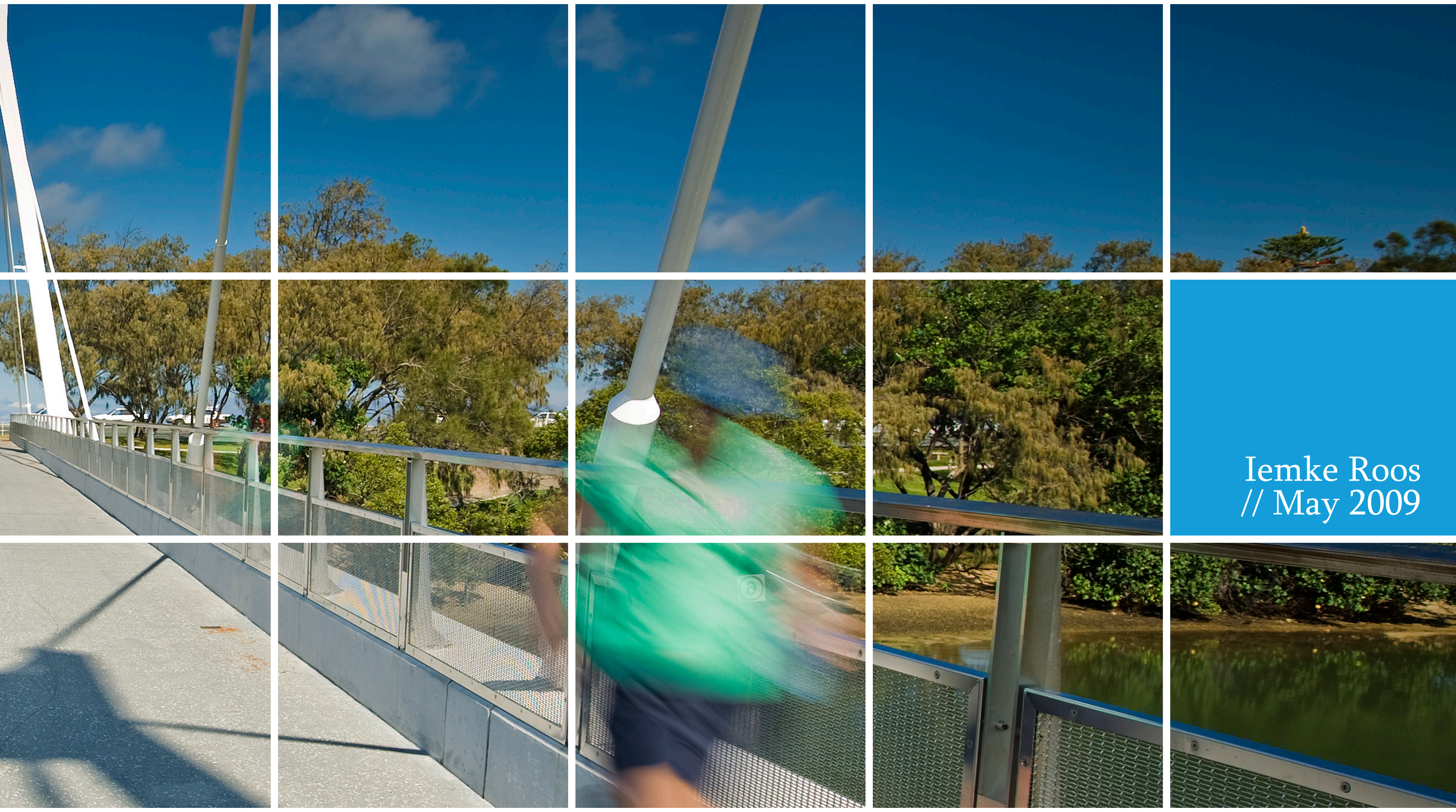
HUMAN INDUCED VIBRATIONS ON FOOTBRIDGES

Application and comparison of pedestrian load models



Iemke Roos
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Application & comparison of pedestrian load models

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The Master thesis is a part of the study Civil Engineering at Delft University of Technology in the Netherlands. This study was completed between October 2008 and May 2009 at Arup in Brisbane, Australia.

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Summary

The last few decades the community demands for more interesting bridges. Improvements in material properties, design methods, building techniques and the involvement of architects led to longer and slender footbridges. These bridges tend to be more sensitive to dynamic forces induced by pedestrians, resulting in vibrations of the bridge deck. These vibrations can in some cases attain high proportions, especially when the walking pace of the pedestrians approaches the natural frequency of the bridge. Such a case could result in a situation where the pedestrian feel uncomfortable or even unsafe. This topic has thus become an important issue for the Serviceability Limit State of footbridges.

Some Codes of practice nowadays refer to this topic but it is still a developing field. Dynamic analyses during the design phase have become inevitable. This report compares and evaluates three load models described in the codes (or proposal for the codes): Proposal Annex C (to EN 1991-2:2003), the British National Annex (to EN 1991-2:2003) and the Australian Standard (AS 5100.2-2004). All three codes have different approaches to this topic. Proposal Annex C considers walking pedestrians (single and groups) and crowds and represent all of them by non moving harmonic loads. The British National Annex also considers joggers which have total different walking pattern. The fundamental difference with the load models described in Proposal Annex C is that the loads are represented by moving harmonic loads. So does the Australian Code, but this one only considers the model of a single pedestrian.

Computer models have been used to analyse two existing footbridges (the Goodwill Bridge and the Milton Road Bridge, both located in Brisbane, Australia) according to these codes. The real behaviour under pedestrian loads of these bridges is known and could thus be compared to the responses generated with the analyses. A modal analysis on both bridges has shown that both bridges are susceptible to vibrate, as their Natural Frequencies lie within the walking and jogging frequency range. The responses are expressed in maximum acceptable accelerations or displacements, depending on the considered code. The load models show scattered responses. The ones generated with the British National Annex seem to correspond most to the actual behaviour of the bridges. However these load models can be subjected to improvements and should also be used with care. Errors in the output can easily occur during the analysis because of the bridge model or the chosen time step of the analysis.

The analyses have shown that the responses greatly differ between the British National Annex and Proposal Annex C: the ones generated with the latter are mostly higher than the ones generated with the British National Annex. Measurements on the Goodwill Bridge have shown that the real accelerations lie in between these values. These values can also be categorised in different sensitivity groups. The noticed vibrations on both bridges could all be categorised in 'Just perceptible' or 'Clearly Perceptible'. Whereas the

responses from Proposal Annex C could be categorized as 'Annoying', the responses of the British National Annex (expect the crowd load) and the Australian Standard could be categorised in 'Just perceptible' or 'Clearly Perceptible'. Taking this aspect into consideration leads to the conclusion that the load models from the British National Annex seem to best represent the real behaviour of the bridges.

The parameters influencing the load properties have been analysed and improvements have been suggested there where it was needed. Especially the Dynamic Load Factors should be revised. The ones from Proposal Annex C should take into consideration that the loads are actually representing moving pedestrians. The ones from the British National Annex lead to smaller accelerations than the ones that occur in reality.

Generally it can be concluded that footbridges should be evaluated more in according to their use than is the case now, the input as well as the comfort criteria. The comfort criteria are in all cases based on one value which defines the acceptability of vibrations. However it has been shown that different degrees of sensitivity could be defined for different types of bridge users or for different situations (walking pedestrians, joggers, people standing still, crowds). The comfort criteria could hence be more nuanced according to the expected use of the bridge.

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1 Introduction

Everyone who has ever walked over a bridge has probably felt it or even seen it: small movements of the deck, which is going up and down, caused by traffic, pedestrians or even wind. These vibrations are usually small and only perceptible with a clear reference point or when standing still on the bridge. The magnitude in which this phenomenon occurs depends on many factors: the length of the bridge, the stiffness of the bridge, the type of load, the magnitude of the load, the place of the load and many more.

The last few decades, there has been a tendency to longer and more slender footbridges. This is mainly the result of improvements in material properties, designing methods, building techniques but also the involvement of architects which has led to bridges with distinguishing forms that do not always follow the logical structures that engineers used to design. A few examples are shown in Figure 1.1 and Figure 1.2.

Figure 1.1 Nescio Bridge (Amsterdam, The Netherlands)

Figure 1.2 Millennium Bridge (London, United Kingdom)

As the structures of footbridges change, so do their natural frequency and damping: the natural frequency tends to lie in such ranges that vibrations can occur, which can be perceived as annoying or even unsafe by users of the bridges. This phenomenon is reinforced by the fact that modern footbridges have a lower damping: vibrations are not being dissipated that fast anymore.

To fully understand the response of the bridge it is essential to model the loads correctly. Pedestrian loads are difficult to model because of their hazardous aspects: weight of the pedestrian, walk velocity, number of pedestrians, distribution of the pedestrians over the bridge etc. Researches in this field have been done earlier in this field and thus different sets of load models have been set up.

The objective of this thesis project is to apply and compare several load models described in the codes of practice intended for practical engineering application. To validate the load models, the computer generated responses are compared to the real behaviour of two bridges. An evaluation of the load models is given and improvements are proposed in order to make these load models more accurate and practical.

To familiarize the reader with dynamics, a general introduction to dynamics of bridges and its interaction with human behaviour is presented in chapter 2. The following chapter gives a short presentation of the bridges that are being analysed in this thesis. Chapter 4 is a review of parts of certain codes that deal with pedestrian vibration loads and requirements. Chapter 5 and 6 present the analyses according to the codes and the results. Chapter 7 gives a general evaluation of the codes and proposes some improvements. Finally the conclusion and the recommendations are presented in Chapter 8.

2 Dynamics of footbridges and human interaction

This chapter presents the dynamics of footbridges induced by human activity, like walking and running. One should note that wind can also induce a dynamic load on the bridge. This is not part of this study and thus will not be explained. Vibrations of a footbridge can have influence on pedestrians in such a way that pedestrians have to adjust their pace or feel uncomfortable.
The first paragraph presents some general information about the current knowledge of bridge dynamics. The second paragraph presents the way pedestrians can be a source of vibration for certain footbridges. The last paragraph deals about the sensitivity of pedestrians due to vibrations.

2.1 Dynamics of footbridges

The last few decades, the improvement of technology has permitted engineers to design slimmer bridges: steel quality increased resulting in smaller cross-sections and longer spans, Finite Element Methods helped engineers to design more accurately and the quality of the building technology also increased. This has lead engineers to design leading edge bridges, like the Millennium Bridge in London for instance (Figure 2.1), which have lower stiffness, lower mass and lower damping.

