Traffic Loading on Highway Bridges

BY

COLM CAREY

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D.I.T. BOLTON STREET

Abstract:

When a vehicle passes over a bridge it passes a static load along with a dynamic load onto the bridge. The dynamic load is due to the "bouncing effect" caused by the interaction between the vehicle's tyres and the bridge's uneven surface. Bridges are designed to a lifetime load effect which is generally the largest static load effect multiplied by a dynamic amplification factor (DAF).

The Eurocode 1: Part 3 (2003)[1] (Annex B) states that,
"(1) A stress history should be obtained by analysis using recorded representative real traffic data, multiplied by a dynamic amplification factor (φ).
(2) This dynamic amplification factor should take into account the dynamic behaviour of the bridge and depends on the expected roughness of the road surface and on any dynamic amplification already included in the records".

"In Western countries the mean allowance for dynamic amplification is up to about 30% (Cooper in the UK recommends 27% and the United States AASHTO code specifies 30%) (O'Brien 2007)[2]. This method of designing bridges is very conservative as it does not take into account the large probability that the worst case static load, that of a multi-truck event, will not occur with the worst case dynamic load, that of a single truck event.

In this study Weigh-In-Motion (WIM) data will be used from a typical European route, motorway A6 (near Auxerre). This motorway was choosen as it is seen to represent a typical European freight truck route. WIM is a measurement system that records the weight of vehicles axles, there speed and overlapping data contineousily over the required recording time. 1000 days of bi-directional traffic will be generated for free flowing and 85 days congested traffic using Monte Carlo simulation for bridge spans between 20 and 60 metres. Using these traffic generations it will be possible to work out what bridge spans should be designed for

free flow traffic (with dynamic loading) and what bridges should be designed for congested flow (no dynamic load).

Dawe (2003)[3] describes, “For the National Application Document (NAD) for use with ENV 1991-3: 1995 for designing road bridges in the UK was published by the British Standards Institution in 2000 analyses were carried out for various spans ranging from 5 to 200 metres. Congested traffic conditions were considered for all spans, but flowing traffic was only considered for spans up to 50 metres as congested traffic governed the loading beyond this point.”

As about 90% of bridges in Europe are under 40m and therefore are designed using loading due to free flow traffic multiplied by a DAF, it becomes apparent the need for an accurate value of DAF. “An assumption inherent in much previous research in this area is that free flowing traffic with coincident dynamic effects is more critical than congested traffic (which has practically no dynamic effect) for short- to medium length bridges. Given that about 90% of bridge stock is of this length, this assumption has critical implications for the expenditure on bridge rehabilitation (Caprani and Rattigan 2006)[4].”

By graphing load effect versus bridge span for congested traffic flow (0% DAF) and free flow with varrying DAF values versus bridge span, it will be possible to analyse the importance of using an accurate DAF value when designing or refurbishing bridges.

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Glossary of Terms:

- **Bending moment**

A bending moment exists in a structural element when a moment is applied to the element so that the element bends. Moments and torques are measured as a force multiplied by a distance so they have as unit newton-metres (N.m)

- **Characteristic value**

A characteristic value is a value that has a certain percentage of occurring in a structures design life. For the Eurocode 1:3 a characteristic value of 1000 year return period (or probability of exceedance of 5% in 50 years) for traffic on the main roads in Europe is utilised.

- **C.L.o.F.T.s.**

Critical Length of Flow Traffic Switch, i.e. the bridge length at which congested flow starts to govern the traffic flow regime.

- **Congested flowing traffic**

Congested traffic flow is basically traffic jam scenario, where vehicles are bumper to bumper moving at slow speeds usually 0-10km/h.

- **DAF (dynamic amplification factor)**

A dynamic load can have a significantly larger effect than a static load of the same magnitude due to the structure's inability to respond quickly to the loading (by deflecting). The increase in the effect of a dynamic load is given by the dynamic amplification factor (DAF):

$$DAF = \frac{u_{max}}{u_{static}}$$

where u is the deflection of the structure due to the load.

- **Dead load**

Dead loads are weights of material, equipment or components that are relatively constant throughout the structure's life. Permanent loads are a wider category which includes dead loads but also includes forces set up by irreversible changes in a structure's constraints - for example, loads due to settlement, the secondary effects of prestress or due to shrinkage and creep in concrete.

- **Deflection**

In engineering mechanics, deflection is a term that is used to describe the degree to which a structural element is displaced under a load. The deflection of a member under a load is directly related to the slope of the deflected shape of the member under that load and can calculated by integrating the function that mathematically describes the slope of the member under that load.

- **Design life**

The design life of a component or product is the period of time during which the item is expected by its designers to work within its specified parameters; in other words, the life expectancy of the item.

- **Dynamic load**

These are loads that display significant dynamic effects. Examples include impact loads, waves, wind gusts and strong earthquakes. Because of the complexity of analysis, dynamic loads are normally treated using statically equivalent loads for routine design of common structures.

- **Extreme load**

The worst or most extreme traffic loading that could reasonably be expected to occur in the lifetime of a bridge (i.e. 120 years) was to be taken as the extreme design loading in limit state terms, namely 1.5 times the normal loading

- **Free flowing traffic**

Free flowing traffic is where traffic moves at a steady flow of usually 80-100km/h.

- **HA loading and HB loading**

In the BS5400: Part 3 traffic flow was grouped into two categories, HA and HB. HA represented normal traffic loads and HB represent very heavy abnormal loads.

- **Impact factor (IM)**

Used in the American standard AASHTO to calculate the dynamic response of a bridge under vehicle loads. (IM = DAF – 1).

- **Live load**

Live loads, sometimes referred to as probablistic load include all the forces that are variable within the object's normal operation cycle.

- **Load Effect 1**

Bending moment at the centre of a simple supported bridge.

- **Load Effect 2**

Bending moment over the central support of a two-span bridge.

- **Load Effect 3**

Left-hand support shear force for a simply supported bridge.

- **Load Model 1 (LM 1)**

Used in the Eurocode 1: Part 3 to describe, concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.

- **Load Model 3 (LM3)**

Used in the Eurocode 1: Part 3 to describe, a set of assemblies of axle loads representing special vehicles (e.g. for industrial transport) which can travel on routes permitted for abnormal loads. It is intended for general and local verifications.

- **Monte Carlo simulation**

Monte Carlo methods are a class of computational algorithms that rely on repeated random sampling to compute their results. Monte Carlo methods are often used when simulating physical and mathematical systems. Because of their reliance on repeated computation and random or pseudo-random numbers, Monte Carlo methods are most suited to calculation by a computer. Monte Carlo methods tend to be used when it is infeasible or impossible to compute an exact result with a deterministic algorithm.

- **Multi-truck events**

Where more than one truck is present one a bridge at one time.

- **Natural frequency**

The fundamental frequency (also called a natural frequency) of a periodic signal is the inverse of the pitch period length. The pitch period is, in turn, the smallest repeating unit of a signal. One pitch period thus describes the periodic signal completely. The significance of defining the pitch period as the smallest repeating unit can be appreciated by noting that two or more concatenated pitch periods form a repeating pattern in the signal. The natural frequency depends on two system properties; mass and stiffness.

- **Resonant frequency**

In physics, resonance is the tendency of a system to oscillate at maximum amplitude at certain frequencies, known as the system's resonance frequencies (or resonant frequencies). At these frequencies, even small periodic driving forces can produce large amplitude vibrations, because the system stores vibrational energy.

- **Return period**

A return period also known as a recurrence interval is an estimate of the interval of time between events like an earthquake, flood or river discharge flow of a certain intensity or size. It is a statistical measurement denoting the average recurrence interval over an extended period of time, and is usually required for risk analysis (i.e. whether a project should be allowed to go forward in a zone of a certain risk) and also to dimension structures so that they are capable of withstanding an event of a certain return period (with its associated intensity).

- **Second moment of area (I)**

The second moment of area, also known as the area moment of inertia or second moment of inertia, is a property of a shape that is used to predict its resistance to bending and deflection which are directly proportional

- **Shear force**

A force that causes a deformation of an object in which parallel planes remain parallel but are shifted in a direction parallel to themselves.

- **Single truck event**

Where only one truck is present on a bridge at one time.

- **Static load**

These are loads that build up gradually over time, or with negligible dynamic effects. Since structural analysis for static loads is much simpler than for dynamic loads, design codes usually specify statically-equivalent loads for dynamic loads caused by wind, traffic or earthquake.

- **Stiffness (K)**

Stiffness is the resistance of an elastic body to deflection or deformation by an applied force. It is an extensive material property.

- **Structural damping**

Structures energy absorption capabilities. The cause of the energy dissipation may be due to many different effects such as material damping, joint friction and radiation damping at the supports.

- **WIM systems**

Weigh-in-motion (WIM) devices are designed to capture and record truck axle weights and gross vehicle weights as they drive over a sensor. Unlike older static weigh stations, current WIM systems do not require the subject trucks to stop, making them much more efficient.

- **Young's modulus (E)**

In solid mechanics, Young's modulus (E) is a measure of the stiffness of a material. It is also known as the Young modulus, modulus of elasticity, elastic modulus (though the Young's modulus is actually one of several elastic moduli such as the bulk modulus and the shear modulus) or tensile modulus. It is defined as the ratio of stress over strain in the region in which Hooke's Law is obeyed for the material. This can be experimentally determined from the slope of a stress-strain curve created during tensile tests conducted on a sample of the material.

List of Symbols:

- **φ** is the dynamic amplification factor.
- **n** is the number of axles multiplied by the weight (kN) of each axle in each group.
- **e** is the axle spacing (m) within and between each group.
- ***v*** is the speed of a single motor vehicle (m/s).
- **V** is a parameter, namely: a conventional speed of 10 miles per hour.
- **T** is the fundamental period of the bridge (Hz).
- **L** is the length of the element (m).
- **K** is the stiffness of an object (N/m).
- $\mathbf{E_{tot}}$ is the maximum total load effect experienced by the bridge from a loading event.
- $\mathbf{E_{stat}}$ is the maximum static load effect form a loading event.
- $\mathbf{D_{sta}}$ is the maximum static deflection (m).
- $\mathbf{D_{dyn}}$ is the deflection due to the dynamic effects (m).
- **I** is the second moment of area (mm^4).
- **E** is the Young's modulus (kN/mm^2).
- **d** is the bridge decks depth (m).
- **b** is the bridge decks width (m).
- **m** is the mass of the member (kN).
- $\mathbf{f_n}$ is the natural frequency in hertz (1/seconds) (Hz).
- **Δ** is the static deflection (m)
- **g** is the gravitational force and its value at the Earth's surface, denoted *g*, is approximately expressed below as the standard average of 9.8 m/s^2.
- **π** or Pi is one of the most important mathematical constants, approximately equal to 3.14159.

- f_i

The impact stress, caused by a load that is applied or removed suddenly (N/mm^2). The stress is the force divided by area of material.

- f_e

The static live stress, is the stress induced by the static load (a static load is one which does not vary) caused by live loads (N/mm^2).

1. Introduction:

In recent years the live load a bridge has to react against has increased dramatically. Large, multi-axle freight trucks are common sight on our highways nowadays. A lot of highway bridges in Europe were constructed in the 1950's and 1960's to repair the transport network throughout Europe after the destruction of World War II. Traffic flow when these bridges were being design would have been considerable less than the traffic we see on highways throughout Europe today. As Europe's economy has grown over the last few decades so has its need to supply goods and merchandise. This has resulted in groups of larger, heavily loaded freight trucks travelling on Europe's highways and across its bridges.

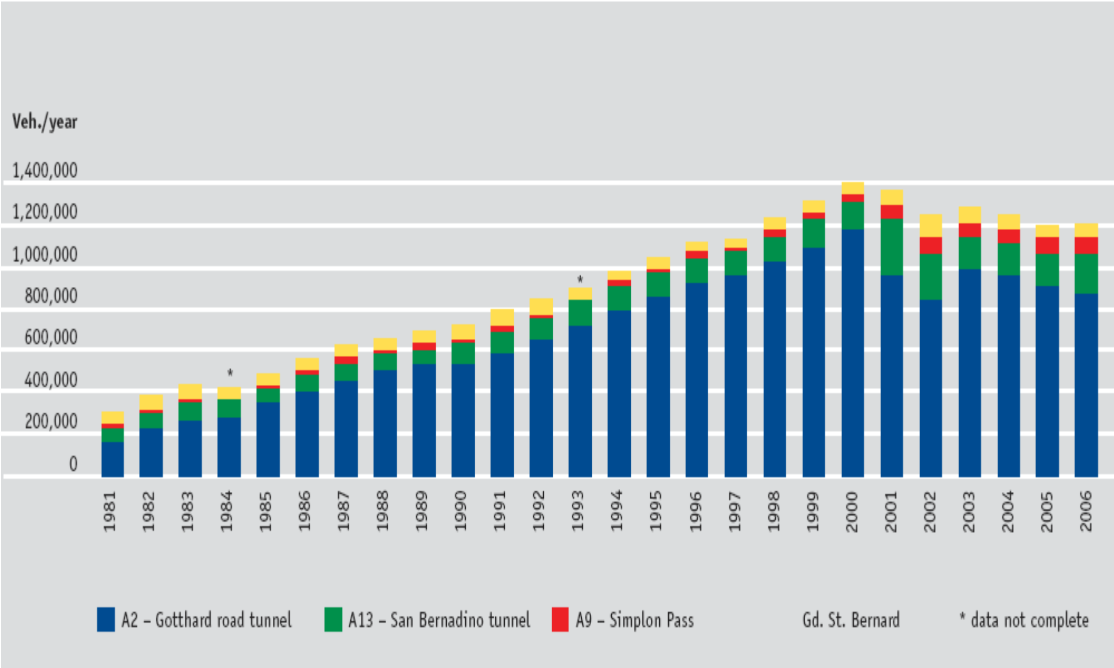


Fig.1: Transalpine goods traffic 1981 to 2006: number of heavy goods vehicles per annum[5].

This greatly increases the probability of a number of trucks being on the one bridge at the one time, which results in a large static load on the bridge. Along with this increased static load, a bridge also has to react against an imposed dynamic load. The dynamic load increases as the quality of the bridge pavement decreases.

As no one could have predicted the growth in the size and amount of large heavy freight trucks on Europe’s highways, the question arises “How safe are these bridges and can they cope with the loading they are subjected to?”

There has been a large amount of research carried out in recent years on the topic of traffic loading on highway bridges to answer this question. Large amounts of field data has been collected with the development of WIM systems. Despite this there are still lots of uncertainness surround traffic loading. This can be attributed to the large characteristic value bridges are designed to. In the Eurocode 1: Part 3 (E.C.) the characteristic value is a 1000 year return period, this equates to an extreme load which has a 1 in 250,000 chance of occurring during a bridges design life (explained in Section 2.2). WIM systems are very expensive to run and therefore only limited real traffic records can be obtained. Because of this, a method to extrapolate the recorded WIM data to obtain the extreme static load in a 1000 year return period was needed. Researchers in U.C.D. have written a program to accurately calculate the extreme static load a bridge will be subjected to (Caprani, 2006)[6].

Despite the advances in predicting the extreme static load a bridge should be designed to, there is still uncertainty surrounding the extreme dynamic load with a probability of occurrence, of 1 in 250,000 in a bridge’s design life. Some of the current design codes are based on tests carried out over 50 years ago. These tests are based on bridge responses due to single trucks crossing a bridge. This concept is theoretically flawed and therefore the dynamic loads that are calculated form current design codes can be overestimated.

This thesis will investigate the history of bridge dynamics due to traffic loading. Also the theory behind how a bridge reacts dynamically due to single and multi-truck events will be looked into. The main body of the thesis will investigate the importance of using an accurate DAF in bridge engineering. Also the question of, at what length of bridge should be designed for congested traffic will be investigated.

2. Theory:

2.1 Traffic Flow:

Traffic flow for highway bridges can be looked at in two categories, congested flow and free flow. Congested traffic flow is basically traffic jam scenario, where vehicles are bumper to bumper moving at slow speeds usually 0-10km/h. Free flowing traffic is where traffic moves at a steady flow of usually 80-100km/h.

Traffic loading can also be looked at in two categories, static loading and dynamic loading. When considering static loading due to traffic on a bridge, we treat the bridge as a beam and the vehicles on the bridge at any particular time as dead loads. The dynamic loading a bridge is subjected to is not so easy to work out as there are many variables associated with dynamic loading.

2.2 Bridge Design Life:

All bridges are design to a design life. This design life is usually 120 years and the loading the bridge is designed to is called the design load. The BS5400 utilises a characteristic value of 5 per cent of occurrence in 120 years. A characteristic value of 5 per cent of occurrence in 120 years means that there is a 1 in 2400 chance of the particular load value being exceeded in one year. (Dawe, 2003)[3] describes how the characteristic value was derived for the BS 5400: part 3, "the worst or most extreme traffic loading that could reasonably be expected to occur in the lifetime of a bridge (i.e. 120 years) was to be taken as the extreme design loading in limit state terms, namely 1.5 times the normal loading. Work done on the calibration of the partial factor for the steel design codes BS 5400: Part 3 had shown that for longer spans the 95 per cent characteristic load (i.e. that load with a 5 per cent chance of occurring in 120 years) derived from surveys of actual traffic was approximately the same as the then current serviceability loading (i.e. 1.2 times nominal HA) as specified in BS 5400: Part 2[7]. Using the same statistical model it was shown that the ultimate design loading would occur approximately once in 200000 years and that the nominal unfactored loading would occur once in 120 years."

In the Eurocode 1: Part 3 the design life is also stated as 120 years for bridges but it states that bridges should be designed to a characteristic value of 5% in 50 years, Table 2.1 "1000 year return period (or probability of exceedance of 5% in 50 years) for traffic on the main roads in Europe) for Load Model 1 and 2. A probability of 5% in 50 years corresponds to means that there is a 1 in 1000 chance of the particular value being exceeded in one year. Therefore the Eurocode characteristic value has a lot greater probability of occurring than that of the previous BS5400 characteristic value. This would suggest that the design load calculated from the Eurocode will be less than the design load calculated from BS5400.

>>>>>have to work on this idea.

2.3 British Standard's Traffic Load Models (HA and HB):

In the BS5400: Part 3 traffic flow was grouped into two categories, HA and HB. HA represented normal traffic loads and HB represent very heavy abnormal loads. These have been replaced in the current Eurocode with HA changed to Load Model 1 and HB changed to Load Model 3.

Loading System:

There are two types of loading:
A: The type HA loading (normal traffic):
Formula design loading for bridges. It consists of a uniformly distributed lane loading, together with one knife edge load.

B: The type HB loading (abnormal vehicle):

Exceptional design loading for bridges. A bridge is calculated for type HA loading and checked for HB loading, which represents abnormally heavy vehicles. When considering the effects of this loading a reduced partial load factor is applied to the HB load and the coexistent HA loading.

Type HA Loading:

-Two carriageway lanes shall always be considered as occupied by full HA loading (100 per cent).

-All other lanes shall be considered as occupied by one-third of the full lane loading (33 1/3 per cent).

-HA loading shall be applied to two lanes – either the remainder of the lane occupied by the HB vehicle plus an adjacent lane, or the remainder of the two lanes straddled by the HB vehicle, or the remainder of one straddled lane plus an adjacent lane.

-All other lanes shall be loaded to 1/3 HA load.

Type HB loading:

-The HB load may be in any position, occupying one lane or straddling two. No other loading shall be considered in the 25m in length at each end of the vehicle.

Load Values:

The type HA loading consists of a and b, or c:

(a) A uniformly distributed lane loading. For loaded lengths up to 30m, the value shall be 30kN per m of notional lane. For greater length (L) it shall be:

$$151 \times (1/L)^{0.475} \qquad \text{Eq. 1}$$

but not less than 9kN per m of notional lane.

(b) One knife edge load (axle load) of 120kN, uniformly distributed across the width of the national traffic lane.

(c) A single nominal wheel load, as an alternative to (a) + (b). The load shall be 100kN and distributed over either a circular area of 0.34m or a square of 0.3m sides. The HA wheel load is applied to members supporting small areas of roadway, where the proportion of the distributed load and knife edge load which would otherwise be allocated to it is small.

The type HB loading is a unit loading representing a single abnormally heavy vehicle. The loading is composed of 4 axle loads, each with a weight expressed in units (1 unit = 10kN). The number of units of HB loading normally required is 45 units (450kN per axle).

Impact factor:

An impact factor of 1.25 is taken into account in the HA loading. No impact factor is used with the HB loading

2.4 Eurocode1: Part 3 Traffic Load Models

Eurocode 1: Part 3 (4.3 Vertical loads – Characteristic values)
There are four types of loading systems:

Load Model 1 (LM1):
Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.

Load Model 2 (LM2):
A single axle load applied on specific tyre contact areas, which covers the dynamic effects of the normal traffic on short structural members

Load Model 3 (LM3):
A set of assemblies of axle loads representing special vehicles (e.g. for industrial transport) which can travel on routes permitted for abnormal loads. It is intended for general and local verifications.

Load Model 4 (LM4):
A crowd loading intended only for general verifications.

Load Model 1:
For LM1 one lane is load with a UDL of 9kN/m2, all other lanes to be loaded to 2.5kN/m2.

Load Model 3:
Instead of a single abnormal truck model used in BS5400 for the HB model, the Eurocode LM3 has a list of "special vehicles". The basic models of special vehicles correspond to various levels of abnormal loads that can be authorised to travel on particular routes of the European highway network

Total weight	Composition	Notation
600 kN	4 axle-lines of 150 kN	600/150
900 kN	6 axle-lines of 150 kN	900/150
1200 kN	8 axle-lines of 150 kN or 6 axle-lines of 200 kN	1200/150 1200/200
1500 kN	10 axle-lines of 150 kN or 7 axle-lines of 200 kN + 1 axle line of 100 kN	1500/150 1500/200
1800 kN	12 axle-lines of 150 kN or 9 axle-lines of 200 kN	1800/150 1800/200
2400 kN	12 axle-lines of 200 kN or 10 axle-lines of 240 kN or 6 axle-lines of 200 kN (spacing 12m) + 6 axle-lines of 200 kN	2400/200 2400/240 2400/200/200
3000 kN	15 axle-lines of 200 kN or 12 axle-lines of 240 kN + 1 axle-line of 120 kN or 8 axle-lines of 200 kN (spacing 12 m) + 7 axle-lines of 200 kN	3000/200 3000/240 3000/200/200
3600 kN	18 axle-lines of 200 kN or 15 axle-lines of 240 kN or 9 axle-lines of 200 kN (spacing 12 m) + 9 axle-lines of 200 kN	3600/200 3600/240 3600/200/200

Table 1: Table A1- Classes of special vehicles (E.C.1:3)[1]

	Axle-lines of 150 kN	Axle-lines of 200 kN	Axle-lines of 240 kN
600 kN	n = 4x150 e = 1.50m		
900 kN	n = 6x150 e = 1.50m		
1200 kN	n = 8x150 e = 1.50m	n = 6x200 e = 1.50m	
1500 kN	n = 10x150 e = 1.50m	n = 1x100 + 7x200 e = 1.50m	
1800 kN	n = 12x150 e = 1.50m	n = 9x200 e = 1.50m	
2400 kN		n = 12x200 e = 1.50m n = 6x200 + 6x200 e = 5x1.5 + 12 + 5x1.5	n = 10x240 e = 1.50m
3000 kN		n = 15x200 e = 1.50m n = 8x200 + 7x200 e = 7x1.5 + 12 + 6x1.5	n = 1x120 + 12x240 e = 1.50m
3600 kN		n = 18x200 e = 1.50m	n = 15x240 e = 1.50m n = 8x240 + 7x240 e = 7x1.5 + 12 + 6x1.5

NOTE
n number of axles multiplied by the weight (kN) of each axle in each group
e axle spacing (m) within and between each group

Table 2: Table A2 – Description of special vehicles (E.C.1:3)[1]

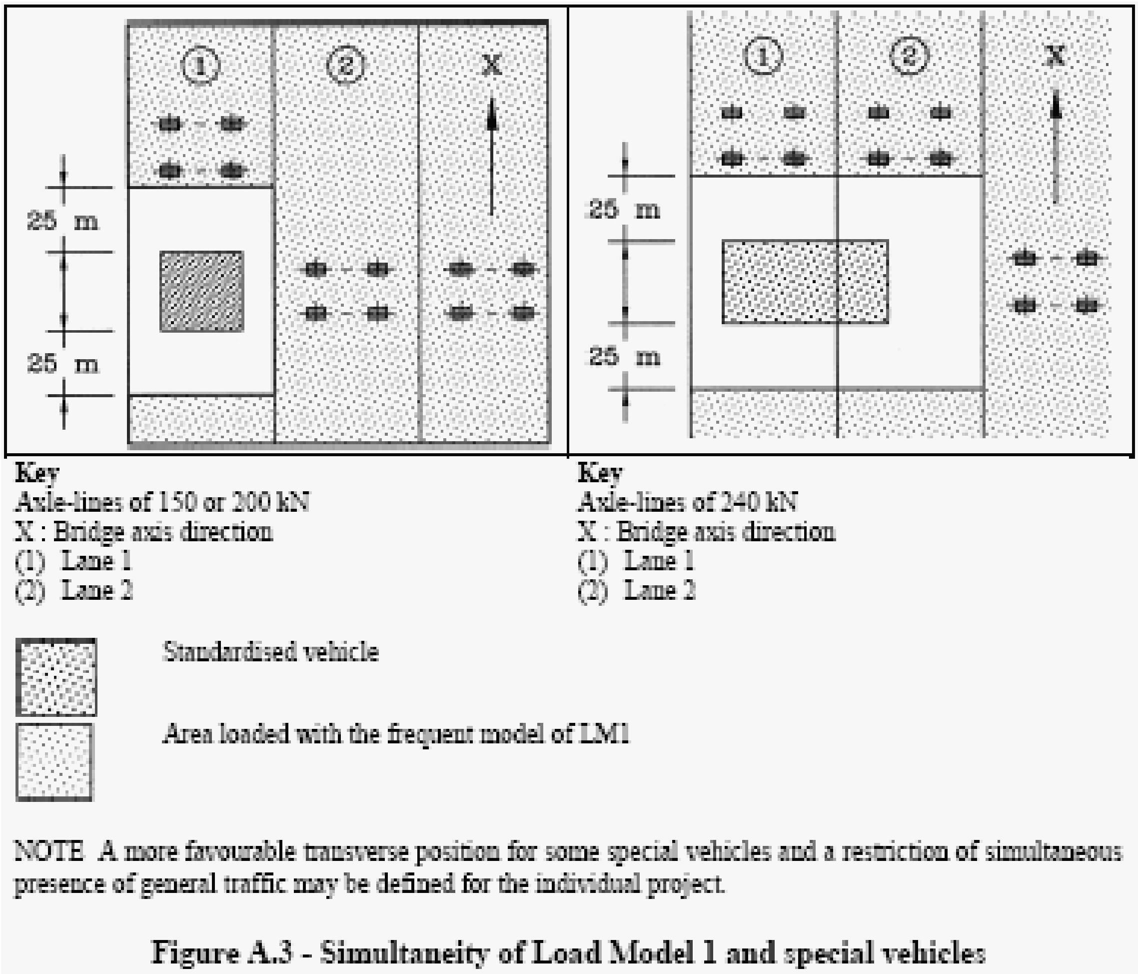


Fig. 2: Figure A.3 – Simultaneity of Load Model 1 and special vehicles (E.C.1:3)[1]

No other loading shall be considered in the 25m in length at each end of special vehicles.

When we compare the present Eurocode to the previous BS design code:

<u>HA is now called LM1:</u>

HA: A uniformly distributed lane loading. For loaded lengths up to 30m, the value shall be 30kN per m of notional lane. For greater length (L) it shall be:

$151 \times (1/L)^{0.475}$ Eq. 1

but not less than 9kN per m of notional lane. With a lane with of 2.3 to 3.8 metres.
If we compare say a 30m bridge with 3m wide lanes

HA lane 1: 10kN/m2
All other lanes: 3.33kN/m2

LM1 lane 1: 9kN/m2
All other lanes: 2.5kN/m2

We can see that there is a reduction in loading for normal traffic flow for bridges up to 30m.

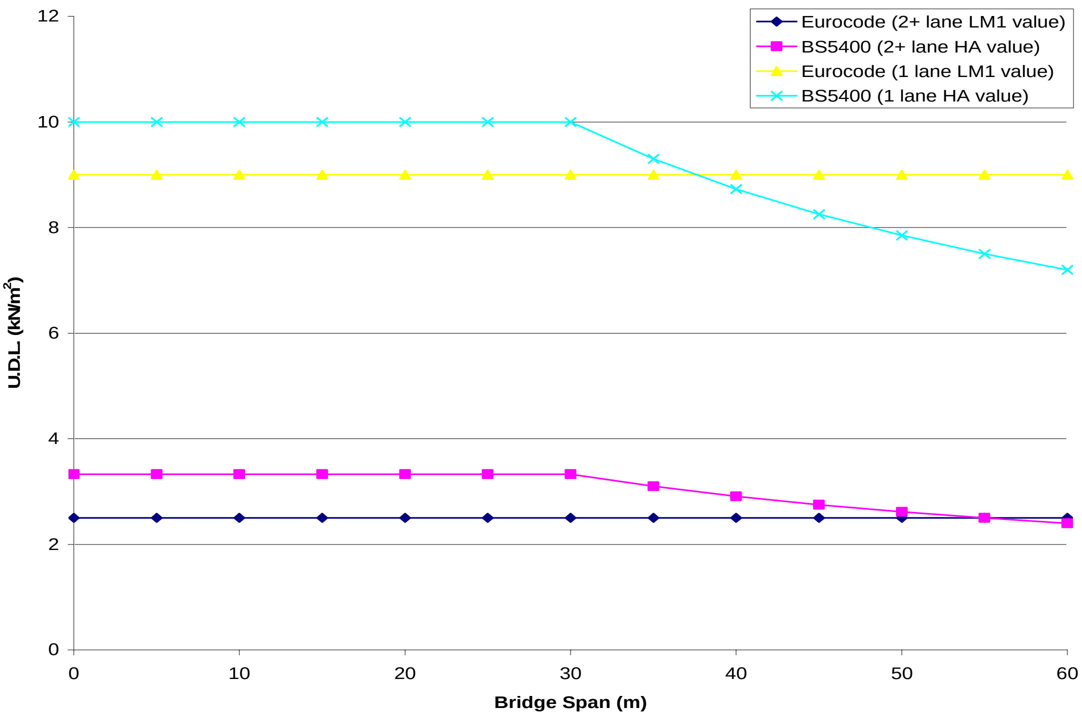


Fig. 3: Comparison between the BS5400: Part 2 HA loading values to the E.C.1:3 LM1 loading values.

For the first lane of loading the HA value is greater than the LM1 value up to bridge lengths of 37.4m. For lanes two and more the HA value is greater than the LM1 value up to bridge lengths of 55.0m.

