thickness – this is not consistent with fundamental engineering principles. An arch carries load through a combination of compressive axial load and bending moments. Its capacity is sensitive to the ring thickness and indeed the original work (Pippard, 1948), discussed earlier, on which the modified MEXE method is based does reflect this sensitivity to arch ring thickness. However, as discussed, the modified MEXE method treats the total crown thickness, i.e. the combined thickness of the arch ring and the fill at the crown, as a single parameter in determining the provisional axle load for a bridge and therefore any variation in the percentage of the total crown thickness which is attributed to the ring thickness is not reflected in the resulting bridge capacity.

Anglesea Bridge, Figure 10, is a very shallow bridge with a span to rise ratio of 10. It was not assessed using the modified MEXE method due to its span to rise ratio being outside the scope of the method. It is also outside the span to rise limit (≤ 6) quoted for the rigid block method (LimitState 2011). Shallow bridges result in large horizontal thrusts and both the three hinge limit analysis and rigid block methods are based on fixed abutments – an assumption that is potentially not justifiable for large abutment thrusts. Logically this limitation should also apply to the three hinge limit analysis method. However for comparative purposes with the two elastic methods the allowable axle loads for Anglesea Bridge for all of the methods, bar the modified MEXE method, are reported in Table 4 and Figure 11. Given the above the assessment rating achieved using both the 2D and 3D elastic methods are sensibly lower than those arrived at using both the three hinge limit analysis method and the rigid block method.

6. DISCUSSION OF ASSESSMENT RESULTS

The five assessment methods considered can be broadly classified as empirical (the modified MEXE method), limit state analysis methods (three hinge limit analysis, rigid block analysis) and elastic (2D and 3D elastic methods). Within the latter two classes there is a rational hierarchy in the assessment methods in so far as the rigid block analysis approach (in which the load for the formation of four hinges is determined) yields a higher rating that the 3-hinge limit analysis, and the 3D elastic method (by virtue of representing the true dispersion and resistance of the arch barrel to load more completely) yields a higher rating than the 2D elastic analysis approach.

Currently the modified MEXE method sits at the core of the assessment guidelines as set out in BD 21/01. This is due mainly to its simple and straightforward application, as well as its long history of use. Notwithstanding this, the results from the modified MEXE method are difficult to rationalise. In its current form this approach does not fully reflect the work on which it based, for example it is sensitive to the total crown thickness rather than the arch barrel ring thickness, and its origins are not fully traceable to earlier work from which it is derived. Current guidelines for its implementation, as presented in Figure 1, state that MEXE assessment ratings for bridges may be unconservative if the height of fill is greater than the ring thickness. In the case of the bridges assessed in this paper this is demonstrated to be the case, relative to the four other assessment approaches. For the other bridges an alternative assessment is specified only if the modified MEXE method results in a rating that is less than 40/44 tonnes GVW. Furthermore if this is found to be the case and if the alternative approach results in a lower capacity, the restricted rating is based on the original modified MEXE assessment rather than the minimum of the two ratings. The clear implication is that the modified MEXE method is a conservative approach. The assessment results for Glennavelagh

Bridge, discussed above, show that, relative to other assessment methods, this is not necessarily the case. It is difficult to advocate continued use of the modified MEXE method in its current form. This may be an unpalatable finding for bridge owners and managers who understandably require an easily and efficient first level screening tool for their bridge populations.

In Figure 13 the assessment ratings for the modified MEXE method are excluded as are the results for the shallow profile Anglesea Bridge. Immediately there is more consistency with the assessment ratings across all bridges and changes in ring thickness are reflected in increased/reduced capacity as expected. Within assessment method classes the higher ratings are achieved, as expected, with either the rigid block method or the 3D elastic method than their counterparts. Furthermore the limit state analyses produce higher ratings for bridges whose profile reflects a line of thrust when axles are at the critical location near to the quarter span, while the elastic methods produce higher ratings for bridges with three-centred arch profiles or semi-circular profiles. The exception to this was Whistle Bridge, a segmental bridge of ashlar limestone construction to which a compressive strength of 14.2 MPa was assigned. In this case, the elastic methods produced higher results than the limit analysis methods despite the segmental profile. The line of thrust determined by limit analysis methods is dependent on the geometry of the bridge; and while limited by the compressive strength of the materials the stresses that are reached for the critical line of thrust are typically very low (Heyman, 1982). Hence these methods are less sensitive to the compressive strength of the masonry than the elastic methods which directly compare the compressive strength required to resist the applied load to the compressive strength of the material. In the case of Whistle Bridge the compressive strength was sufficiently high to yield greater results from the elastic methods than the limit analysis methods.

The results discussed above and illustrated in Figure 13 could provide the basis for the first stages of a hierarchical assessment framework which would aim to maximise the assessment rating achieved for each bridge. A strategy for assessment method selection is identified in Table 5 and is based on the shape, compressive strength and span to rise ratio associated with the arch. The assessment approach that produced the highest rating, limit analysis or elastic analysis, is identified for each bridge. In general limit analysis approaches are most appropriate for segmental bridges while elastic methods are more appropriate for semi-circular or three centred arches. However, the compressive strength has an impact on the optimum method for segmental shaped bridges. Where the compressive strength is high (greater than 10 MPa) the elastic analysis approach may give higher ratings. It is noted further that the usual limits of applicability for each of the individual methods should be considered, for example limit analysis approaches are not advisable for relatively flat profiles and indeed an upper bound on the span to rise ratio of 6 is advocated (LimitState, 2011). Also, limit analysis approaches assume rigid abutments and so may not be appropriate for flexibly sprung arches.

7. CONCLUSIONS

Eleven different bridge geometries and profiles have been assessed using five different assessment approaches with the objective of reviewing and rationalising current assessment guidelines for masonry arch bridges. The bridges chosen are representative of the largest majority of stone arch bridges on the Irish National Roads network.

It was found that it is difficult to rationally advocate the continued use of the modified MEXE method. The arguments for its continued use are primarily its ease of use and the implicit assumption that it is, within the specified limits of applicability, a conservative assessment approach. In the authors' opinion the lack of full traceability to the original work on which it is based, and its inability to capture the sensitivity of arch bridge capacity to the ring thickness alone far outweigh these positives. It is anticipated that discontinuation of its use would renew research vigour, interest and financial support for the identification of a new and more appropriate first level conservative screening approach for masonry arch bridge assessment.

Considering limit analysis and elastic methods alone, a more consistent set of ratings were achieved and limit analysis methods are advocated for bridges whose profiles match the expected line of thrust when the load is applied at its critical location, while elastic analysis methods are more appropriate for rating bridges that deviate from this profile, i.e. three-centred or semi-circular arches. Within these classes the four hinge rigid block limit analysis method resulted in higher ratings than the three hinge method, and the three dimensional elastic method produced higher ratings than the two dimensional elastic method. This is consistent with an expectation of convergence on actual bridge capacity as assessment approaches more completely capture the true bridge response.

The results presented are part of an ongoing research programme directed towards establishing a hierarchical framework of assessment algorithms for masonry arch bridges where increasing analysis effort is reflected in convergence to the expected bridge capacity. An initial attempt at defining this approach is presented. The next stage in the research programme is the quantification of the degree of conservatism associated with the limit analysis and elastic methods described by comparison with nonlinear three dimensional finite element models that include the fill material, surrounding soil-structure interaction and constitutive models that enable progressive cracking and failure of the complete structural system. It is intended that this would form the basis of a more realistic and reliable methodology for the assessment of arch bridges.

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REFERENCES

Boothby TE (2001) Load rating of masonry arch bridges. *Journal of Bridge Engineering*, **6**(2), 79-86.