Figure 2.1 Millennium Bridge (London, United Kingdom)

Figure 2.2 Solférino Bridge (Paris, France)

A few footbridges have encountered some dynamic problems because of this: at the opening of the Millennium Bridge in 2001, the bridge began to sway laterally. The hundreds of people walking over the bridge at that moment had to adjust their pace to be able to stand in equilibrium. This phenomenon on this scale had not been foreseen. The same phenomenon occurred to the Solférino Bridge in 1999 in Paris, just after its inauguration (Figure 2.2), while a dense crowd walked over it.

This paragraph presents how a bridge can be described as a vibrating system, i.e. an oscillator and which parameters influence the system.

2.1.1 Footbridges as oscillators

A footbridge can only act as an oscillator if a dynamic load is applied. A dynamic load is a load that varies over the time, in contrary to a static load which stays constant over the time. In the case of this study, we only consider pedestrian loads which, as we will see in the next paragraph, can have considerable influences on footbridges.
An oscillator can have one or more degrees of freedom ("DOF"). An oscillator with one degree of freedom can be represented as is shown in Figure 2.3, where:

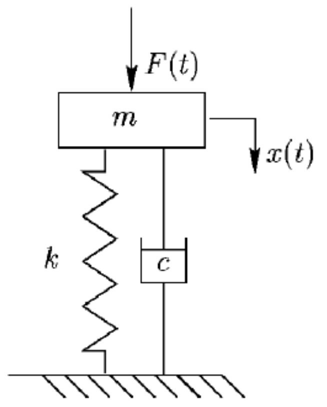

Figure 2.3 Representation of a simple oscillator

m = mass of the object (in our case the mass of a footbridge)
k = stiffness of the support (this could for example represent the stiffness of the main girders of a bridge)
c = damping of the structure (caused by friction between members of the structure or the use of dampers)
F(t) = external force on the footbridge (in our case pedestrians), variable in time
x(t) = displacement of the mass in the time

The sum of the vertical forces should be equal to the mass multiplied by its acceleration:

$$\sum F = m \cdot a$$

The acceleration is the second integral of the displacement:

$$a = \ddot{x}(t)$$

When pushing the oscillator downwards as shown in Figure 2.4, we find:

$$F(t) - F_{spring}(t) - F_{damping}(t) = m \cdot \ddot{x}(t) \quad (1)$$

We know that:

$$F_{spring}(t) = k \cdot x(t)$$

$$F_{damping}(t) = c \cdot \dot{x}(t)$$

Equation *(1)* becomes:

$$F(t) - k \cdot x(t) - c \cdot \dot{x}(t) = m \cdot \ddot{x}(t)$$

Rearranged, this leads to an important formula for dynamics:

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = F(t)$$

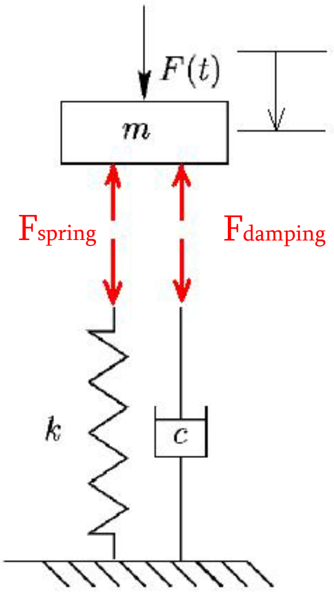

Figure 2.4 Forces in system when pushing the mass downwards

This is a second order differential equation, with which the behaviour of a structure (with 1 DOF) can be described.

F(t) can be any dynamic load, in the form of a harmonic excitation, a pulse or a random excitation.

Such a system as presented on the former page has an important property: it has a natural frequency. When no load is applied and the mass has a given displacement, the mass will tend to oscillate in its natural frequency. This harmonic vibration dissipates in time with the damping. Would the damping be absent, then the mass would continue oscillating with the same amplitude.
Another important aspect of such a system is that the dynamic external load can be in coordination with the natural frequency: in that case, as the load is helping the system, relatively high oscillations can occur which can cause damage to the system.

Multiple degrees of freedom (MDOF)
The same principle applies to multiple degrees of freedom systems, like in Figure 2.5 where masses are connected together by springs and dampers.

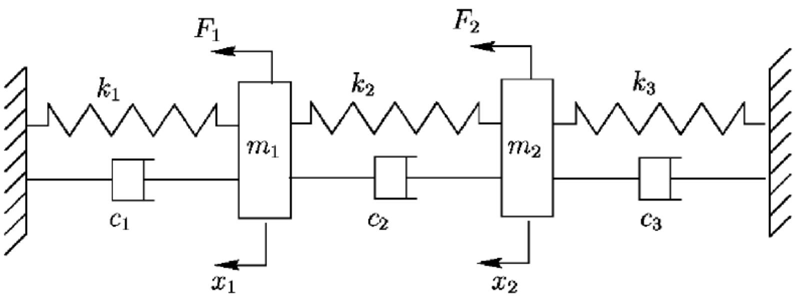

Figure 2.5 Representation of system with 2 degrees of freedom

The equation of motion resulting of a n-DOF system is similar to the 1-DOF system:
$M\underline{\ddot{x}(t)} + C\underline{\dot{x}(t)} + K\underline{x(t)} = \underline{F(t)}$
where:

M	Mass matrix of the system	(n * n)
C	Damping matrix of the system	(n * n)
K	Stiffness matrix of the system	(n * n)
$\underline{\ddot{x}(t)}$	Acceleration vector of the system	(n * 1)
$\underline{\dot{x}(t)}$	Velocity vector of the system	(n * 1)
$\underline{x(t)}$	Displacement vector of the system	(n * 1)
$\underline{F(t)}$	External force vector of the system	(n * 1)

One can see that footbridges can be seen as oscillators with one or more degrees of freedom. The parameters that influence the system are being discussed in the next paragraph. The external load is in our case the pedestrian load which is being discussed in paragraph 2.2.

2.1.2 Parameters

As has been shown in the former paragraph, the mass, the stiffness and the damping are the most important parameters and have the most influence on the response of the system. The mass and the stiffness can be determined quite easily. The damping however is more difficult to predict.

Mass and stiffness

When modelling a bridge in a Finite Element program, one should pay attention how the mass and stiffness should be spread out. This can have substantial influence on the way a bridge reacts.

It has been shown that handrails can have a substantial influence on the stiffness of a footbridge: 50 % of its stiffness can sometimes be counted as effective. The same applies for the decks: a part of the stiffness can sometimes be effective. However, this is dependant of the bridge and should be assessed for each one.

Damping

The damping of the bridge is difficult to determine. The main source of damping on footbridges is interaction / friction with other elements. Damping is essential to dissipate energy from the structure. Without damping, vibrations do not dissipate. Every structure has some damping, but as the design and construction technology are improving, the damping seems to decrease. That's one of the reasons of the higher vibrations on footbridges like the Millennium Bridge and the Solférino Bridge.

Damping can be split in three categories:

- Critical damping;
- Over-damping;
- Under-damping.

Usually, we speak about damping ratio, noted by ζ, which is dependant of the viscous damping coefficient *c* and the critical damping: $\zeta = \dfrac{c}{2\sqrt{km}}$

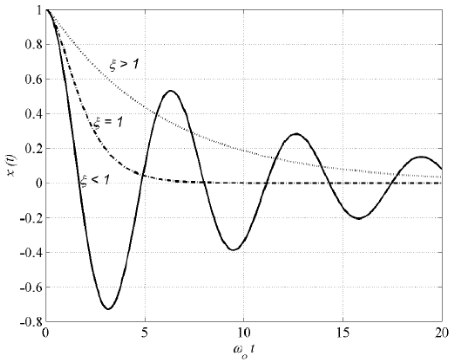

Figure 2.6 Illustration of the 3 damping modes

Figure 2.6 illustrates the three categories: when the damping ratio is smaller than 1, the vibrations dissipate slowly and theoretically never deem out. When the damping ratio is 1, the vibrations dissipate rapidly: in this case the damper is strong enough to avoid the mass to vibrate. The mass is directly going back to its original position. When the damper ratio is higher than 1, the same phenomenon occurs, but slower.

Damping is particularly important when the frequency of the dynamic load is in the neighbourhood of the natural frequency of the system: in that case resonance can occur and may cause structural damages. Figure 2.7 illustrates this phenomenon.

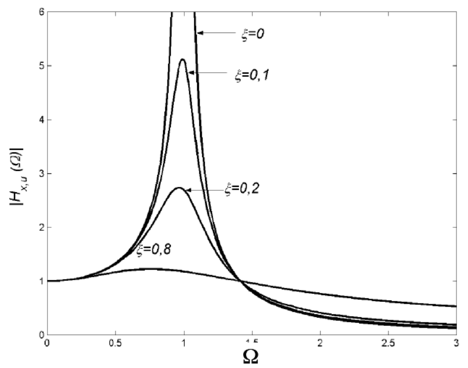

Figure 2.7 Resonance curve

The x-axis represents the relative pulsation Ω: this is the relationship between the load frequency and the natural frequency:
$\Omega = \omega / \omega_R$.
The y-axis represents the amplification factor of the displacement due to a static load F_0:
$x_{static} = F_0 / k$.
Ω = 0 and thus ω = 0 rad/s means a static load and the amplification factor is 1, so the displacement is equal to the static displacement.

The displacement is shown for a few damping cases. One should note that at around Ω = 1, the amplification becomes much higher. This is the moment where resonance occurs: the frequency of the dynamic load is nearby the natural frequency of the system. Without damping, the amplification is theoretically infinitely high. With a damping ratio of 0.2 it is already much smaller. Note that this graphic is just an example. The degree of amplification is dependant of the structure.

There have been several studies regarding to the damping of footbridges. In 1995 Bachmann et al published “Vibration problems in Structures: Practical Guidelines” in which the damping ratio of 43 footbridges has been compared. The main reason of difference in damping ratio seems to be in the construction type (Table 2.1).

Table 2.1 Common values of damping ratio ζ for footbridges[1]

Construction type	Min.	Mean	Max.
Reinforced concrete	0.008	0.013	0.020
Prestressed concrete	0.005	0.010	0.017
Composite	0.003	0.006	--
Steel	0.002	0.004	--

It can be concluded that light steel structures have a lower damping ratio than heavier concrete structures. Other papers mentioned that the number of spans could also influence the damping ratio, but no concrete data about that has been found.