It is more difficult to compare the abnormal loading classed HB in the BS5400 and LM3 in the Eurocode, as the Eurocode uses different “special vehicles”, see Fig. 4, for different classes of bridges where as the BS5400 used one abnormally loaded vehicle for all bridge types. All that can be noticed is that the BS5400 the normal HB model consists of 4 axles weighing 450kN which gives a total truck weight of 1800kN. There also is a “special truck” weighing 1800kN in the LM3 which lists eight different trucks whose weights range from 600kN to 3600kN. The largest LM3 truck weighs 3600kN which is twice the weight of the HB truck weight of 1800kN where as there is not the same degree of difference if we compare the HA model to the LM1 model. This indicates that general traffic weights on highway bridges have stayed the same over the last 30 years whereas abnormally heavy vehicles weights have increases substantially.

Also in the Eurocode the LM3 is multiplied by a DAF which can be as large 1.36 (for bridges greater than 20m in length). In the BS5400, there is a DAF of 1.25 for HA loading but a DAF is not used with HB loading.

2.5 Dynamic Loads on Bridges due to Traffic Loading

Static loading for bridges can be worked out quite accurately but there are many conflicting views among bridge engineers in relation to the dynamic loading a bridge is subjected to. The dynamic load a bridge is subjected to is caused by the vehicles tyres reacting with the bridges uneven surface as the vehicle travels across the bridge. There are many variables to consider when looking at dynamic loading for bridges such as:

- vehicle speed
- vehicle weight
- vehicle tyre pressure
- vehicle suspension
- weight of bridge
- surface roughness
- bridge length
- bridge dampening
- bridge frequency
- period of loading.

2.5.1 How vehicle speed effects Bridge Dynamic Loading

The basic theory behind dynamic loading for bridges is that there is an additional response to a load when it is moving across the bridge than when the same load is stationary, or static. Another way of wording this theory is that when a load has a speed on a bridge, there is also an associated additional response or dynamic response.

When we see the dynamic response in this light it becomes logical that the DAF is a function of the vehicle speed that induces the loading event. Indeed one of the first equations for dynamic loading contained velocity as the principal function of the equation, “the impact effect must depend on the speed of the vehicles. In fact, the Final Report of the 2nd Congress for Bridge and Structural Engineers 1929, contains the following formula framing this principle:

$$f_i/f_e=1.5/(1+(V/3v)) \qquad \text{Eq. 2}$$

in which v is the speed of a single motor vehicle, expressed in miles per hour, whilst V is a parameter, namely: a conventional speed of 10 miles per hour, f_i is the impact stress and f_e is the static live stress. This formula is obtained empirically from observations, as a curve of best fit” (Freudenthal, 1947)[8].

(Chaallal and Shahawy, 1998)[9] describe testing they carried out to evaluate the dynamic effects of moving vehicles on the response of bridges, “test vehicles (two-axle trucks and three-axle tractor-semitrailer combinations) made approximately 1900 runs over 15 bridges at speeds varying between 12 km/h and 32 km/h. The following observations were made: the DAF generally increased with the speed parameter:

$$SP = vT/2L \qquad \text{Eq. 3}$$

where v is the speed of the vehicle, L is the span length, and T is the fundamental period of the bridge. The largest value of the DAF for displacements was 1.63, and only 5% of measured DAF exceeded 1.40. The largest value of the DAF for moments (strains) was 1.41 and only 5% of measured DAF exceeded 1.286, which was the value specified by the impact formula in the AASHTO standard specification for highway bridges at the time” (AASHTO 1989)[10].

(Inbanathan and Wieland, 1987)[11] presented an analytical investigation on the dynamic response of a simply-supported box girder bridge due to a moving vehicle for speeds of 19 km/h and 38 km/h. Some of the findings reported were:

1. The feet of vehicle mass on the bridge response is more significant for high speeds.

2. The stresses developed by a heavy vehicle moving over a rough surface at high speeds exceed those recommended by current bridge design codes.

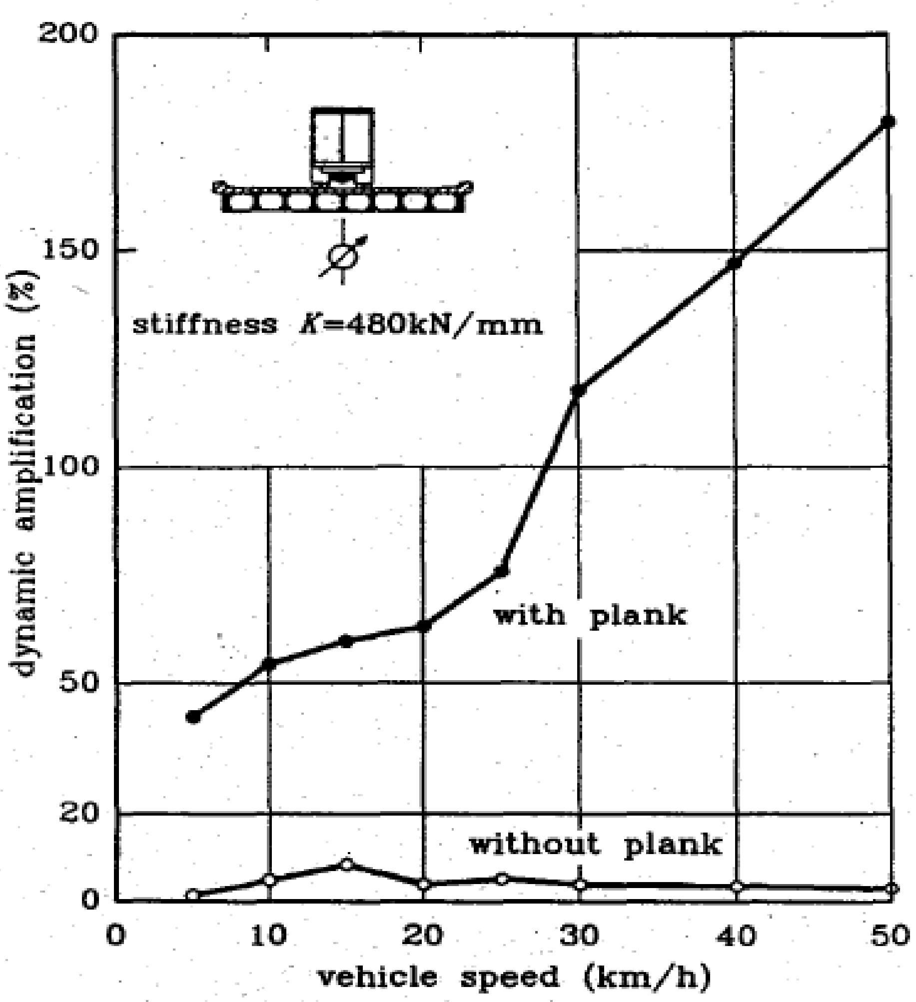


Fig. 4: The amplification effect produced by roadway irregularities increases with the bridge stiffness – stiff bridge (Cantieni, 1983)[12].

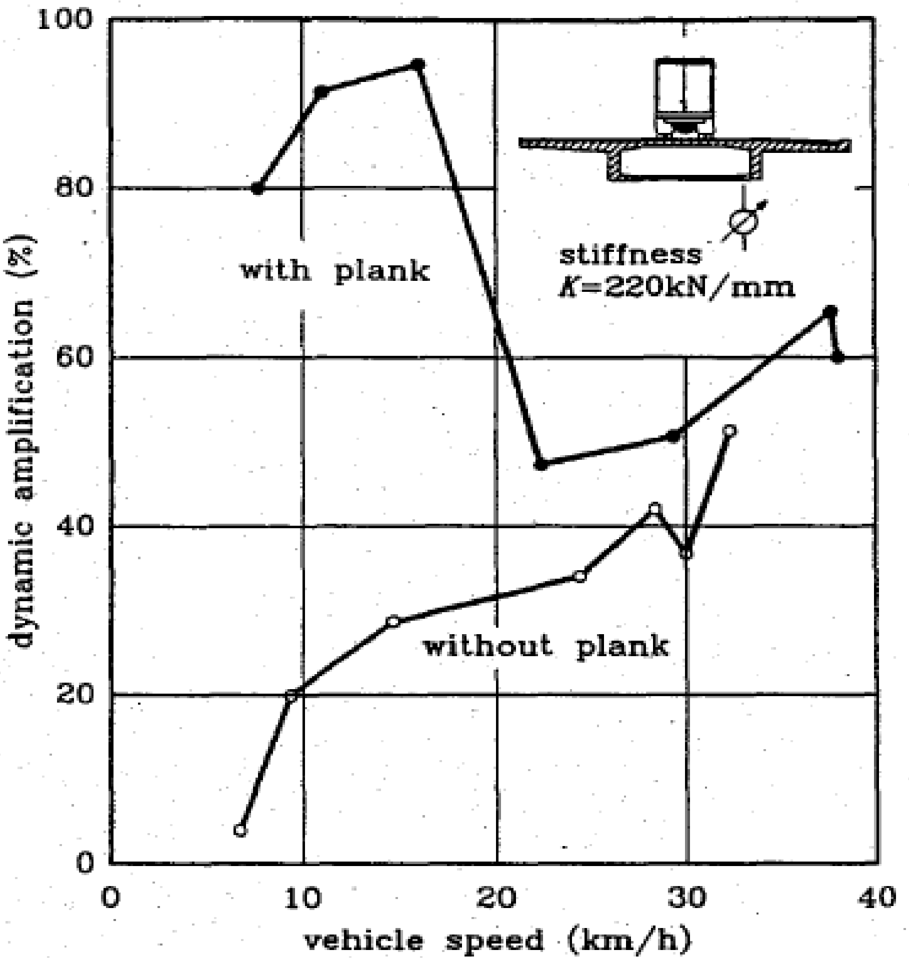


Fig. 5: The amplification effect produced by roadway irregularities increases with the bridge stiffness – flexible bridge (Cantieni, 1983)[12].

2.5.2 How vehicle weight effects Bridge Dynamic Loading:

When a vehicle travels across a bridge it transmits a load onto the bridge. The greater the vehicles load the greater the load transmitted onto the bridge. As the load the bridge is subjected to increases, so to those the bridges responses, such as shear forces, bending moments and deflections.

The dynamic amplification factor (DAF) is the total load effect divided by the static load effect:

$$DAF = E_{tot} / E_{stat} \quad \text{Eq. 4}$$

where E_{tot} is the maximum total load effect experienced by the bridge from a loading event and E_{stat} is the maximum static load effect for the same event. Intuitively as the vehicle crossing a bridge increase, so to will maximum total load effect and the maximum static load effect.

The Dynamic Load Allowance (IM), or Impact factor is used by the AASHTO instead of the DAF. We can relate the DLA to the DAF by:

$$IM + 1 = DAF \quad \text{Eq. 5}$$

$$IM = D_{dyn} / D_{sta} \quad \text{Eq. 6}$$

Where D_{sta} is the maximum static deflection and D_{dyn} is the additional deflection due to the dynamic effects. The static load effect will increase with increasing vehicle weight, this in turn will decrease the IM value and also the DAF value. Therefore as the vehicles weight increase the DAF value decreases. (Hwang and Nowak, 1991)[13] show this graphically in the following three figures:

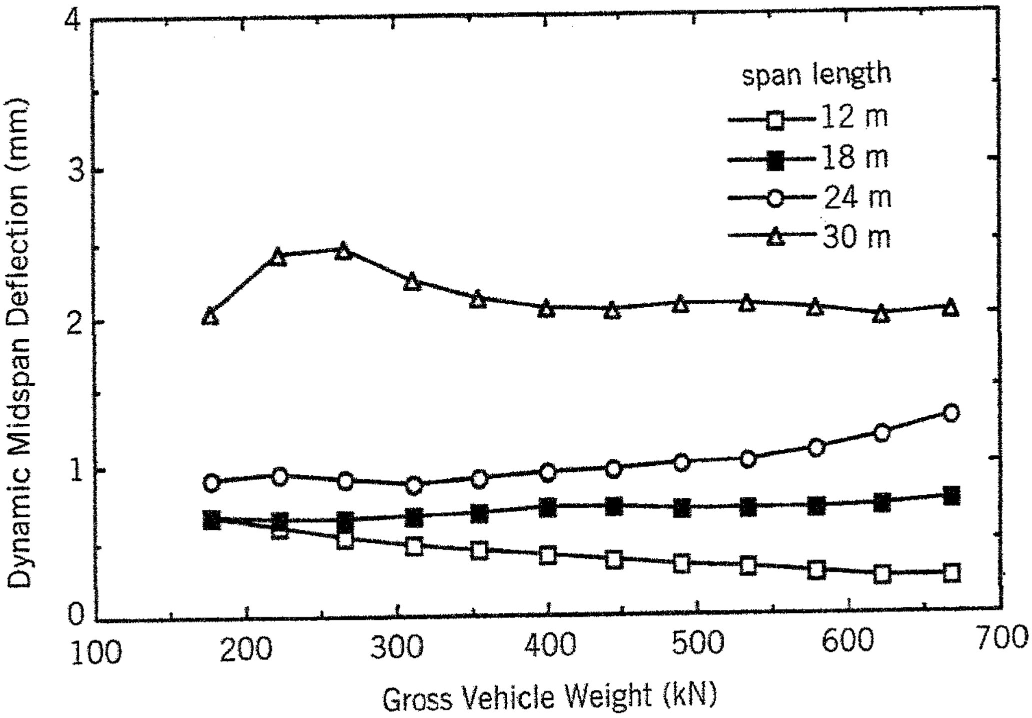


Fig. 6: Dynamic bridge response (Hwang and Nowak, 1991)[13].

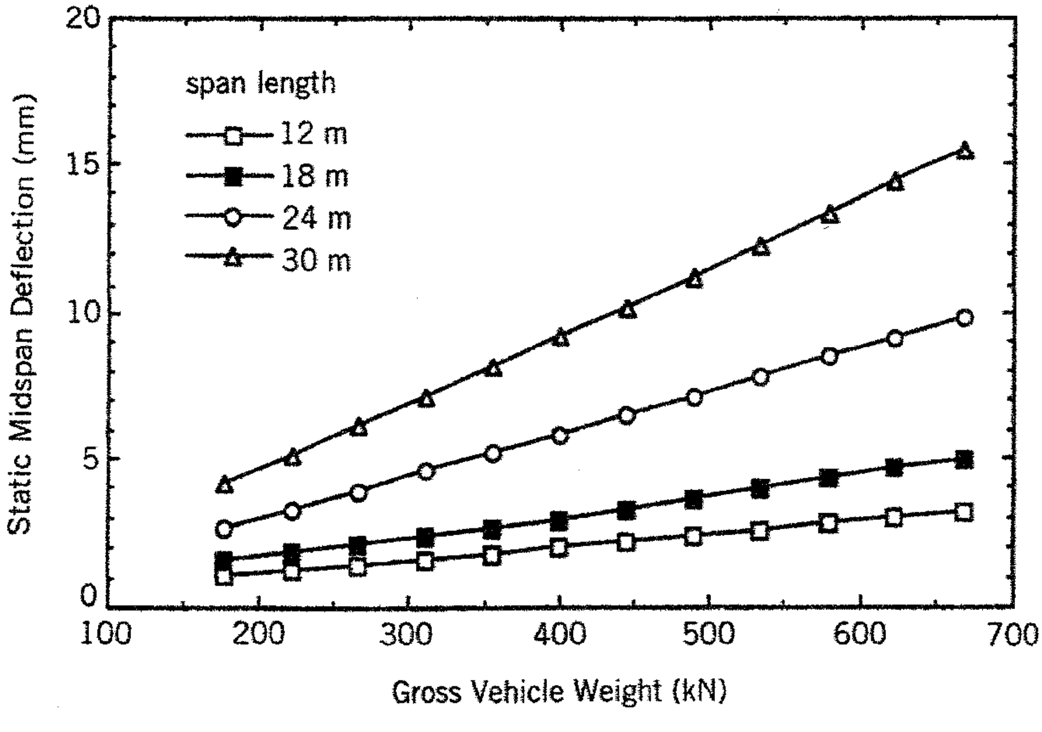


Fig. 7: Static bridge response (Hwang and Nowak, 1991)[13].

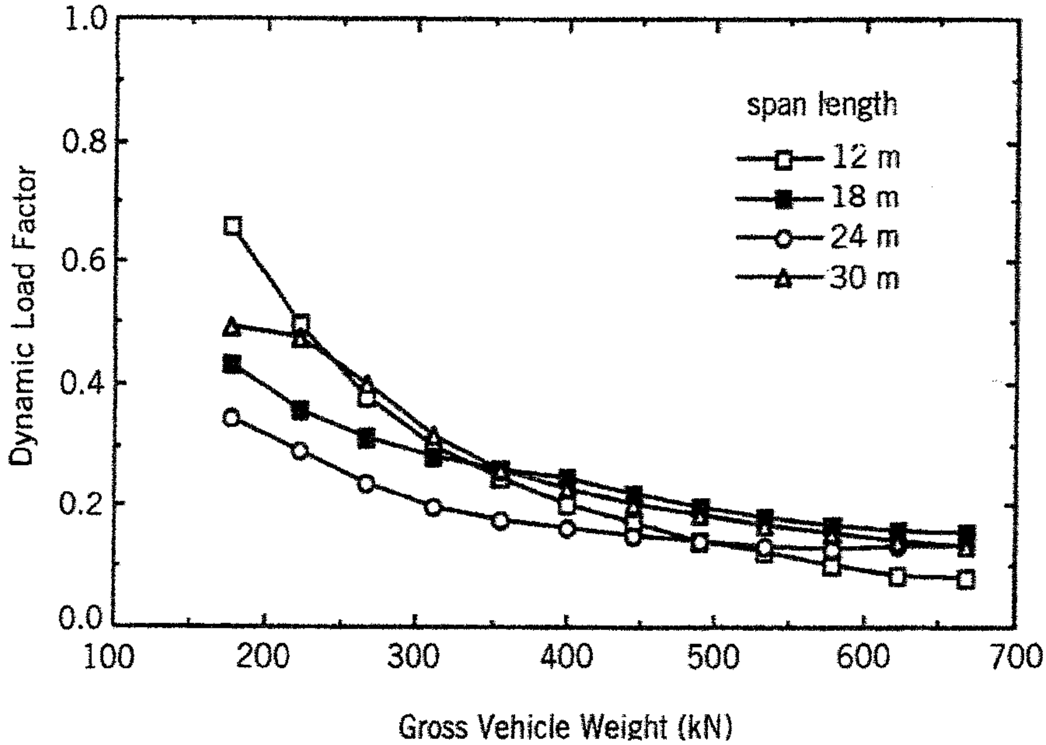


Fig. 8: Dynamic load allowance (Hwang and Nowak, 1991)[13].

Fig. 7 shows that there is little or no change in the dynamic mid-span deflection with increasing gross vehicle weight. Fig. 8 points out the obvious, that for increasing gross vehicle weight, the static mid-span deflection increases. When we look at Eq. 5 we see that if the static deflection increases whilst the dynamic deflection remains constant, the IM will decrease. From Eq. 6 we can also say that if the IM value decreases so to those the DAF value.

2.5.3 How vehicle tyre pressure and vehicle suspension effect Bridge Dynamic Loading:

As the dynamic load is dependent on the vehicle-bridge interaction, it stands to reason that the vehicle tyre pressure and vehicle suspension should play a major role on the size of the dynamic load passed onto a bridge as the vehicle crosses the bridge. It would be logical to presume that the greater the dampening quality of the vehicle's suspension system, the less dynamic load will be passed onto the bridge. Also as the vehicle tyres are the only contact points between the vehicle and the bridge, the pressure in the tyres and the tyre stiffness logically should also be a factor in the size of the dynamic load produced in a vehicle crossing.

Various studies into the effects of vehicle tyre pressure and vehicle suspension on dynamic loading of bridges have been carried out. (Whitmore, 1970)[14] studied the dynamic effects of heavy vehicles moving on pavement roadway. Roadway profile irregularities (using a profilometer and a spectral density technique), and vehicle characteristics (mass distribution, suspension system, speed) were examined.

The objectives were:

- to investigate the effect of roadway profile and vehicle characteristics on the dynamic loadings,
- to isolate specific parameters influencing loads transmitted to the pavement,
- to develop a technique capable of predicting the dynamic loads from the aforementioned parameters.

The following observations were made:

1. The force at the tire-roadway interface could be measured on the vehicle or on the roadway, and was found to increase with tire pressure and suspended mass.

2. Damping of the dynamic loads depended on the vehicle mass and suspension system.

(Chaallal and Shahawy, 1998)[9] state in their work "Suspension systems of vehicles can have a significant influence on the DAF in the case of initial vibrations of these vehicles. Also, it was found that the DAF increases with tire pressure."

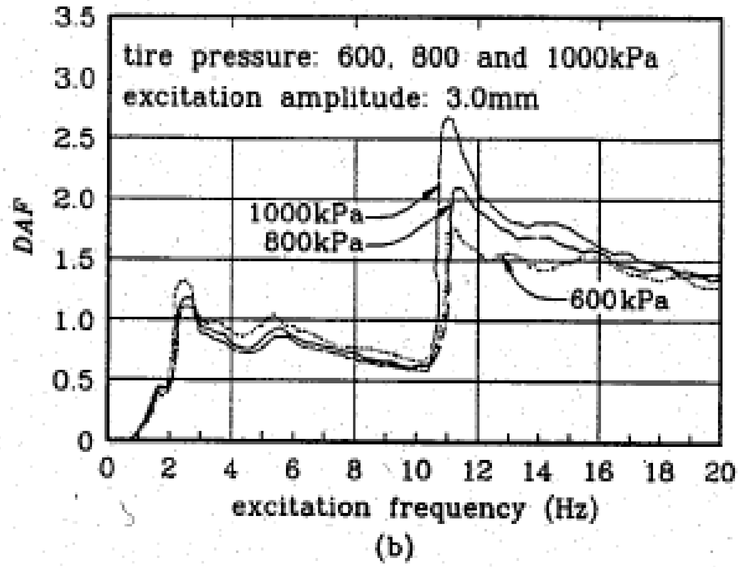


Fig. 9: Effect of tyre pressure on DAF (Tilly, 1986)[15].

In large scale testing carried out by the AASHTO in 1962 to investigate how dynamic loads caused by vehicle-bridge interaction effect bridge structures. Test vehicles (two-axle trucks and three-axle tractor-semi-trailer combinations) made approximately 1900 runs over 15 bridges at speeds varying between 12 km/h and 32 km/h. One observation in relation to vehicle suspension was that the interleaf friction in the suspension system of the trucks has an important effect on the dynamic response of the bridge.

(Agarwal and Billing, 1990)[16] reported "dynamic response and displaying complex vibrations in torsional and flexural modes. The first four modes of the bridge were found to be in the 2 to 5 Hz range corresponding to the resonant frequencies of commercial traffic suspension systems." From the basics of dynamics we can say that if a structure, in this case the bridge, and the excitation force, in this case the vehicle-bridge interaction, have same frequency, resonance will occur. If resonance occurs, the amplitude of vibration will increase substantially, which will cause much larger dynamic responses in the bridge structure. Therefore if the bridge and the vehicle's suspension system have the same resonant frequencies, the dynamic load due to the bridge-vehicle interaction will be considerably greater than otherwise anticipated.

From these findings we can say that vehicle tyre pressure and vehicle suspension have a large effect on the dynamic load induced by a vehicle crossing a bridge.

2.5.4 How the weight of a bridge effects Bridge Dynamic Loading:

Over the last half century bridge structures have become lighter and lighter. More emphases are being placed on the aesthetics of bridges and how they fit into there natural surroundings, which has resulted in the reduction in the size of bridge members in particular the bridges deck. As the bridges have become lighter, they have also become more susceptible to dynamic loads. We can see from Eq. 7 and Eq. 8 that as the size of the bridge members decreased, so to do the bridges stiffness (k). We can see from comparing Fig. 4 to Fig. 5 that if the stiffness of a bridge decreases, it becomes more susceptible to the applied dynamic loading it is subjected to.

$$K = 4EI / L \qquad \text{Eq. 7}$$

where

I is the second moment of area of the beam,

E is the Young's modulus,

L is the length of the element

$I = bd^3 / 12$ Eq. 8

where

b is the bridge decks width,

d is the bridge decks depth.

2.5.5 How bridge surface roughness effects Bridge Dynamic Loading:

One of the principle factors in the size of the dynamic load caused by a vehicle crossing a bridge is the bridges surface roughness. The rougher the bridge pavement is the greater the dynamic load induced. Roadway surfaces are by no means perfectly smooth, therefore a vehicle's suspension must react to roadway roughness by compression and extension of the suspension system. This oscillation creates axle forces that exceed the static weight during the time the acceleration is upward, and is less than the static weight when the acceleration is downward. "In the majority of field tests, roadway imperfections and irregularities were found to be a major factor influencing bridge response" (Eyre and Tilly, 1977)[17]. To illustrate this point it is useful to look at Fig. 11.

Many researchers in the field of bridge dynamics along with testing dynamic loading due to vehicles driving across the bridges surface also place planks, usually 20-50mm thick, on the bridges surface and record the induced dynamic load when the vehicle drives over the plank. These tests are important as many bridges have a lateral indentation on the bridge deck as one enters the bridge. This can be due to poor compaction of the soil under the bridge deck in this area. This indentation cause's excitation in the vehicles suspension systems as the vehicle enters the bridge and this is simulated by the plank placed on the bridges surfaces in research work.

Fig. 10 shows tests carried out by (Cantieni, 1984)[18, 19] on concrete bridges. In 73 tests, the vehicle moved along an undisturbed pavement, in another 69 tests, the vehicle crossed a plank, 50mm thick, 300mm wide, placed at the point where the deflection was recorded.

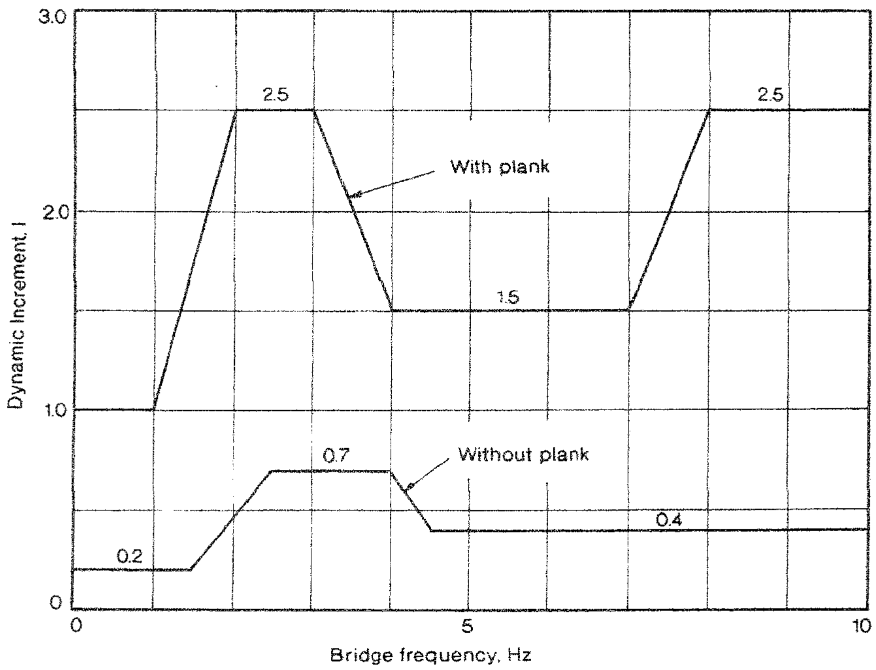


Fig. 10: Upper bounds to values of I observed by (Cantieni, 1984)[18, 19] for truck passages with and without a 50mm plank.

From Fig. 10 we can see that the bridge surface which the vehicle crosses over impacts on the induced dynamic load greatly. In fact most of the modern design codes, including the Eurocode have bridge surface roughness is a principle factor in determining DAF.

Eurocode 1: Part 3 (2003) (Annex B) states that,

> "(1) A stress history should be obtained by analysis using recorded representative real traffic data, multiplied by a dynamic amplification factor φ.
>
> (2) This dynamic amplification factor should take into account the dynamic behaviour of the bridge and depends on the expected roughness of the road surface and on any dynamic amplification already included in the records".

2.5.6 How bridge length effects Bridge Dynamic Loading:

Most modern design codes in there equations for calculating the dynamic component of traffic loading have the bridges length as the only variable function. This indicates the importance of the length of a bridge to how it behaves under traffic loading. Most of these design codes see Fig. 20 have the DAF value decreasing with increasing bridge length. Some design codes suggest the bridges of a certain length, usually around 50m (NAD ENV1991:3), are governed by congested traffic flow and hence there is no need to use a DAF. "For long span bridges, the impact problem becomes even more complex, since it is recognized that for such bridges maximum loading occurs with traffic stationary, and, consequently, it is suggested that further allowance for impact is not necessary" (Buckland, 1991)[20].

We can see that from Eq. 7 that if the bridge length increases the stiffness, k, decreases. If the stiffness decreases the bridge becomes more susceptible to dynamic loading. This suggests that with increased length the DAF value should increase.

If a bridge is to be design as congested flow, with no dynamic factor, at a certain length it suggests that when vehicles are on a bridge there speed decreases. If free flowing traffic travels at approximately 80-100km/h and congested traffic flows at approximately 0-10km/h and bridges of 50m are to be designed for congested flow, this suggests a fall in speed of 80-90km/h over the 50m of bridges length. This suggests a theoretical deceleration of approximately 1.7km/h for every meter travelled along the bridge, thus speed is decreasing with increasing bridge length. If we look at Fig.8 which shows DAF increases with increasing speed, and also take into account that speed decreases with increasing bridge length, it indicates that DAF will decrease with increasing bridge length.

(Coussy et al., 1989)[21] presented a theoretical study of the effects of random surface irregularities on the dynamic response of bridges under suspended moving loads. A single degree of freedom oscillator was used for each axle of the vehicle, while the bridge was represented by an elastic beam with constant flexural rigidity and linearly distributed mass. The profile of the roadway was described by a random process with spectral density taken from previously reported experimental data. The authors concluded that the DAF decreases with span length but not as strongly as given by the codes of different countries. This was explained by the coupling of bridge and vehicle motions. Results of this investigation also suggest that the DAF is independent of the span length in the absence of surface irregularities.

2.5.7 How a bridge's dynamic characteristics effects Bridge Dynamic Loading:

The extent of structural damping depends mainly on the structures energy absorption capabilities. The cause of the energy dissipation may be due to many different effects such as material damping, joint friction and radiation damping at the supports. It seems obvious that if a bridge has high damping characteristics, the bridges dynamic response will be reduced.