Boothby TE and Fanning PJ (2004) Load rating of masonry arch bridges: Refinements. *Journal of Bridge Engineering*, **9**(3), 304-307.

Brencich A and De Francesco U (2004) Assessment of multispan masonry arch bridges. Part I: A simplified approach. *Journal of Bridge Engineering*, **9**(6), 582-590.

Brencich A, De Francesco U and Gambarotta L (2001) Elastic no tensile resistant-plastic analysis of masonry arch bridges as an extension of Castigliano's method. *Proc. 9th Canadian Masonry Symposium*, Fredericton, Canada.

Bridle RJ and Hughes TG (1990) An energy method for arch bridge analysis. *Proc. Inst. Civ. Eng.*, Part 2, **89**, 375-385.

British Standards Institution (1964) *BS 648:1964 Schedule of Weights of Building Materials*, London, British Standards Institution.

Cavicchi A and Gambarotta L (2006) Two-dimensional finite element upper bound limit analysis of masonry bridges. *Computers and Structures*, **84**, 2316-2328.

Choo BS, Coutie MG and Gong NG (1991) Finite-element analysis of masonry arch bridges using tapered elements. *Proc. Inst. Civ. Eng.*, Part 2, **91**, 755-770.

Crisfield MA (1984) *A finite element computer program for the analysis of masonry arches, Laboratory Report 1115*, Crowthorne, Transport and Road Research Laboratory.

Crisfield MA and Packham AJ (1987) *A mechanism program for computing the strength of masonry arch bridges, Research Report 124*, Crowthorne, Transport and Road Research Laboratory.

Davey N (1953) *Tests on Road Bridges, National Building Studies Research Paper No. 16*, London, HMSO.

Department of Transport (1997) BA 16/97 The Assessment of Highway Bridges and Structures. *Design Manual for Roads and Bridges*, Volume 3, Section 4, Part 4, London, HMSO.

Department of Transport (2001) BD 21/01 The Assessment of Highway Bridges and Structures. *Design Manual for Roads and Bridges*, Volume 3, Section 4, Part 3, London, HMSO.

Drosopoulos GA, Stavroulakis GE and Massalas CV (2006) Limit analysis of a single span masonry bridge with unilateral frictional contact interfaces. *Engineering Structures*, **28**, 1864-1873.

Duffy L (2004) Development of Eirspan: Ireland's bridge management system. *Proc. Inst. Civ. Eng. Bridge Engineering*, **157**(BE3), 139-146.

European Parliament and Council of the European Union. Directive 1996/53/EC of the European Union and of the Council of 25 July 1996 laying down for certain road vehicles circulating within the Community the maximum authorized dimensions in national and international traffic and the maximum authorized weights in international traffic. *Official Journal of the European Communities*, L235, 17.9.1996, 59–75.

Falconer RE (1987) The assessment of masonry arches using commercial software. *Proc. 3rd Int. Conf. on Structural Faults and Repair*, Forde MC (Ed.), London.

Fanning PJ and Boothby TE (2001) Three-dimensional modelling and full-scale testing of stone arch bridges. *Computers and Structures*, **79**(29-30), 2645-2662.

Fanning PJ and Boothby TE (2003) Experimentally-based assessment of masonry arch bridges. *Proc. Inst. Civ. Eng. Bridge Engineering*, **156**(BE3), 109-116.

Fanning PJ, Sobczak L, Boothby TE and Salamoni V (2005) Load testing and model simulations for a stone arch bridge. *Bridge Structures*, **1**(4), 367-378.

Ford TE, Augarde CE and Tuxford SS (2003) Modelling masonry arch bridges using commercial finite element software. *Proc. 9th Int. Conf. on Civil and Structural Engineering Computing*, Egmond aan Zee, Netherlands.

Gifford and Partners (2003) *ARCHTEC Verification of Structural Analysis, Report No. B1660A/V10/R03*, Southampton, Gifford and Partners.

Gilbert M and Melbourne C (1994) Rigid block analysis of masonry structures. *The Structural Engineer*, **72**(21), 356-361.

Gong, NG, Choo, BS, and Coutie, MG (1993) Crack and contact problems in masonry arch bridges. *Proc. 5th Int. Conf. Computing in Civil and Building Engineering*, 801-808, New York, ASCE.

Harvey WJ (1988) Application of the mechanism analysis to masonry arches. *The Structural Engineer*, **66**(5), 77-84.

Heyman J (1982) *The Masonry Arch*, Chichester, Ellis Horwood.

House of Commons (1998) *The Road Vehicles (Authorised Weight) Regulations 1998*. SI 1998/ 3111, London, HMSO.

Hughes TG (1995) Analysis and assessment of twin-span masonry arch bridges. *Proc. Inst. Civ. Eng. Structures and Buildings* **110**, 373-382.

Hughes TG, Hee SC and Soms E (2002) Mechanism analysis of single span masonry arch bridges using a spreadsheet. *Proc. Inst. Civ. Eng. Structures and Buildings* **152**(4), 341-350.

Kamiński T (2010) Tests to collapse of masonry arch bridges simulated by means of FEM. *Proc. 5th Int. Conf. on Bridge Maintenance, Safety, Management and Life-Cycle Optimization*, Frangopol DM, Sause R and Kusko CS (Eds.), 1420-1427, London.

LimitState (2011) *LimitState:RING Manual, Version 3.0*, Sheffield, Limit State Ltd.

Martín-Caro JA, Martínez JL and León J (2004) A first level structural analysis tool for the Spanish railways masonry arch bridges. *Proc. Arch Bridge IV, Advances in Assessment, Structural Design and Construction*, Roca P and Molins C (Eds.), 192-201, Barcelona.

McKibbins LD, Melbourne C, Sawar N and Sicilia Gaillard C (2006) *Masonry arch bridges: condition appraisal and remedial treatment, CIRIA C656*, London, CIRIA.

Melbourne C and Gilbert M (1995) The behaviour of multiring brickwork arch bridges. *The Structural Engineer*, **73**(3), 39-47.

Melbourne C, Gilbert M and Wagstaff M (1997) The collapse behaviour of multispan brickwork arch bridges. *The Structural Engineer,* **75**(17), 297-305.

Melbourne C, Tomor AK and Wang J (2004) Cyclic load capacity and endurance limit of multi-ring masonry arches. *Proc. Arch Bridges IV, Advances in Assessment, Structural Design and Construction*, Roca P and Molins C (Eds.), 375–384, Barcelona.

Melbourne C, Wang J, Gilbert M and Smith C (2009) *Review of Assessment Methods for Masonry Arch Highway Bridges*, CSS Masonry Arch Research Project Final Draft Report.

Military Engineering Experimental Establishment (1963) *Military Load Classification of Civil Bridges by the Reconnaissance and Correlation Methods, Solog Study B38,* Christchurch, Signals Research and Development Establishment.

Molins C and Roca P (1998a) Capacity of masonry arches and spatial frames. *Journal of Structural Engineering*, **124**(6), 653-663.

Molins C and Roca P (1998b) Load capacity of multi-arch masonry bridges. *Arch Bridges*, Sinopoli A (Ed.), 213-222, Rotterdam, Balkema.

Molloy E (1988) *Report on inspection, assessment and rehabilitation of masonry arch bridges*, Dublin, Department of the Environment.

National Roads Authority (1998) *National Road Needs Study Final Report*, Dublin, National Roads Authority.

National Roads Authority (2009) *Stage I Assessment Methodology Report*, Dublin, National Roads Authority.

Network Rail (2006) *The Structural Assessment of Underbridges, Guidance Note NR/GN/CIV/025*, London, Network Rail.