[1] H. Bachmann *et al*, “Vibration Problems in Structures: Practical Guidelines”, 1995

2.1.3 Natural vibration modes

A mode of vibration is a characteristic pattern or shape in which a bridge vibrates. Footbridges can have many vibration modes which can be determined by a modal analysis. The actual vibration of a footbridge is generally a combination of all the vibration modes. Some examples of vibration mode are shown in Figure 2.8.

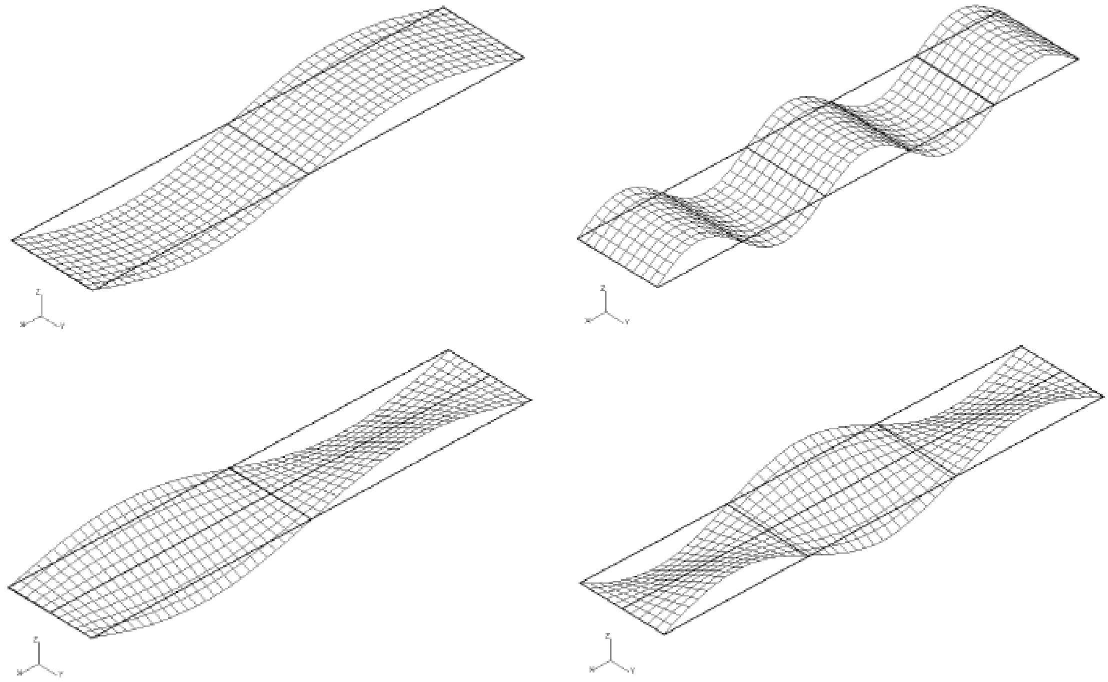

Figure 2.8 Examples of vibration modes

Generally, a bridge will get in the vibration mode that requests the less energy, coming from pedestrians. Energy can also have other sources, like wind. In 1940, the Tacoma Narrows Bridge collapsed because of its extreme flexibility, both vertically and in torsion during a strong wind. As can be seen in Figure 2.9 the response of the bridge was a so called flutter mechanism, which is a combination of bending and torsional vibrations: the bridge was effectively divided in two half spans, which vibrated out-of-phase with each other. The structural damages were such that it eventually collapsed.

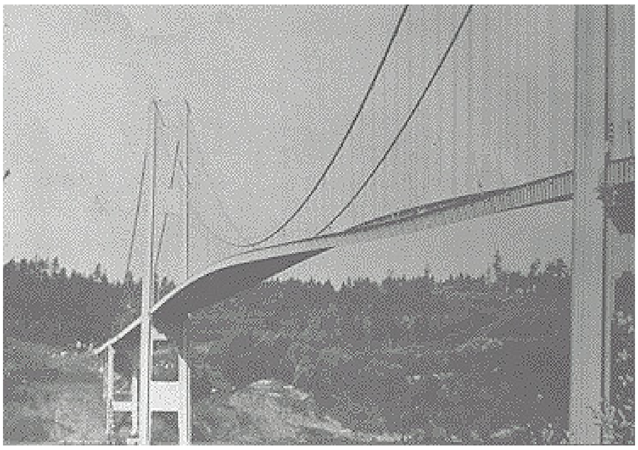

Figure 2.9 Tacoma Narrows bridge in bending and torsional vibration

Even though the energy source is totally different than this thesis handles and the result was an extreme situation, it demonstrates that knowing the vibration shape modes is essential to understand the behaviour of a bridge. Note that each vibration mode has its own natural frequency. Thus, one could state that the vibration modes which natural frequency lies in the range of the one of pedestrian loads is the most susceptible to occur.

2.2 Pedestrians as source of vibration

As seen before, pedestrians are more and more susceptible to create vibrations on footbridges as the design and building technologies constantly improve and slender and longer footbridges are being built. In first instance, this paragraph presents the forces exercised by pedestrians when walking or running. Then, the interaction between pedestrians and footbridges will be clarified. To finalise, the attention will go the modelling of pedestrians loads.

2.2.1 Forces induced by pedestrians

The centre of gravity of the human body is located at about 55% of its height and makes a sinusoidal motion during walking, both in vertical and horizontal directions. The force thus has three components: a vertical, a longitudinal and a lateral. The vertical component is the largest: up to 40% of the body weight. The lateral and longitudinal components are considerably smaller.

2.2.1.1 Vertical component

Early studies on pedestrian induced forces on footbridges were carried out by Blanchard et al. in 1977. Walking, running or jumping each produce a different loading curve over time as well as frequencies in which the oscillations can occur (Figure 2.10). Note that during walking always at least one foot is in contact with the ground.

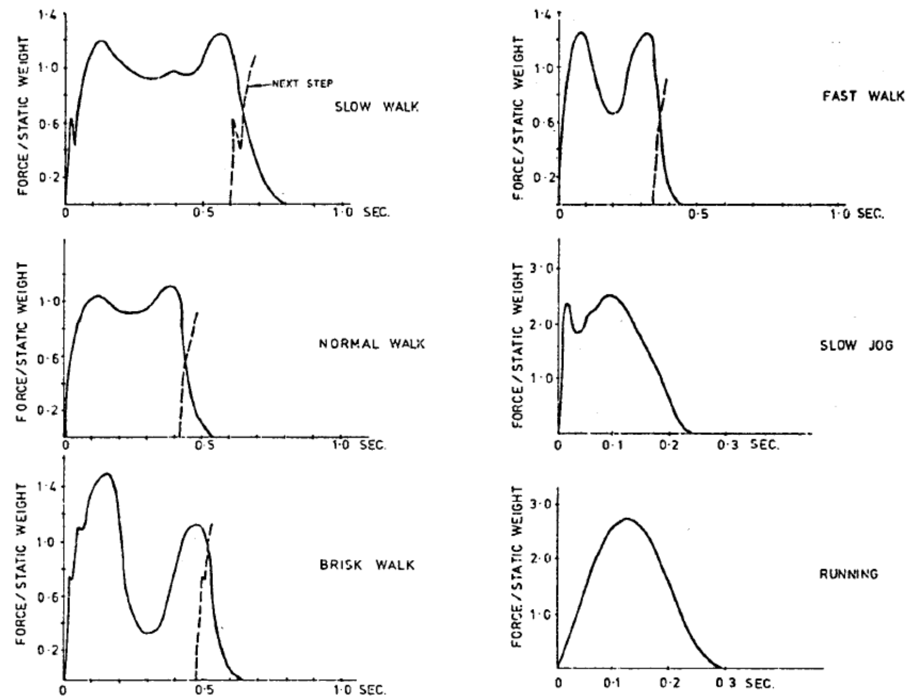

Figure 2.10 Loading curves of one pedestrian for different types of steps (Blanchard, 1977)

During walking, the vertical component shows a characteristic double hump, which is the result of the impact of the heel on the ground (first one) and the push off (second one). The maximums increase with increasing step frequency. Note that the next footfall begins just before the other one has finished (Figure 2.11). This phenomenon does not occur while running, as both foots can be off the ground at the same moment and the time that the foot is on the ground is relatively short, as shown in Figure 2.11.

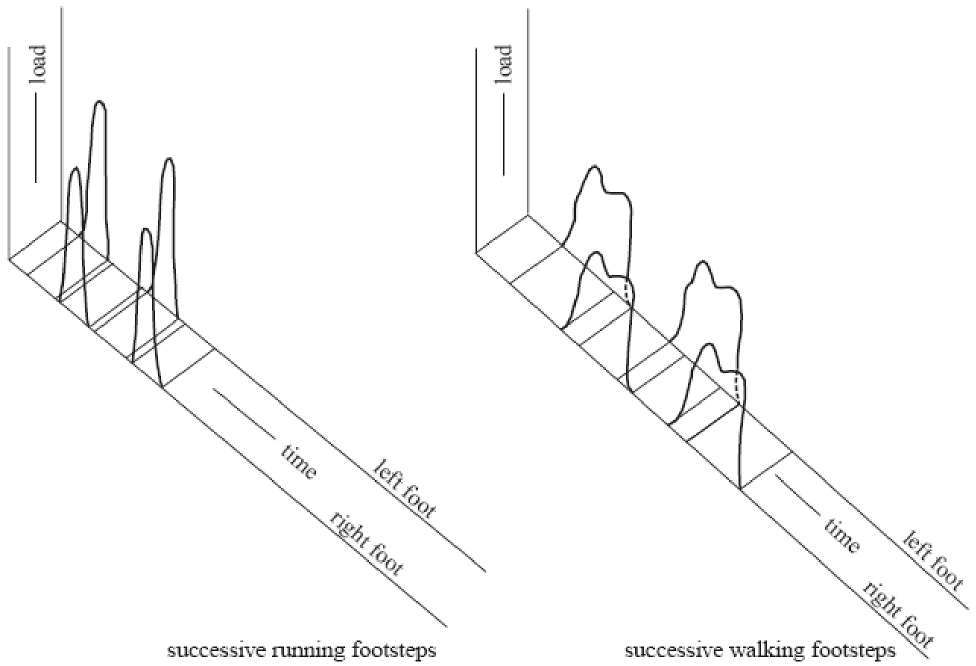

Figure 2.11 Patterns of running and walking forces

During normal walking the vertical forces are centred at a frequency in the range of 1.3 - 2.4 Hz, corresponding to the pace rate. For running the frequencies lie in the range 2 – 3.5 Hz. Normal walking follows a normal distribution with a mean value of 2.0 Hz and a standard deviation of 0.173 Hz, according to a research of Matsumoto (1978).