(Eyre and Tilly, 1977)[17] carried out tests on 23 bridges having spans of 17 to 213 m, consisting of steel box girders with steel decks, steel box girders with concrete decks and steel plate girders with concrete decks. "The authors found that the measured damping values increased with the amplitude of vibration and could reach values four times higher than the values for small amplitudes. They also noticed that damping tended to increase with frequency, resulting in values of damping higher for single-span bridges than multi-span bridges."

Damping values for 225 bridges located in Europe were obtained in various field tests and are summarized in Table 3, along with damping values for 19 out of 27 bridges recently tested in Ontario (Billing, 1984)[22]. Note that the damping values obtained by Billing are relatively small compared to the damping values reported by Tilly. This is probably due to the use of different methods for the evaluation of damping (logarithmic decrement, half power bandwidth, etc). Although high levels of damping reduce dynamic response, more research is required before the exact influence of damping on the DAF can be ascertained.

Type of Bridge	Span Length (m)	Number of bridges	Average damping	Lowest damping
Concrete, Switz., UK, Belg. (Tilly, 1986)	10-85	213	0.079	0.020
Composite, steel-concrete UK (Tilly, 1986)	28-41	12	0.084	0.055
Prestressed Concrete (Billing, 1984)	8-42	4	0.022	0.008
Steel (Billing, 1984)	4-122	14	0.013	0.004

Table 3: Typical values of measured damping of highway bridges (Tilly and Billing 1986, 1984).

(O'Connor and Shaw, 2000)[23] carried out tests on 198 concrete bridges and found values of damping indicated by the following values of the logarithmic decrement: minimum, 0.019: mean, 0.082; maximum, 0.360.

2.5.8 How a bridge's frequency effects Bridge Dynamic Loading:

The natural frequency of a system or structure is the frequency at which a mechanical system will vibrate freely. A pendulum, for example, always oscillates at the same frequency when set in motion. More complicated systems, such as bridges, also vibrate with a fixed natural frequency. For simple structure such as a beam the natural frequency depends on two system properties; mass and stiffness. For a single degree of freedom oscillator, a system in which the motion can be described by a single coordinate:

$$f_n = (1/2\pi)(k/m)^{1/2} \qquad \text{Eq. 9}$$

k = stiffness of the member

m = mass of the member

f_n = natural frequency in hertz (1/seconds)

Another way of expressing the natural frequency of a member is in terms of the static deflection, Δ, by observing,

$$k\Delta = mg \qquad \text{Eq. 10}$$

Thus, Eq. 9 can be expressed in terms of the static deflection, Δ, as

$$f_n = \frac{1}{2\pi}\sqrt{\frac{g}{\Delta}} \qquad \text{Eq. 11}$$

A key aspect of structural dynamic analysis concerns the behaviour of a structure at "resonance." If a varying force with a frequency equal to the natural frequency is applied a structural member the vibrations can become violent, this phenomenon is known as resonance. The natural frequency of vibration of a structure corresponds to that structure's resonant frequency. If a structure is subjected to vibration at its natural frequency, the displacements of that structure will reach a maximum ("resonance"). The greater the displacement, the greater the stresses that are developed in the framing members and connections of the structure. The Tacoma Narrows Bridge is an example of the destructive effects that resonance can have on a structure.

The frequency of a bridge is a very important factor when analysing how a bridge responses to dynamic loads caused vehicle crossings. Most freight trucks suspension systems have a natural frequency of 2-5Hz. When a truck crosses a bridge its suspension interacts with the bridge, as the trucks tyres travel across the bridges uneven deck it begins to "bounce". The suspension system of the truck dampens the effect of the uneven surface and truck tyre interaction but causes the truck to initially "bounce" or vibrate at a given frequency. This frequency is called the natural frequency for the trucks suspension system. Most freight trucks suspension systems have a natural frequency of 2-5Hz. Therefore to avoid resonance, bridges should not be designed to have a natural frequency between 2 and 5 Hz.

(O'Connor and Shaw, 2000)[23] also describe testing carried out on 226 bridges from 1958 to 1981, "In 1958, a decision was made to standardise test procedures, and from then until 1981 static and dynamic load tests were carried out on 226 slab and girder highway bridges. Much of this (and later) work has been described by R. Cantieni and his associates (Bez, Cantieni and Jacquemoud 1987; Cantieni 1984a, 1984b, 1987, 1992; Krebs and Cantieni 1997). Cantieni (1984a and 1984b) reported first the measurement of natural frequencies (f_n) for 224 bridges, with spans from 13 to 86m. He plotted f (Hz) against the maximum span L and derived the best-fit curve:

$$f = 90.6 \text{ x } L^{-0.923} \qquad \text{Eq. 12}$$

The standard deviation of departures from this curve was +- 0.8f(Hz)".

Further testing of bridges by Centre de Recherches Routieres (Brussels), Ministry of Public Works (Liege), EMPA (Switzerland) and MTCO and MTQ generated Fig.4 with a best-fit curve:

$$f = 82 \text{ x } L^{-0.9} \qquad \text{Eq. 13}$$

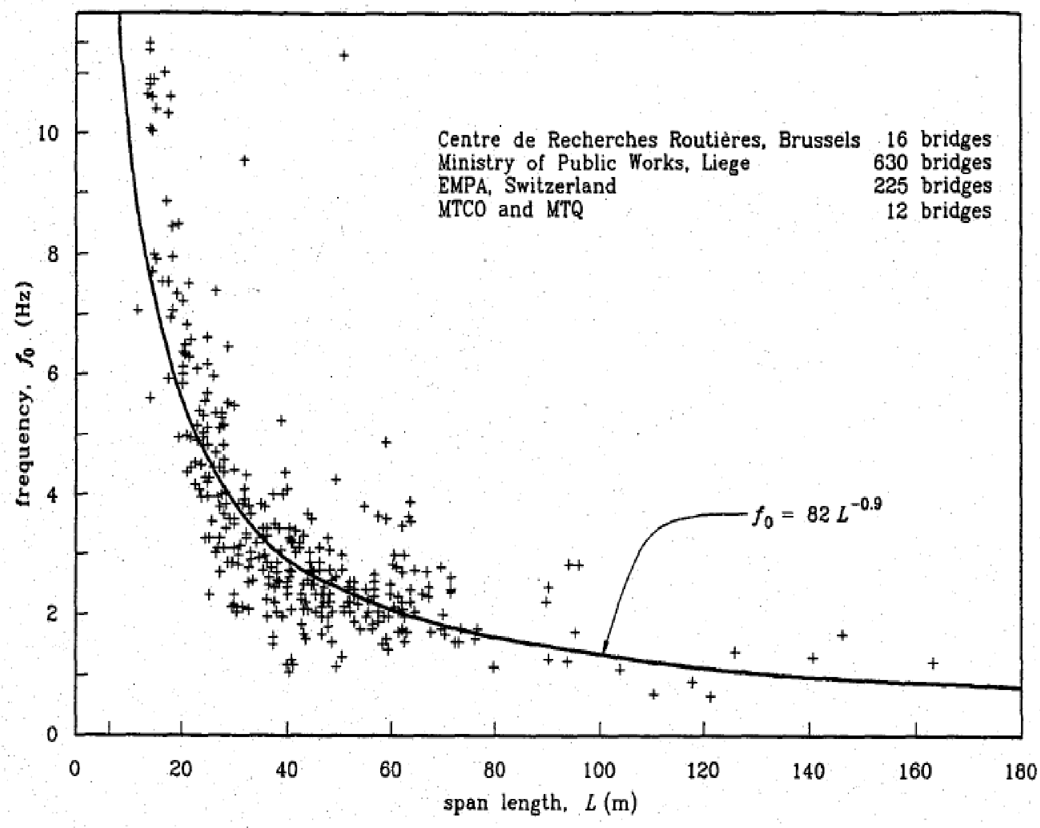


Fig. 11: Fundamental frequencies versus span length for 898 highway bridges (Taly, 1998)[24].

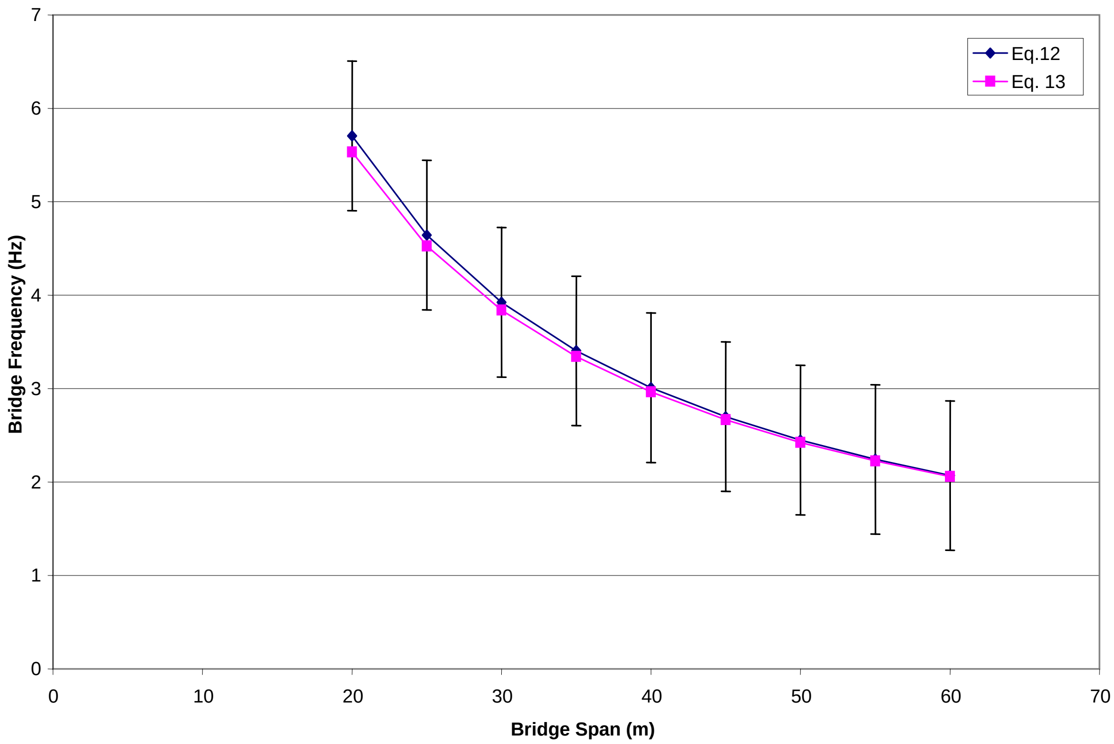


Fig. 12: Bridge Frequency versus Bridge Span (comparison between Eq. 12 and Eq. 13).

From Fig. 12 we can see that there is little difference between Eq. 12 and Eq. 13. For bridges between 20-60m in length there natural frequency is between 2-6Hz. As approximately 90% of the bridges in Europe are between 20-60m long we can say that most bridges in Europe have a natural frequency between 2-6Hz. As most bridges have a frequency in the same region as trucks suspension systems (2-6Hz) the probability of resonance occurring in a bridge's design life for a single truck crossing is quite high.

From Eq. 9 we can see that the frequency increases with increasing stiffness and decreases with increasing mass. From Fig.7 and Fig.8 we can see that the dynamic response decreases with increasing stiffness. Also from section 2.5.4 we can see that the dynamic response decrease with bridge weight. This indicates if we neglect resonance, the dynamic response of a bridge decreases with increasing bridge natural frequency.

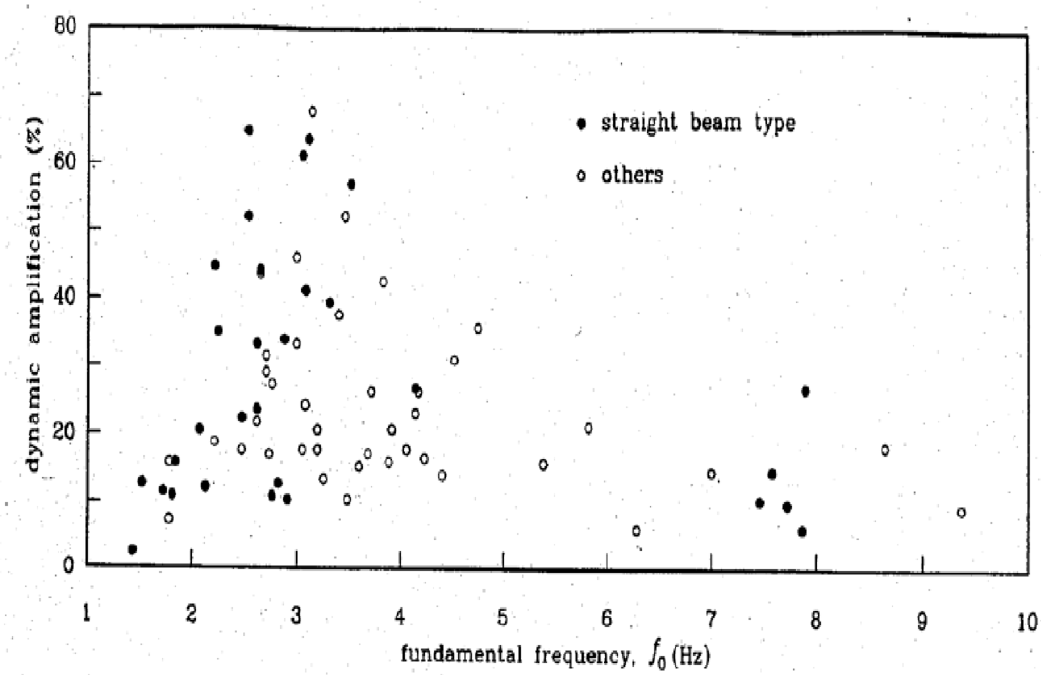


Fig. 13: Effect of bridge geometry on the dynamic amplification (Cantieni, 1983)[9].

This relationship is difficult to see from Fig.13. This is due to the fundamental frequency of the majority of highway bridges fall between 2-6 Hz. "The natural vibration frequency of a bridge has a considerable influence on its dynamic response. It is now well established that the majority of modern highway bridges have fundamental frequencies in the range of 2 to 5 Hz, corresponding to the resonant frequencies of commercial vehicles" (Eyre and Tilly, 1977)[17]. Because of this bridges with a natural frequency between 2-5 Hz are more susceptible to dynamic loading.

This can be seen in Fig. 14 which illustrates the design codes form different regions of the world, “The ordinate axis represents the load increase or dynamic load allowance (DLA) and the abscissa is the fundamental frequency of the structure. In cases where the specification value is a function of span length (e.g. AASHTO (1996)), the frequency is estimated using an empirically based formula. Note the widely variability for DLA. This variability indicates that the worldwide community has not reached a consensus about this issue” (Paultre et al., 1992)[25].

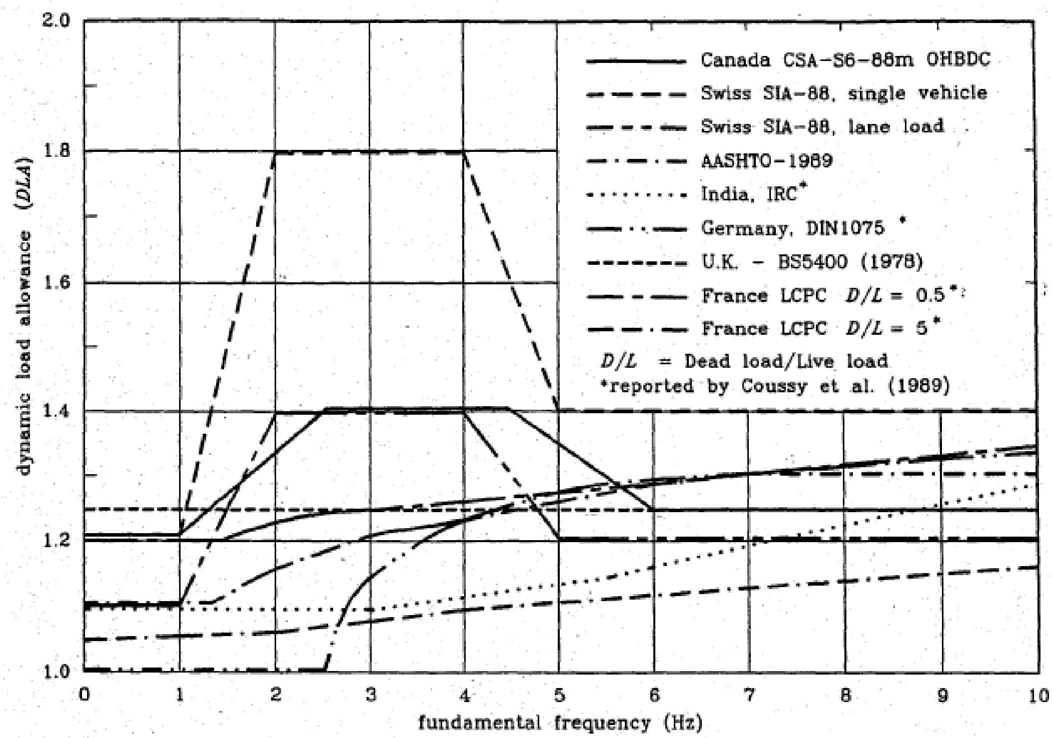


Fig. 14: Dynamic load allowance (DLA) versus fundamental frequency for different national codes (Barker and Puckett, 1997)[26].

Since it is virtually impossible to assess the influence of the various factors, most design codes prefer rather simple and straight-forward methods, not necessarily physically “correct” as regards the impact values. Most design codes deal with the impact factor either by a coefficient decreasing with increasing span length or by a constant coefficient already included in the “static” live load. Recent research results seem to indicate the impact factor to be mainly dependent upon the natural frequency of the structure with peak values between 2 and 5 Hz.

2.5.9 How the period of loading effects Bridge Dynamic Loading:

As the speed and weight of vehicles increased, the load applied to the bridge from the vehicle becomes more of an instant load. Normal free flowing traffic travels at approx 80km/h, say if we look at an arbitrary truck travelling at 80km/h over a 20m long highway bridge. The period that the truck applies a load to the bridge is:

Truck speed = 80km/h
= 22.22m/s

therefore the truck is actual on the bridge for

(20 / 22.22) = 0.9 seconds

As the period of loading is so short the load may be considered as a sudden or actual impact load. Intuitively it can be said that a load applied instantly to a beam causes much larger stress than the same load applied statically to the beam.

2.5.10 Conclusions from Theoretical Investigation into Bridge Dynamic Loading:

	Variable in vehicle- bridge interaction	**Effect on Dynamic Response of Bridge**
1	vehicle speed	increases with increasing vehicle speed
2	vehicle weight	decreases with increasing vehicle weight
3	vehicle tyre pressure	increases with increasing vehicle tyre pressure
4	vehicle suspension	increases substantially if vehicle suspension has frequency in the range of 2 to 5 Hz
5	weight of bridge	decreases with increasing weight of bridge
6	surface roughness	increases with increasing surface roughness
7	bridge length	decreases with increasing bridge length
8	bridge damping	decreases with increasing bridge damping
9	bridge frequency	decreases with increasing frequency, (more importantly, if frequency of the bridge is in the range 2 to 5 Hz, the dynamic response increases dramatically)
10	period of loading.	decreases with increasing period of loading

When considering the live load on bridges due to traffic, the live load can be categorised into loads, the static load due to the vehicles axle weight and the dynamic load due to the vehicle-bridge interaction. The static load can be worked out accurately but the dynamic load has many variables and is therefore more complicated to calculate. Recent research has indicated that the dynamic load should be worked out as a function of the bridges natural frequency but as the natural frequency is quiet hard to predict when designing a bridge, most design codes use the bridges length to calculate the dynamic load. This dynamic load, DAF, is multiplied by the extreme static load to calculate the design load.

As there are so many variables influencing the dynamic load and dynamic response of the bridge, there are many conflicting views and how the dynamic load should be calculated. “The impact formulas provided by most current design codes are not consistent in physical units and lack a solid theoretical basis, of which the application should not be extended to bridges travelled by vehicles at high speeds. A more rational approach is to relate the impact factor, which is non-dimensional, defined as the ratio of the driving frequency of the moving vehicles to the vibration frequency of the bridge”(Taly, 1998)[24].

3. Historic Review:

The primary propose of a bridge is to carry some form of load from one point to another. Bridges have been built since ancient times, and the main loading that bridges had to react against in ancient times was its own self weight. As transport needs increased and developed, bridges have multiplied and improved.

Bridge development until recently has been closely related to previous experience, "If it worked before why change things?" But as the loading on bridges and bridge spans increased dramatically, so did Engineers desire to understand how a bridge responses to different types of loading.

As with most research over the last century, research into loading of bridges was sparked by the advent of war. Up until the beginning of the twentieth century, pedestrians and horse and carts were the main external vertical loads that a bridge would be subjected to. Even at this stage some bridge engineers understood that responses, such as deflection, shear force and bending moment, of the bridge were greater than the static loads being applied. This addition in the response was initially named an "impact load". Although there was no mention of it in the design codes for bridges of the time some bridge engineers as early as 1857, Gerber, were developing formulas to deal with this impact load. Impact loading was first looked at for railway bridges, with a formula to account for impacting loading introduced by an American engineer, C.C. Schneider in 1887.
"Schneider's formula was known as the "Pencoyd" formula and was as follows:

$$f_i = f_e(300/(L + 300)) \qquad \text{Eq. 14}$$

where f_i was the impact stress, to be added to the live static stress, was the calculated static live stress, and L was the length of the loaded distance, in feet, which produces the maximum stress" (Dawe, 2003)[3].

Engineers realised that this additional response was due to moving loads crossing the bridge structure. If a load travelled across a bridge at speed it generated a larger response to that of the same load applied to the bridge statically. This additional load was due to the dynamic behaviour of the bridge due to the moving load. Bridge engineers of the time realised it was important to consider the effects of this dynamic behaviour of highway bridges. They noticed that stress increased due to the dynamic response of the bridge to moving loads. As bridge engineers design bridges to react against applied stresses, it was important for them to understand and be able to calculate the increase in stress due to moving loads.

There was little, if any, agreement among engineers in the middle of the nineteenth century over the effect of a moving load on a beam. “While some assumed that a load moving with a high speed acts like a sudden applied load and may produce deflections larger than those corresponding to the static action, others argued that at very high speeds there was insufficient time for the load to drop through the distance of the expected dynamical deflection” (Timoshenko, 1952)[27].

“Dynamic tests on beams by Willis, James, and Galton during the 1850s showed that deflections increased with increases in speed and that the dynamic deflections were two or three times larger than the static deflections obtained at higher speeds. However, experimental investigation of actual bridges did not show the effects of speeding in such a marked way” (Dawe, 2003)[3].

In the United States, around 1900, lack of understanding of the impact phenomenon made it somewhat customary to post, even on new and sturdy bridges, signboards warning traffic to cross at a walking pace. Signs worded “Warning! Walk your horses. Penalty: $5.00 fine” or “$5.00 fine for crossing faster than a walk” were common.

In 1914 with the beginning of World War 1 there was a large increase of motor vehicles transporting goods and machinery throughout UK. To deal with this the Ministry of Transport put forward a research program to nationalise loading for highway bridges. The static load could be worked out quite accurately but there was lots of questions surrounding the value of impact loading that should be used. In recent years the term impact load as been change to dynamic amplification factor (DAF) and is defined as the ratio of total to static load effect:

$$DAF = \varphi_{stat} / \varphi_{tot} \qquad \text{Eq. 15}$$

Where φ_{stat} is the static response and φ_{tot} is the total response by the bridge to the applied load.

Initially the dynamic loading was looked at in three ways:

1. dynamic load can be defined as a sudden applied load
2. dynamic load can be defined as a load whose period of application as shorter than the fundamental period for the structure on which the load is applied
3. dynamic load can be related to the bridge-vehicle interaction.

1. Dynamic load can be defined as a sudden applied load

See section 2.5.9.

2. Dynamic load can be defined as a load whose period of application as shorter than the fundamental period for the structure on which the load is applied.

Considering only the fundamental mode of vibration, G.G. Stokes (1819-1903) was the first to show that the magnitude of dynamic deflection depended on the ratio of the period of the beams fundamental mode of vibration of the time taken by the moving force to cross the span. Progress in this field was made by Homersham Cox, who concluded in 1849 from energy considerations that dynamic deflection was limited to twice the static deflection.

3. Impact can be related to the bridge-vehicle interaction.

The third effect is the interaction between the vehicles tyres the bridges uneven surface. As a vehicle transverses a bridge its tyres move along move the bridges pavement and any imperfection in the pavement will result in a 'bouncing' reaction in the vehicle. The vibration of the vehicle induces vibrations in the structure. The magnitude of stresses induced the bouncing effect is dependent on a number of factors:

- The condition of the bridges pavement, the more uneven a bridges surface is, the greater the dynamic stress induced. Even if a bridge surface is relatively smooth, it will still induce some dynamic response.
- The speed at which the vehicle is travelling.
- The relative masses of the vehicle and the bridge.
- The natural frequency of the structure.
- The damping characteristics of the bridge.

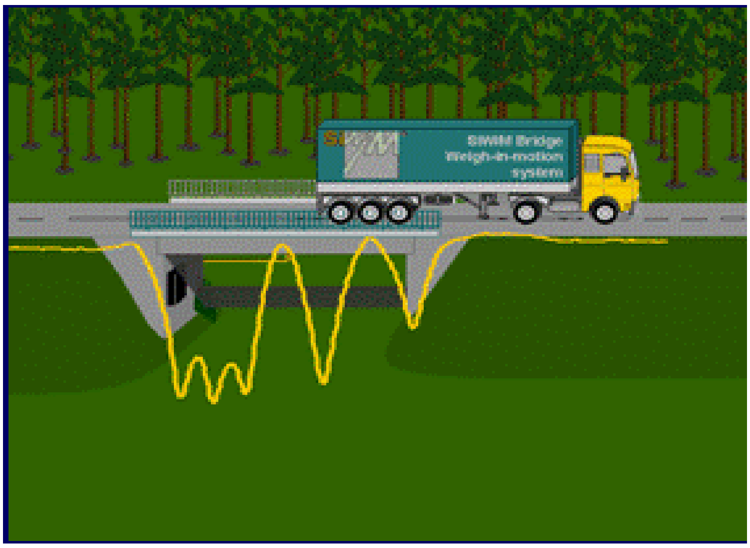


Fig. 15: Static mid-span response of bridge to truck crossing[28].

Figure 15 shows the mid-span response of a highway bridge due to a truck crossing, the peaks represent the trucks axles. Along with this static response there is an associated dynamic response. Form Figure 16 we can see how the dynamic response oscillates about the static response.

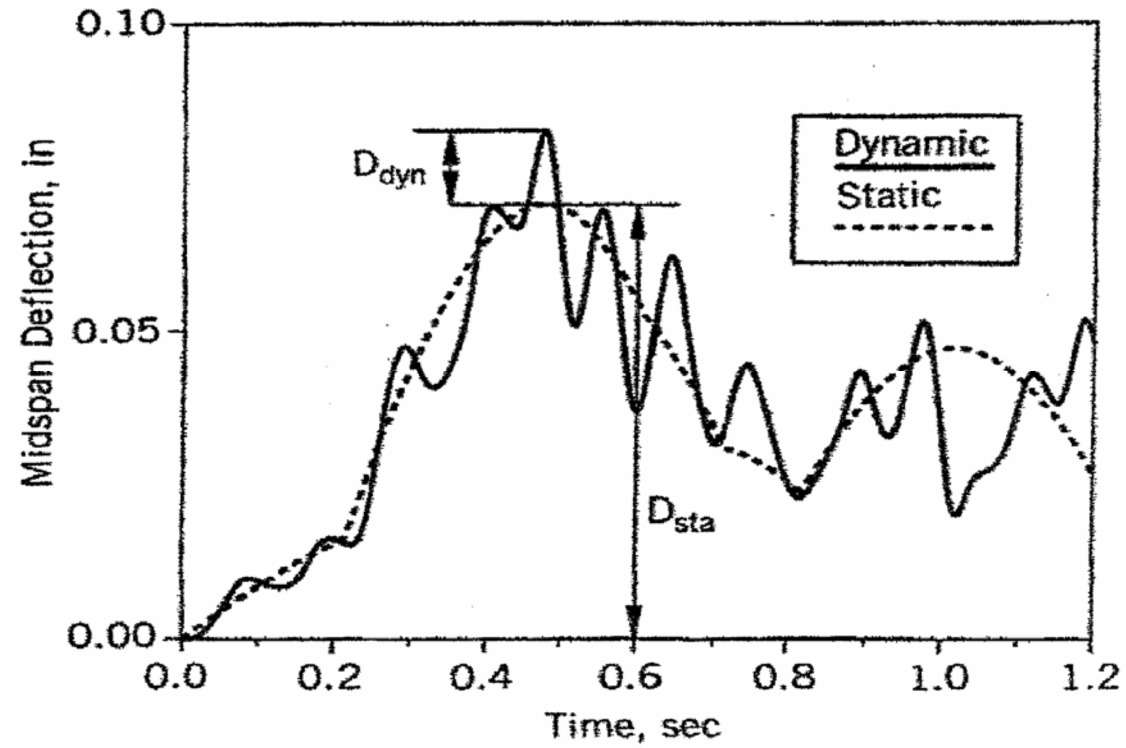


Fig. 16: Static and Dynamic mid-span response to truck crossing (Hwang and Nowak, 1991)[13].

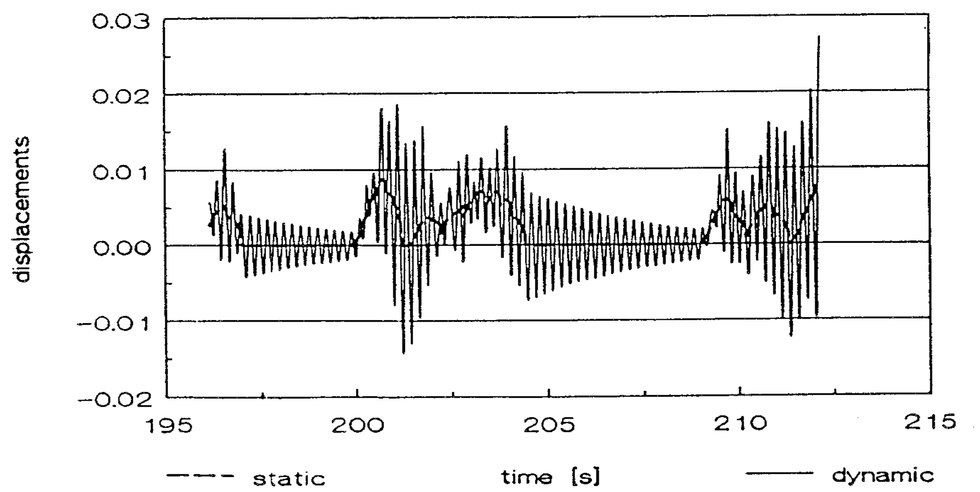


Fig. 17: Static and Dynamic displacements for 20 metre bridge (lanes, roughness class good) (Vrouwenvelder and Waarts, 1993)[29].