Ng KH and Fairfield CA (2004) Modifying the mechanism method of masonry arch bridge analysis. *Construction and Building Materials*, **18**, 91-97.

Ng KH, Fairfield CA and Sibbald A (1999) Finite-element analysis of masonry arch bridges. *Proc. Inst. Civ. Eng. Structures and Buildings*, **134**, 119-127.

Orbán Z (2007) UIC Project on assessment, inspection and maintenance of masonry arch railway bridges. *Proc. 5th Int. Conf. on Arch Bridges,* Lourenço PB, Oliveira D and Portela V (Eds.), 3-12, Madeira.

Pippard, AJS (1948) The Approximate Estimation of Safe Loads on Masonry Bridges. *The Civil Engineer in War*, **1**, 365-372.

Pippard AJS and Ashby RJ (1939) An Experimental Study of the Voussoir Arch. *Journal Inst. Civ. Eng.*, **10**, 383-405.

Ponniah DA and Prentice DJ (1998) Load-carrying capacity of masonry arch bridges estimated from multi-span model tests. *Proc. Inst. Civ. Eng. Structures and Buildings* **128**, 81-90.

Robinson DJ, Johnston R and Taylor SE (2010) Finite element analysis of a skewed flexi-arch. *Proc. BCRI Joint Symposium*, 505-512, Cork.

Senker JH (2007) Pennsylvania stone arch bridges: Evaluation and assessment. *Proc. 5th Int. Conf. on Arch Bridges*, Lourenço PB, Oliveira D and Portela V (Eds.), 257-265, Madeira.

Sustainable Bridges (2007) *Masonry Arch Bridges Background document D4.7*, Sustainable Bridges, EU FP6.

Thavalingam A, Bicancic N, Robinson JI and Ponniah DA (2001) Computational framework for discontinuous modelling of masonry arch bridges. *Computers and Structures*, **79**, 1821-1830.

Tóth AR, Orbán Z and Bagi K (2009) Discrete element analysis of a stone masonry arch. *Mechanics Research Communications*, **36**, 469-480.

Wang J and Melbourne C (2010) Mechanics of MEXE method for masonry arch bridge assessment. *Proc. Inst. Civ. Eng. Eng. and Comp. Mech.*, **163**(EM3), 187-202.

Woolfenden PA (1993) Modelling the Masonry Arch. Improving Modern Bridge Assessment using Non-Linear Finite-Element Software Package (MAFEA). *Proc. 2nd Int. Conf. on Bridge Management*, 254-263, Guildford.

Xiang H (1993) *Bridges in China*, Shanghai, Tongji University Press.

Table 1
Bridge geometries

Name	Shape	Span (m)	Rise (m)	Span/rise	Ring thickness (m)	Depth fill (m)	Width for assessment (m)
Temple	Segmental	3.0	0.68	4.4	0.380	0.050	6.53
Oghermong LB	Segmental	7.8	2.00	3.9	0.270	0.400	3.60
Oghermong UB	Segmental	7.8	2.00	3.9	0.550	0.120	3.60
Owenmore LB	Segmental	8.6	2.28	3.8	0.264	0.496	3.82
Owenmore UB	Segmental	8.6	2.28	3.8	0.440	0.320	3.82
Glanbehy	Segmental	13.4	3.40	3.9	0.625	0.150	6.40
Windy	Segmental	10.7	1.97	5.4	0.670	0.300	4.05
Whistle LB	Segmental	6.2	1.31	4.8	0.255	0.655	3.65
Whistle UB	Segmental	6.2	1.31	4.8	0.435	0.475	3.65
Anglesea	Segmental	15.2	1.53	10.0	0.800	0.300	3.12
Glennagevlagh LB	Semi-circular	3.1	1.53	2.0	0.276	0.234	3.60
Glennagevlagh UB	Semi-circular	3.1	1.53	2.0	0.460	0.050	3.60
Glanlough	Semi-circular	2.4	0.94	2.6	0.490	0.100	6.40
Killeen	Three-centred	9.3	2.65	3.5	0.480	0.250	3.15
Griffith	Three-centred	9.5	2.71	3.5	0.446	0.126	3.92

Table 2
Material properties

Name	Masonry					Fill
	Compressive strength (MPa)	Tensile strength (MPa)	Density (kN/m³)	Young's modulus (GPa)	Poisson's ratio	Density (kN/m³)
Temple	4.5	0.23	22.8	3	0.3	18
Oghermong LB	4.5	0.23	22.8	3	0.3	18
Oghermong UB	4.5	0.23	22.8	3	0.3	18
Owenmore LB	7.0	0.35	22.0	4	0.3	18
Owenmore UB	7.0	0.35	22.0	4	0.3	18
Glanbehy	7.0	0.35	22.0	4	0.3	18
Windy	10.5	0.53	22.0	5	0.3	18
Whistle LB	14.2	0.71	22.0	13	0.3	18
Whistle UB	14.2	0.71	22.0	13	0.3	18
Anglesea	14.2	0.71	22.0	13	0.3	18
Glennagevlagh LB	7.0	0.35	22.0	4	0.3	18
Glennagevlagh UB	7.0	0.35	22.0	4	0.3	18
Glanlough	7.6	0.38	22.8	4	0.3	18
Killeen	15.0	0.75	22.0	10	0.3	17
Griffith	15.0	0.75	22.0	10	0.3	17

Table 3
Partial load factors applied to axles

	Critical axle	Other axle
Single axle	3.4	–
Double axle	3.4	1.9
Double axle with axle lift-off	4.35	0.95

Table 4
Allowable axle load per axle for a double axle with 1.8 m spacing (tonnes)

Name	MEXE	Three hinge	Rigid block	2D elastic	3D elastic	Max:Min ratio	
						Incl. MEXE	Excl. MEXE
Temple	18.5	23.3	28.5	22.0	23.5	1.54	1.30
Oghermong LB	12.0*	5.1	8.0	4.0	5.0	3.00	2.00
Oghermong UB	13.5	16.2	21.5	16.0	19.0	1.59	1.34
Owenmore LB	15.0*	4.2	12.5	5.0	6.0	3.57	2.98
Owenmore UB	16.5	10.3	23.2	13.5	16.5	2.25	2.25
Glanbehy	8.5	15.7	20.1	13.0	17.0	2.36	1.55
Windy	18.0	35.7	43.4	25.0	40.5	2.41	1.74
Whistle LB	25.0*	8.6	11.2	12.5	14.5	2.91	1.69
Whistle UB	27.0*	22.5	28.9	37.0	51.0	2.27	2.27
Anglesea	–	50.0	77.6	19.0	21.0	4.08	4.08
Glennagevlagh LB	23.5	5.2	6.8	7.0	8.0	4.52	1.54
Glennagevlagh UB	26.5	10.9	13.0	23.0	23.0	2.43	2.11
Glanlough	43.5	30.1	34.7	65.0	74.0	2.46	2.46
Killeen	7.5	3.9	7.3	5.5	9.0	2.31	2.31
Griffith	7.5	5.2	8.3	9.0	12.0	2.31	2.31

* MEXE result may be unconservative
– Profile too flat for MEXE assessment

Table 5
Analysis approach giving highest assessment rating excluding MEXE

Name	Shape	Masonry compressive strength > 10 MPa	Span/rise > 6	Limit analysis	Elastic analysis
Temple	Segmental			x	
Oghermong LB	Segmental			x	
Oghermong UB	Segmental			x	
Owenmore LB	Segmental			x	
Owenmore UB	Segmental			x	
Glanbehy	Segmental			x	
Windy	Segmental	yes		x	
Whistle LB	Segmental	yes			x
Whistle UB	Segmental	yes			x
Anglesea	Segmental	yes	yes		x
Glennagevlagh LB	Semi-circular				x
Glennagevlagh UB	Semi-circular				x
Glanlough	Semi-circular				x
Killeen	Three-centred	yes			x
Griffith	Three-centred	yes			x