2.2.1.2 Horizontal components

The horizontal force components are considerably lower than the vertical force. The pattern of these forces are shown in Figure 2.12 (walking) and Figure 2.13 (running).

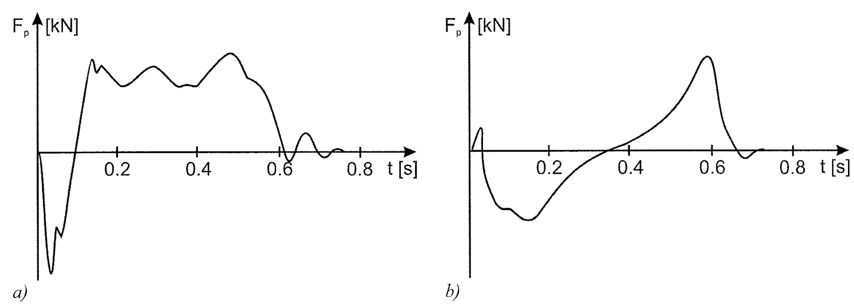

Figure 2.12 Horizontal forces from walking: a) lateral, b) longitudinal

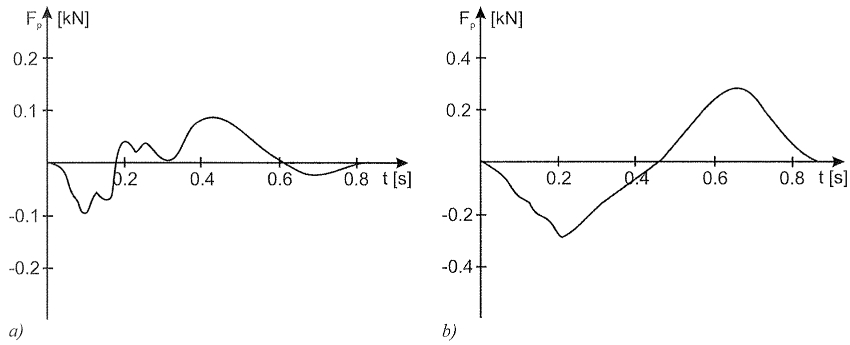

Figure 2.13 Horizontal forces from running: a) lateral, b) longitudinal

Horizontal forces are periodic with half of the walking frequency, with a mean value of 1 Hz, as the force changes of direction by each step. The longitudinal force is characterised by the retarding and the pushing walking period. The lateral force is caused by the lateral oscillation of the body. Runners seem to have a larger lateral stability so that the lateral force is smaller as during walking.

2.2.2 Interaction between pedestrians and footbridges

It is well known that there is a certain degree of interaction between pedestrians and the structure of footbridges. Two phenomena can be distinguished. One concerns the change of properties of the footbridge when humans are using the bridge. The other phenomenon is the synchronisation of the walking pattern between pedestrians and synchronisation of humans with the structure, under certain circumstances.

2.2.2.1 Dynamic properties of footbridges under human-induced loads

A change in dynamic properties is the more likely to happen to light structures where human loading can have significant impact on the structure compared to a non loaded structure: the mass and the damping can increase and thus this can have effect on the natural frequency of the footbridge, as seen in paragraph 2.2.1.

2.2.2.2 Synchronisation between pedestrians

Synchronisation between pedestrians is mainly dependant on the pedestrian density on a footbridge. Figure 2.14 shows different density situations. At low densities pedestrians are free to walk without obstacles (other pedestrians). When the path becomes denser, pedestrians are less free to choose their pace and adjust to the surrounding. The first restrictions occur at about 0.6 person per m². At this stage passing becomes more difficult. At a density of 1.0 person per m² the freedom of movement is greatly inhibited. When the density reaches about 1.5 persons per m², walking has become very difficult and pedestrians are greatly dependant on other users of the bridge. The velocity of the pedestrians decreases as the density increases.

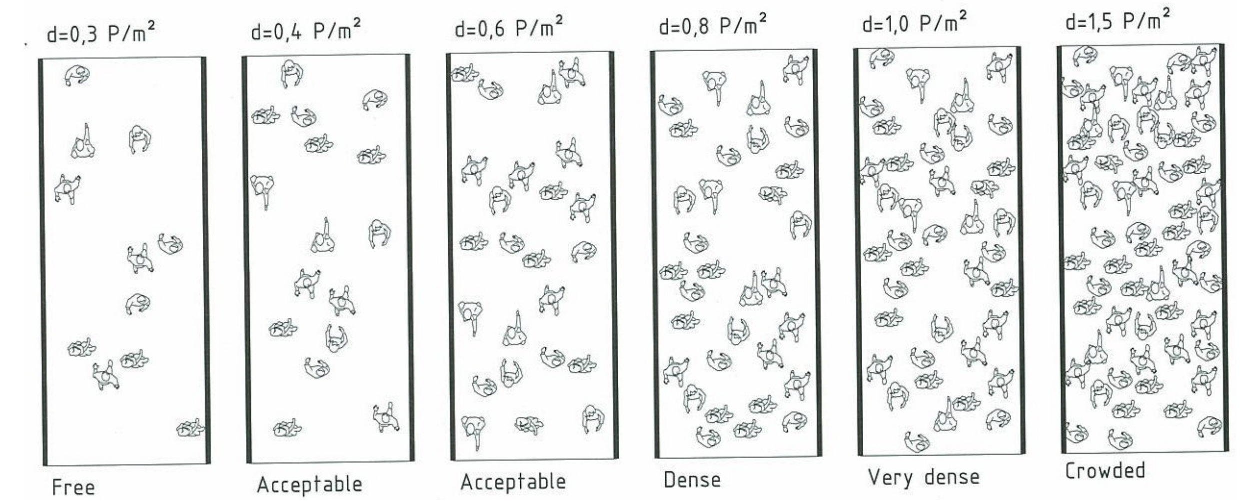

Figure 2.14 Different types of pedestrian densities

Hence it becomes clear that synchronisation of pedestrians is more likely to occur at higher densities, when people are not able to walk freely and are dependant on other pedestrians. The density of pedestrians also influences their velocity and consequently their dynamic forces on the bridge.
Synchronisation between runners is less likely to occur, as the velocity is quite high and thus the density is lower.

2.2.2.3 Synchronisation between bridge and pedestrians

Synchronisation between pedestrians and the structure is called Lock-in. It expresses the phenomenon by which a pedestrian crowd, with frequencies randomly distributed around an average value and with random phase shifts, will gradually coordinate at common frequency of the footbridge and enters in phase with the footbridge.

Lock-in in transverse direction is the most likely to occur and is also known as Synchronous Lateral Excitation (SLE). This is due to the fact that pedestrians are much more sensitive to lateral vibrations than to vertical vibrations (more information in paragraph 2.3). A good example of a bridge on which such a phenomenon occurred is the Millennium Bridge in London, which vibrated severally laterally during opening in June 2000 when hundreds of people were walking over it. The maximum density has been estimated on between 1.3 and 1.5 persons per m².

Dallard et al. investigated the Millennium Bridge and concluded that during SLE the produced dynamic force by the pedestrians was proportional to the lateral velocity of the deck: $F_L(t) = kv_L(t)$. By investigating more bridges, he found out that SLE could occur to any bridges with a lateral frequency of 1.3 Hz under the condition that sufficient number of people would cross the bridge at the same time.

2.2.3 Modelling of pedestrian loads

This paragraph illustrates the basics of modelling pedestrian loads. Load models for walking differ from the ones for running, as seen in paragraph 2.2.1 and illustrated in Figure 2.15.

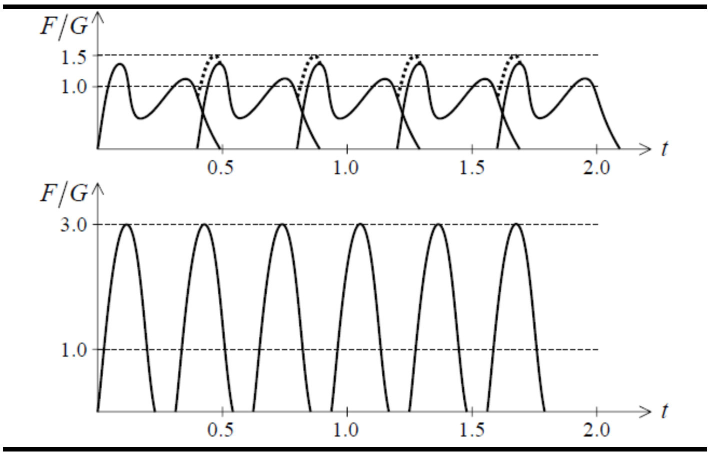

Figure 2.15 Loadings due to walking (up) and running (down)

The walking load is much more complex than the running load. This paragraph focuses on walking pedestrian loads. One can differentiate two types of models: the time domain model and the frequency domain model.

Time domain models consist of modelling waveforms and determine the attenuation model that best fits the real load pattern.

For the frequency domain model, one solves the spectrum of the load waveform that best fits the attenuation model. This research only focuses on the time domain model, as it is the most commonly used for this type of research so far. The basic models presented in this paragraph come from existing literature.

2.2.3.1 Vertical load model of walking pedestrian

Vertical forces due to human footsteps can be divided into different sinusoidal oscillations by a Fourier transformation:

$$F(t) = F_0 + \sum_i F_i \sin\left(2\pi i f_s t - \varphi_i\right)$$

where:

F_0	Mean or static load (person's weight)
F_i	Load component for frequency $i \cdot f_s$
f_s	Step frequency
φ_i	Phase angle of load component F_i

Usually the first three harmonics are taken into account, as the fourth harmonic and higher do not have much influence anymore on the pattern of the load model. The ratio of force amplitude to the static load is defined as Dynamic Load Factor (DLF). Different results of investigations done by Bachmann among others show that the values of the DLF's scatter greatly. This is mainly due to uncertainties during the measurements, but also to the fact that there are differences between persons. This also applies for the phase angles. Note that these DLF's have been determined by walking on a rigid floor.