Fig. 18 shows results of test carried out by (Vrouwenvelder and Wart, 1985)[29] "simulations carried out for short periods only (24 periods of 5 min per hour) and extrapolated to a full day. Fig17 gives a typical simulation results for a 2 lane bridge with L = 20 metres and roughness class "good".

Over the last century there have been many changes in the views of researchers into the correct approach that should be taken to obtain an accurate DAF value. Because of this there have been many changes in the design codes for bridge engineering to the value of DAF that should be used for highway bridges. Traffic loading on Highway bridges by (Dawe, 2003)[3] provides us with an in-depth historic review into the changes in concepts and values surrounding DAF since the idea was first conceived.

A brief outline of this is presented below:

- At the end of WW1, in 1918, the Ministry of Transport was established and became responsible for issuing the first national loading rules for bridges.

- <u>Report on the administration of the Road Fund (1921-1922)</u>

The train of vehicles was based on the heaviest commonly occurring vehicles such as might be used by agricultural contractors, except that all the actual axle weights were increased by 50 per cent to allow for impact. This might appear to be somewhat excessive, especially in view of the slow speed of such vehicles, but perhaps reflects the lack of knowledge at the time about impact effects.

- <u>BS 153 British standard unit loading for highway bridges (1925)</u>

The impact effect was calculated from a simple formula which took account of the loaded length of the member concerned and the number of lines of traffic supported by that member with a max value of 0.7.

- BS 153 Standard specification for girder bridges (revision) (1937)

New rules for assessing impact were introduced with the impact factor reducing with increasing span. For bridges carrying one lane of traffic the maximum impact factor was 60 per cent whilst for bridges carrying two or more lanes it was 50 per cent. It was assumed that the surface of the carriageway on and immediately adjacent to the bridge was maintained in good condition.

- Code of practice for simply supported steel bridges (1949)

The committee had received considerable evidence that the effect of impact was considerably less than had been previously allowed and therefore recommend a radical reduction in the allowance in cases where the Ministry of Transport Standard Loading, which included a 50 per cent allowance, was not being used. For slabs, stringers and girders of less than 100ft span an impact allowance of 50 per cent was to be added only to the individual concentrated load producing the greatest bending moment or shear. For spans greater than 100ft impact could be ignored.

- BS 153: Part 3A (1954)

An allowance for impact was included in the derivation of the normal loading from the train of vehicles. This amounted to a 25 per cent increase in the load of any one axle of one vehicle, or any single pair of adjacent wheels of two vehicles travelling abreast.
No impact factors were applied to Abnormal loading (HB loading), since it was assumed that these were slow moving vehicles.

- BS 5400:Part 2 (1978)

A 25 per cent impact allowance was added to the weight of one axle or pair of adjacent wheels in the trains, the position being chosen to give the worst load effect.

- Departmental Standard BD 37/38 (1989)

An impact factor of 1.8 was applied through OPTAX computer program to one axle of the vehicle being considered in the single vehicle case only. OPTAX automatically applied the factor to the axle which had the most effect on the bending moment or shear force. The values of 1.8, was extracted from TRRL lab. Report LR 722 which gave details of impact loads measured under the rear wheel of a 2-axle rigid vehicle traversing 30 motorway bridges. A factor of 1.8 was adopted as the extreme value of impact effects obtained form the tests, ignoring a much higher value which was rejected as being a freak result. The decision to apply an impact factor only to the single vehicle case was felt to be justified because of the low probability of the impact effects of several vehicles in convoy being in phase. The various computer runs produced an envelope of the worst bending moments and shears encompassing all the then legal C&U vehicles, inducing the 38-tonne articulated vehicle, and inducing a 1.8 impact factor where appropriate

Year	**Body**	**DAF**
1921	Report on the administration of the Road Fund	1.5
1925	BS 153- British standard unit loading for highway bridges	1.7
1937	BS 153- Standard specification for girder bridges (revision)	1.5
1949	Code of practice for simply supported steel bridges	1.5
1954	BS 153: Part 3A	1.25
1978	BS 5400:Part 2	1.25
1989	Departmental Standard BD 37/38	1.8

Table 4: History of DAF values used.

Fig. 18: Graph of the history of DAF

The historic review is based on DAF values used in the UK but there is a large variance between DAF values used throughout the world. As there are a large number of variables associated with DAF, it is quite difficult to derive a theoretical method to find an accurate DAF value. To compute a DAF value bridge researchers usually test existing bridges, running a single truck across bridges of different spans. (Dawe, 2003)[3] describes how the UK's Transportation Research Labourite (TRL) tested 28 existing highway bridges over three stretches of road with good, medium and bad surface profiles.

(O'Connor and Shaw, 2000)[23] describes testing carried out by the American Association of State Highway and Transportation Officials (AASHTO) between 1956 and 1960, and reported between 1961 and 1962. "It is fair to describe it as probably the greatest civil engineering research project ever carried out; one of its major objectives was to observe the dynamic behaviour of bridges, all of 15.2m span, with four of pre-stressed concrete, four of reinforced concrete and eight of composite steel and concrete construction. Fifteen of these were studied under approximately 1900 passages of 70 to 80 vehicles of ten different types, with axle hop frequencies from 10 to 13.2 Hz, and frequencies for body bounce, under various conditions, from 1.7-5.6 Hz. The maximum recorded values of the dynamic increment, I, were 0.63 from deflection measurements (with 88% between 0.1 and 0.4), and 0.41 from strains (90% between 0.05 and 0.30). These values are relatively low, and undoubtedly contributed to the relatively low values of I (0.30 or less) specified in the AASHTO design code. Although the number of passages and the vehicle is large, the number of bridges still constitutes a relatively small sample."

Around the world over the last century researchers have tested existing bridges to find there appropriate DAF values. By graphing the DAF values calculated form existing against bridge span, an empirical formula to best fit the graph can be obtained. As only a small number of bridges were looked at in each country and due to the different methods of testing, different DAF values were calculated for different countries.

The applied impact factors are given in the table below:

L	GERMANY	FRANCE - BELGIUM		ITALY	NETHERLANDS		USA	SWITZERLAND	JAPAN
in m.	$1.4 - 0.008L$	Steel $1 + \frac{0.80}{1+0.2L}$	Concrete $1 + \frac{0.64}{1+0.2L}$	$1 + \frac{(100-L)^2}{100\ (250-L)}$	Steel $1 + \frac{40}{100 + L}$	Concrete $1 + \frac{3}{10 + L}$	$1 + \frac{50}{L + 125}$ (L in feet) When L is in metres $1 + \frac{15.24}{L + 38}$	$1 + 0.05\ \frac{L + 100}{L + 10}$	Steel $1 + \frac{20}{50 + L}$
0	1.40	1.80	1.64	1.40	1.40	1.30	1.30(*)	1.50	1.40
10	1.32	1.27	1.21	1.34	1.36	1.15	1.30	1.28	1.33
20	1.24	1.16	1.13	1.28	1.33	1.10	1.26	1.20	1.29
30	1.16	1.11	1.09	1.22	1.31	1.08	1.22	1.16	1.25
40	1.08	1.09	1.07	1.17	1.29	1.06	1.20	1.14	1.22
50	1.00	1.07	1.06	1.13	1.27	1.05	1.17	1.13	1.20
60	1.00	1.06	1.05	1.09	1.25	1.04	1.16	1.12	1.18
70	1.00	1.05	1.04	1.05	1.24	1.04	1.14	1.11	1.17
80	1.00	1.05	1.04	1.02	1.22	1.03	1.13	1.10	1.15
90	1.00	1.04	1.03	1.01	1.21	1.03	1.12	1.10	1.14
100	1.00	1.04	1.03	1.00	1.20	1.03	1.11	1.09	1.13

*) Maximum value 1.30.

Norway, Sweden and United Kingdom Impact factor included in the axle loads.

Fig. 19: DAF values for different countries (OECD 1979)[30].

From Fig. 20 we can see that there are large differences in the DAF values used in different parts of the world.

Germany	1.4 - 0.008L
France	1 + 0.8/(1 + 0.2L)
Belgium	1 + 0.8/(1 + 0.2L)
Italy	1 + (100 - L)^2 / (100(250 - L))
Netherlands	1 + 40/(100 + L)
USA (IM)	15.24 / L + 38.1 <= 0.3
Switzerland	1 + 0.05(L + 100)/ (L +10)
Japan	1 + 20/ (50 + L)
Eurocode	1.4- L/500 >= 1

Table 5: Regions and the applicable DAF (2 lane, steel bridge).

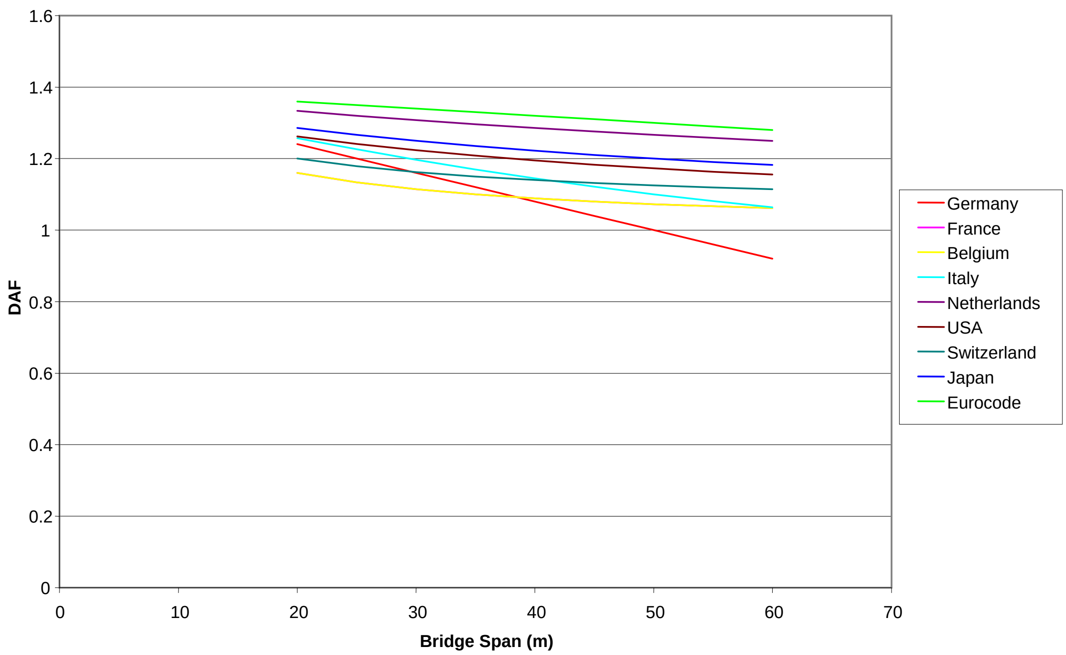


Fig. 20: Different Regions DAF versus Bridge Span (2 lane, steel bridge)

In the current design codes for bridges in Europe, Eurocode 1 part 3, has an equation for DAF in Annex A.3:

(3) Depending on the models under consideration, these models may be assumed to move at low speed (not more than 5 km/h) or at normal speed (70 km/h).

(4) Where the models are assumed to move at low speed, only vertical loads without dynamic amplification should be taken into account.

(5) Where the models are assumed to move at normal speed, a dynamic amplification should be taken into account. The following formula may be used :

$$\varphi = 1{,}40 - \frac{L}{500} \quad \varphi \geq 1$$

where :

L influence length (m)

From Fig. 20 we can see that the Eurocode value for bridge loading is larger than any of the other codes. In fact if the present Eurocode value is compared to the present AASHTO value there is a large difference. The present AASHTO equation for Impact is as follows:

$$I = \frac{15.24}{L + 38.1} \leq 0.3 \qquad \text{Eq. 16}$$

where I is the impact factor. To obtain the DAF value form the AASHTO value I, add 1 to the I value.

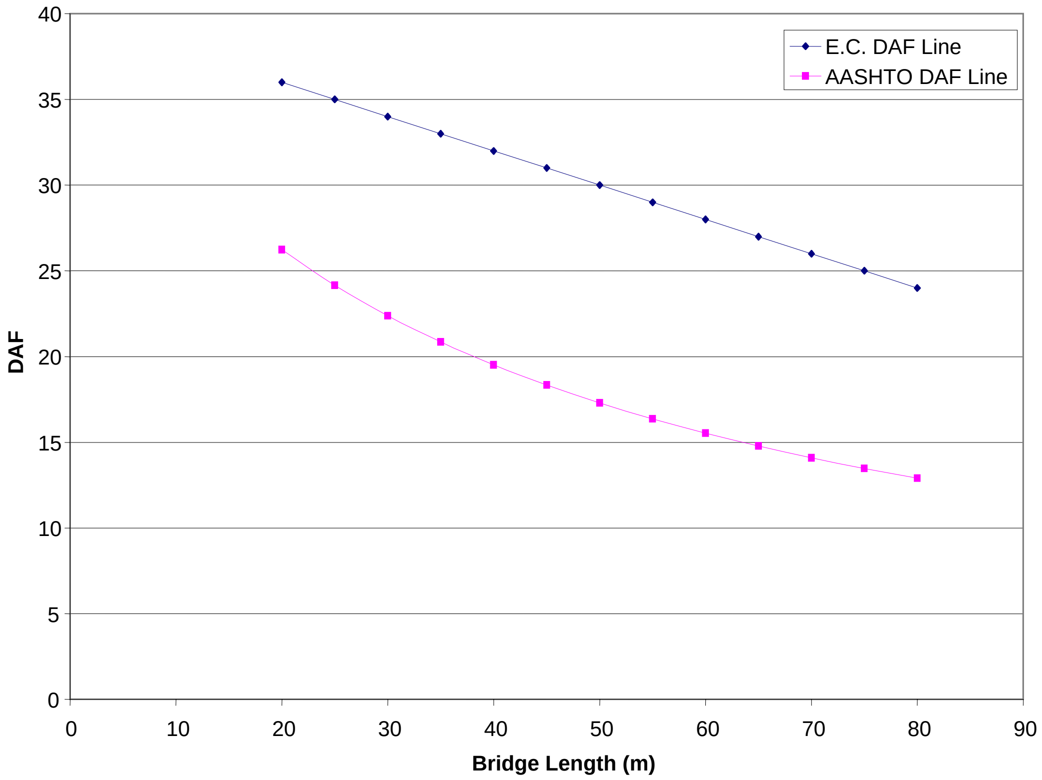


Fig. 21: Bridge Length versus E.C.1:3 and AASHTO DAF

Bridge Length (m)	**Percentage difference between E.C. and AASHTO DAF**
20	27%
25	31%
30	34%
35	37%
40	39%
45	41%
50	42%
55	44%
60	45%
65	45%
70	46%
75	46%
80	46%

Table 6: Percentage difference between E.C.1:3 and AASHTO DAF

Conclusion:

There are large differences in the DAF values used throughout the last century and to date. These large differences represent a general lack of knowledge about the topic. Analysis and Design of Bridges by (Yilmaz and Wasti, 1984)[23] describes, "considering 20 countries whose experience in bridge design is recognized, the dynamic coefficient diagrams according to the bridge span are located within a wide band shown in Fig. 22".

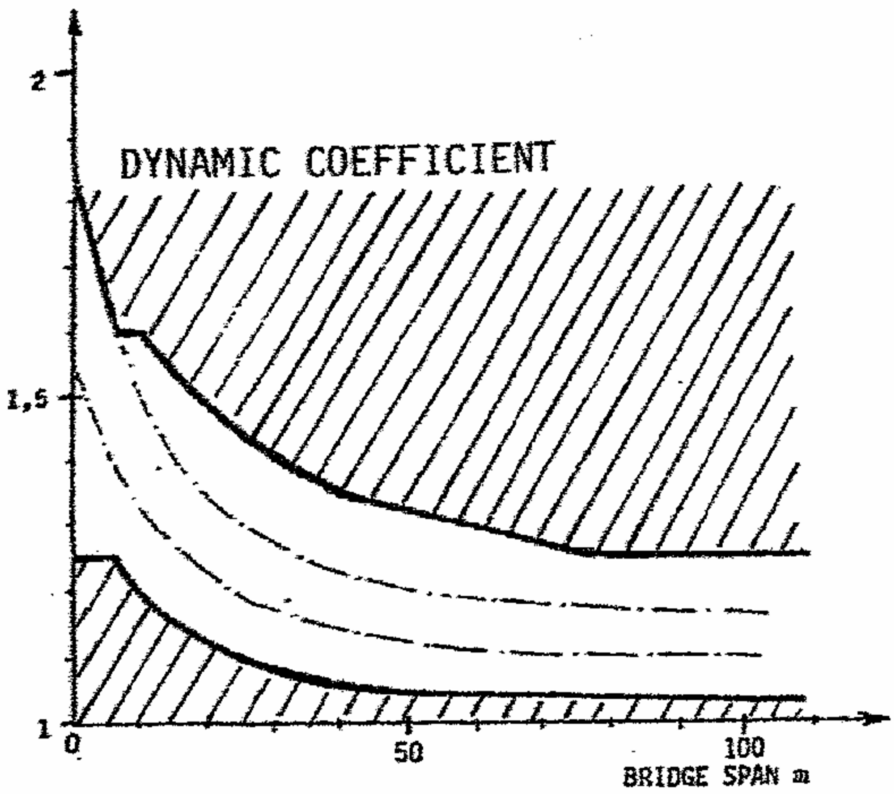


Fig. 22: DAF versus Bridge Span from 20 different countries design codes (Freudenthal, 1947)[8].

The wide band shown in Fig. 22 proves that further research is needed to acquire an accurate equation for DAF.

4. Purpose of Research:

From Fig. 22 we can see that there is a lack of knowledge surrounding the dynamic loading a bridge is subjected to during its design life. Many countries throughout Europe have their own equations for calculating DAF and hence there is a large variance in the design loads bridges are built to in different countries in Europe and the world.

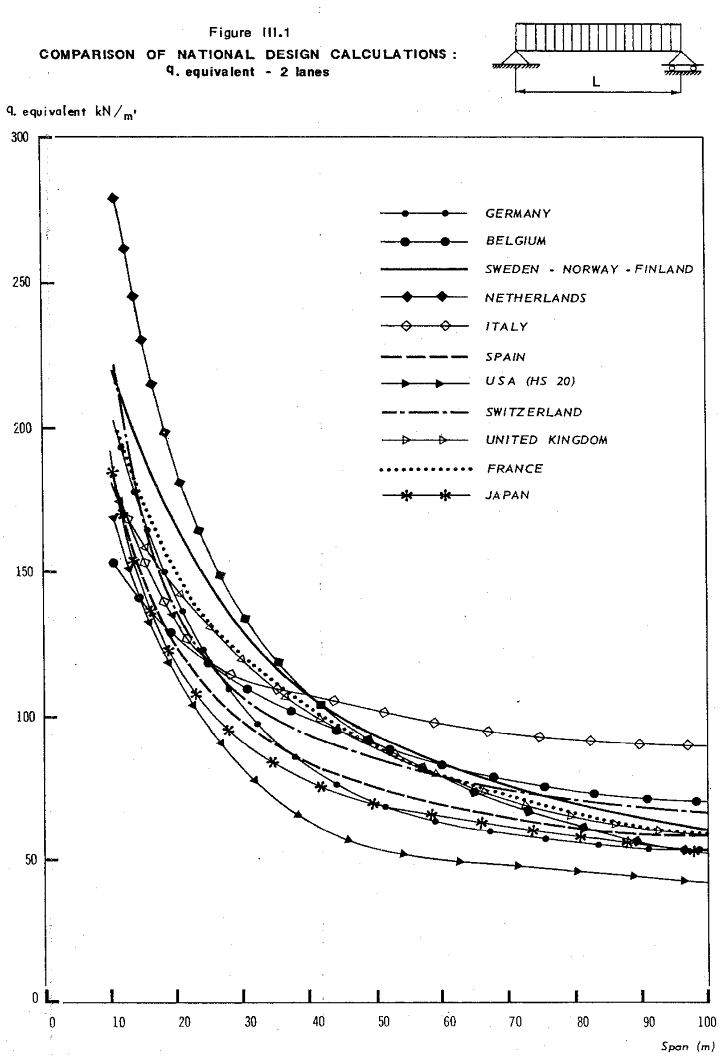


Fig. 23: Comparison of National Design Calculations (OECD 1979)[30].

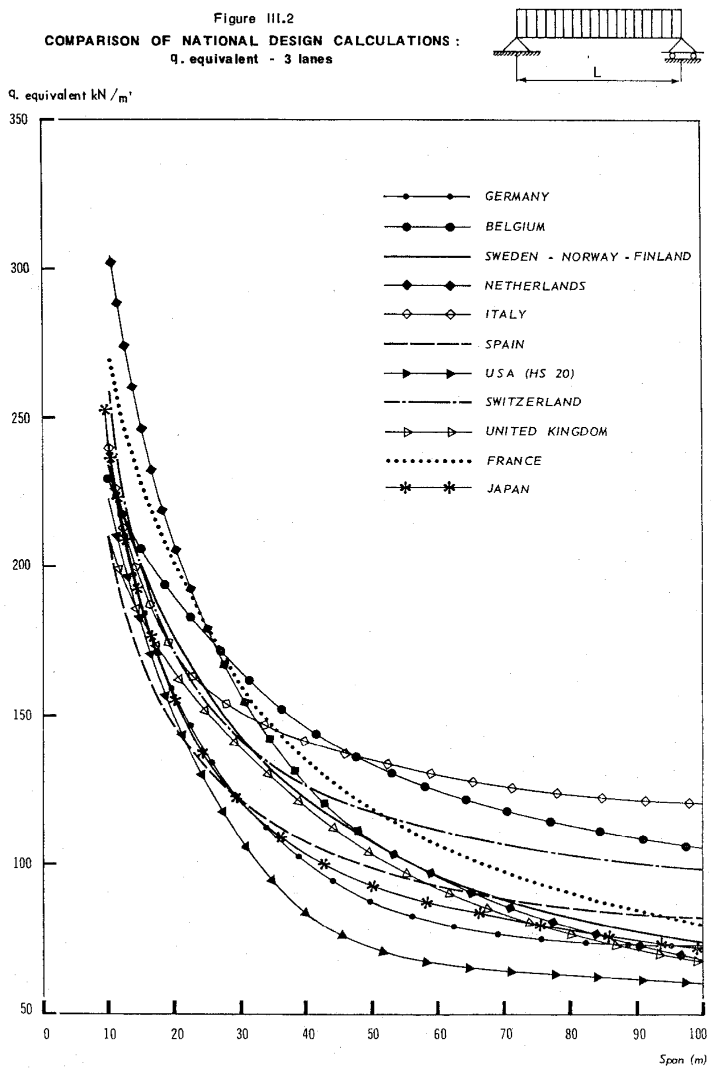


Fig. 24: Comparison of National Design Calculations (OECD 1979)[30].

As the Eurocode is being introduced throughout Europe and bridge loading capacities are being gauged to class bridges so truck routes can be decided, it is an important time to investigate the true nature of bridge dynamic loading. Most bridges are designed with a DAF value of approx 1.3 whereas recent research has suggested a DAF of approx 1.06.

(Caprani and Rattigan, 2006)[4] analysed static and dynamic tests carried out on The Mura River bridge in Slovenia. This study produced Fig. 2, which shows that The Mura River bridge has an, "expected level of lifetime dynamic interaction, for this site and bridge, is a DAF of about 1.06. This is significantly less than the DAF allowed for in the Eurocode of about 1.13 for such a bridge."

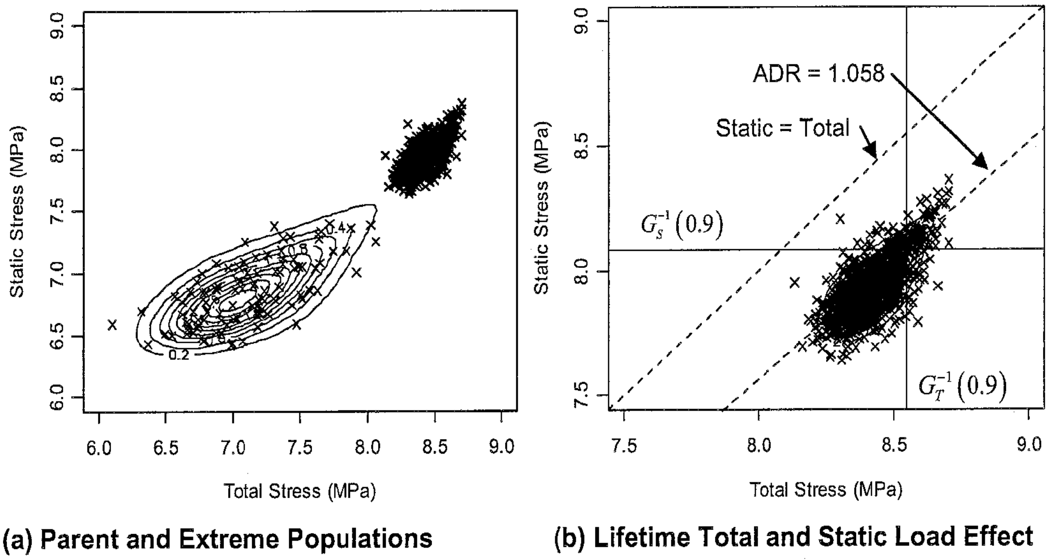


Fig. 25: Multivariate Extreme Value Extrapolation for Lifetime DAF (Caprani and Rattigan, 2006)[4].

"The very slow lifetime dynamic allowance found for the Mura River bridge, if found to be general, will alter the governing loading scenario for the vast majority of bridges. These points are summarised in Fig. 26" (Caprani and Rattigan, 2006)[4].

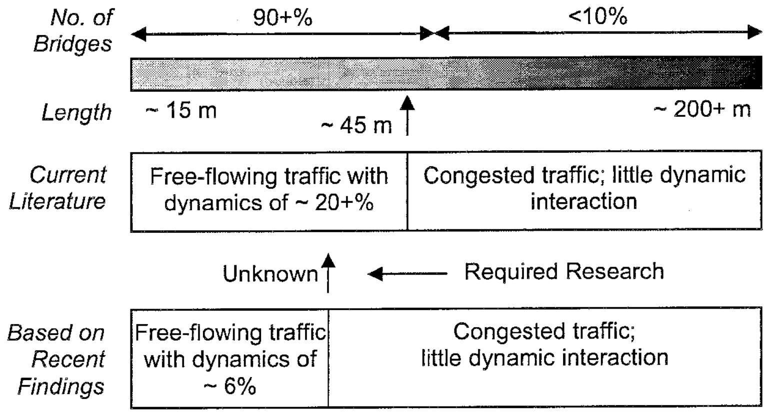


Fig. 26: Governing loading scenarios for different bridge lengths (Caprani and Rattigan, 2006)[4].

The majority of testing carried out on bridges to find the dynamic response of a bridge was preformed with a single truck crossing, this is due to the complications involved in testing with more than one vehicle. “Usually, it is sufficient to consider only one heavy vehicle on the bridge at one time. Nevertheless studies in the United States indicate spacing between vehicles in the same lane of the order of 30 metres, and in multilane bridges with very heavy truck traffic, it does happen that two or more vehicles combine to cause stresses at a specific point that are somewhat greater than would result from a single vehicle” (Dawe, 2003)[3]. From a single truck event, larger dynamic responses will occur, as firstly there is a greater chance of resonance and secondly as there is less static load on the bridge during single truck crossings. But we do not design bridges for single truck events.

If option A, single trucks extreme static load multiplied by a single truck event DAF is greater than, option B, a multi-truck event extreme static load multiplied by a multi-truck event DAF for a given bridge length then option A should be used as the design load. The DAF for single truck event is about 1.28 (DAF values from test carried out by Dawe[3] and we can see from Fig. 32, that for a single truck, the max gross weight is going to be under 500kN. From 1000 year return period results, described in Section 2.2, for free flowing traffic, even for bridges as small as 20 metres there are up to 3 truck events. These 3 truck events contain large trucks up to three 5-axle trucks with a combined static weight of over 1200kN. From this we can see that the extreme static load is governed by more than one truck being present on a given bridge.

Taking a DAF value for a single truck event and multiplying this DAF by the extreme static load, which is a multiple truck event, is therefore unrealistic as these two events can not occur at the one time. Therefore it is theoretically flawed to use a DAF obtained from test carried out on single truck events. The DAF used by different countries design codes are based on tests carried out using single truck crossing. Researchers from around the world carried out thousands of tests on bridges made from different materials, of different lengths and with varying surface roughness. The dynamic response recorded from these tests was graphed versus mainly bridge length. From Fig. 27 we can see the general trend that appeared. A safety factor was added on to this general trend. From this an empirical formula was derived to best fit the general trend plus safety factor. This empirical formula is used to calculate the DAF in design codes with the bridge length as the only variable in the formula.

By graphing extreme load effects due to congested flow for different bridge lengths, it will be possible to obtain the extreme static load for different bridge lengths. Putting the extreme load effects due to free flow for different bridge lengths on the same graph and multiplying the free flowing line with different variables (DAF) it will be possible to obtain the points of intersection of the congested and free flow lines. As the congested flow are travelling at traffic jam speeds, i.e., 5km/h the will not generate any DAF. The free flowing traffic travel at normal speeds of approximately 80km/h and therefore induce dynamic responses in the bridge when crossing.

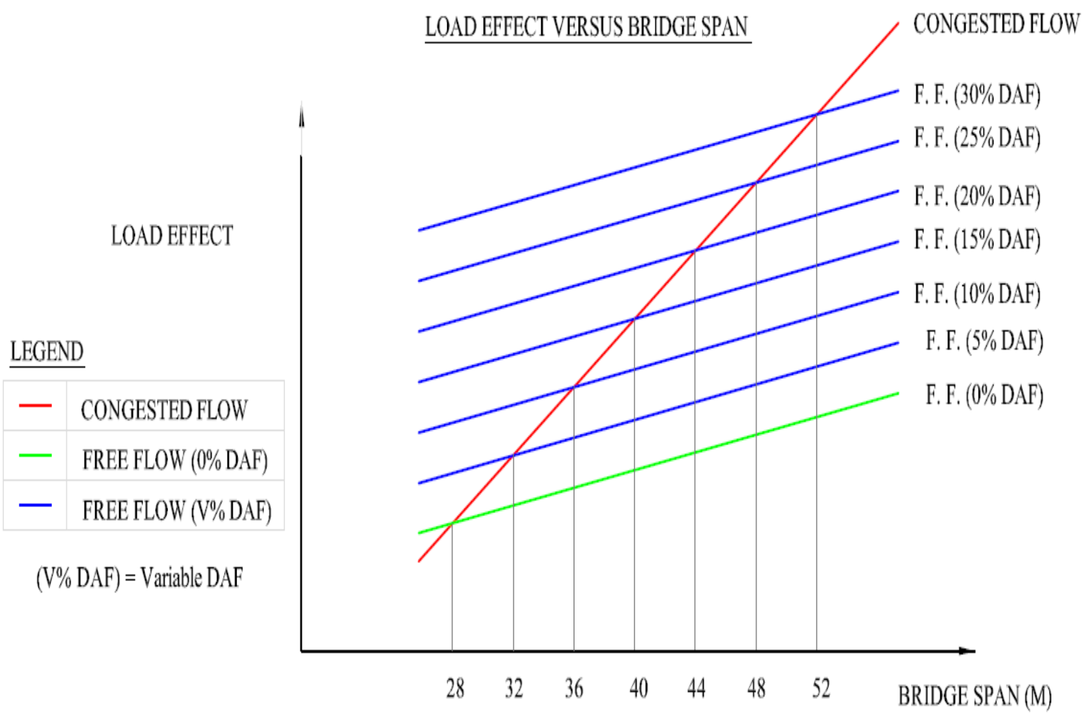


Fig. 28: Load Effect versus Bridge Span (Graph Possibility A).