List of Figures

Figure 1

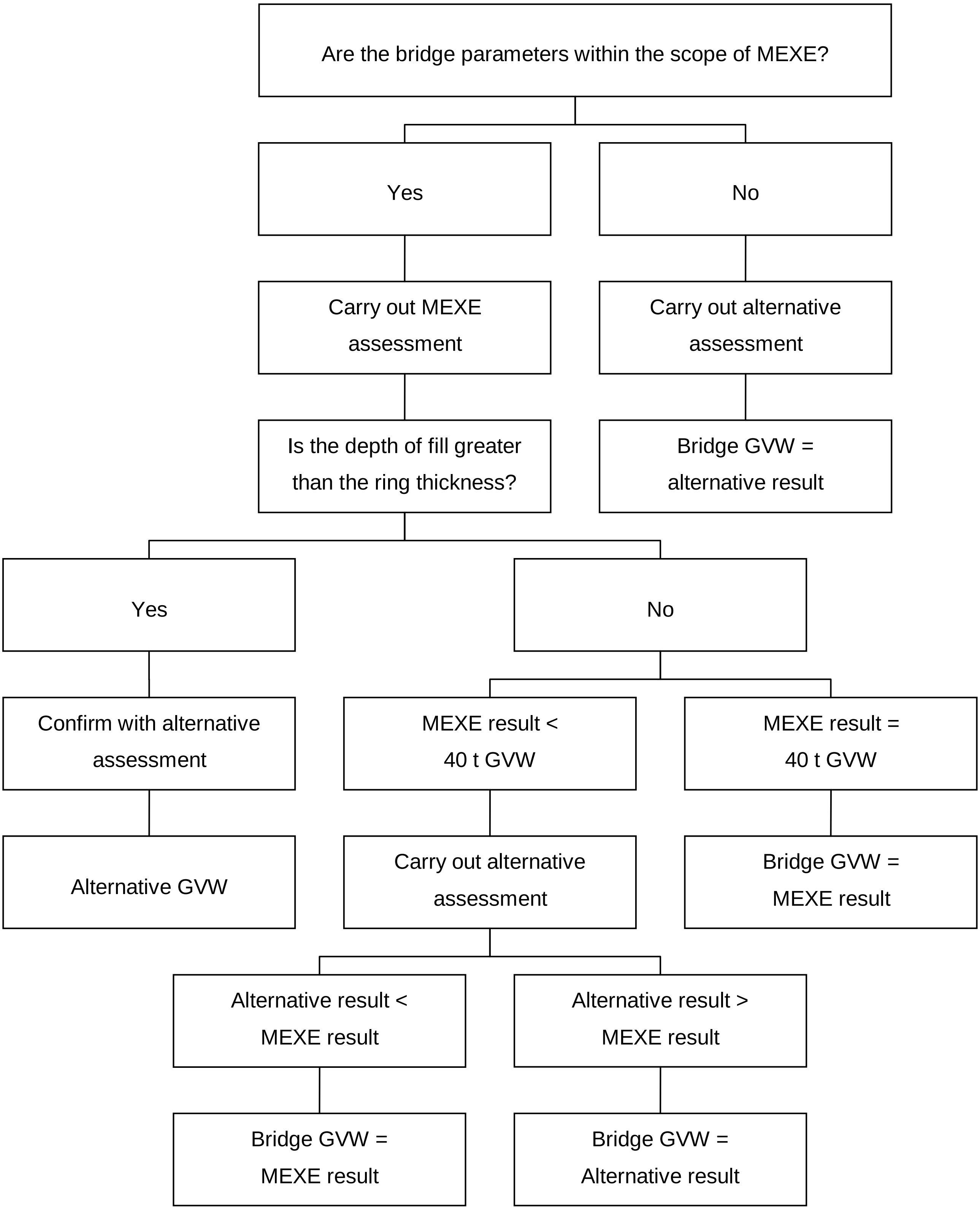
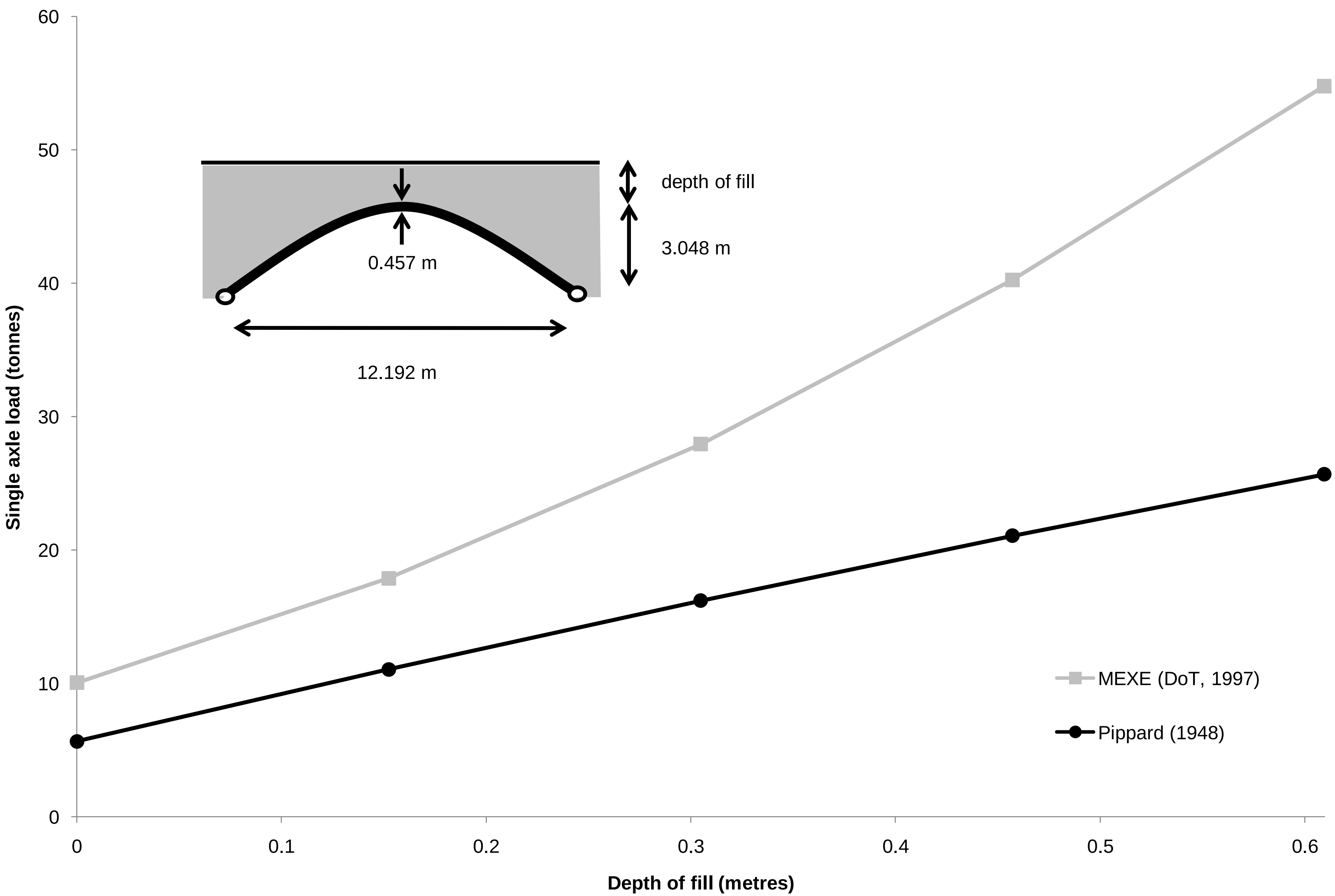