Bachmann's coefficients are mostly used. For walking at a frequency of 2 Hz, the following force amplitudes and phase angles are mainly used:

$F_1 = 0.4\ F_0$ (DLF = 0.4)

$F_2 = F_3 \approx 0.1\ F_0$ (DLF = 0.1)

$\varphi_2 = \varphi_3 \approx \pi/2$

This leads to the following formula for vertical load:

$$F(t) = F_0 + 0.4F_0 \sin(2\pi f_s t) + 0.1F_0 \sin(4\pi f_s t - \pi/2) + 0.1F_0 \sin(6\pi f_s t - \pi/2)$$

The numerical approach and the real force as described by Bachmann have been plotted in Figure 2.16, together with the real load model.

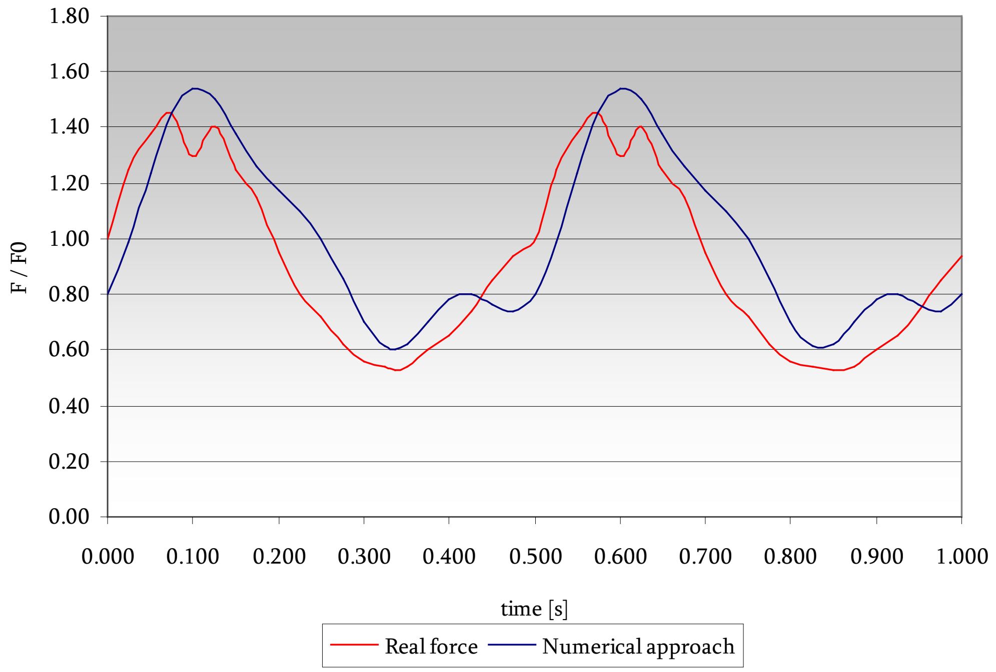

Figure 2.16 Representation of real force (according to Bachmann) and Numerical approach of force, resulting from walking (f = 2 Hz)

2.2.3.2 Horizontal load model of walking pedestrian

Even if the horizontal forces caused by a pedestrian are much smaller than the forces in vertical direction, attention has to be paid to it. It can in some situations be one of the sources of serious problems, like the ones occurred on the Millennium Bridge. The longitudinal forces do not have much effect on the vibration of footbridges and will therefore not be further treated. The forces in lateral direction however will be.

Horizontal load can be represented by Fourier's transformation, like the vertical load. The frequency of the lateral load is half of the vertical load, corresponding to the lateral oscillation of the centre of gravity of the body. In order to be able to represent the load according to the vertical frequency, the solution generally used is to modify the presentation in the following form:

$$F(t) = \sum_{i=1/2}^{n} F_i \sin(2\pi i f_s t)$$

where i has the (non-whole) values of 1/2, 1, 3/2, 2, etc. The phase shifts are close to 0 and therefore do not appear in the expression. For walking at a frequency of 2 Hz, the following force amplitudes are mainly used:

$F_{1/2} = F_{3/2} \approx 0.05\ F_0$ (DLF = 0.05)

$F_1 = F_1 \approx 0.0.1\ F_0$ (DLF = 0.01)

This leads to the following formula for vertical load (sum of the 1st to the 4th harmonic):

$$F(t) = 0.05F_0 \sin(2\pi \cdot \tfrac{1}{2} \cdot f_s t) + 0.01F_0 \sin(2\pi \cdot 1 \cdot f_s t) + 0.05F_0 \sin(2\pi \cdot \tfrac{3}{2} \cdot f_s t) +$$
$$+ 0.01F_0 \sin(2\pi \cdot 2 \cdot f_s t)$$

The representation of the real force and the numerical approaches are given in Figure 2.17.

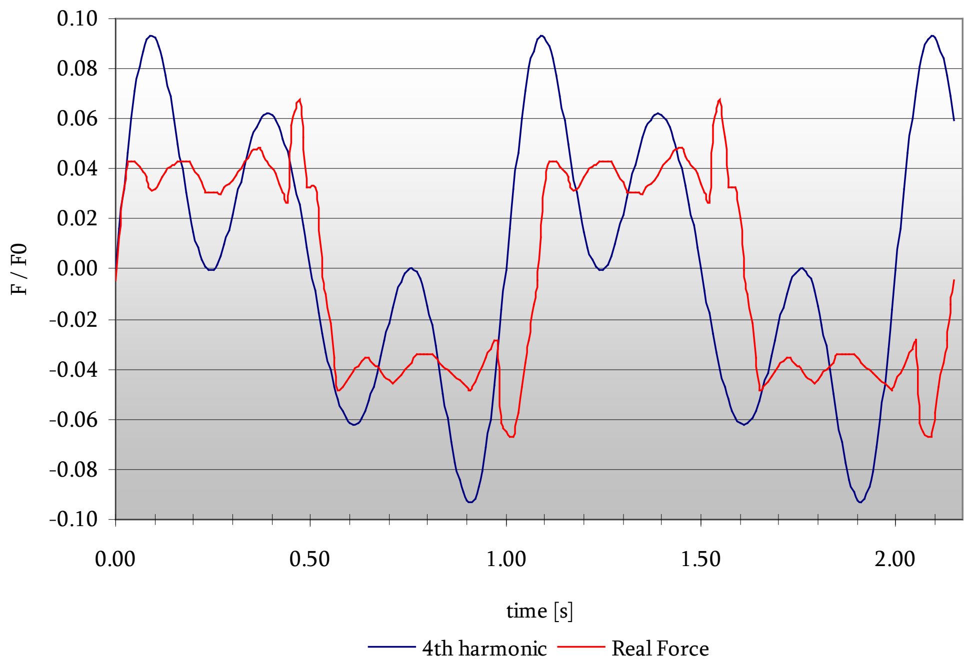

Figure 2.17 Representation of real force and Numerical approach of lateral force, resulting from walking (f_s = 2 Hz)

2.2.3.3 Load models for group of pedestrians

A footbridge is rarely submitted to forces due to one pedestrian only. Groups of pedestrians are much more common. Each person has its own characteristics (weight, speed, frequency, initial phase shift, location on the bridge) which make the system much more complicated. Moreover, dependant of the number of pedestrians on a bridge, people tend to walk more or less synchronous with each other (see paragraph 2.2.2), during which the natural pace of pedestrian changes. This behaviour is nearly impossible to model correctly.

According to the statistics, the load can be increased with the square of the number of pedestrian on the bridge: $\sqrt{N}$. This means that a number of $\sqrt{N}$ are walking synchronously. However, this is only the case if initially none of the motions on the bridge are synchronous. In reality a (small) part of the crowd will be synchronised.

Different codes and guidelines have set up load models for group of pedestrians. These models can be found in Chapter 4 of this report.

2.3 Sensitivity of pedestrians to vibrations of footbridges

Both vertical and horizontal vibrations can be perceived as a disturbing effect during the stay on the bridge and can therefore considerably influence the serviceability limit state of the bridge. Perception of vibrations is a rather complicated topic, as it has many influencing factors where human psychology plays an important role. Each person senses a vibration differently, but this is also dependant of the moment when the vibrations are perceived, the eventual sounds from the structure or even the height above the ground. Pedestrians can also get used to vibrations over the time and acceptance regarding to vibrations can rise.

It is important to distinguish the sensitivity in horizontal direction and the one in vertical direction. Pedestrians seem to be much more sensible to horizontal vibrations than to vertical ones. Therefore these two cases are being explained separately.

2.3.1 Sensitivity to vertical vibrations

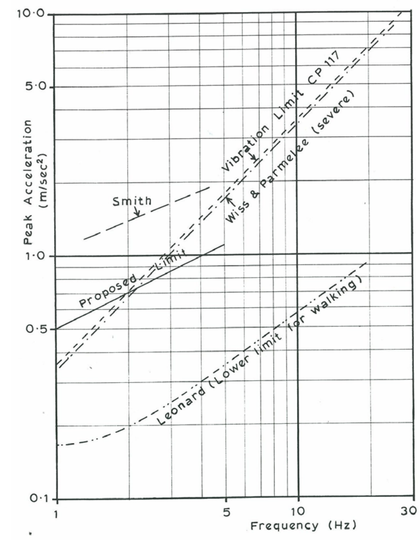

Figure 2.18 Tolerance thresholds and vibration limits

Experiments have shown that pedestrians are more sensitive to vibration when standing still than when walking. Leonard[2] obtained with the help of laboratory experiments a lower bound of comfort threshold specifically for pedestrians on bridges, shown in Figure 2.18. As can be seen, the comfort criteria are being expressed in accelerations of the vibrations. The acceleration, the speed and the displacement of the vibrations are closely related with each other through the dynamics formula stated in paragraph 2.1.1. One will therefore find the comfort criteria according to one of these factors.

[2] D.R. Leonard, "Human tolerance levels for bridge vibrations", Ministry of Transport RRL Report No 34, Road Research Laboratory, Harmondsworth, 1966.

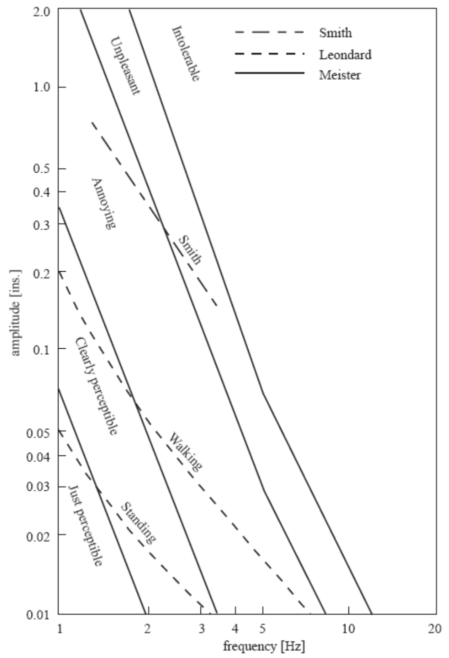

Figure 2.19 Human perception to vertical vibrations

Smith[3] investigated large amplitude vibrations which affected the normal walking considerably and found an upper bound of pedestrian tolerance (also shown in Figure 2.18). A proposed limit was set approximately midway between the Leonard and Smith curves, which has been used for the British Standard BS 5400.