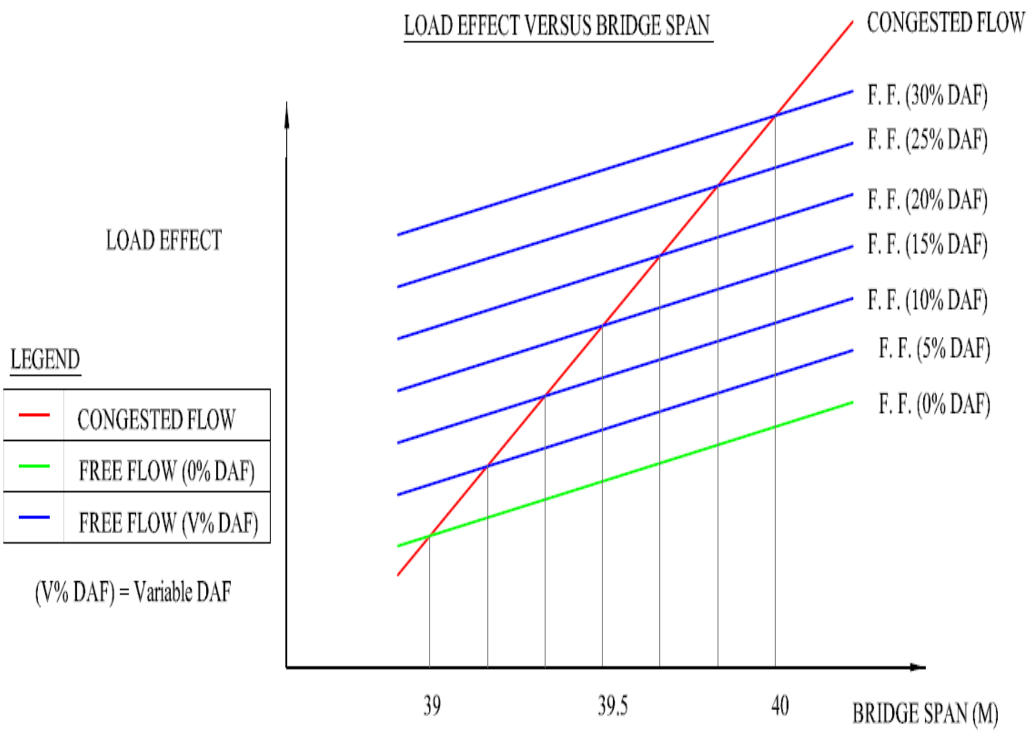


Fig. 29: Load Effect versus Bridge Span (Graph Possibility B)

By taking the intersection points of the free flow multiplied by variable DAF and the congested flow lines and graphing these intersection points against the DAF a graph such as Fig. 28 or Fig. 29 will be produced. The greater the slope of these graphs the greater the importance of the DAF value used in designing bridges. Basically the greater the difference in the slope between free flowing traffic and congested traffic, the less important the DAF is. Also this graph tells us that any highway bridge with a length to the left of the line on the graph should be designed as free flowing and any bridge with length to the right of the line should be design as congested flow with no DAF value. This is true as this line indicates the point where congested flow starts to govern, in the congested line on Fig. 28 and Fig. 29. Congested flow starts to govern free flow traffic when the congested line intersects the free flow line and any bridge length past this point will have a greater congested extreme load than an extreme free flow load multiplied by DAF.

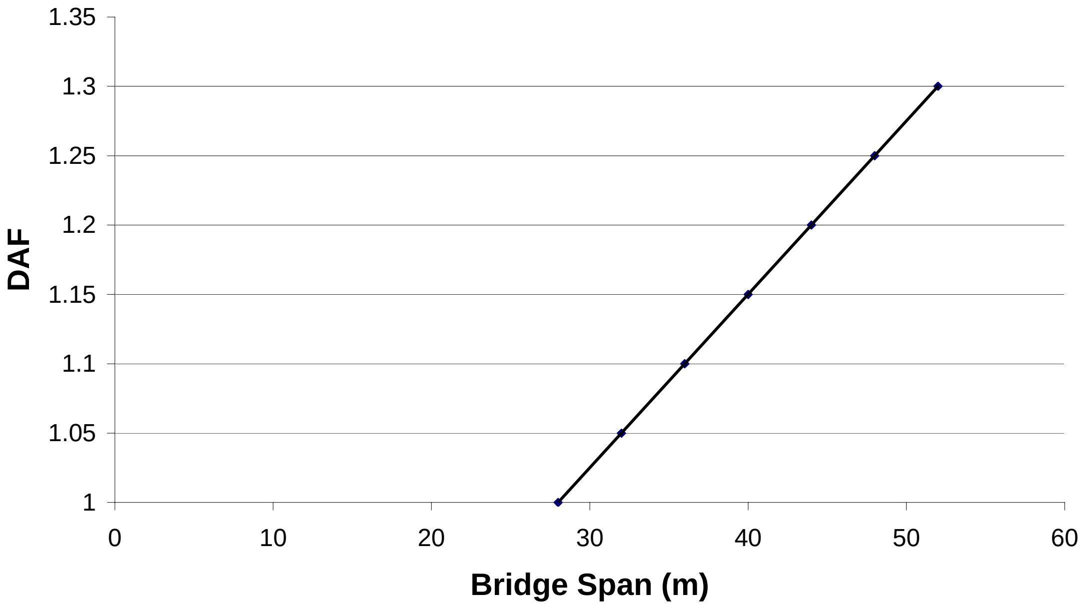


Fig. 30: DAF versus Bridge Span Breaker (Graph Possibility A).

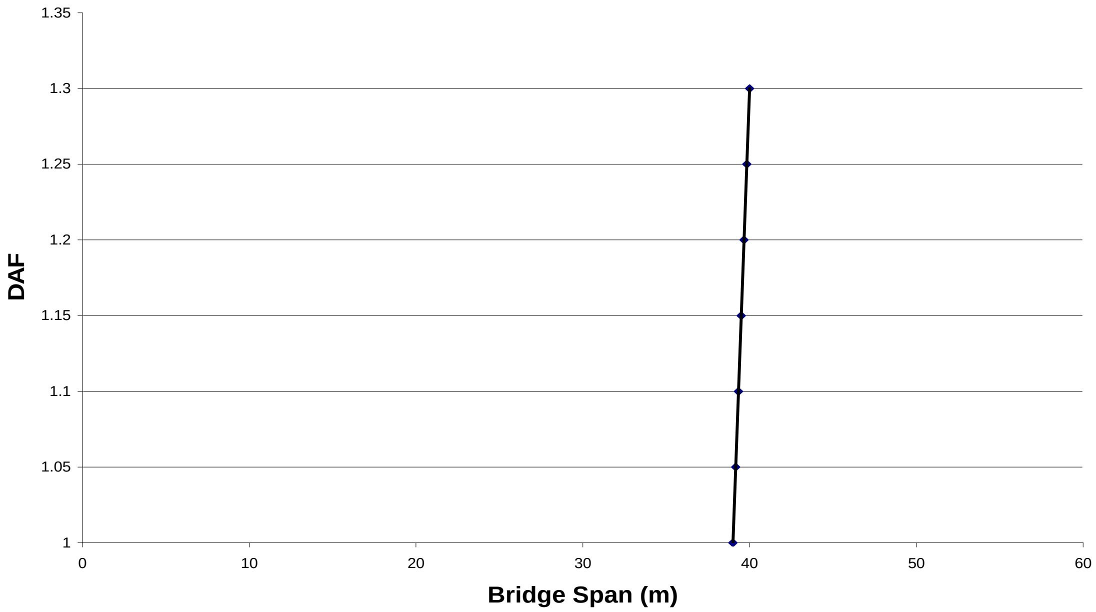
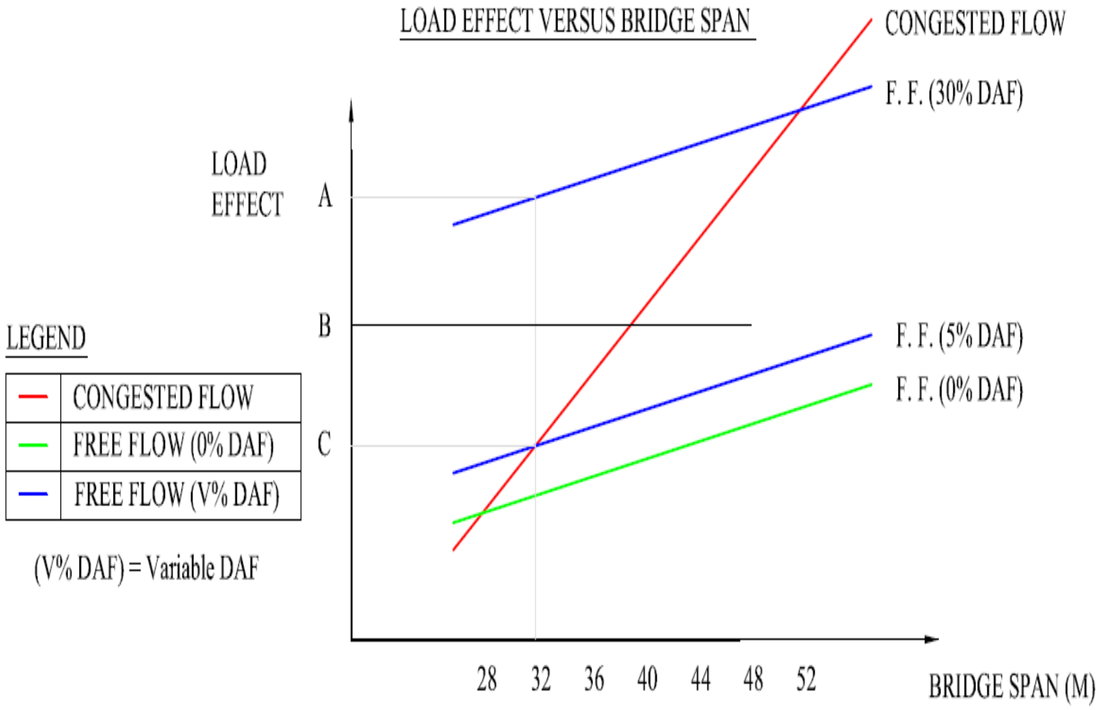


Fig. 31: DAF versus Bridge Span Breaker (Graph Possibility B).



- **<u>Analysing 32m bridge:</u>**
- A = Conservative Load Effective
- B = Load Effect Bridge can resist
- C = Suggested non conservative Load Effect

Fig. 32: Load Effect versus Bridge Span (Importance of Accurate DAF).

The E.U. is planning to raise the legal limit of freight trucks from 40 tonne to 60 tonne on certain routes. Bridges along these designated routes will have to be tested to see if they will be able to cope with the extra loading that may occur.

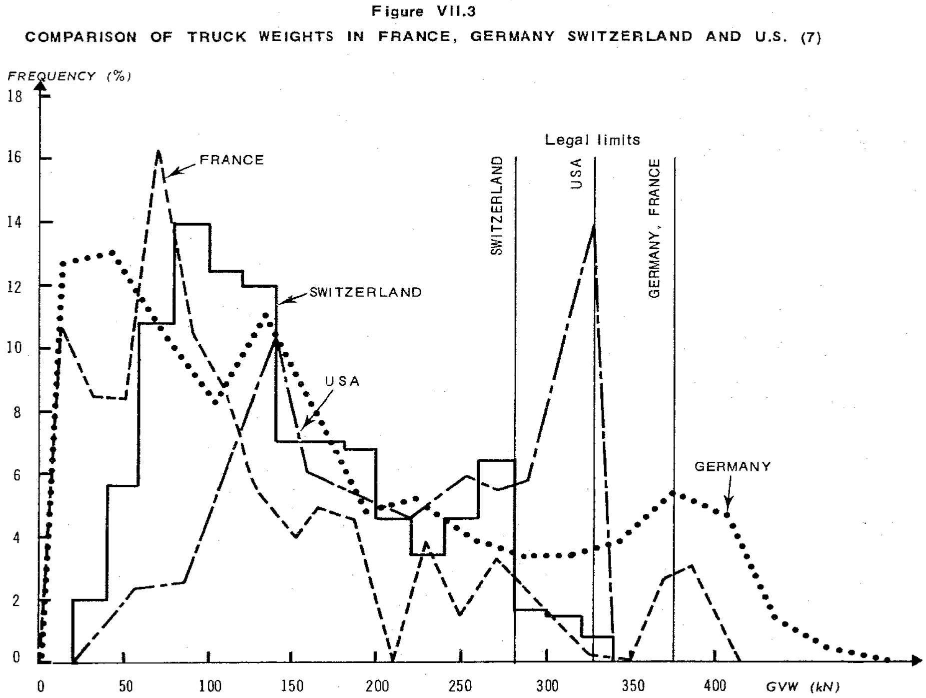


Fig. 33: Comparison of Truck Weights in France, Germany, Switzerland and U.S.[31].

From Fig. 33 we can see that the legal limit for trucks is not well adhered to and the legal limit for trucks is often broken. This suggests that the bridges that these overweight trucks are crossing can take more weight than that to which they were design for. This could be down to a number of reasons like, there is a safety factor calculated into every structure, the legal limits for trucks falls below the load the bridges were designed to, or bridges are over designed for the loads they are subjected to. Work carried out by (Caprani and Rattigan, 2006)[4], Fig. 25, suggests that bridges maybe over designed and bridges could take greater loads than previously predicted. These findings may prove very important as, with the increase in loading due to the legal weight limit of trucks increasing, many of Europe's bridges may have to be replaced or strengthened unnecessarily at great costs.

Fig. 30 shows how important using an accurate DAF value can be. If a 32 metre bridge's strength is being tested and the tests prove the extreme load effect the bridge can cope with is "B", and if a DAF of 1.30 is used, the bridge will have to be replaced or strengthened. But if a DAF of 1.05 is used the bridges strength is acceptable.

5. Main Body of Work:

The traffic data that was used in this work was recorded on the A6 motorway near Auxerre in France in 1986. This motorway was chosen as it is a key freight route and is seen to be a good representative of the traffic flow on a European route. Traffic data used to produce the Eurocode 1: Part 3 "Traffic loads on bridges" was also recorded on the A6 motorway. The data was recorded for a one week period in May 1986, there is four lanes on the motorway, two lanes in each direction, and Weigh-In-Motion (WIM) data from the four lanes was recorded, with a daily average flow of 6744 trucks. Only truck data, or vehicles over 8 tonne are used in traffic loading for bridge engineering as trucks weigh a lot more than cars, the effect they have on highway bridge is considerably greater, "According to an AASHTO study, one 40-ton truck crossing does as much damage to the bridge as 9,600 cars." The recorded data was inputted into a computer program which is capable of returning 1000 year return period truck loading events.

5.1 Description of Program

As a truck drives over a WIM systems, the WIM system records gross weight, axle and group axle load, truck speed, axle spacing and vehicle classification. WIM systems are very expensive to run and therefore only limited recording times are financially viable. The WIM data is used in a computer program developed by U.C.D. which processes the WIM data and produces extreme load effects for a given return period. This program is described in detail in work carried out by (Caprani, 2006)[6]. A brief outline of the program is described here. The program has 3 phases Generate.exe, Simulate.exe and Analyse.exe.

5.1.1 Generate.exe

The Generate.exe phases takes the one week of recorded traffic data and extrapolates this data out for a given time period. The input data file for Generate.exe traffic file is called “GTin”:

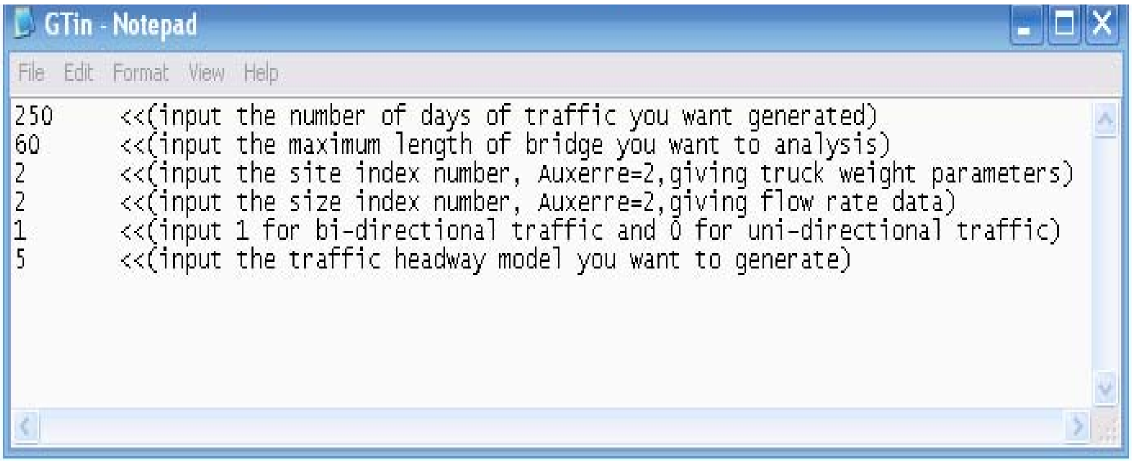
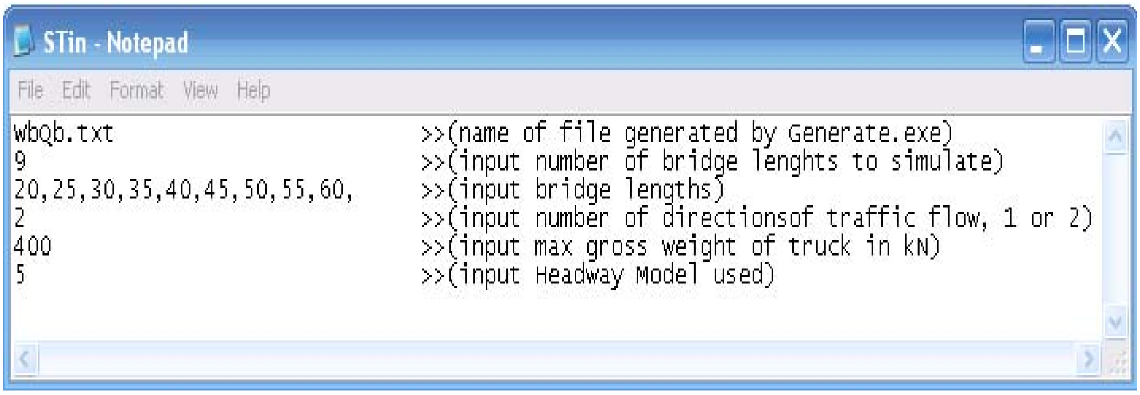


Fig. 34: GTin file.

In the GTin file the user inputs the number of days they would like to generate traffic for, the max length of bridge to be generated, the site specific parameters (for this study only number 2 for the Auxerre site is used), the number of lanes (in this study only two lanes of traffic were studied) and the Headway Model to be used. The Headway Model is a method of controlling the headways of the trucks. In this study we use two Headway Models, the “HeDS” Headway Model “0”, and the “congested” Headway Model “5”. The HeDS Headway Model generates free flowing traffic travelling with speeds and headways governed by the recorded WIM data. In the congested Headway model the headway between trucks in restricted to 5m with a 10% standard deviation. This restriction was put in place to generate traffic in a traffic jam scenario. The Eurocode 1: Part 3 and work carried out by (Dawe, 2003)[3] indicates that in a traffic jam scenario the headways are generally between 2 and 5 metres. By using a 5 metre headway for congested flow in this study the static load a bridge will be subjected to will be conservative as less truck axles will fit on a bridge at one time the greater the headway used. The speed was not restricted as only the static loads are recorded by the WIM system and the speed of a truck has no barring on the static weight of the truck.

5.1.2 Simulate.exe

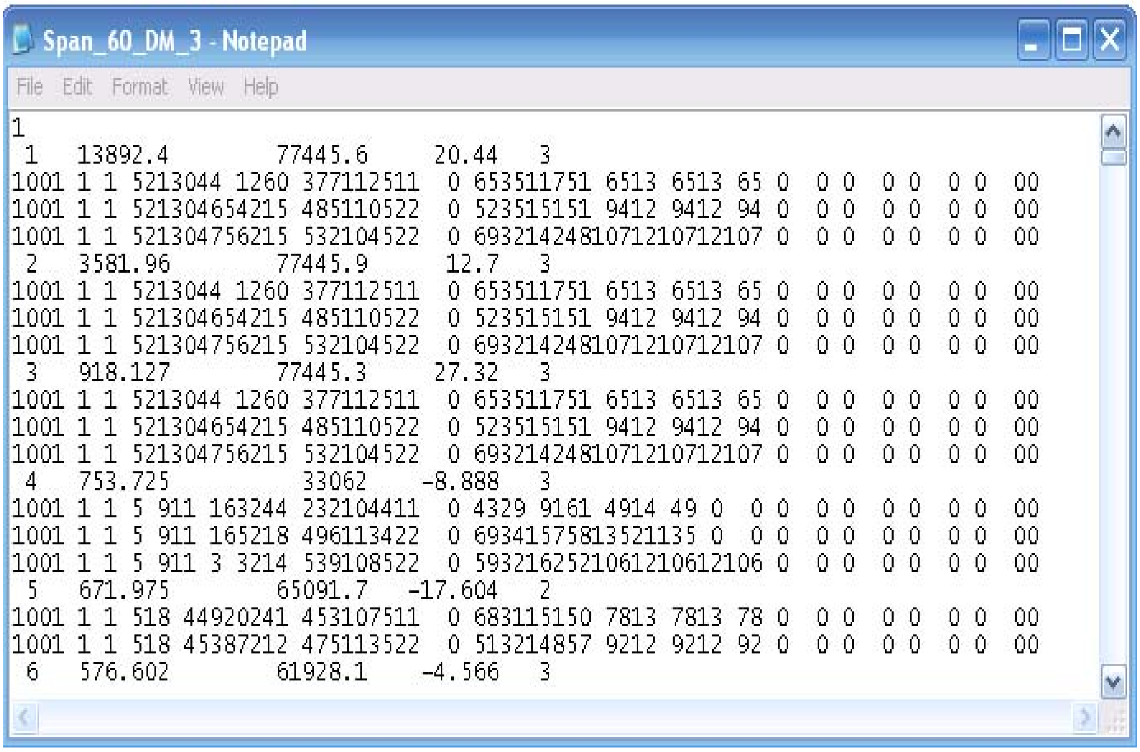
The Simulate.exe phase of the program works as the name suggests. The trucks are simulated crossing a bridge of given length and the daily maximum, the truck event with the largest static load for each day is outputted in “Span_bridge length_DM_number of file” files. The input data file for Simulate.exe is called “STin”:




```
wbQb.txt                         >>(name of file generated by Generate.exe)
9                                >>(input number of bridge lenghts to simulate)
20,25,30,35,40,45,50,55,60,      >>(input bridge lengths)
2                                >>(input number of directionsof traffic flow, 1 or 2)
400                              >>(input max gross weight of truck in kN)
5                                >>(input Headway Model used)
```

Fig. 35: STin file.

The “Span_bridge length_DM_number of file” files output the largest daily static load for a given truck event. These files contain all the information of the truck event and look as follows:

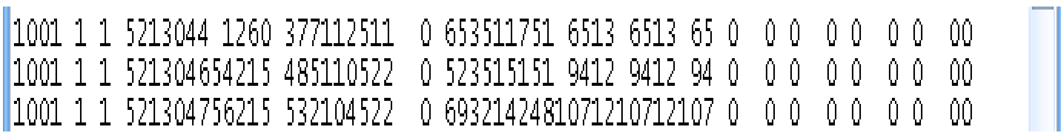


```
Span_60_DM_3 - Notepad
File  Edit  Format  View  Help
1
 1   13892.4        77445.6     20.44   3
1001 1 1 5213044 1260 377112511  0 653511751 6513 6513 65 0  0 0  0 0  0 0  00
1001 1 1 521304654215 485110522  0 523515151 9412 9412 94 0  0 0  0 0  0 0  00
1001 1 1 521304756215 532104522  0 69321424810712107121O7 0  0 0  0 0  0 0  00
 2   3581.96        77445.9      12.7   3
1001 1 1 5213044 1260 377112511  0 653511751 6513 6513 65 0  0 0  0 0  0 0  00
1001 1 1 521304654215 485110522  0 523515151 9412 9412 94 0  0 0  0 0  0 0  00
1001 1 1 521304756215 532104522  0 6932142481071210712107 0  0 0  0 0  0 0  00
 3   918.127        77445.3     27.32   3
1001 1 1 5213044 1260 377112511  0 653511751 6513 6513 65 0  0 0  0 0  0 0  00
1001 1 1 521304654215 485110522  0 523515151 9412 9412 94 0  0 0  0 0  0 0  00
1001 1 1 521304756215 532104522  0 6932142481071210712107 0  0 0  0 0  0 0  00
 4   753.725          33062    -8.888   3
1001 1 1 5 911 163244 232104411  0 4329 9161 4914 49 0  0 0  0 0  0 0  0 0  00
1001 1 1 5 911 165218 496113422  0 69341575813521135 0  0 0  0 0  0 0  0 0  00
1001 1 1 5 911 3 3214 539108522  0 5932162521061210612106 0  0 0  0 0  0 0  00
 5   671.975        65091.7   -17.604   2
1001 1 1 518 44920241 453107511  0 683115150 7813 7813 78 0  0 0  0 0  0 0  00
1001 1 1 518 45387212 475113522  0 513214857 9212 9212 92 0  0 0  0 0  0 0  00
 6   576.602        61928.1    -4.566   3
```

Fig. 36: Simulate.exe output file.

The single number on the top left of each truck event indicates the day (line 2-first number), and the single number on the top right of each truck event indicates the number of trucks in the event (line 2- last single number).

Also the information about each individual truck is given in the output file. Line 3 to line 6:



```
1001 1 1 5213044 1260 377112511  0 653511751 6513 6513 65 0  0 0  0 0  0 0  00
1001 1 1 521304654215 485110522  0 523515151 9412 9412 94 0  0 0  0 0  0 0  00
1001 1 1 521304756215 532104522  0 6932142481071210712107 0  0 0  0 0  0 0  00
```

Fig. 37: Simulate.exe output file, line 3-6 (individual truck data)

Each one of these lines contains the information on a single truck and can be read on the following page:

Number	Characteristic	Units
1 to 4	Head	
5 and 6	Day truck enters bridge	
7 and 8	Month truck enters bridge	
9 and 10	Year truck enters bridge	
11 and 12	Hour truck enters bridge	
13 and 14	Minute truck enters bridge	
15 and 16	Second truck enters bridge	
17 and 18	Second/100 truck enters bridge	
19 to 21	Speed of truck	dm/s
22 to 25	Gross Vehicle Weight	kg/100
26 to 28	Length of Truck	dm
29	Number of Axles	
30	Direction Of Truck	
31	Lane Truck is in	
32 to 34	Transverse Location in Lane	dm
35 to 37	Weight of Axle 1	kg/100
38 to 39	Spacing between Axle 1 - Axle 2	dm
40 to 42	Weight of Axle 2	kg/100
43 to 44	Spacing between Axle 2 - Axle 3	dm
	↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓↓	
72 to 75	Spacing between Axle 8 - Axle 9	dm
76 to 78	Weight of Axle 9	kg/100

Table 7: Description of truck file

5.1.3 Analyse.exe

The next and last phase of the program is Analyse.exe. This phase of the program takes the simulate traffic files and extrapolates the daily maximum results to get a return period results for different load effects. The return period of 1000 years is used in this study which is also the characteristic value used in the Eurocode 1: Part 3. The load effects used in this study are Load Effect1, 2 and 3:

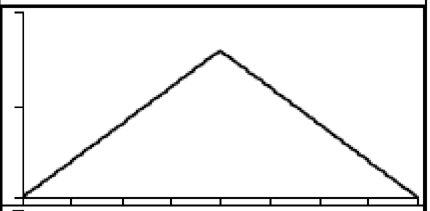
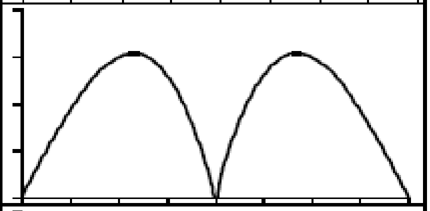
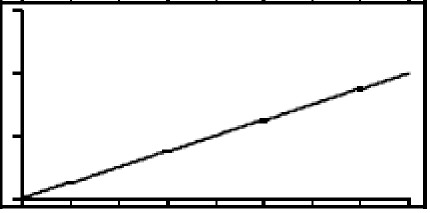
IL Index	Description	Graph
1	Bending moment at the centre of a simply-supported span.	
2	Bending moment over the central support of a two-span bridge.	
3	Right-hand support shear force for a simply-supported span.	

Fig. 38: Load Effects studied in this work.

The input file for Analyse.exe is called "AEin":

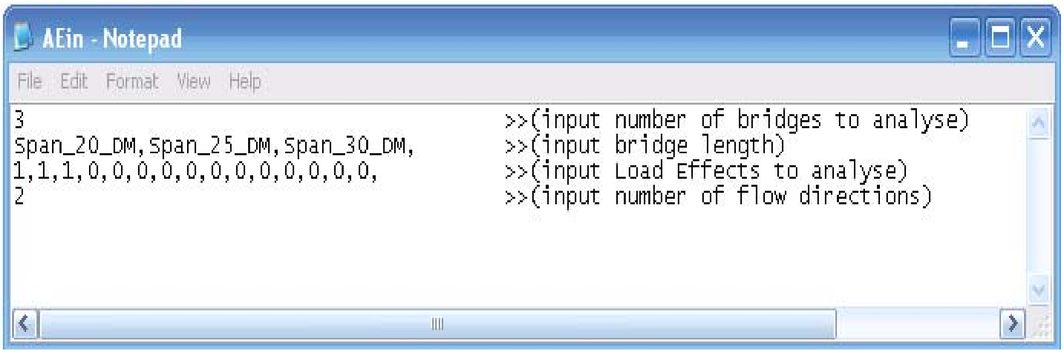


Fig. 39: AEin file

The output files from Analyse.exe are called 'Span_bridge length_DM_D_EV' from which a 1000 year return period values for Load Effect 1, 2 and 3 are obtained. These 1000 year return period values are the extreme static values for congested and free flowing traffic.