Are the bridge parameters within the scope of MEXE?
Yes
No
Carry out MEXE assessment
Carry out alternative assessment
Is the depth of fill greater than the ring thickness?
Bridge GVW = alternative result
Yes
No
Confirm with alternative assessment
MEXE result < 40 t GVW
MEXE result = 40 t GVW
Alternative GVW
Carry out alternative assessment
Bridge GVW = MEXE result
Alternative result < MEXE result
Alternative result > MEXE result
Bridge GVW = MEXE result
Bridge GVW = Alternative result

Figure 2



depth of fill
3.048 m
0.457 m
12.192 m
60
50
40
30
20
10
0
Single axle load (tonnes)
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth of fill (metres)
MEXE (DoT, 1997)
Pippard (1948)

Figure 3

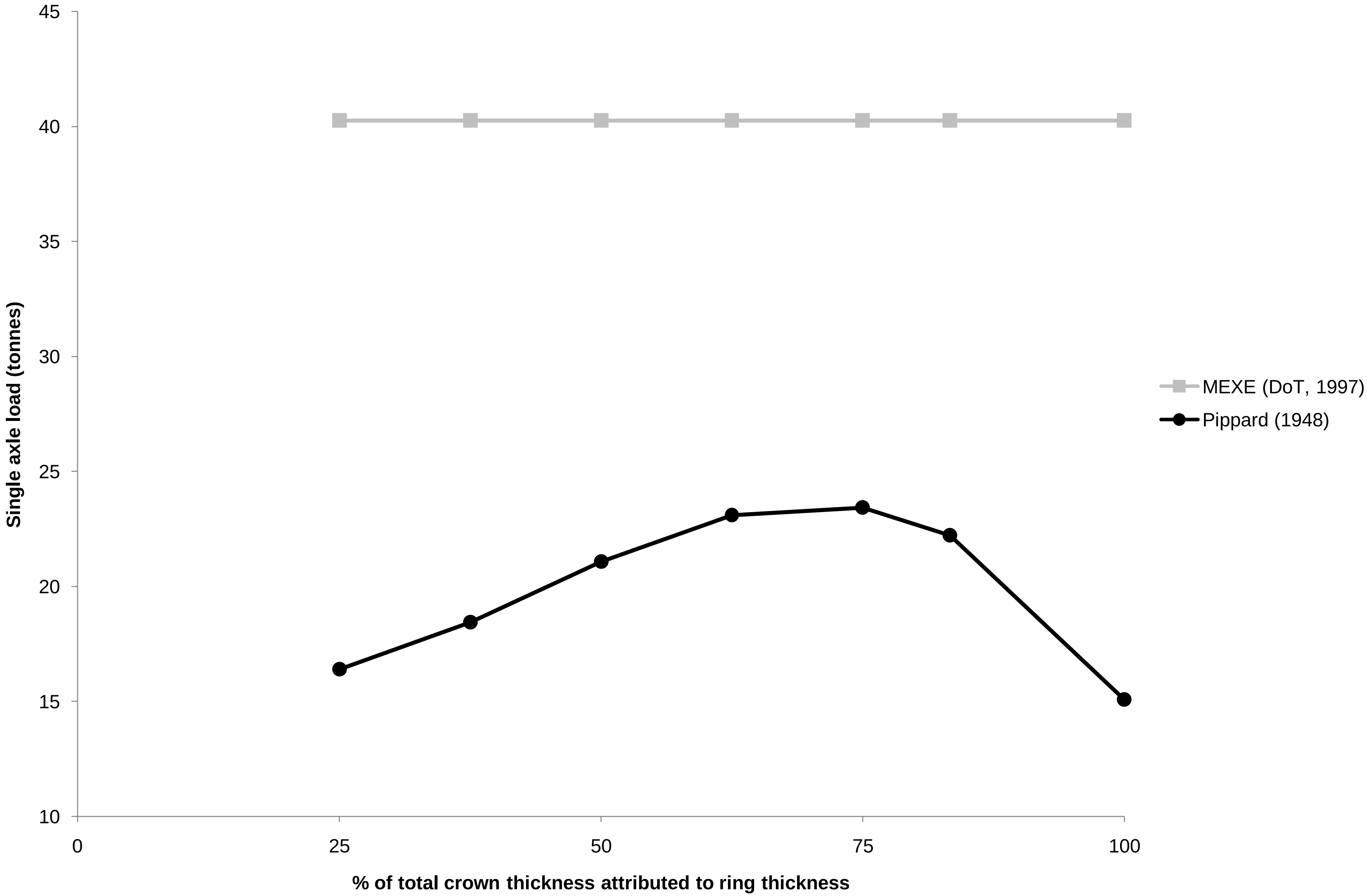
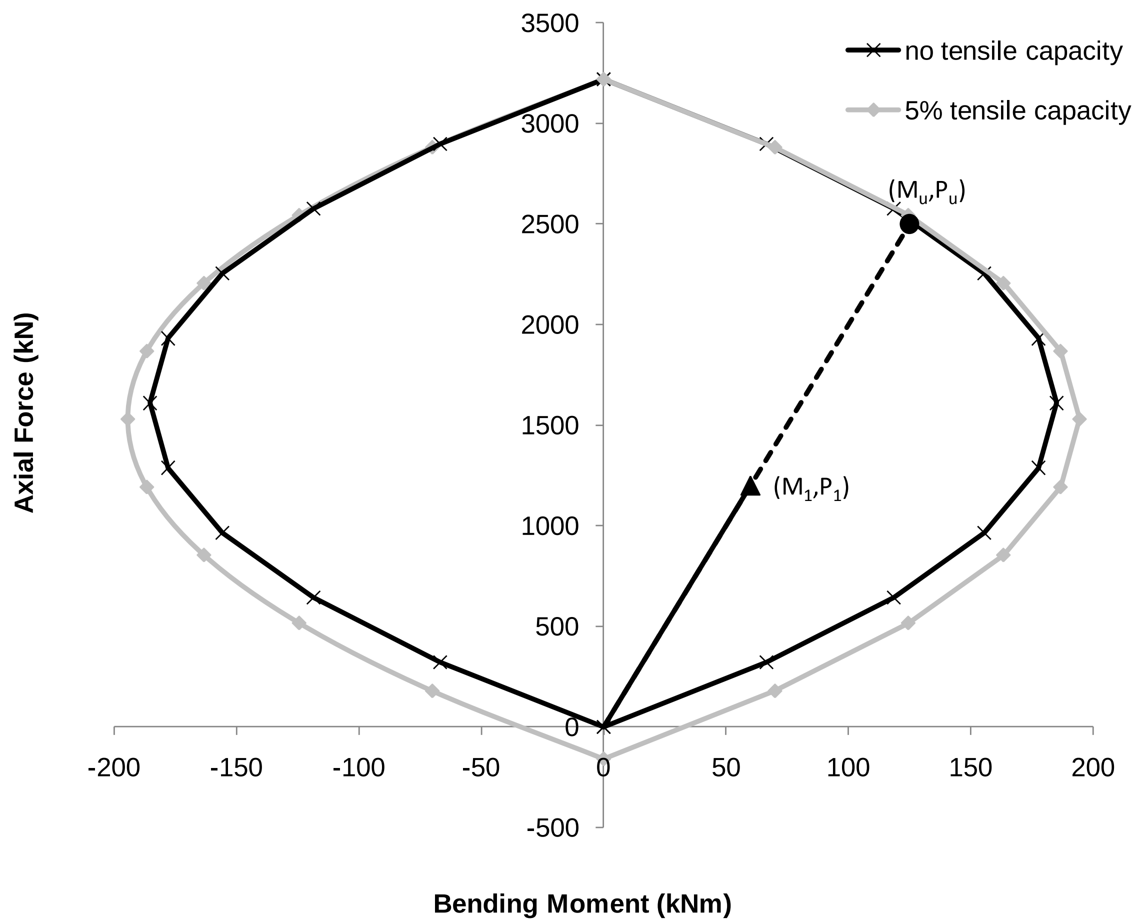
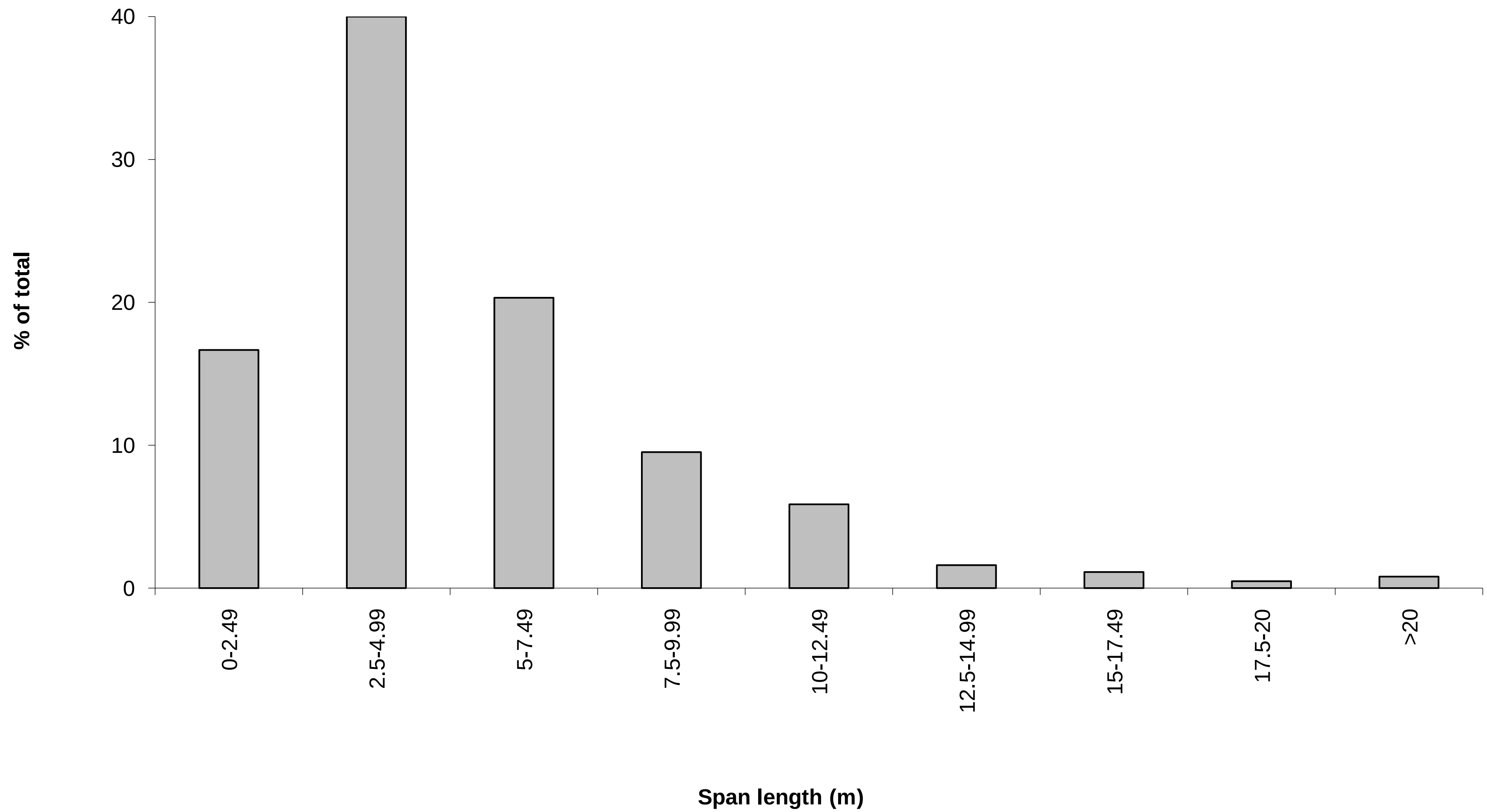