In Figure 2.19, it becomes clear that a standing person is more sensible to vibrations than a walking person. However, this does not imply that standing persons should not feel any vibrations of passing pedestrians. This could lead to an uneconomical design of the bridge. The Smith curve is also represented in the graph.

Lateral vibrations are especially sensible by pedestrians when the natural frequency of the bridge is higher than 4 Hz.

2.3.2 Sensitivity to horizontal vibrations

Pedestrians are much more sensitive in horizontal direction (especially lateral) than in vertical direction. This is due to the fact that a person is more easily brought off balance with horizontal oscillations than with vertical ones. However, data on human perception of horizontal vibrations of bridges are scarce. Like for the vertical comfort criteria, the acceleration can be used to define a limit for the horizontal vibrations.

Pedestrian tend to react differently in crowd situations than in normal situations. In crowd situations walking pedestrians have a lower standard for the lateral vibrations. Higher vibrations could then be permitted.

Lateral vibrations are especially sensible by pedestrians when the lateral natural frequency of the bridge is below 2.5 Hz. Accelerations of about 0.3 m/s² are clearly perceptible by pedestrians and can influence their walking behaviour.

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[3] J.W. Smith, “The vibration of highway bridges and the effects on human comfort”, Ph.D. Thesis, University of Bristol, September 1969.

3 Review of the analysed footbridges

This chapter gives a short structural overview of the essential parts of the bridges that are being analysed for this thesis. For some more detailed information about the bridges, refer to Appendix 1.

3.1 Goodwill Bridge

3.1.1 General information

The Goodwill Bridge is a bridge for pedestrians and cyclists that spans over the Brisbane River. It links the southern part of the Central Business District (CBD) of Brisbane with South Bank which offers many public attractions. It has been opened in 2001 and is nowadays weekly used by approximately 40,000 people. The situation is presented in Figure 3.1.

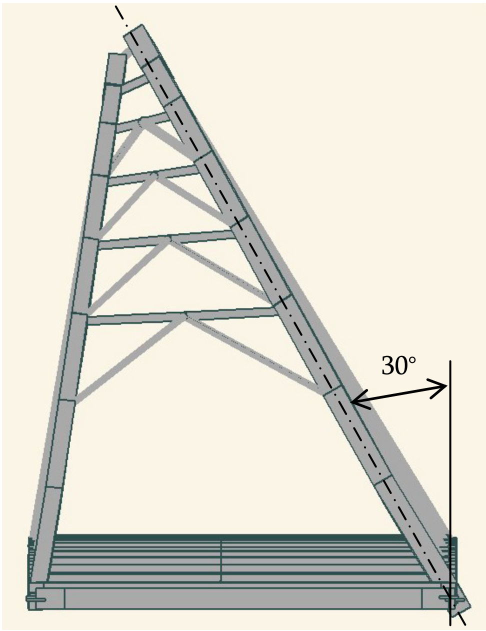
Figure 3.1 View on the Goodwill Bridge, Brisbane

The 450 meter long footbridge has three distinct parts: the Rampart on the South Bank riverside, the Arch as main span over the river and the Pier on the CBD riverside. The Arch spans 102 meter and provides 13 metres clear height above the water level and is the part that is being studied in this research as it is the most susceptible to vibrate under pedestrian loads.

3.1.2 Structural information

The Arch consists of a dual arch frame. The two arches have the particularity that they are not placed on the same angle: the upstream arch is nearly vertical whereas the downstream arch is inclined under an angle of 30 degrees. This situation is shown in Figure 3.2. The two arches nearly join each other on half the span, as can be seen in Figure 3.3. Trapezoidal sections fabricated from plates have been used for this purpose. The thickness of the plates varies over the length of the bridge. The arches arise 14 meters above the 6.7 meter wide deck. The hangers have a spare web arrangement of circular hollow sections varying in diameter between 114 and 406 mm. The main chords are shaped boxes which house post-tensioned tendons: the upstream tie has been prestressed to approximately 9500 kN, whereas the downstream tie only to 4500 kN.

Upstream side



Downstream side

Figure 3.2 Side View of the model of the Goodwill Bridge without concrete deck (GSA)

The concrete deck is supported by the cross beams which are supported by the chords. Its centre line corresponds to the centre of gravity of the steel structure, which lies at 4.45 m from the upstream chord (the total width of the steel structure is 9.95 m). On the downstream side, in the middle of the span, a small rest area has been added next to the concrete deck.

The cross beams between the chords are equally spaced over the length of the bridge, with a space of 2.5 m between each one of them. These beams are I-shaped (called Universal Beams). The cross beams at the edges of the bridges are fabricated box sections which are specifically made for the purpose of this bridge. These beams carry all loads to the foundation.

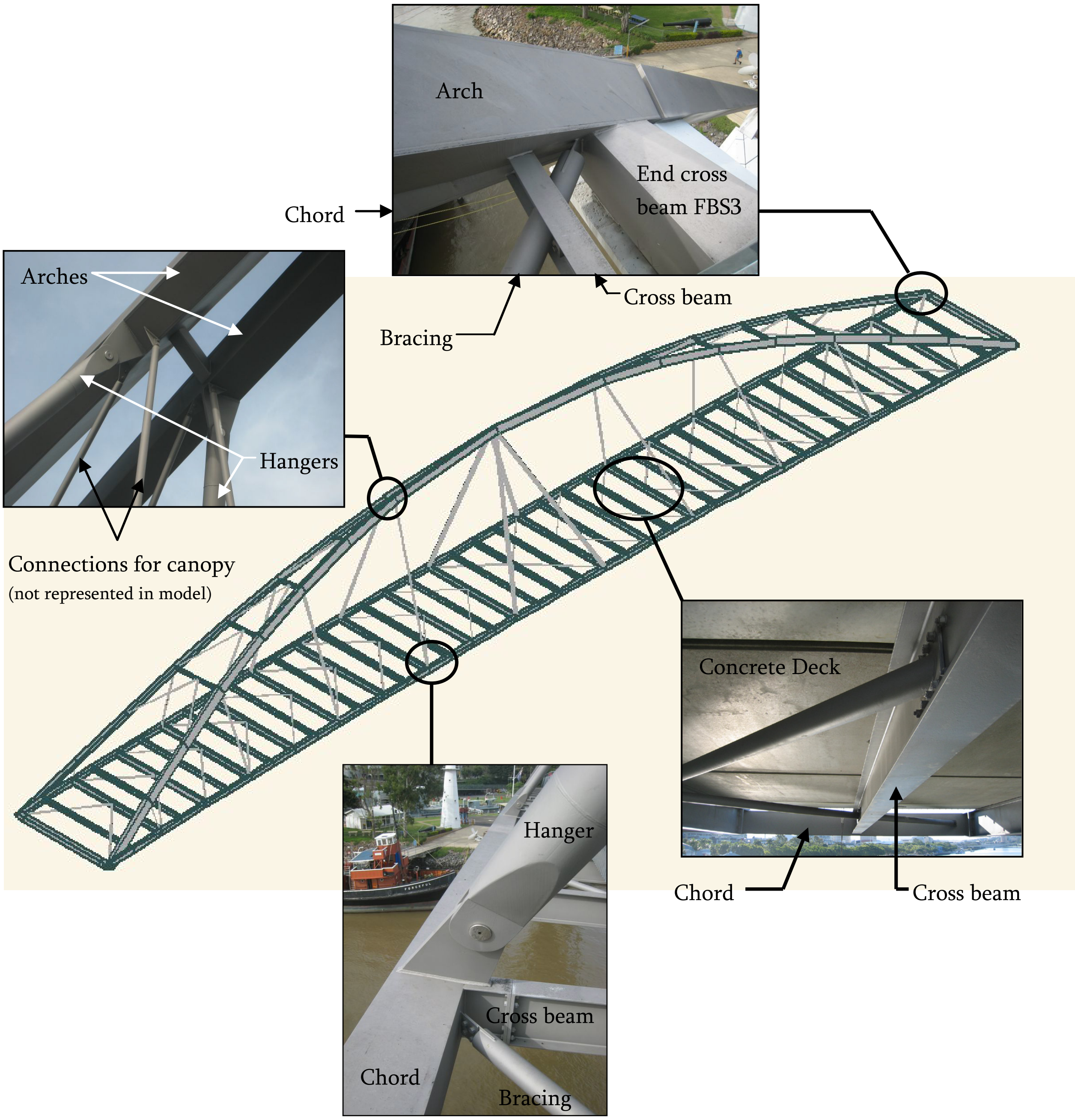


Figure 3.3 Model of Goodwill Bridge without concrete deck (GSA) with clarifying pictures

Figure 3.3 shows how the different elements are connected. Most of the connections are pin joints. Note that the canopy and its connections to the main structures are not represented in the model.

On the Pier side, the Arch is supported by the Pavilion. The Pavilion is the cable-stayed part of the bridge, above the one of the foundations. The situation can be seen in Figure 3.4.