5.2 Analysis of Load Effects 1, 2 and 3 for Multi-Truck Events

To fully understand the load effects for multi truck events a excel spreadsheet was developed. The spreadsheet has 2 inputs, the bridge length and the truck event. By inputting these variables the spreadsheet produces graphs of the 3 load effects versus time of event for individual trucks and combined truck events. The input and output sheets are as follows, the yellow highlighted cells are the input cells:

Bridge Span (m)	60

Truck Event	
No. of Trucks	6

Truck No.	
1	1001 5 1 5 9 25474243 417117411 0 7335142691O113101 0 0 0 0 0 0 0 0 0 00
2	1001 5 1 5 9 25559261 398114511 0 583113059 7012 7012 70 0 0 0 0 0 0 0 0 0 01
3	1001 5 1 5 9 25648260 421106411 0 7433143611O212102 0 0 0 0 0 0 0 0 0 00
4	1001 5 1 5 9 25710233 403116511 0 583312957 7213 7213 72 0 0 0 0 0 0 0 0 0 01
5	1001 5 1 5 9 25948228 438120522 0 623613658 8013 8013 80 0 0 0 0 0 0 0 0 0 00
6	1001 5 1 5 9 3 039215 404111522 0 563213257 7211 7211 72 0 0 0 0 0 0 0 0 0 01
7	
8	

Truck 1							
G.V.W. (kN)	409.077					-	
No. of Axles	4						
Axle No	1	2	3	4	5	6	Total
Weight (kN)	71.613	139.302	99.081	99.081	0	0	409.077
Spacing (m)	0	3.5	6.9	1.3	0	0	11.7

Truck 2							
G.V.W. (kN)	390.438					-	
No. of Axles	5						
Axle No	1	2	3	4	5	6	Total
Weight (kN)	56.898	127.53	68.67	68.67	68.67	0	390.438
Spacing (m)	0	3.1	5.9	1.2	1.2	0	11.4

Truck 3							
G.V.W. (kN)	413.001					-	
No. of Axles	4						
Axle No	1	2	3	4	5	6	Total
Weight (kN)	72.594	140.283	100.062	100.062	0	0	413.001
Spacing (m)	0	3.3	6.1	1.2	0	0	10.6

Truck 4							
G.V.W. (kN)	395.343						-
No. of Axles	5						
Axle No	**1**	**2**	**3**	**4**	**5**	**6**	**Total**
Weight (kN)	56.898	126.549	70.632	70.632	70.632	0	395.343
Spacing (m)	0	3.3	5.7	1.3	1.3	0	11.6

Truck 5							
G.V.W. (kN)	429.678						-
No. of Axles	5						
Axle No	**1**	**2**	**3**	**4**	**5**	**6**	**Total**
Weight (kN)	60.822	133.416	78.48	78.48	78.48	0	429.678
Spacing (m)	0	3.6	5.8	1.3	1.3	0	12

Truck 6							
G.V.W. (kN)	396.324						-
No. of Axles	5						
Axle No	**1**	**2**	**3**	**4**	**5**	**6**	**Total**
Weight (kN)	54.936	129.492	70.632	70.632	70.632	0	396.324
Spacing (m)	0	3.2	5.7	1.1	1.1	0	11.1

Fig. 40: Input page of "Analyse bridge response to multi-truck event" excel spreadsheet

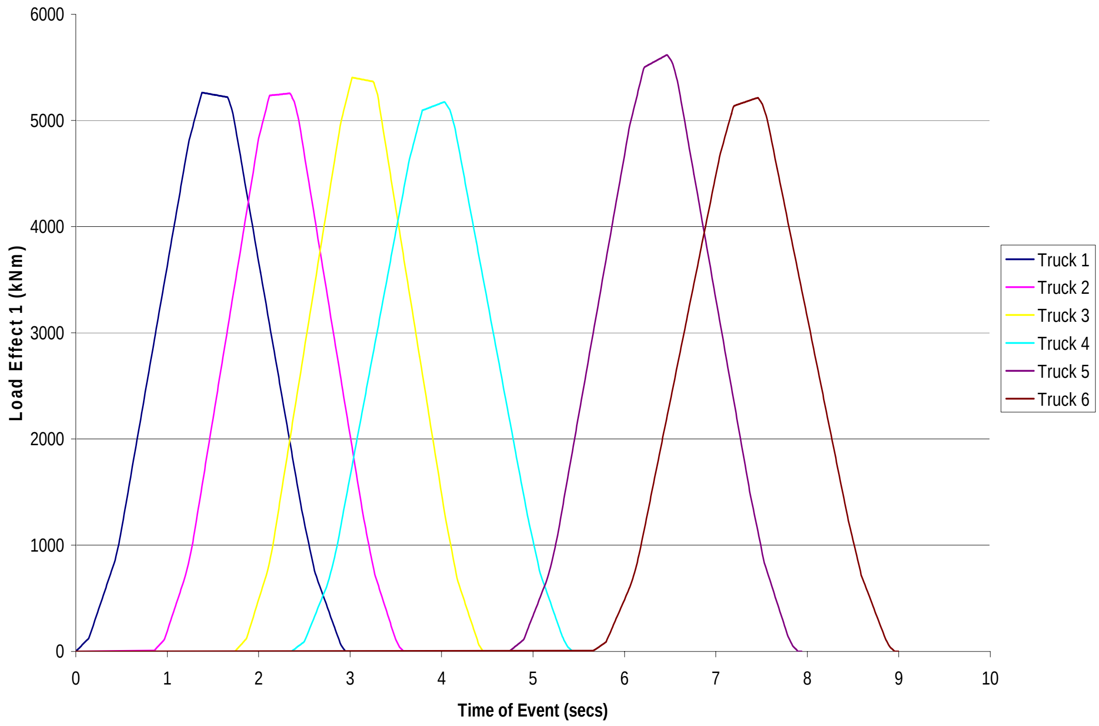


Fig. 41: Load Effect 1 versus Time of Event for individual trucks in a multi-truck event.

Fig. 41 shows the induced mid-span moments due to individual trucks in a multi-truck event crossing a 60 metre bridge. We can see that the trucks enter the bridge at different times and the Load Effect 1 gets larger as the trucks are closer to the middle of the bridge. Logically the largest Load Effect 1 will occur at a time when the most trucks are on the bridge at the one time. From Fig 41 we can see that truck 1 is completely off the bridge when truck 6 enters the bridge. This tells us that when calculating the max Load Effect 1 we should not use the Load Effects due to truck 1 and truck 6. If two individual trucks curves on Fig. 41 cross this means that are present on the bridge at one time. If a “time-line” is drawn on Fig. 41 for any given time, every individual truck curve this time-line crosses contributes to the combined Load Effect at this time. From studying Fig. 41, it is possible to say that largest Load Effect 1 for this multi-truck event will occur somewhere between 2 and 3 seconds after the first truck enters the bridge. We can say this as, any time-line drawn between 2 and 3 seconds will cross over 3 to 4 individual truck curves, which is a lot more than any other time in this multi-truck event.

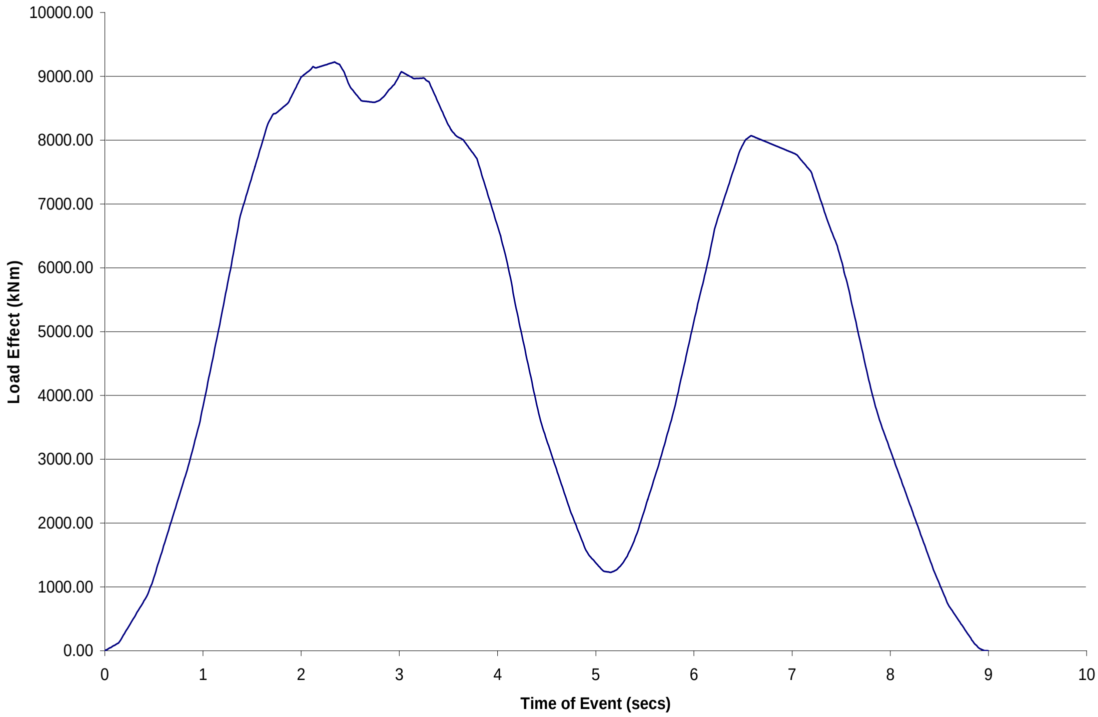


Fig. 42: Load Effect 1 versus Time of Event for combined trucks in a multi-truck event.

Max Moment (kNm)	Time at Max Moment (secs)
9224.62	2.34

Fig. 43: Read out of max Load Effect 1 moment and time of max moment for multi-truck event.

Fig. 42 shows the combined results of Fig. 39. From the read out of Fig. 43 we can see that the max mid-span moment for a 60 metre one span bridge under this particular multi-truck event is 9224.62kNm and this moment occurs after 2.34 seconds of the first truck entering the bridge. Fig. 42 has two major curves, this is the case as there is two groups of trucks in the truck event. The first group of four trucks,

trucks 1 to 4, enter the bridge between 0 and are all off the bridge after approximately 5.5 seconds whereas the second group of trucks, trucks 5 and 6, only enter the bridge at approximately 4.8 seconds, after truck 1 enters the bridge, and the first group are all nearly off the bridge at this time. Because of this truck 5 and 6 do not contribute to the Load Effect 1 moment but cause the second curve in Fig. 42.

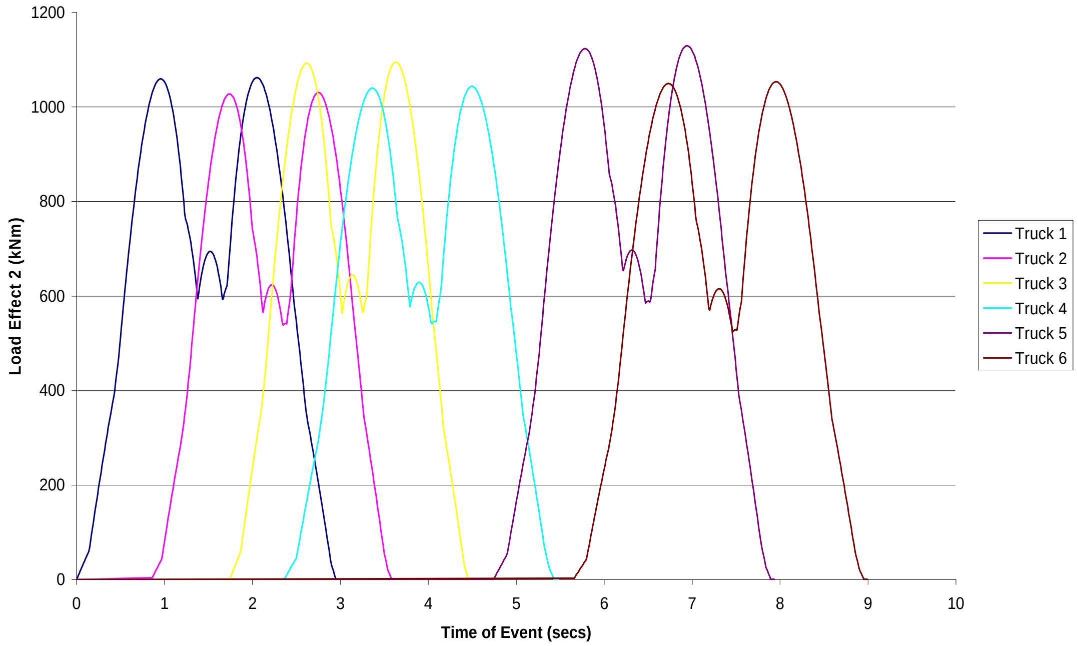


Fig. 44: Load Effect 2 versus Time of Event for individual trucks in a multi-truck event.

Fig. 44 shows Load Effect 2 versus Time of Event for the same multi-truck event as in Fig. 41. We can see by comparing Fig. 41 and Fig. 44 that the curves for the individual truck crossing for Load Effect 2 are not as smooth as the Load Effect 1 curves. The reason for this becomes clearer when Fig. 38 is viewed. If a single load travels across a bridge, the individual truck curves in Fig. 44 would look like Fig. 38 IL 2. From Fig. 40 we can see that truck 1 is a 4 axle truck and is 11.7 metres long. This means that as truck 1 cross the bridge, there are 4 single loads crossing the bridge resulting in the truck 1 curve in Fig. 44.

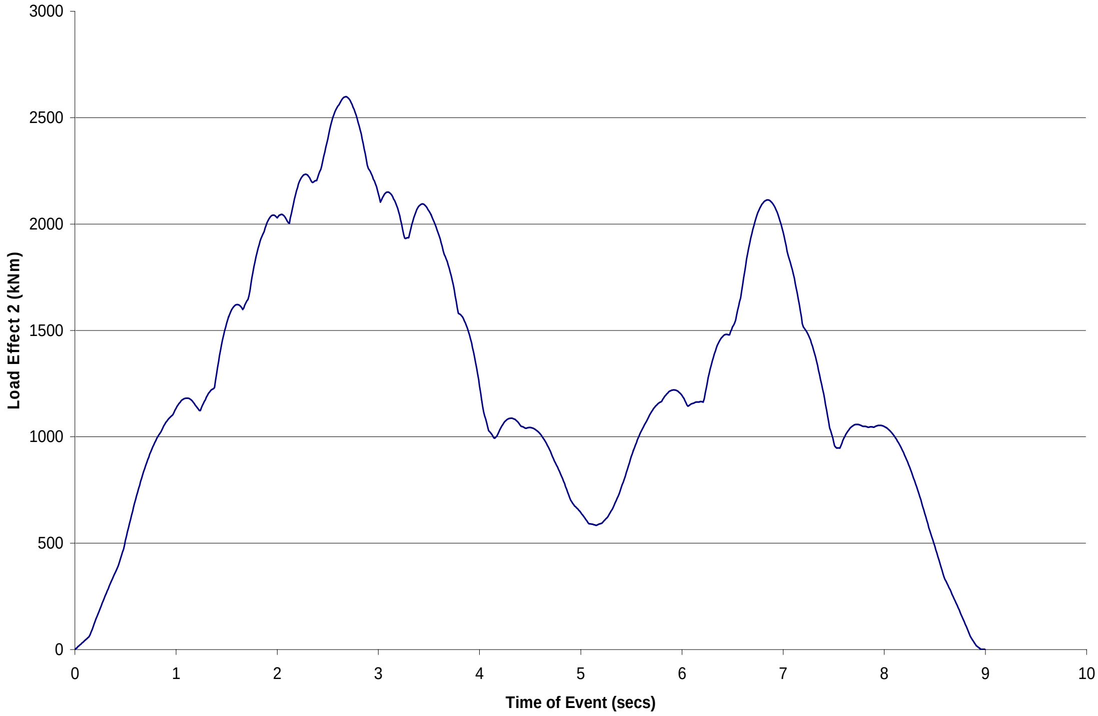


Fig. 45: Load Effect 2 versus Time of Event for combined trucks in a multi-truck event.

Max Moment (kNm)	**Time at Max Moment (secs)**
2598.58	2.68

Fig. 46: Read out of max Load Effect 2 moment and time of max moment for multi-truck event.

As all three Load Effects are based on the one multi-truck event, the time at which the most trucks are present on the bridge at one time is still between 2 and 3 seconds. Therefore the max Load Effect 2 moment should be between 2 and 3 seconds. We can see from Fig. 46 that this is the case, with the max Load Effect 2 moment of 2598.58kNm which occurs at 2.68 seconds after the first truck enters the bridge.

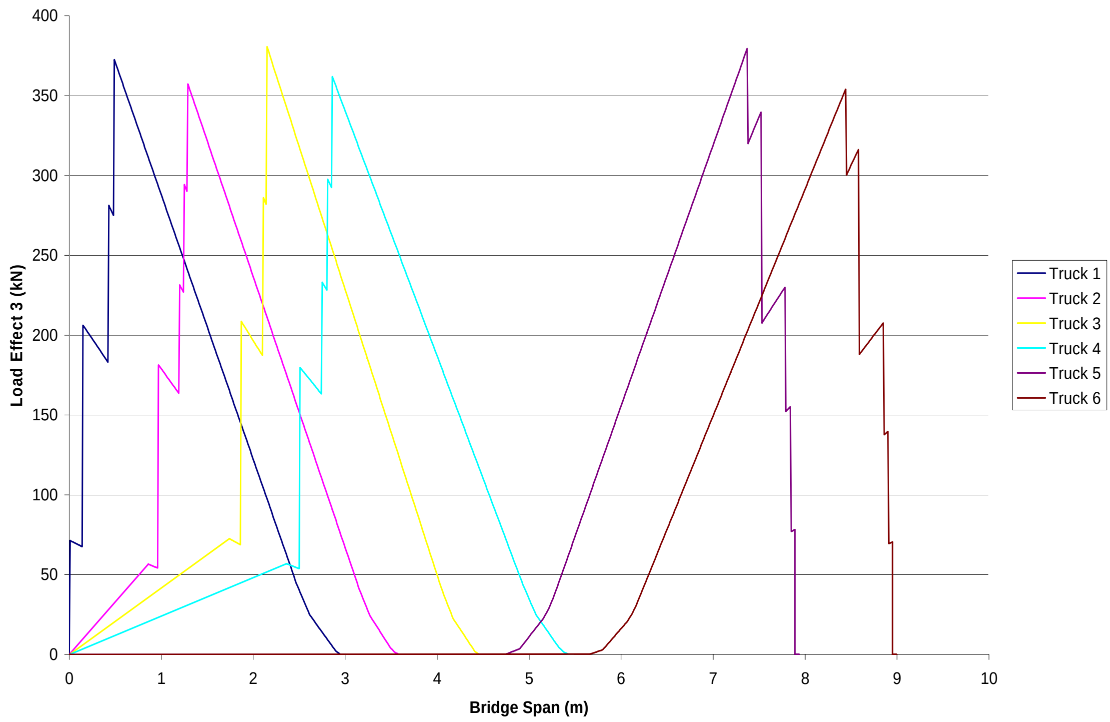


Fig. 47: Load Effect 3 versus Time of Event for individual trucks in a multi-truck event.

Fig. 47 shows the right hand support shear, Load Effect 3. We can see that trucks 1 to 4 are travelling from right to left on the bridge and trucks 5 and 6 are travelling in a different direction, from left to right. The jagged edges on the graph represent the individual trucks axles entering the bridge. If an individual trucks crossing on Fig. 47, say truck 1 is studied, a good understanding of how Load Effect 3 works as regard a trucks crossing. We can see that at 0 seconds truck 1's first axle enters the bridge and there is a jump in Load Effect 3 of approximately 70kn, this suggests that the axle load for axle 1 for truck 1 should be approximately 70kN. From Fig. 40 we can see that this is true as the axle 1 load for truck 1 is 71.613kN.

Initially, for the next 0.2 seconds approximately, there is a drop in Load Effect 3. This is because axle 1 of truck 1 is travelling away from the right hand support of the bridge. The Load Effect 3 decreases the further the applied load is away from the right hand support. At approximately 0.2 seconds there is another jump in Load Effect 3, from approximately 65kN to 210kN as truck 1's second axle enters the bridge. This suggests a load of approximately 145kN which is verified in Fig. 40.

There are four jumps in Load Effect 3 in all which corresponds to the four axles of truck 1. At approximately 0.5 seconds the Load Effect 3 reaches a max for truck 1, this is the time that all four of truck 1's axles are on the bridge.

After 0.5 seconds there is a smooth decrease in Load Effect 3 for truck 1 with respect to time. This is because all truck 1's axles are on the bridge travelling from right to left and therefore there is going to be a steady decrease in Load Effect 3.

For trucks 5 and 6 the trend of truck 1 is reversed as the trucks are travelling from left to right and therefore the Load Effect 3 will increase for trucks 5 and 6 as they travel across the bridge.

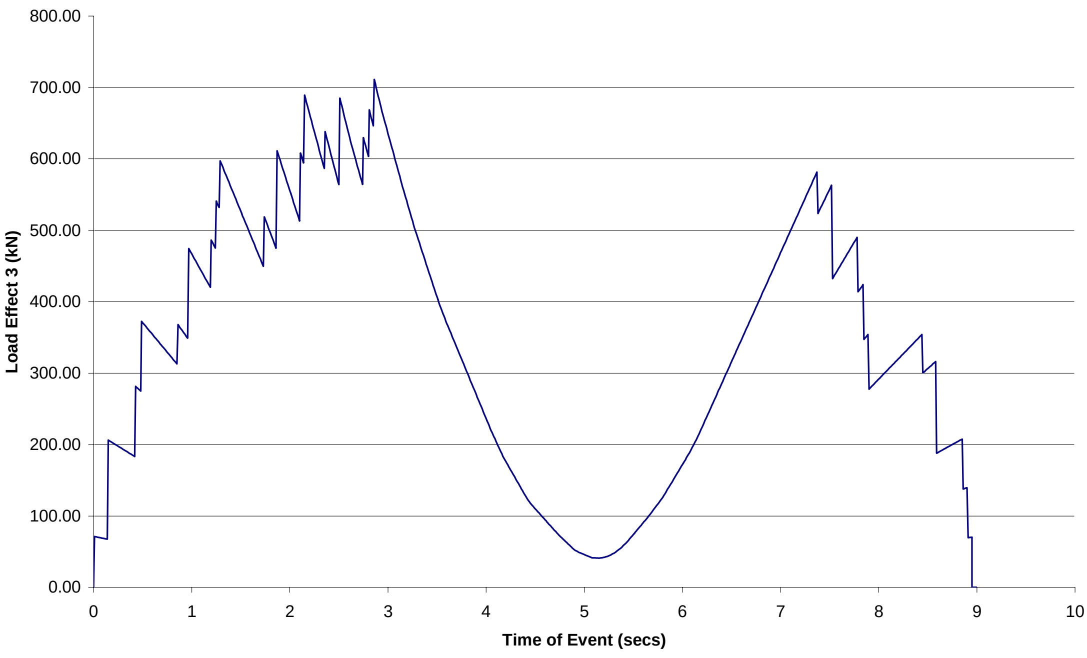


Fig. 48: Load Effect 3 versus Time of Event for combined trucks in a multi-truck event.

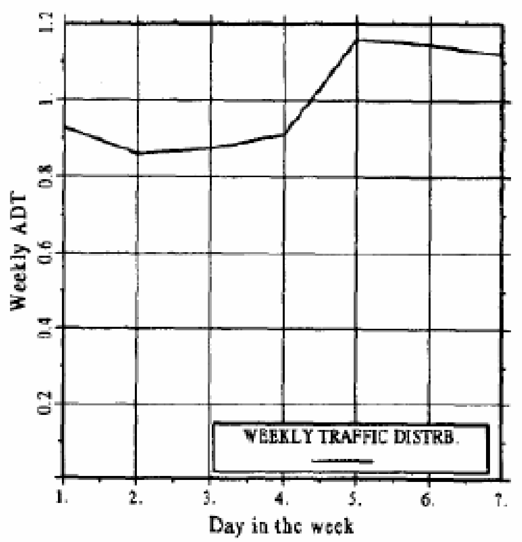
Max Shear Force (kN)	Time at Max Moment (secs)
711.34	2.86

Fig. 49: Read out of max Load Effect 3 moment and time of max moment for multi-truck event.

Fig. 48 shows the combined Load Effect 3 for the six trucks. From Fig 49 we can see that the Max Load Effect 3 occurs between 2 and 3 seconds as expected. The max right hand shear force for this particular mutli-truck event is 711.34kN and occurs at 2.86 seconds after the first truck enters the bridge.

5.3 Truck Flow Density

1000 days of free flowing traffic was generated. Traffic jam scenarios only happen at certain times of the day, usually during rush hour traffic when people are going to and coming home from work. Therefore it would be unrealistic to generate 1000 days of congested traffic.



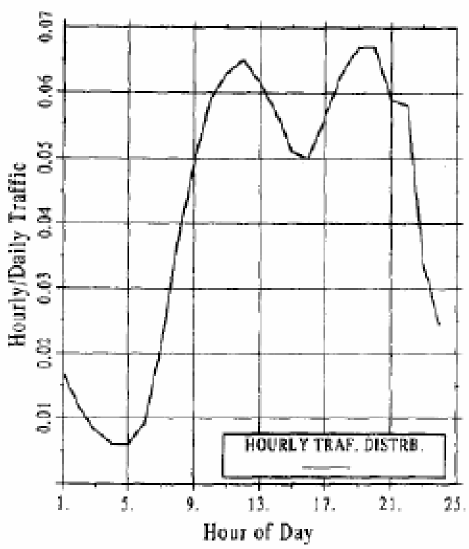


Fig. 50: Weekly and hourly traffic distribution from (Crespo-Minguillon and Casas 1997)[32].

From Fig. 50 we can see that there are variances in traffic flow for different days of the week and for different hours of each day. There are two peaks in the hourly traffic distribution graph, these peaks would represent heavier traffic flow than free flowing traffic. (Dawe, 2003)[3] carried out detailed studies of traffic flow for which the following rule was adopted to calculate the percentage of traffic flow that is congested:

Percentage of the traffic flow in close spaced jams = 2 x hourly flow/1200 Eq. 17

The average daily traffic flow for the Auxerre site was 6756 trucks.

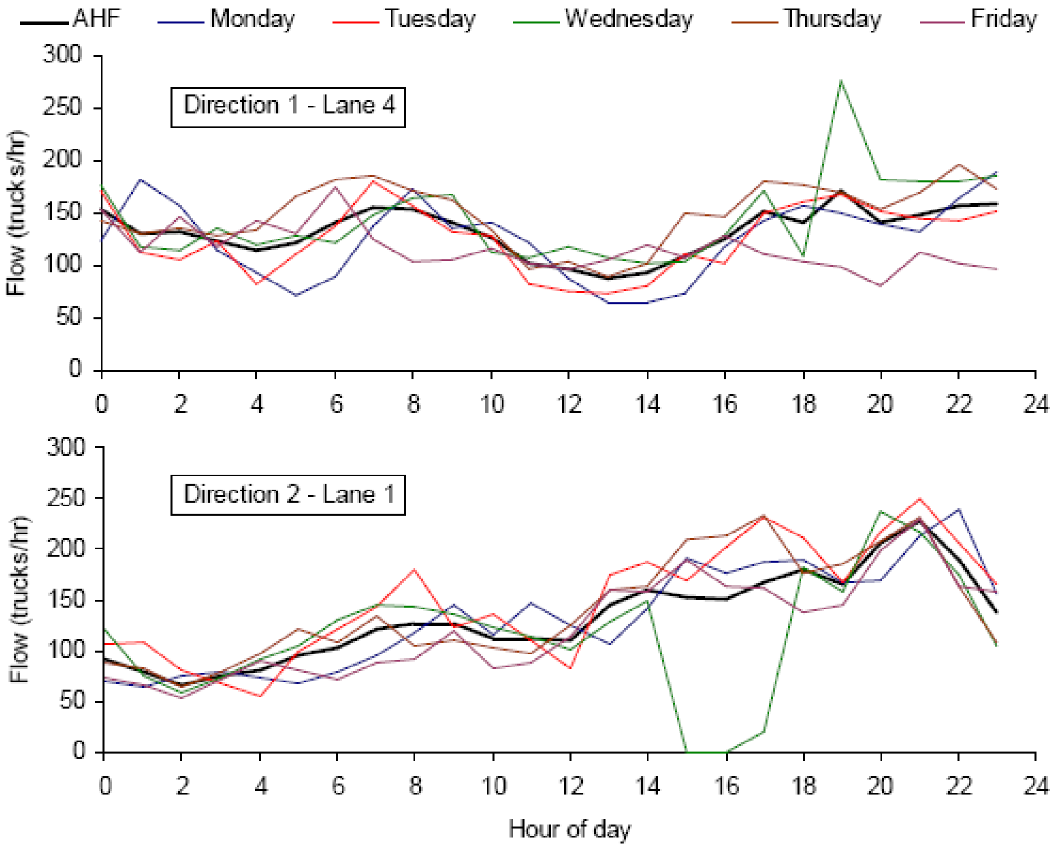


Figure 5.7: Daily variation and AHFs for both directions at Auxerre.

Fig. 51: Daily variation and average hourly flows (AHF) for both directions at Auxerre (Caprani 2006)[4].

The daily average flow recorded form Auxerre was for 4 lanes of traffic. Fig. 51 shows that the maximum flow for a given lane was approximately 275 trucks per hour (Wednesday between 18-20hrs, direction 1, lane 4).

A calculation of the percentage of congested flow for 275 trucks per hour, using Eq. 17 is as follows:

Hourly flow: 275 trucks
Percentage of the traffic flow in close spaced jams = 2 x 275/1200
= 0.46 per cent

This means that the traffic flow in lane 4, direction 1 between the hours 18:00-20:00 for the A6 is congested for 1.7 hours every year, and for 1700 hours in every 1000 years. If we neglect 14:00-16:00 for direction 2, lane 1, the lowest hourly truck flow is approximately 50 trucks per hour. A calculation of the percentage of congested flow for 50 trucks per hour is as follows:

Hourly flow: 50 trucks
Percentage of the traffic flow in close spaced jams = 2 x 50/1200
= 0.08 per cent

This means that for the least amount of hourly truck flow, 50 trucks per hour, the A6 motorway is congested for 0.3 hours every year, and for 300 hours in every 1000 years. It should be pointed out that the A6 motorway is very busy, with a lot of truck flow. For the majority of roads throughout Europe the hourly flow rate of trucks would be considerable less with certain hours of the day containing few if any trucks. This makes the congested model used in this study very conservative, which in turn will lead to a conservative design load.