45
40
35
30
25
20
15
10
Single axle load (tonnes)
0
25
50
75
100
% of total crown thickness attributed to ring thickness
MEXE (DoT, 1997)
Pippard (1948)

Figure 4



3500
3000
2500
2000
1500
1000
500
0
-500
-200
-150
-100
-50
0
50
100
150
200
no tensile capacity
5% tensile capacity
(M_u,P_u)
(M_1,P_1)
Axial Force (kN)
Bending Moment (kNm)

Figure 5



40
30
20
10
0
% of total
0-2.49
2.5-4.99
5-7.49
7.5-9.99
10-12.49
12.5-14.99
15-17.49
17.5-20
>20
Span length (m)

Figure 6

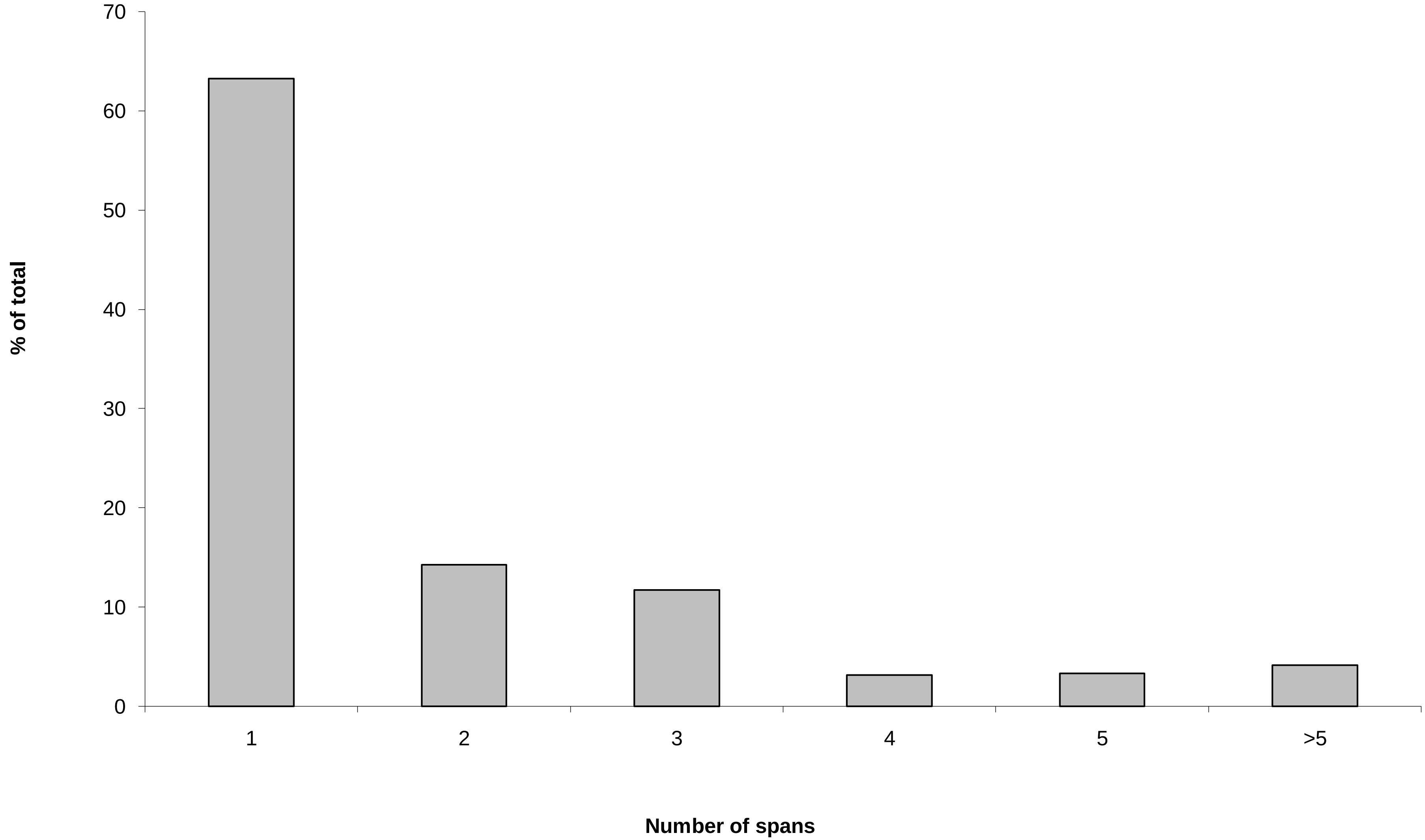


Figure 7
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Figure 8
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Figure 9
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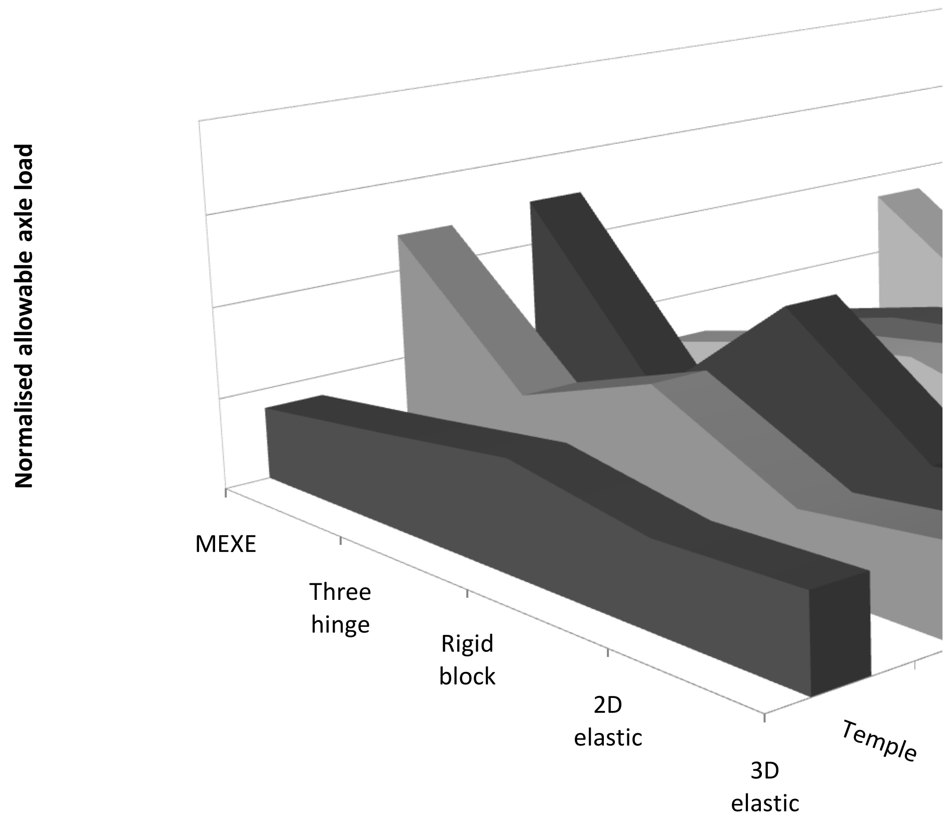
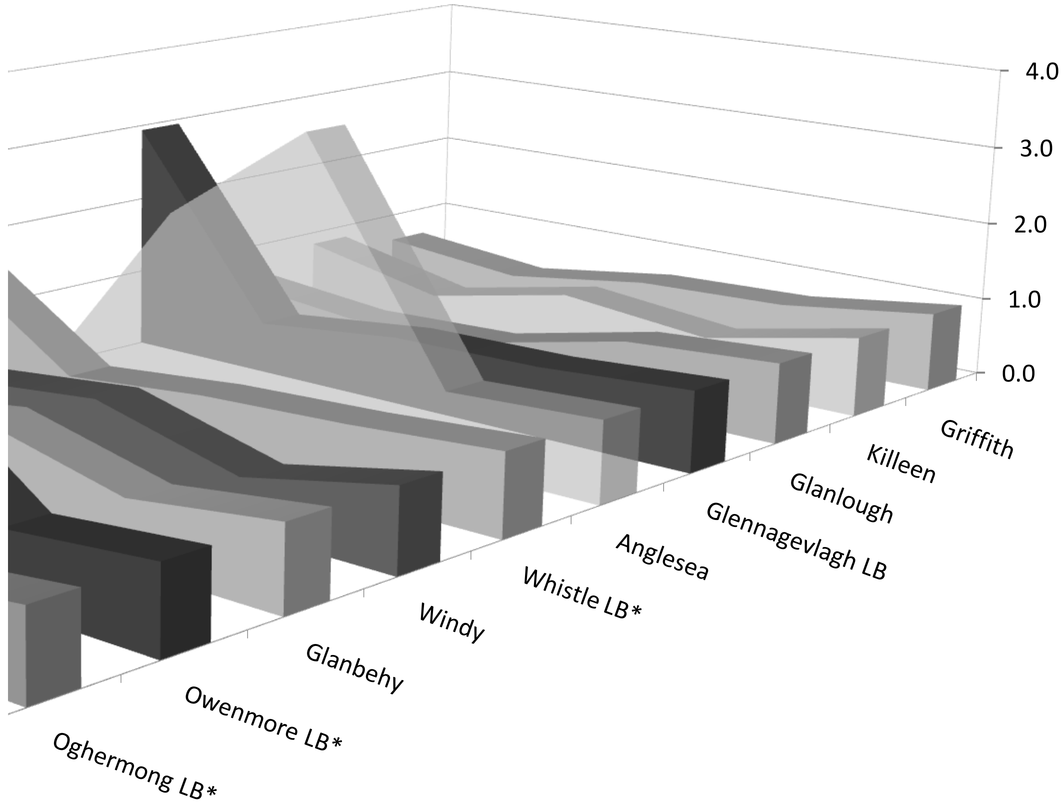
Figure 10
Click here to download high resolution image