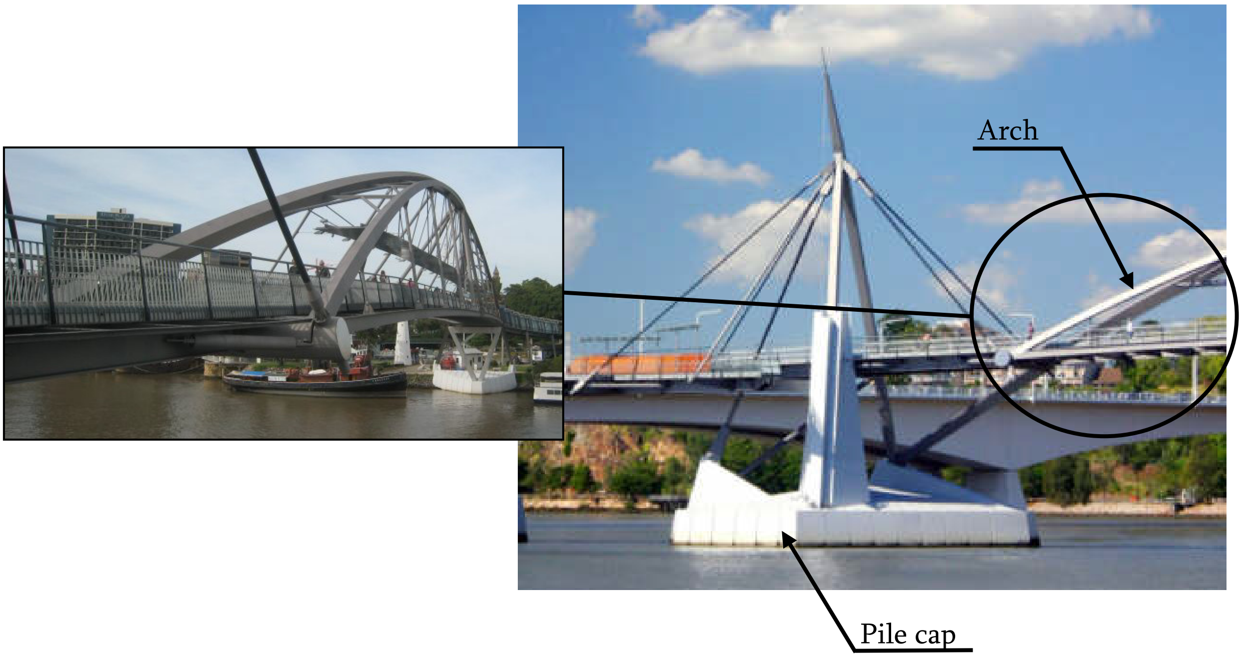


Figure 3.4 Support of the Arch on the Pier side

A set of piles is covered by the concrete pile cap in the river. The piles are founded on the rock layer. The concrete pile cap has been design such that the piles remain under water. The foundation of the bridge has been represented in Figure 3.5.

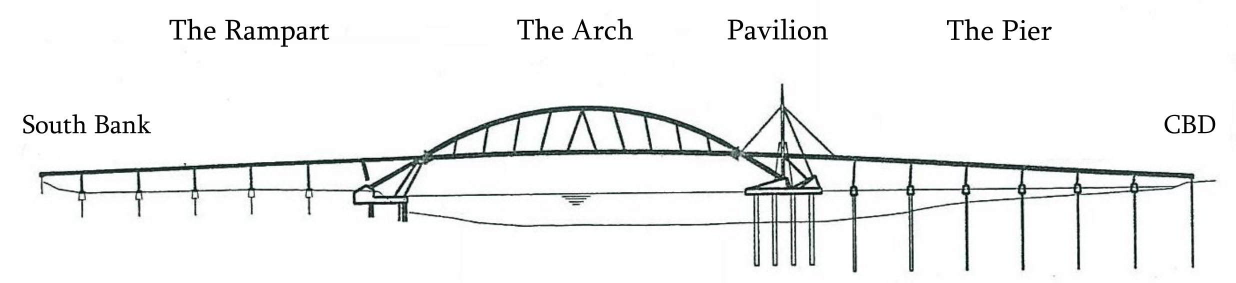


Figure 3.5 Representation of the foundation of the Goodwill Bridge

Appendix 1.1 presents more details (pictures and technical drawings) of the structure of the Goodwill Bridge.

3.1.3 Vibrations

The Arch of the Goodwill Bridge is a slender structure and is the most susceptible to vibrate under human loads. Under normal circumstances, small vibrations can be perceived when standing still on the bridge. These vibrations cannot be felt when walking on the bridge. Users of the bridge

However somewhat higher vibrations have been noticed while large groups of joggers used the bridge during a running competition on the Arch. These vibrations could be felt by people standing still and in some cases by people walking or jogging on the bridge, but were never perceived as annoying. That was the reason for Arup to measure these vibrations during another running competition. Measurements of the vertical acceleration have been made on three strategic spots on the deck of the bridges. These measurements were also made before the competition during a normal use of the bridge.

No clear perceptible vibrations have been felt on the Rampart or the Pier so far. The Arch is therefore the only part of the bridge that is being further analysed in this report.

3.2 Milton Road Bridge

3.2.1 General information

The Milton Road Bridge has been built within the redevelopment project of the 52,500 seats Suncorp Stadium. The redevelopment also involved the construction of substantial new transport infrastructure to encourage the majority of spectators to access the stadium by public transport. The Milton Road Bridge links the Suncorp Stadium with the Milton Train Station. The situation is represented in Figure 3.6.

Figure 3.6 General overview of the Suncorp Stadium and the Milton Road Bridge

The bridge is nearly only used by people going to or coming from the stadium. It is therefore mostly used by larger groups of pedestrians for relative short moments. The 86 meter long and 8.45 meter wide bridge spans over the Milton Road which is known as a busy road from and to the Central Business District of Brisbane. The bridge spans about six meters above the road.

3.2.2 Structural information

The Milton Road Bridge is a truss bridge divided into two main spans of about the same length. The bridge consists of a concrete deck and the rest are steel elements. A general view of the bridge is represented in Figure 3.7.

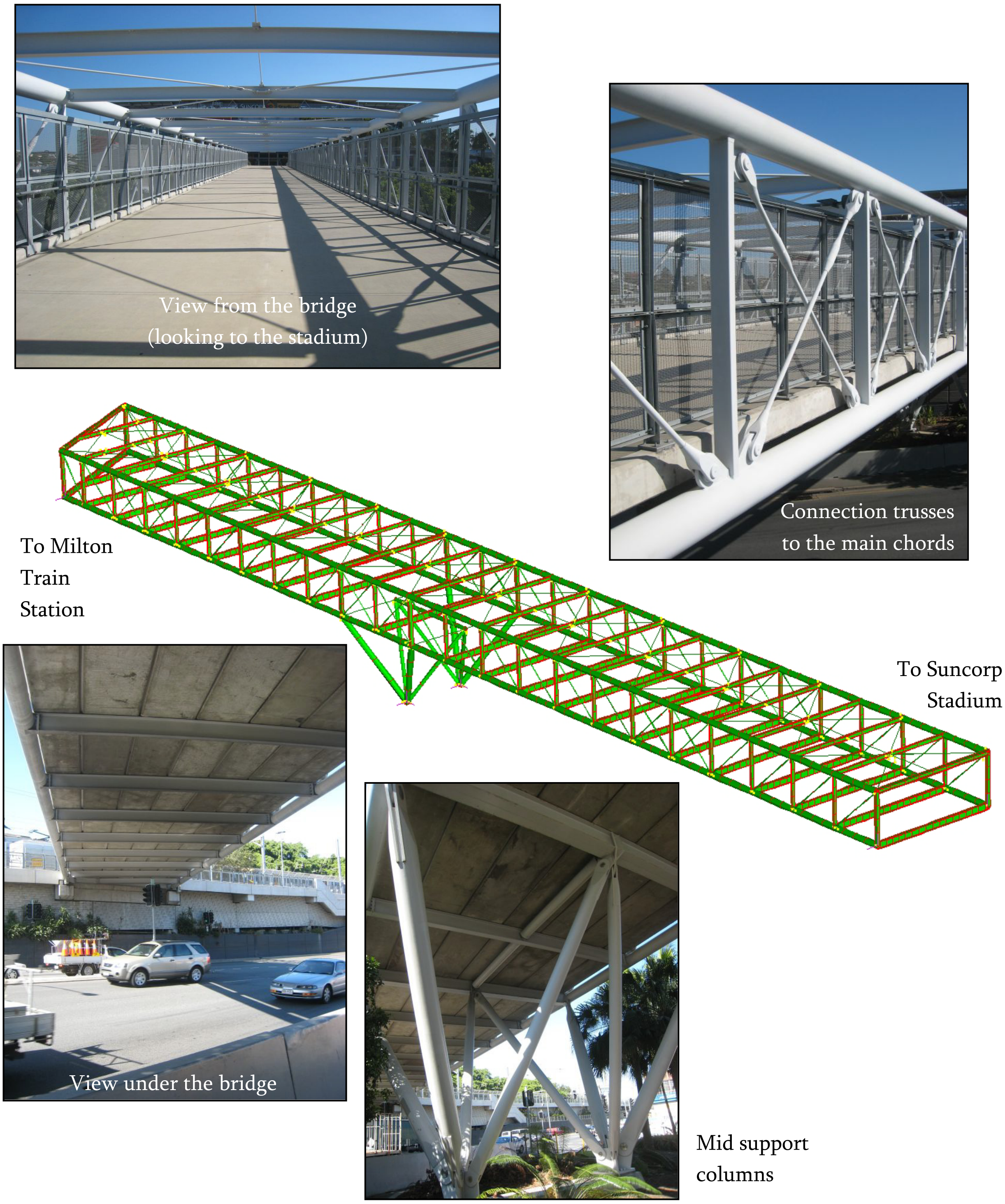


Figure 3.7 Model of the Milton Road Bridge (without concrete deck – in Strand7) with clarifying pictures

Most of the elements are welded except for the bracings and the mid support columns which are connected to the main structures with pin joints. The concrete deck has a varying thickness of 125 to 175 mm. Most of the steel elements consist of Circular Hollow Sections and Rectangular Hollow Sections which are welded together. The cross beams consist of Universal Beams. The upper cross beams have been placed in the top to accommodate the wind bracings. The bridge is supported on three different places: at each end and at the mid support.

3.2.3 Vibrations

Vibrations have been noticed on the bridge by people standing still. However, these have never been perceived as annoying. It does not influence the behaviour of the bridge users. The particularity of this bridge is that vibrations are best perceived when one or two pedestrians are crossing the bridge.