As this study is using an extreme load value for a 1000 year return period, we can see that the A6 motorway will be congested for all 24 hours if the worst individual hours are considered. The more hours a bridge is subjected to congested flowing traffic the greater the probability that the extreme load effect will be larger. For this study 2 hours of every day were chosen to have congested flow, these 2 hours are represented in Fig. 50's two peaks.

Bridges with lengths in the range of 20 to 60 metres were only analysed as the majority of highway bridges (approximately 90%) have lengths within this range. The larger a bridge is, the more trucks can fit on a bridge at the one time, therefore the higher the possible static load. Bridges are designed to an extreme load effect, this load effect is either due to free flowing traffic or congested traffic. The free flowing traffic static load is multiplied by a DAF, where as the congested static load has no DAF.

If we consider a 20 metre bridge, for congested flow, as larger trucks with large static loads have a length of approx 11m, the extreme 1000 year return period can only contain one full truck plus a part of a section truck in each direction. This is because physically this is the max amount of trucks that can fit on a 20 metre bridges with 5 metre gaps between trucks. For a 20 metre bridge, under free flow traffic, it is reasonable to assume that for a 1000 year return period that there will be at least one full truck in each direction. As free flowing traffic's extreme static load is multiplied by a DAF it will, more than likely, be greater than the congested flow's extreme static load. This means that free flowing traffic will govern bridges of 20 metres in length. The gaps in the congested flow model are set to 5 metres and the gaps for the free flow model will be greater, due to driver's behaviour.

Therefore the longer the bridge length gets:

Number of trucks on bridge at one given time for congested model = A
Number of trucks on bridge at one given time for congested model = B
Length of bridge = L

As L increases so to those (A / B)

Therefore if we graph the extreme static load due to congested flow (CF) versus bridge length, and the extreme static load due to free flow multiplied by DAF (FF) versus bridge length, the slope for the CF line will increase at a greater rate than the slope of the FF line. This means that at a certain bridge length the CF will govern FF. This bridge length, corresponding to the intersection point of the CF and FF lines, is named C.L.o.F.T.s., and is very important in bridge engineering. C.L.o.F.T.s. stands for Critical Length of Flow Traffic switch. It tells the engineer that for a given load effect, any bridge with a length less than the C.L.o.F.T.s. length should be designed for free flowing traffic as FF traffic governs the max extreme design load. Any bridge with a length greater than the C.L.o.F.T.s. length should be designed for congested traffic as CF traffic governs the max extreme design load.

5.4 Investigation into dominant traffic flow regime

The first step in investigating the C.L.o.F.T.s. values for the 3 load effects is graphing the extreme static load resulting from congested flow and the extreme static load resulting from free flowing traffic versus bridge length. Note that there is no DAF factored into the free flowing traffic, these graphs are merrily to see the relationship both sets of traffic flow have statically with bridge length. The values are obtained from the output file of Analyse.exe. As the traffic is generated randomly based on actual WIM data, each simulation will produce different extreme values. Each value produce will comply with the Eurocode 1: Part 3 characteristic value of "1000 year return period (or probability of exceedance of 5% in 50 years)". To be confident that none of the 1000 year return period values were neither abnormally too high or too low, five simulations were carried out. This process was carried out for each individual bridge length, this means that, for each simulation, the 1000 year return period values are independent of each other. Therefore there is no correlation between say, simulation 1 for a 20 metre bridge and simulation 1 for a 25 mere bridge. Despite this a definite general trend emerges, which complies with the theoretical views on traffic loading for bridges. For smaller bridges of about 20 metres, there is little difference in the free flow extreme static loads and the congested flow extreme static loads. But as the bridge length increases the extreme static load due to congested flow becomes more dominant and starts to govern at between 30-50 metres. It is important to remember though that there is an additional dynamic load to consider with the static load for free flowing traffic and there is no DAF factored into these graphs:

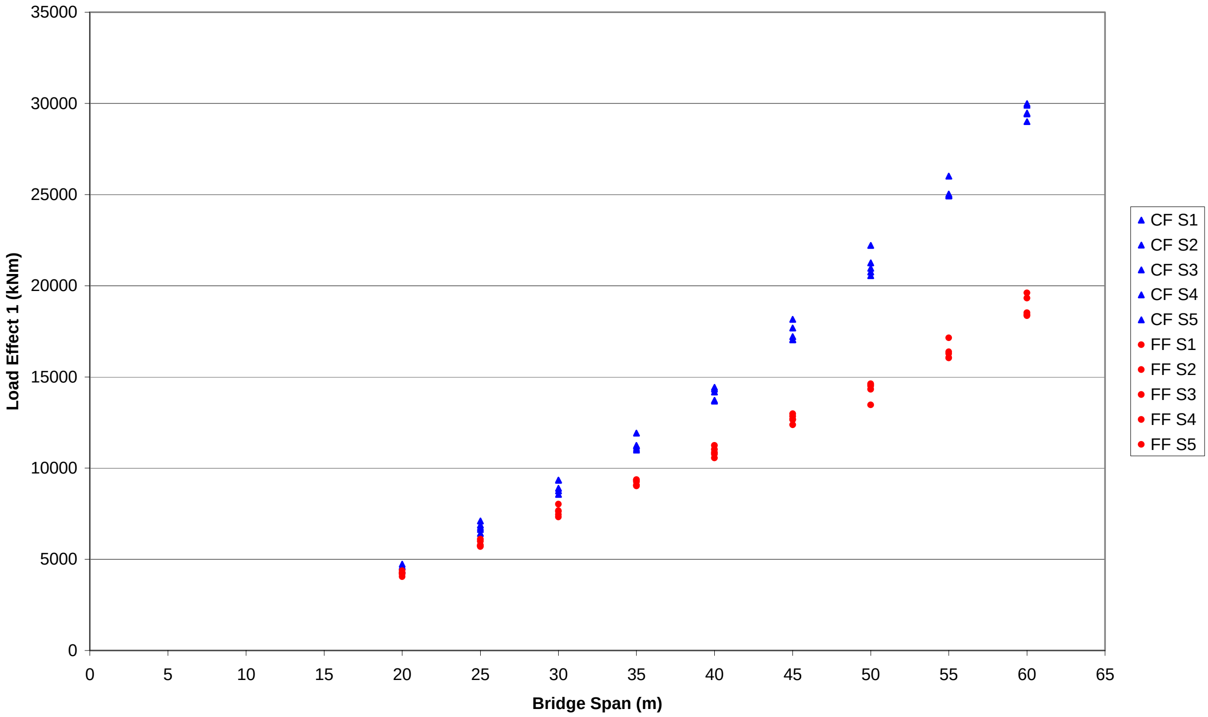


Fig 52: Bridge Length Versus Load Effect 1 (comparison of free flow versus congested flow extreme loads).

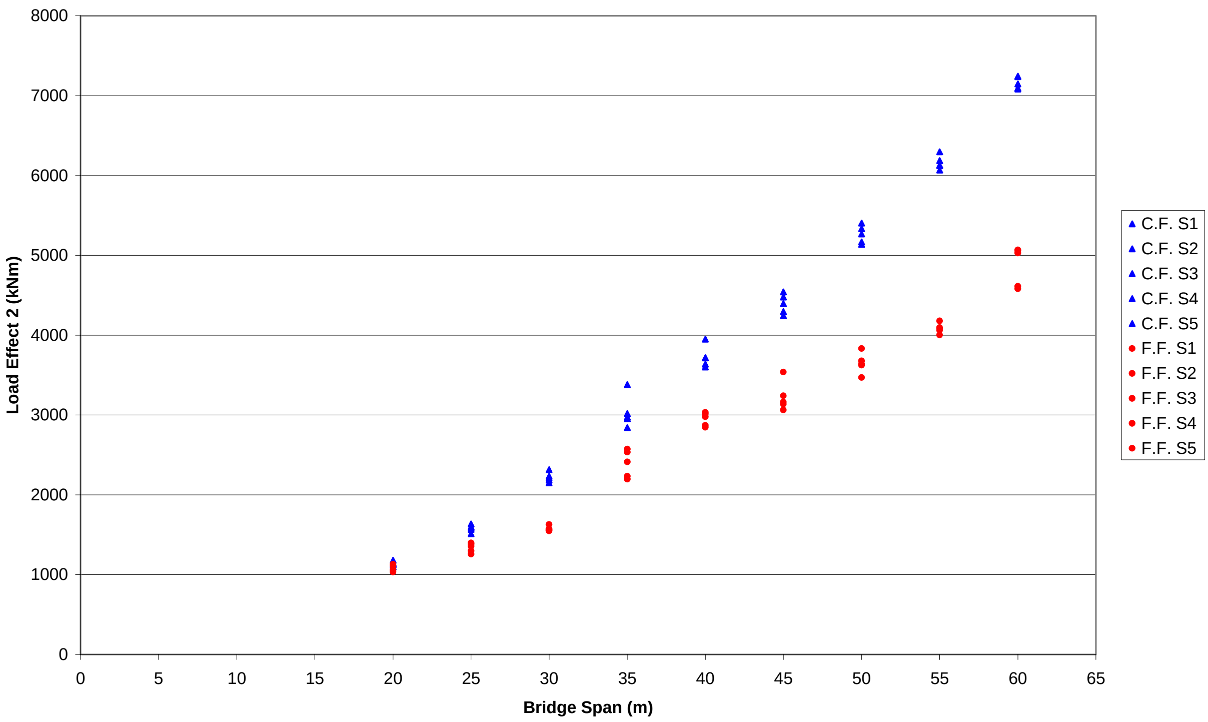


Fig 53: Bridge Length Versus Load Effect 2 (comparison of free flow versus congested flow extreme loads).

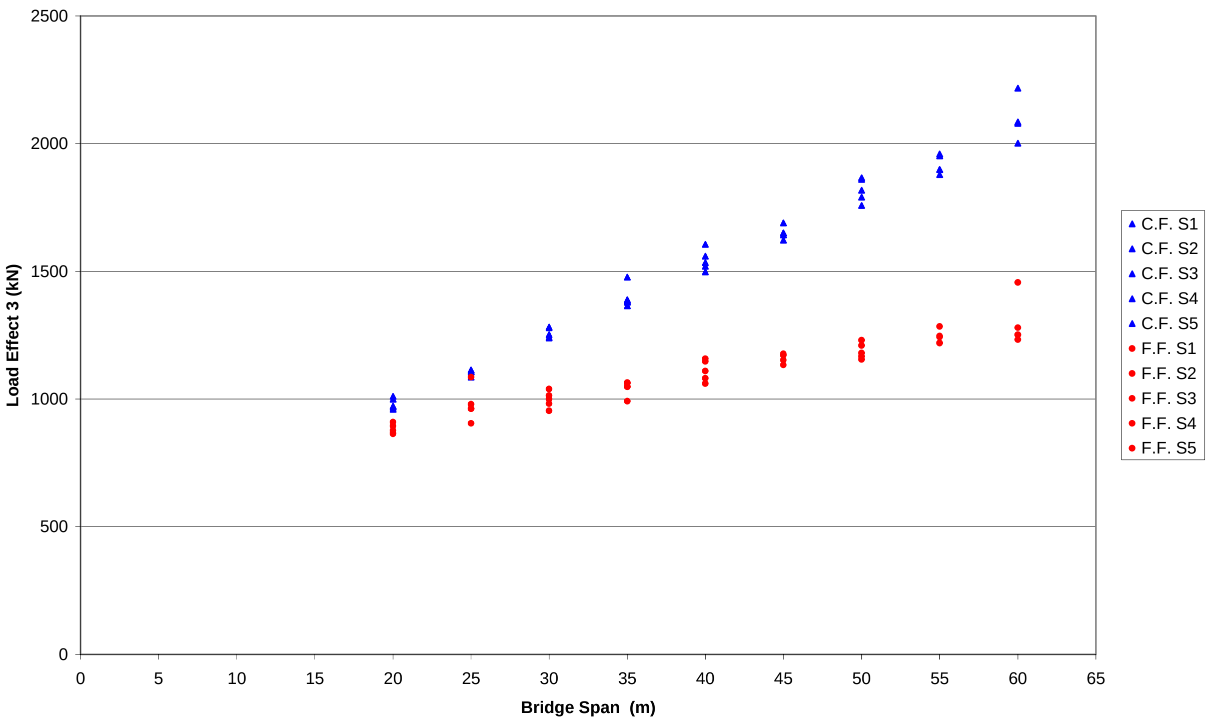


Fig 54: Bridge Length Versus Load Effect 3 (comparison of free flow versus congested flow extreme loads).

5.5 Importance of an accurate DAF

The next step in the study is to investigate the effect of different DAF values to the C.Lo.F.T.s. lengths. This was achieved by, getting the average of the 5 simulations for CF (ACF) and FF (AFF) for each bridge length and multiplying the AFF by different DAF values. The ACF and AFF multiplied by varying DAF, for the 3 load effects, was graphed against bridge length. These new graphs will aid in giving a better understanding of the effect of varying the DAF values. Also Fig.'s 55, 56 and 57 contain AFF multiplied by the Eurocode values for DAF for different bridge lengths.

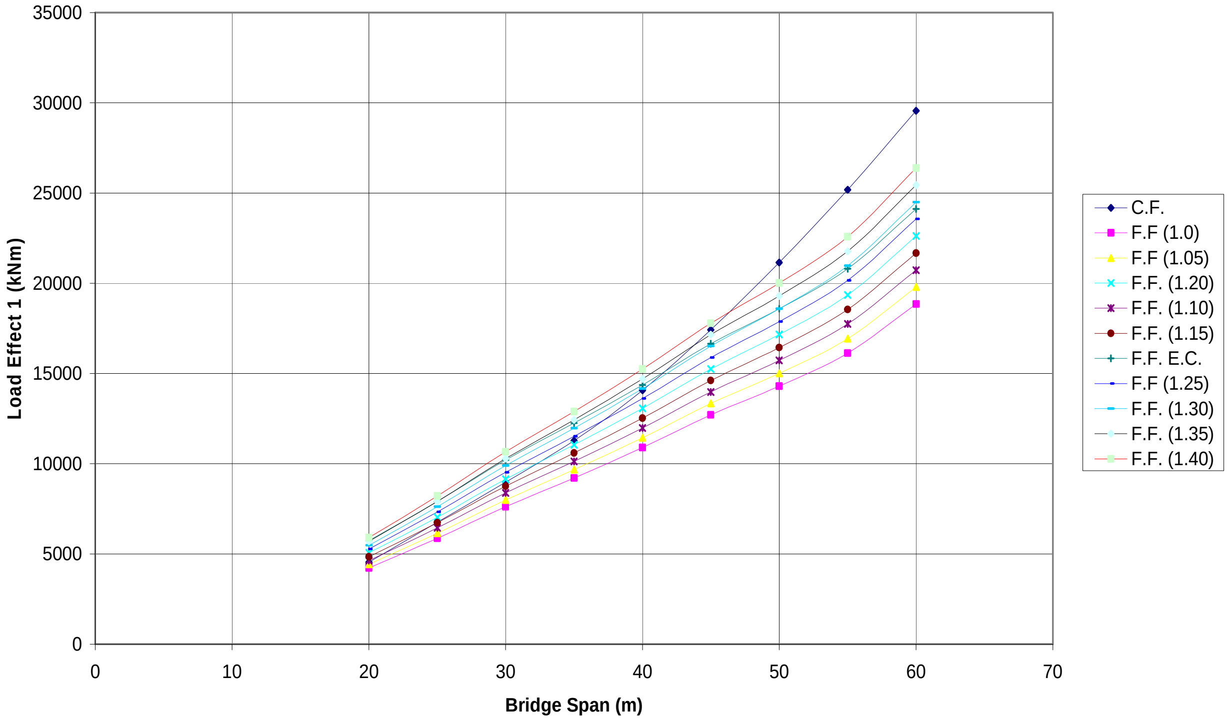


Fig 55: Bridge Length Versus Load Effect 1 (comparison of free flow (variable DAF) versus congested flow extreme loads).

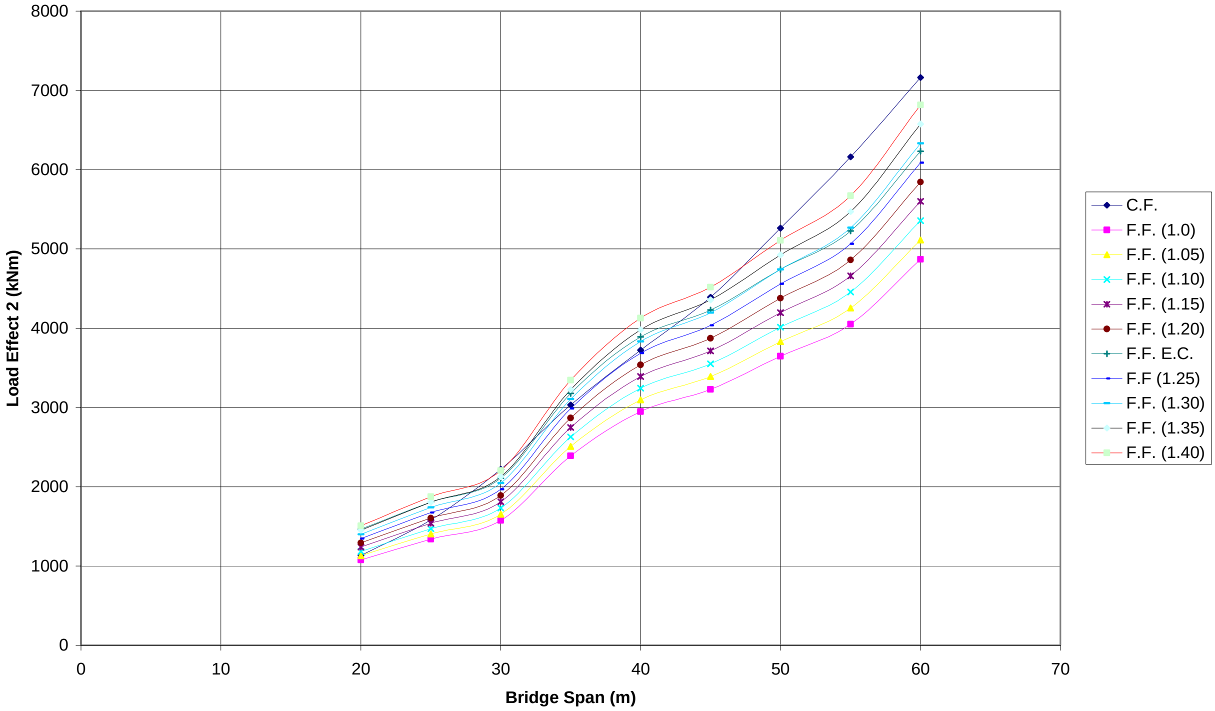


Fig 56: Bridge Length Versus Load Effect 2 (comparison of free flow (variable DAF) versus congested flow extreme loads).

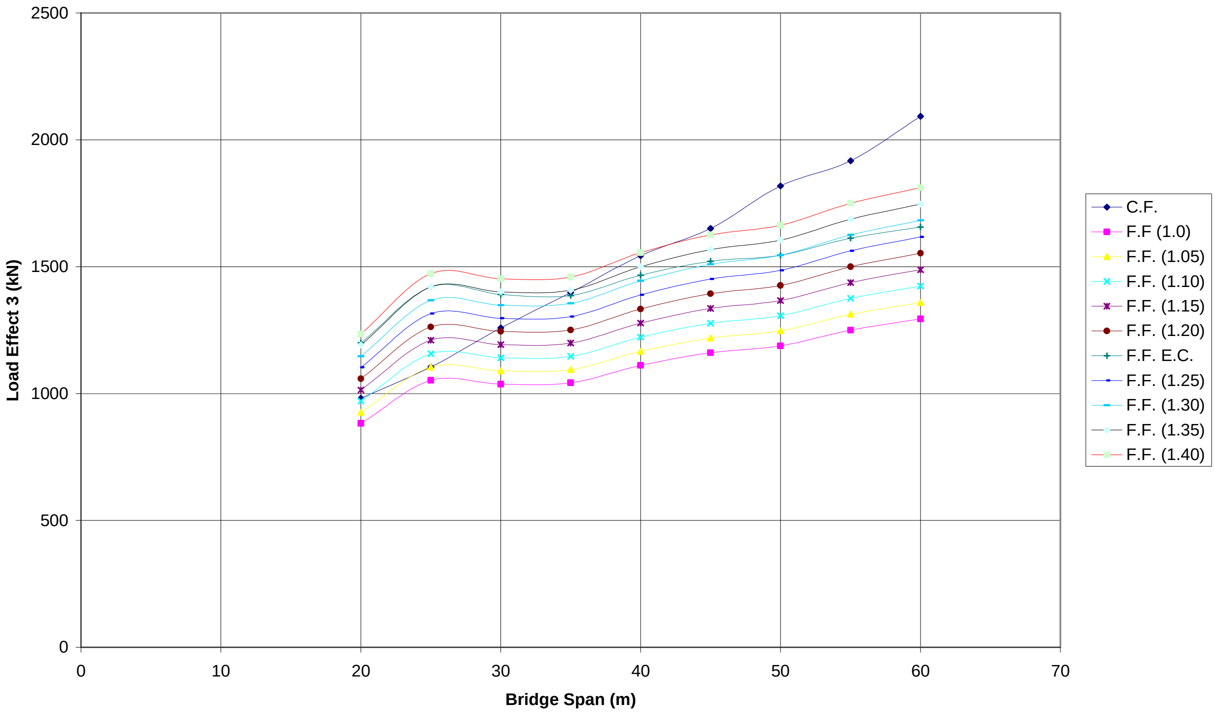


Fig 57: Bridge Length Versus Load Effect 1 (comparison of free flow (variable DAF) versus congested flow extreme loads).

From Fig.'s 55, 56 and 57 the C.L.oF.T.s values can be obtained for the different DAF values. In Fig. 58 the C.L.o.F.T.s. values versus DAF is plotted.

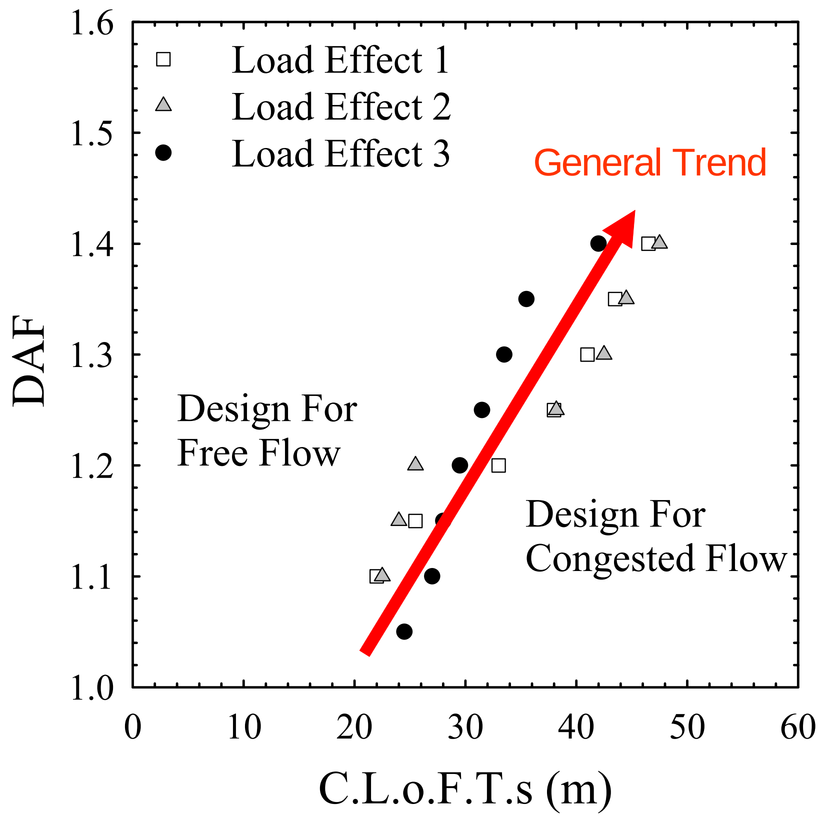


Fig. 58: DAF versus C.L.o.F.T.s. for 3 Load Effects 1, 2 and 3.

Fig. 58 shows the general trend of DAF versus C.L.o.F.T.s.. As discussed earlier, from Fig. 58 we can say that any bridge with length to the left of the general trend curve should be designed for free flowing traffic with associated DAF and any bridge with bridge length to the right of the general trend curve should be designed for congested traffic. Also we can see from Fig. 58 that for DAF values between 1.0 and 1.30 per cent, the C.L.o.F.T.s. lengths fall between 20 and 40 metres. This means that for a variance in DAF of 30% there is a variance of bridge length of approximately 20 metres. From Fig. 58 we can see that even for conventional DAF values of about 1.25 to 1.30, the C.L.o.F.T.s. values lies between bridge lengths of 30-40m. If the DAF is changed by 10% the C.L.o.F.T.s. changes by approximately 10 meters. Even if a substantially high DAF of 1.40 is considered, the C.L.o.F.T.s. value is approximately

40m. This proofs that an allowance for dynamic loading is not required for bridges over a certain length.

Calibrated theoretical models based on experiments carried out on the Maura River Bridge for free flowing traffic, including multi-truck events found the mean DAF to be 1.10 with a standard deviation of 5% (Caprani and Rattigan 2006)[4]. A DAF is randomly read from the normal distribution curve from these tests and multiplied by the free flowing extreme values in Fig.'s 55, 56 and 57. This extreme free flow load is graphed versus bridge length in Fig.'s 59, 60 and 61. The extreme static load due to congested flow, with inter-vehicle gaps of 5, 7.5, 10, 12.5 and 15 meters is also plotted versus bridge length in Fig.'s 59, 60 and 61. In the Eurocode an inter-vehicle gap of 5 meters was used but this may be unrealistic as this inter-vehicle gap represents the gap from the back axle of the lead truck to the front axle of the following truck. With large freight trucks the overhang distance, from the back axle to the back of the track may be quiet large, up to 2.5 meters in some cases. Also there is an overhang distance at the front of the following truck. For this reason it is important to investigate the variance in the C.L.o.F.T.s. values for different inter-vehicle gaps.

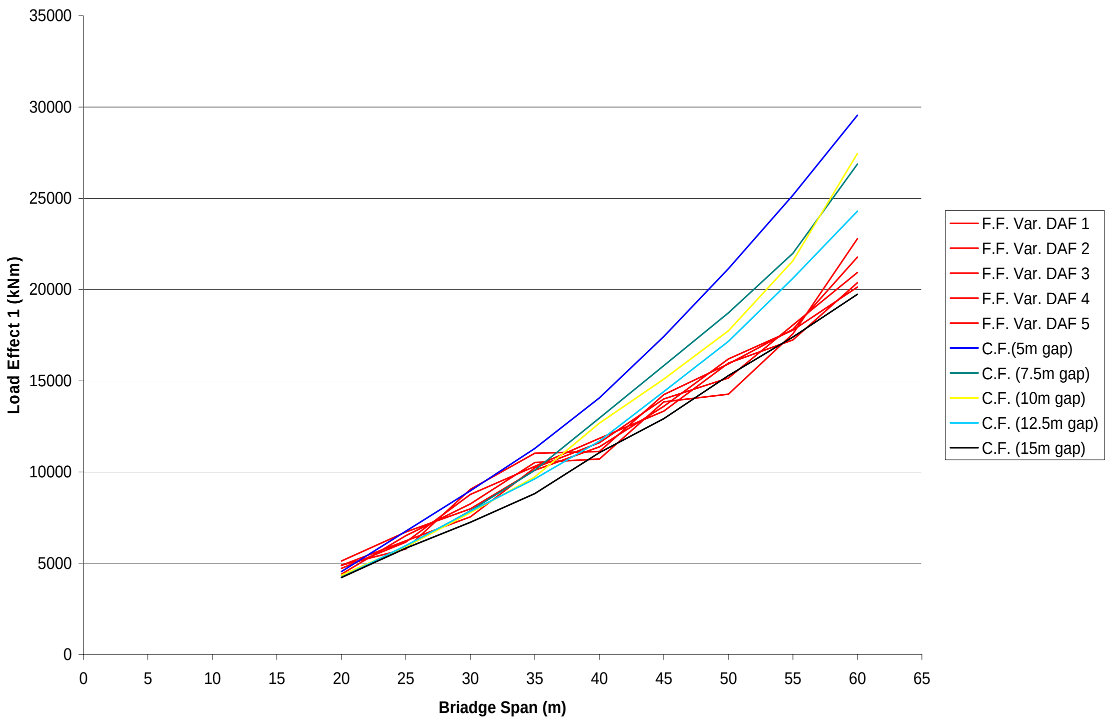


Fig. 59: Bridge Length versus Load Effect 1 (Congested Flow (5-15m inter-vehicle gaps) and Free Flow (varying DAF, mean 1.10 with standard deviation 5%))

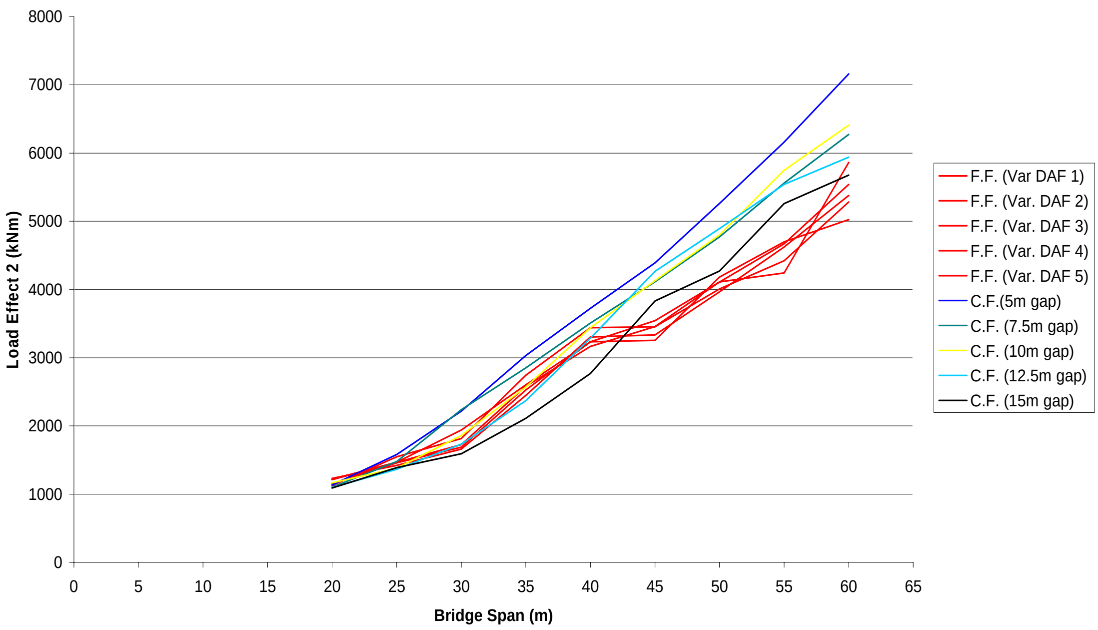


Fig. 60: Bridge Length versus Load Effect 2 (Congested Flow (5-15m inter-vehicle gaps) and Free Flow (varying DAF, mean 1.10 with standard deviation 5%))

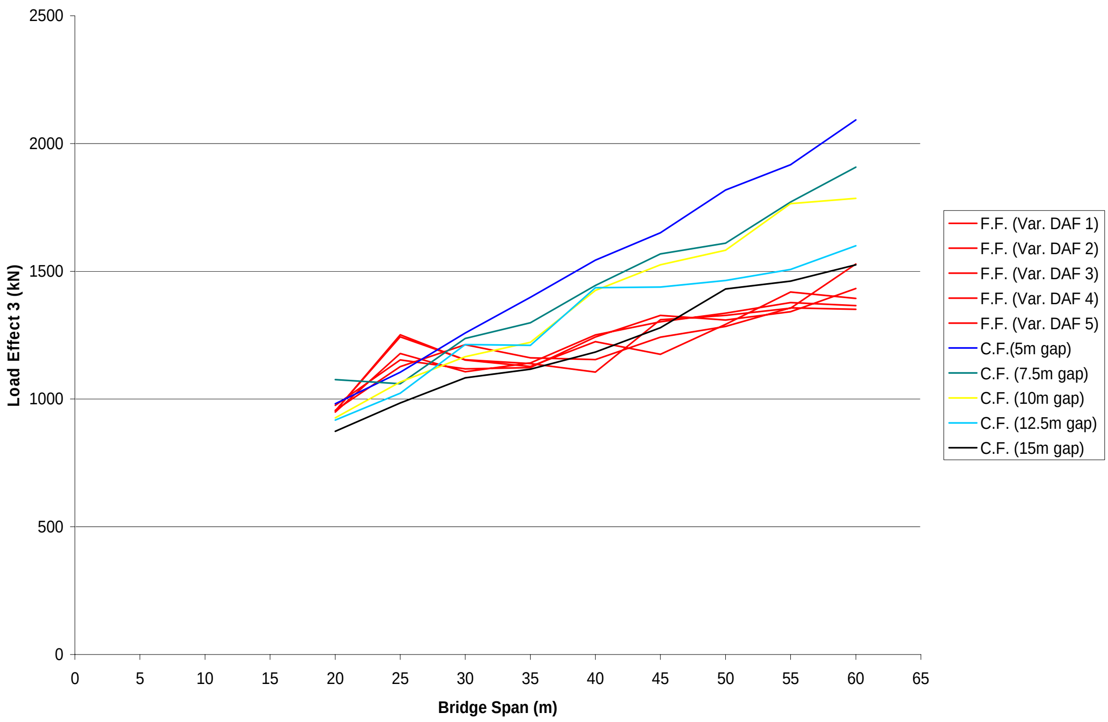


Fig. 61: Bridge Length versus Load Effect 3 (Congested Flow (5-15m inter-vehicle gaps) and Free Flow (varying DAF, mean 1.10 with standard deviation 5%)).