Figure 11

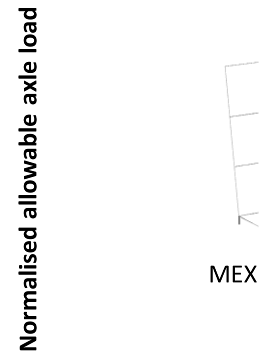


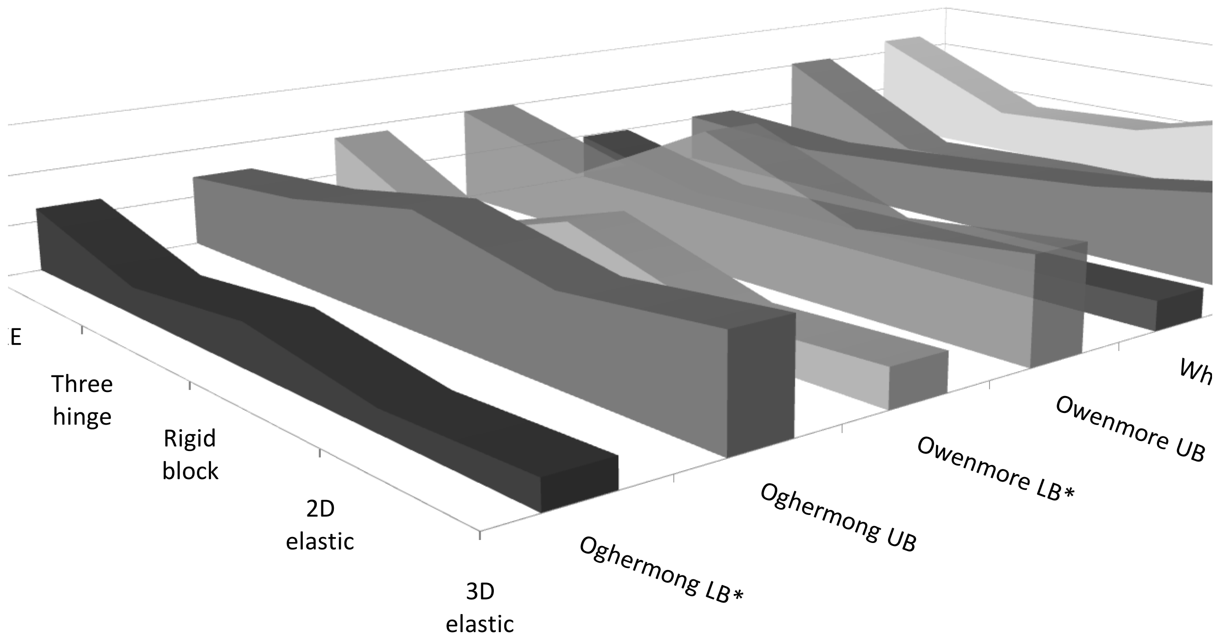
Normalised allowable axle load
MEXE
Three hinge
Rigid block
2D elastic
3D elastic
Temple



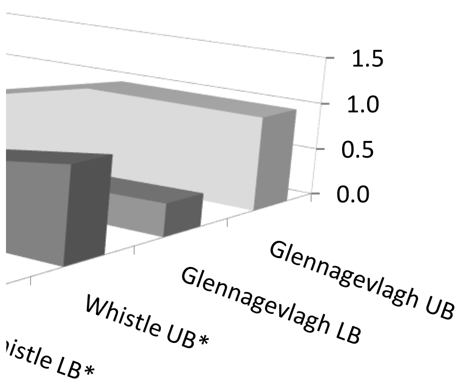
4.0
3.0
2.0
1.0
0.0
Griffith
Killeen
Glanlough
Glennagevlagh LB
Anglesea
Whistle LB*
Windy
Glanbehy
Owenmore LB*
Oghermong LB*

Figure 12



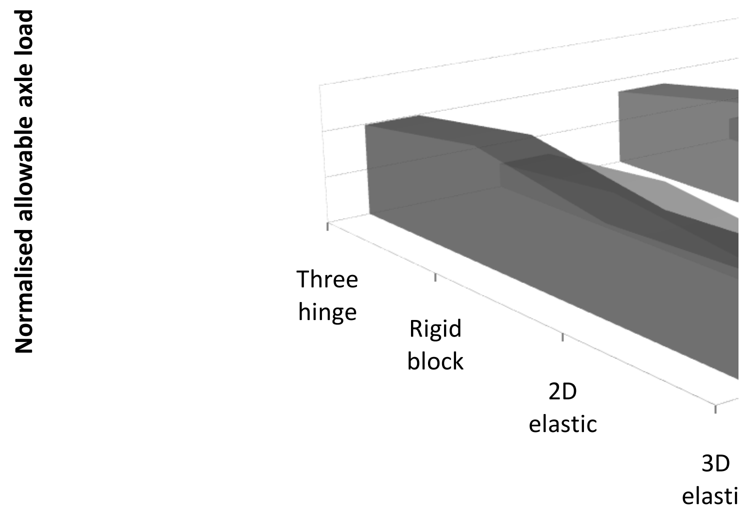


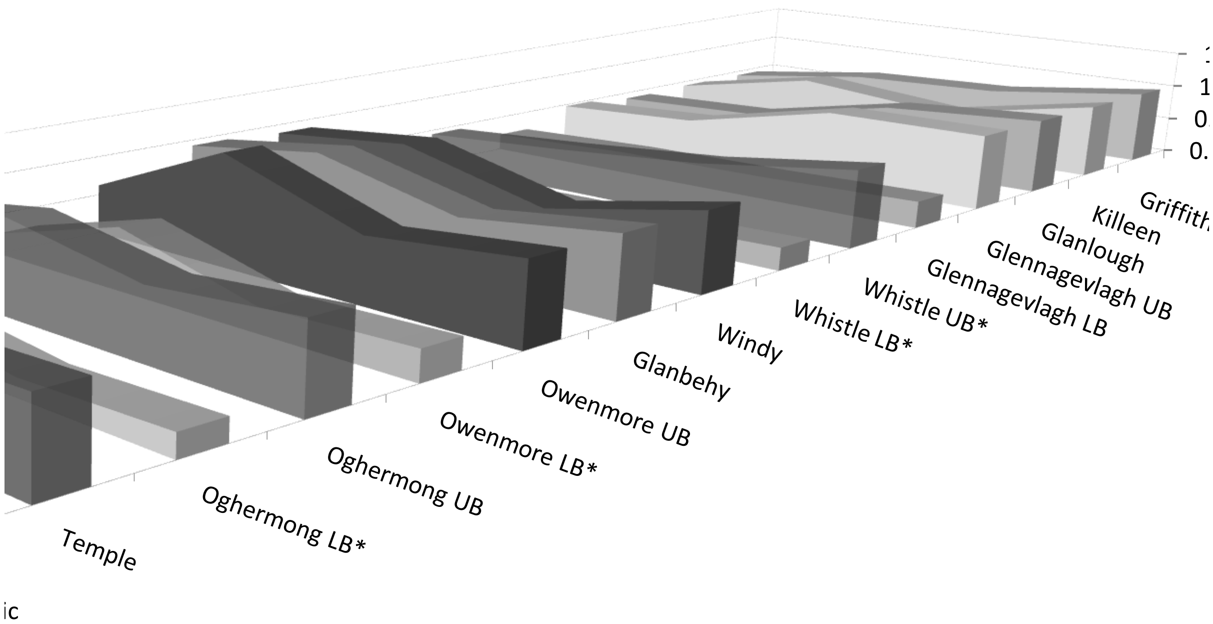
E
Three hinge
Rigid block
2D elastic
3D elastic
Oghermong LB*
Oghermong UB
Owenmore LB*
Owenmore UB
Wh



1.5
1.0
0.5
0.0
Glennagevlagh UB
Glennagevlagh LB
Whistle UB*
istle LB*

Figure 13


Normalised allowable axle load
Three
hinge
Rigid
block
2D
elastic
3D
elasti



Temple
Oghermong LB*
Oghermong UB
Owenmore LB*
Owenmore UB
Glanbehy
Windy
Whistle LB*
Whistle UB*
Glennagevlagh LB
Glennagevlagh UB
Glanlough
Killeen
ic

1.5
1.0
.5
0