The C.L.o.F.T.s. values are read off Fig.'s 59, 60 and 61 and a Histogram of the C.L.o.F.T.s. values for the different inter-vehicle gaps for load effects 1, 2 and 3 can be produced. The Histograms in Fig.'s 62, 63 and 64 contain 1000 C.L.o.F.T.s. values for each inter-vehicle gap for load effects 1, 2 and 3. There are no readings for the congested model with a 15m inter-vehicle gap for load effect 1 in the Histograms in Fig.'s 61 as it was found that the free flow extreme loads completely governed the congested model for bridges with lengths between 20-60 meters.

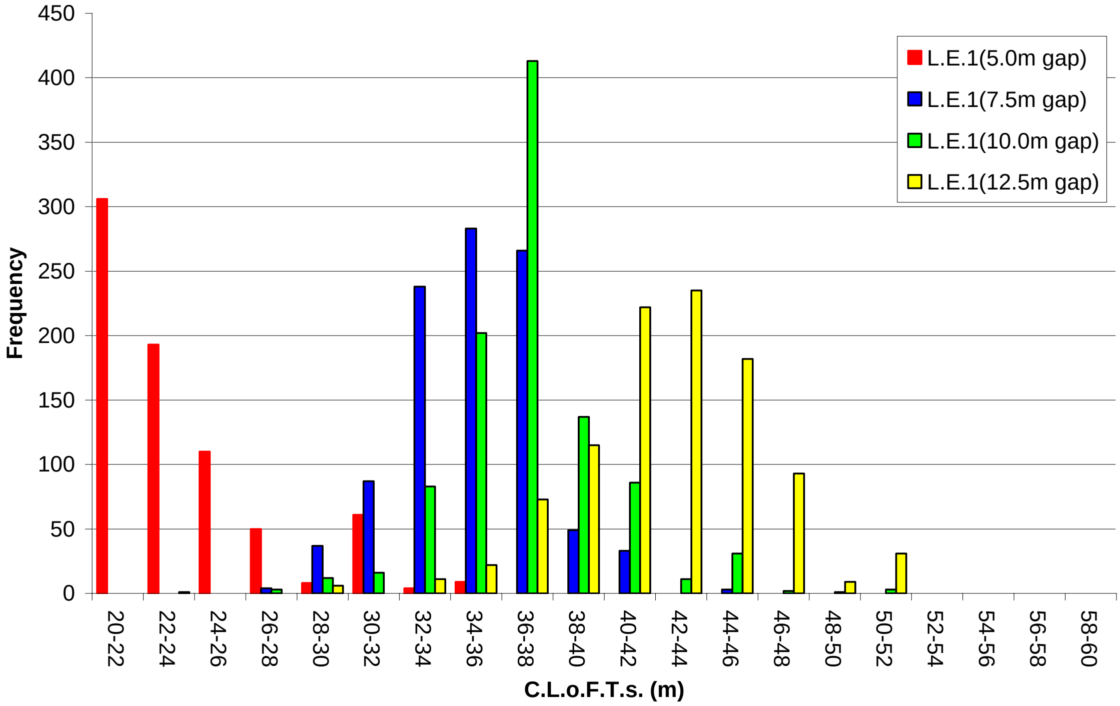


Fig. 62: Histogram of C.L.o.F.T.s. (L.E.1).

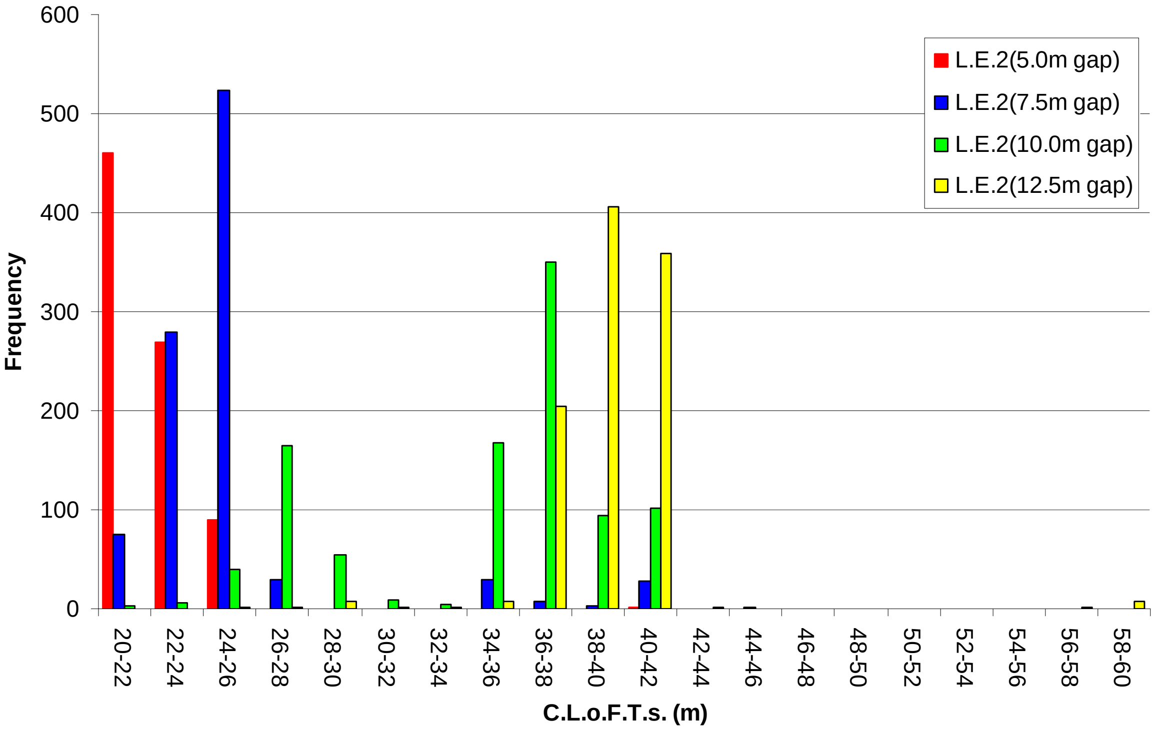


Fig. 63: Histogram of C.L.o.F.T.s. (L.E.2).

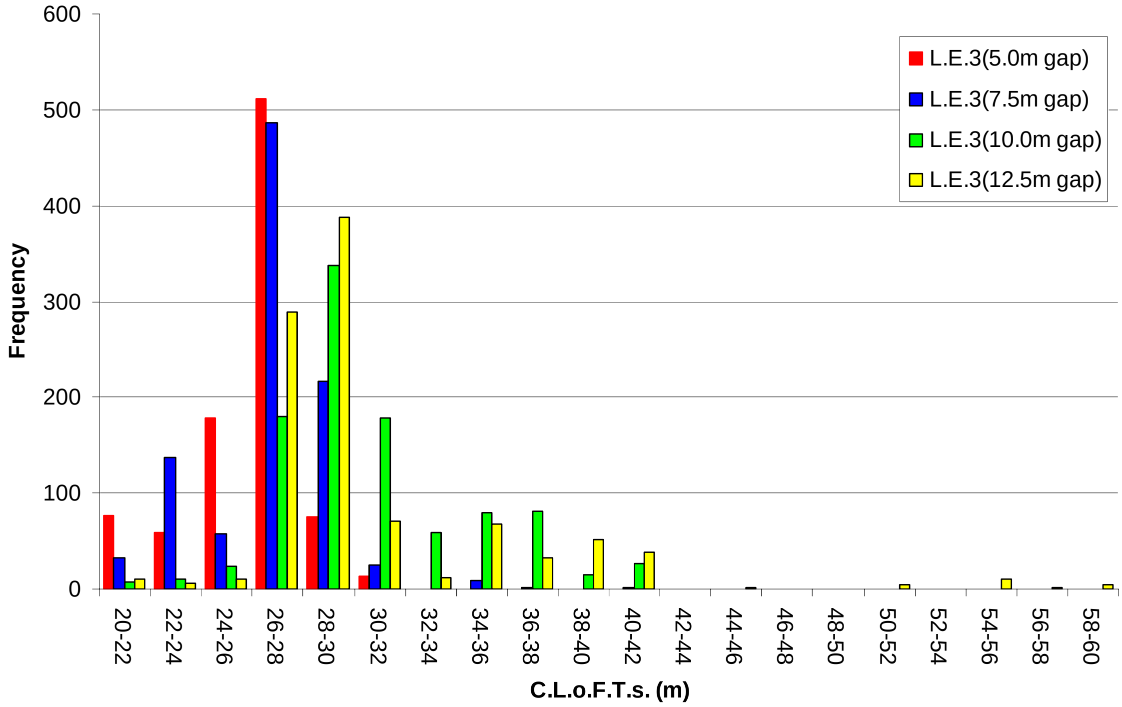


Fig. 64: Histogram of C.L.o.F.T.s. (L.E.3).

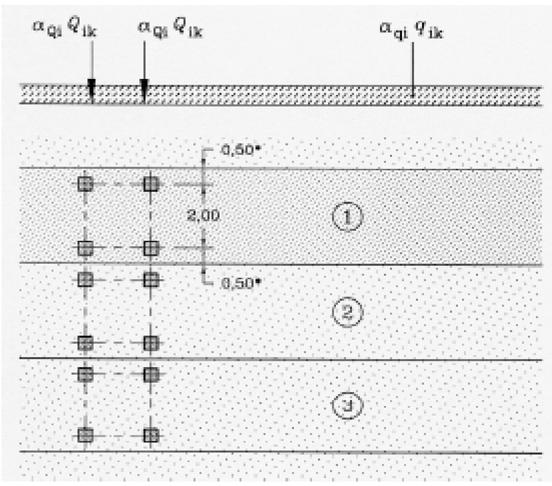
At a certain bridge length congested flow begins to govern. If a conventional congested flow model with a 5 meter inter-vehicle gap is considered, this bridge length maybe as low as 20-22 meters for load effect 1 and 2, and 26-28 meters for load effect 3. This means that DAF can be neglected for bridges above these lengths. The bridge length at which congested flow begins to govern is conventionally taken as between 45-50 meters for the 3 load effects. This is considerably higher than the C.L.o.F.T.s. lengths mentioned above. There maybe a significant reduction in the design loads for bridges if the C.L.o.F.T.s. lengths are used in bridge design.

5.6 Eurocode Load Model 1 loading versus Computer Generated loading

To investigate if there is a reduction in the extreme load due to traffic if the computer generated loading with associated C.L.o.F.T.s. (C.P.) is used instead of the Eurocode bridge design loading, the extreme traffic load for the Eurocode was calculated for bridge lengths 20-60 meters and compared to the extreme traffic loads generated in this research (C.P.). The Eurocode Load Model 1 was used to calculate the (E.C.) extreme loads.

Location	**Tandem system *TS***	***UDL* system**
	Axle loads Q_{ik} (kN)	q_{ik} **(or** q_{rk}**)** (kN/m^2)
Lane Number 1	300	9
Lane Number 2	200	2,5
Lane Number 3	100	2,5
Other lanes	0	2,5
Remaining area (q_{rk})	0	2,5

The details of Load Model 1 are illustrated in Figure 4.2a.



Key
(1) Lane Nr. 1 : Q_{1k} = 300 kN ; q_{1k} = 9 kN/m^2
(2) Lane Nr. 2 : Q_{2k} = 200 kN ; q_{2k} = 2,5 kN/m^2
(3) Lane Nr. 3 : Q_{3k} = 100 kN ; q_{3k} = 2,5 kN/m^2
* For w_l = 3,00 m

Fig. 65: Eurocode 1: 3 Figure 4.2a – Application of Load Model 1.

Bridges with lengths 20-60m, with 2 lanes 3.0m wide each were compared for Load Effects 1, 2, and 3. To calculate the Load Model 1 U.D.L. system a U.D.L. of 9.0kN/m2 was placed on one lane and a U.D.L. of 2.5kN/m2 was placed on the other lane. To calculate the Tandem system a two axle truck with each axle weighing 300kN was put on one lane and a two axle truck with each axle weighing 200kN was put on the second lane, side by side. These trucks were stepped across the bridges of different lengths in 0.01 meters increments and the extreme load case was recorded for each load effect. The U.D.L. system and the Tandem system loads were added to give an extreme design load for the Eurocode Load Model to for the three load effects.

The extreme C.P. loads were taken from Fig.'s 59, 60 and 61, with the average free flow multiplied by DAF load values used up until a bridge length corresponding to the C.L.o.F.T.s. value. Above this length the congested flow (with 5m inter-vehicle gap) load values were used. The congested flow (with 5m inter-vehicle gap) load values were used in the comparison with the Eurocode as this is the inter-vehicle gap that the Eurocode used and also this will give the lowest percentage difference as the extreme congested load values for the 5m inter-vehicle gap are greater than those of the 7.5, 10.0, 12.5 and 15.0m inter-vehicle gap congested models.

Fig.'s 66, 67 and 68 show the extreme load values for the Eurocode load Model 1 (E.C.) and the new computer generated traffic (C.P.) extreme load values for Load Effect 1, 2 and 3.

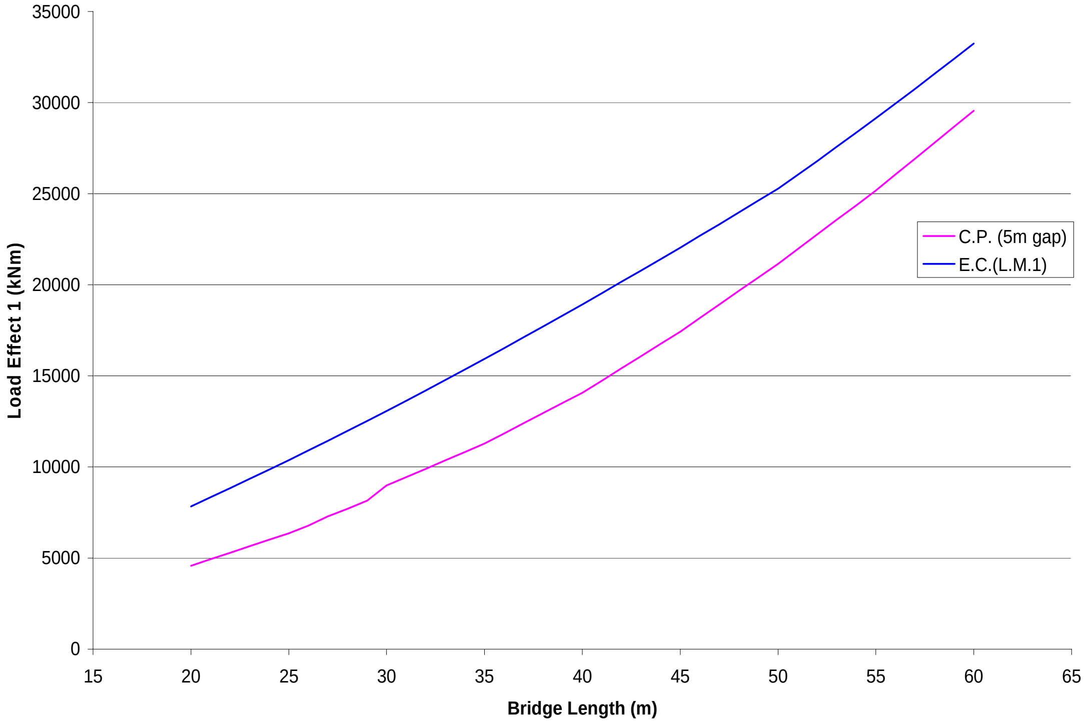


Fig. 66: Bridge Length versus Load Effect 1.

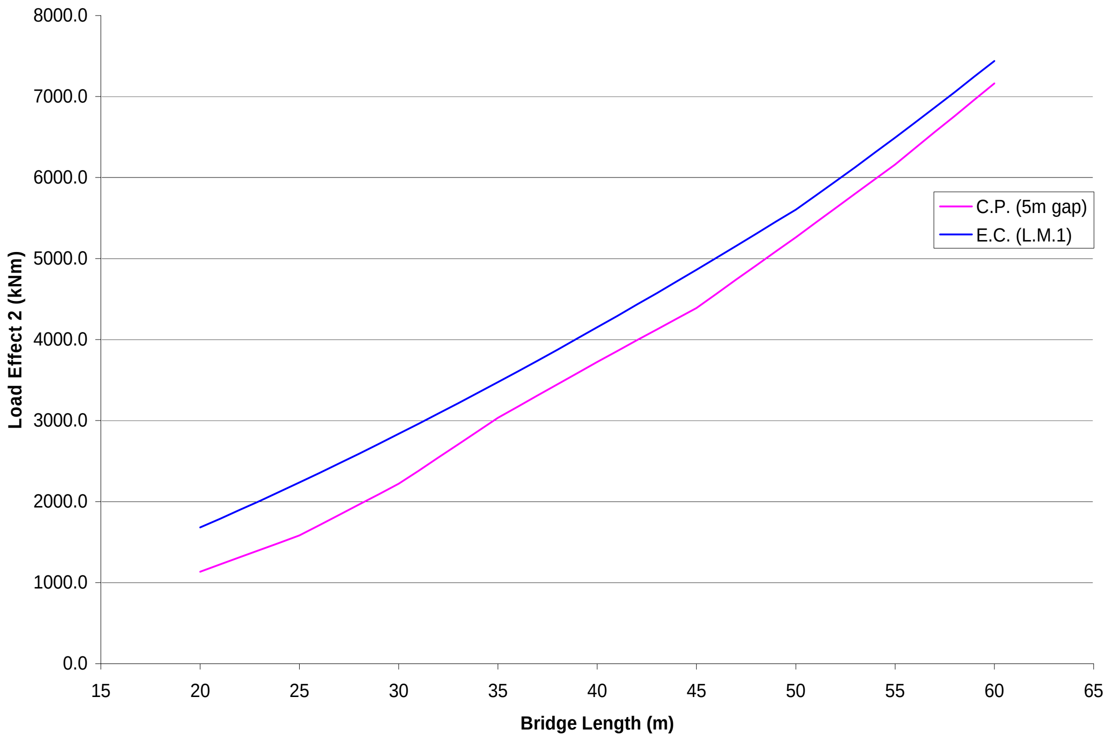


Fig. 67: Bridge Length versus Load Effect 2.

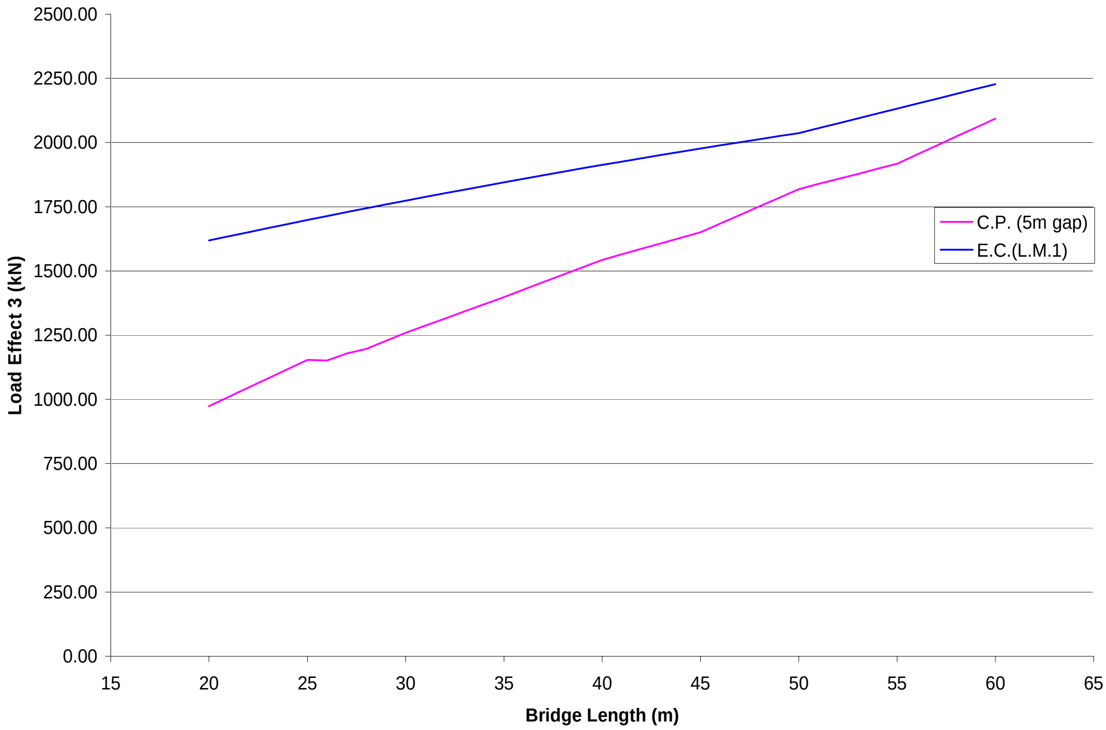


Fig. 68: Bridge Length versus Load Effect 3.

Fig. 69 shows the percentage difference between the Eurocode extreme load values and the C.P. extreme values. For 20 meter bridges the percentage difference is as high as 42-33% whereas for bridge lengths of 60 meter the percentage difference is 11-4%. As the C.L.o.F.T.s. lengths for (C.P.) are between 20-22 meters for load effect 1 instead of the 45-50 meters C.L.o.F.T.s. length used in the Eurocode, it is expected that for bridge lengths between 22-50 meters there should be a large percentage difference. This is because for bridge lengths between 22-50 meters the loading used for load effect 1 in C.P. is congested flow with no DAF whereas the loading used by the Eurocode between 22-50 meters for load effect 1 is free flow multiplied by DAF. From 20-22 and 50-60 meters bridge lengths the loading regime is the same for both the C.P. loading and the Eurocode loading. As the C.P. extreme load values have the required characteristic value stated in the Eurocode, 1000 year return period, Fig. 69 shows that, especially for highway bridges under 40 meters, the design traffic load values for the Eurocode maybe over estimated for some bridges. This can have serious cost implications for the rehabilitation of European bridges, especially in view of the proposed change in the legal weight limit of trucks in Europe.

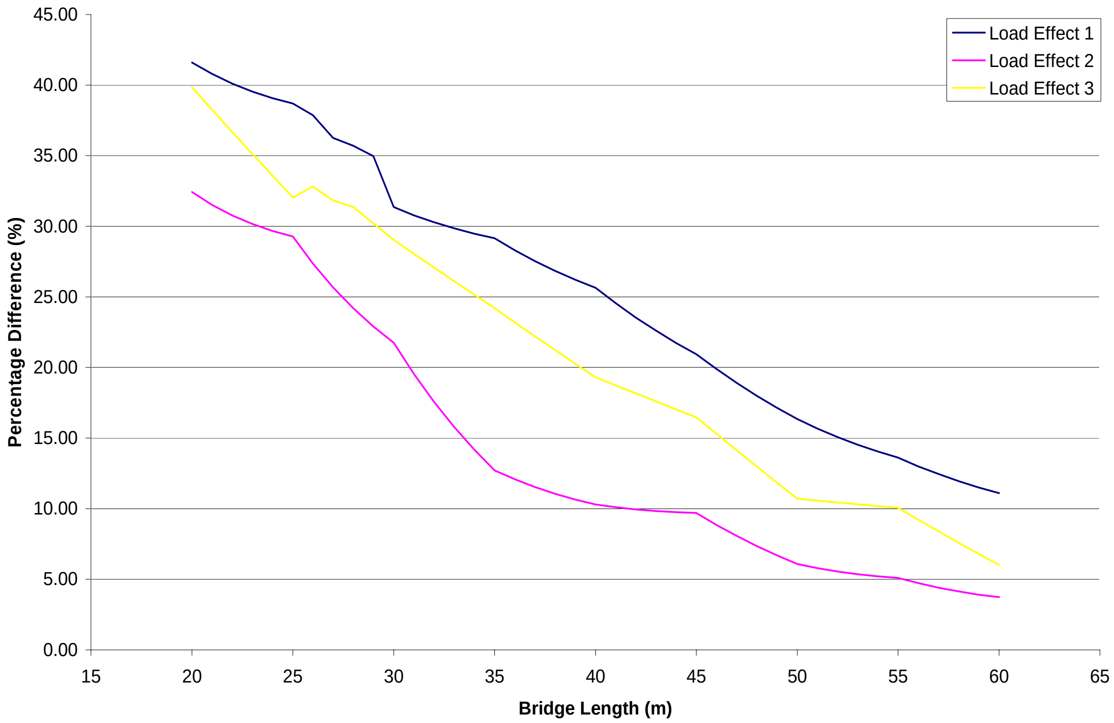


Fig. 69: Percentage Difference Eurocode (Load Modal 1) Versus Computer Prediction (C.P.).

7. Conclusions:

This thesis investigated the importance of using an accurate DAF value in bridge engineering and also the critical loading traffic regime, be it free flow or congested, for a range of bridges. General results found in this research are as follows:

- The inter-vehicle gap used in the congested flow model is very important in calculating the C.L.o.F.T.s. values for different bridge lengths. The greater the congested flow model gap the greater the C.L.o.F.T.s. values. This is because the slope for congested flow, in Fig. 59, 60 and 61, is reduced with an increase in the inter-vehicle gap used in the congested model. As the free flow model is independent of the inter-vehicle gap used in the congested model (free flow model inter-vehicle gaps are generated randomly from recorded onsite data) its slope remains the same in Fig. 59, 60 and 61.
- The C.L.o.F.T.s. value may be considerable lower than previous expected, the recognised C.L.o.F.T.s. value of 45-50 meters may actually be as low as 24-26 meters. This means that congested flowing traffic governs the critical loading traffic regime for bridges over 26 meters.
- Extreme traffic design loads calculated form the Eurocode Load Model 1 may be overestimated by up to 40% for some bridge lengths.
- Using an accurate DAF value is very important as a change in DAF of 10% corresponds to a change in C.L.o.F.T.s. length of approximately 10 meters, thereby increasing the design traffic load for bridges in this 10 meter gap.

8. Recommendations for Further Research:

Currently there is a lot of research being carried out in the field of traffic loading on highway bridges. This can be accredited to the wide variance in the design codes and view points from different countries relating to this topic. Some ideas for further research in this field are as follows:

- An investigation into the probability of different inter-vehicle gaps occurring in congested flow could prove very important as inter-vehicle gaps are important in calculating C.L.o.F.T.s. values.
- An investigation into the percentage of time that congested flowing traffic dominants could be important as at the moment a very rudimentary approached is used.
- As some current research in this field surrounds weather heavily loaded freight trains (up to 165 tonnes) dominate bridge loading, an interesting research topic would be to model some of these freight trains and obtain the load effects for one of these trucks crossing and compare it to congested flows extreme loading.
- Investigate if the extreme dynamic load should be added to the extreme static load for load effects 1 and 2 as some on site recordings suggest the worst case dynamic load occur in the first and last 10% of the bridges length and the worst case static load occurs in the centre on a bridge length for load effects 1 and 2. An investigation as to how this would effect the overall extreme design load may prove interesting.
- Load effects 1, 2 and 3 are calculated in the C.P. model using 1 influence line. As there are usually 3 or 4 beams supporting a highway bridge, it would be more accurate to calculate the 3 load effects from the influence lines for each beam in a bridge. This would be a more complex as torsional effects would also have to be accounted for but this method would produce more accurate results and a better understanding of traffic loading on bridges.

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