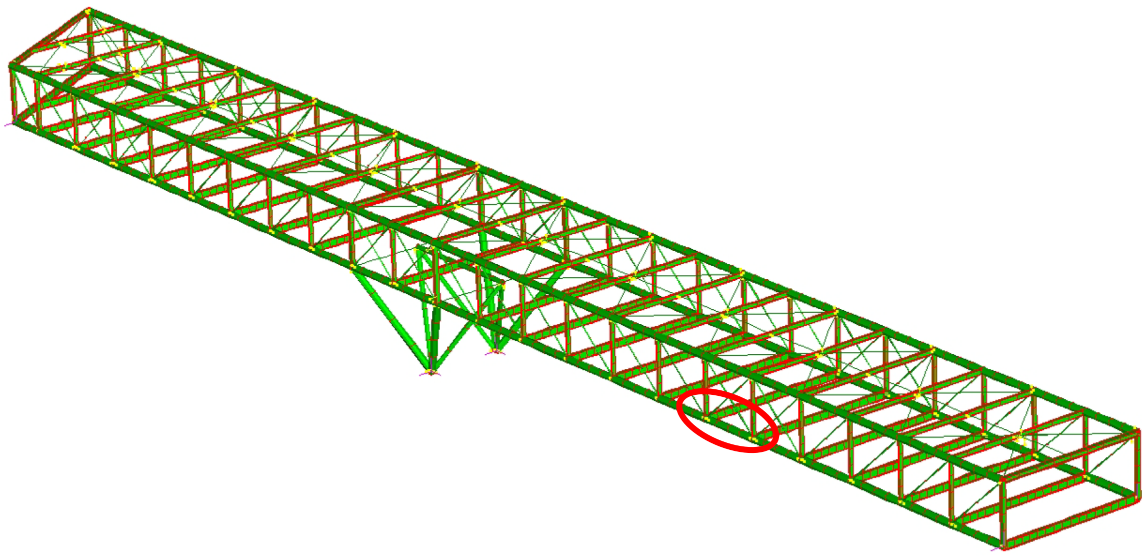


Figure 3.8 Area on the bridge where the vibrations can best be felt

Figure 3.8 shows the place where these vibrations are best perceived when standing still. Unlike on the Goodwill Bridge, there is under certain circumstances no particular reason to stop at this place or at any other place on the bridge. This bridge is essentially used by larger crowds between the stadium and the train station. In these cases perceptible vibrations have never been felt.

4 Review of the codes

As can be understood from the former chapters, the dynamic behaviour of footbridges can be a critical design parameter. More and more codes are trying to give designers guidance in this subject. The dynamic phenomenon is however relatively complex and yet not well known. The codes can therefore differ from each other in methodology. This chapter reviews the existing codes in Europe and Australia.
The Eurocode is presented in the first paragraph. Some parts of the Eurocode have been left to the National Annexes. The second paragraph deals about a proposed Annex in 2001 and the third paragraph about the UK National Annex. The Australian Standard will be covered in the fourth paragraph. The last paragraph will shortly compare the different methodologies and requirements.

4.1 Eurocode

The Eurocode is a set of building codes developed by the European Committee for Standardisation. However the National Standards of each European country still prevails at the moment of writing this paper, the Eurocode can be considered as the building standard for Europe. Three parts of the Eurocode deal about pedestrian loads or structural requirements:

- Eurocode 0 (EN 1990:2002 “Basis of Structural Design”): Annex A2.4.3.2 gives the comfort criteria for pedestrians;
- Eurocode 1 (EN 1991-2:2003 “Actions on Structures”): paragraph 5.7 deals with pedestrian loads on bridges;
- Eurocode 3 (EN 1993-2:2006 “Design of Steel Structures”): paragraph 7.9 deals with the performance criteria for pedestrian bridges.

4.1.1 Eurocode 0

Eurocode 0 defines the comfort criteria. As mentioned earlier in paragraph 2.3 of this report, pedestrians are sensitive to vibrations, more in horizontal direction than in vertical direction. The Eurocode therefore state a limit to these vibrations. Eurocode 0 also states when a dynamic analysis should be performed.

Eurocode 0 states that the comfort criteria should be defined in terms of maximum acceptable acceleration. The amplitude of the vibrations are directly related to the acceleration. Even though the acceptable acceleration criteria can be defined by the national annexes, some recommended maximum values are given. These values are given in Table 4.1.

Table 4.1 Recommended maximum values of the acceleration of any part of the deck according to EN 1990:2002 Annex A2.

Vertical vibrations	0.7 m/s²
Horizontal vibrations (normal use)	0.2 m/s²
Horizontal vibrations (crowd conditions)	0.4 m/s²

A distinction has been made between normal use and crowd conditions for the horizontal vibrations. As mentioned in paragraph 2.3, the perception of pedestrians of crowd situations is significantly different than in normal circumstances: less attention is paid to vibrations and higher vibrations are accepted.

Annex A2.4.3.2 also states when a verification of the comfort criteria should be performed. Table 4.2 shows at which fundamental frequencies of the deck the comfort criteria should be checked.

Table 4.2 Fundamental frequencies of the deck at which a verification of the comfort criteria should be assessed, according to EN 1990:2002 Annex A2.

Vertical vibrations	< 5 Hz
Horizontal and torsional vibrations	< 2.5 Hz

4.1.2 Eurocode 1

Paragraph 5.7 of Eurocode 1 mentions three points of attention:

- Depending on the dynamic characteristics of the structure, the relevant natural frequencies (corresponding to vertical, horizontal, torsional vibrations) of the main structure of the bridge deck should be determined from an appropriate structural model.
- Forces exerted by pedestrians with a frequency identical to one of the natural frequencies of the bridge can result into resonance and need to be taken into account for limit state verifications in relation with vibrations.
- Appropriate dynamic models of pedestrian loads and comfort criteria should be defined.

The interesting part of the code is that the dynamic models which should be applied are not given. It has been left over to the National Annexes. This is caused by the complexity of the topic and the fact that a lot of research is yet still being made in this field..

4.1.3 Eurocode 3

Paragraph 7.9 mentions: For footbridges and cycle bridges with excessive vibrations could cause discomfort to users, measures should be taken to minimise such vibrations by designing the bridge with appropriate natural frequency or by providing suitable damping devices.

4.2 Proposal Annex C for Eurocode 1

In 2001 a proposal annex has been made for the Eurocode. This Annex has never been officially approved, but has been used as guidance for designers. After this proposal, the regulations regarding vibrations of footbridges have been left over to the National Annexes. This annex (called Annex C) gives guidance in:

- the way to determine the natural frequencies and the structural damping;
- dynamic load models which should be applied.

4.2.1 Assessment of natural frequencies and structural damping

The assessment of the particular vertical and horizontal frequency should take into account the mass of any permanent load. The mass of pedestrians should be taken into account only for very light decks and where unfavourable.
Some recommended values of the damping ratio for fundamental loads are given (see Table 4.3). Note that δ is the logarithmic decrement due to structural damping. The damping ratio is $\zeta = \dfrac{\delta}{2\pi}$.

Table 4.3 Recommended values of the damping ratio for fundamental modes (Proposal Annex C)

Material of construction	**δ**
Steel	0.03
Steel and concrete composite	0.04
Concrete	0.05
Timber	0.06 to 0.12
Aluminium alloy	0.02
Glass or Fibre Reinforced Plastic	0.04 to 0.08

4.2.2 Load models

Annex C proposes three load models due to normal walk of pedestrians.

4.2.2.1 Dynamic load model of a single pedestrian (DLM 1)

DLM 1 consists of one pulsating force with two components:
a vertical component (N) : $Q_{pv} = 280\sin(2\pi f_v t)$
a horizontal component (N) : $Q_{ph} = 70\sin(2\pi f_h t)$

where:

f_v is the natural vertical frequency of the bridge, that is the closest to 2 Hz;
f_h is the natural horizontal frequency of the bridge, that is the closest to 1 Hz.

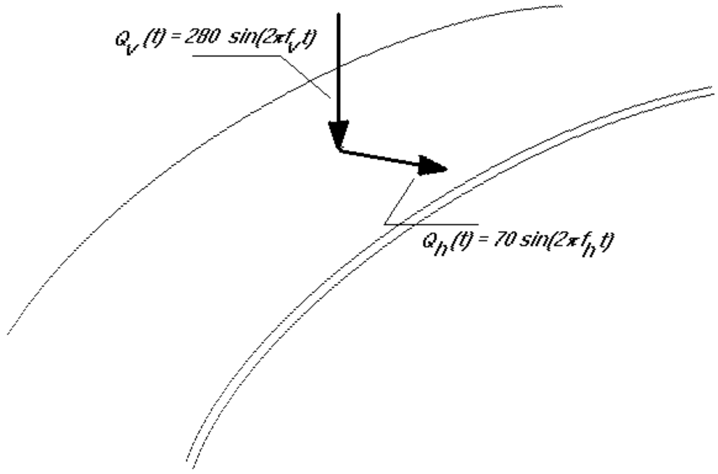


Figure 4.1 Representation of dynamic model of a single pedestrian (DLM 1) according to Annex C

DLM1 corresponds to the action of a pedestrian of 700 N weight and 0.9 f_v velocity. The part of the action of the pedestrian on the vertical harmonic mode under consideration is assumed as 40% of his weight (280 = 700 × 0.4). This corresponds with the first harmonic of the numerical approach of the load, as given in paragraph 2.2.3. The horizontal component corresponds to: $F(t) = 0.1F_0 \sin(2\pi \cdot \frac{1}{2} \cdot f_v t)$, which is slightly higher than the first harmonic of the numerical approach given in paragraph 2.2.3.
This load model should be applied at the most unfavourable location on the footbridge deck.

4.2.2.2 Dynamic load model of a group of pedestrians (DLM 2)

DLM2 consists in one pulsating force with two components to be taken into account separately:
for vertical vibrations (N) : $Q_{gv} = 280\, k_v(f_v) \sin(2\pi f_v t)$
for horizontal vibrations (N) : $Q_{gh} = 70\, k_h(f_h) \sin(2\pi f_h t)$
where:
f_v is the natural vertical frequency of the bridge, that is the closest to 2 Hz;
f_h is the natural horizontal frequency of the bridge, that is the closest to 1 Hz;
$k_v(f_v)$ is given in Figure 4.3a;
$k_h(f_h)$ is given in Figure 4.3b.

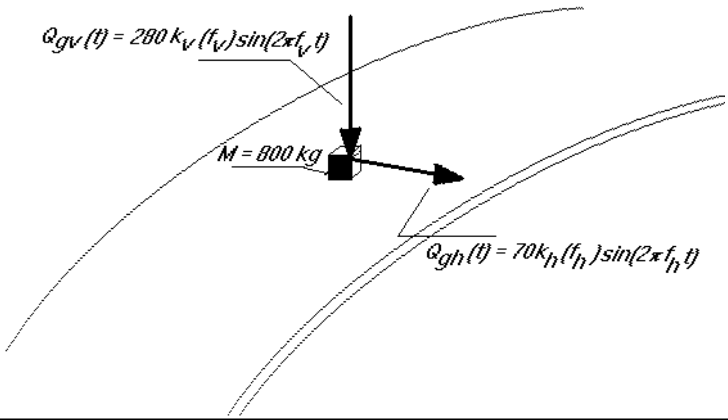


Figure 4.2 Representation of dynamic model of a group of pedestrians (DLM 2) according to Annex C

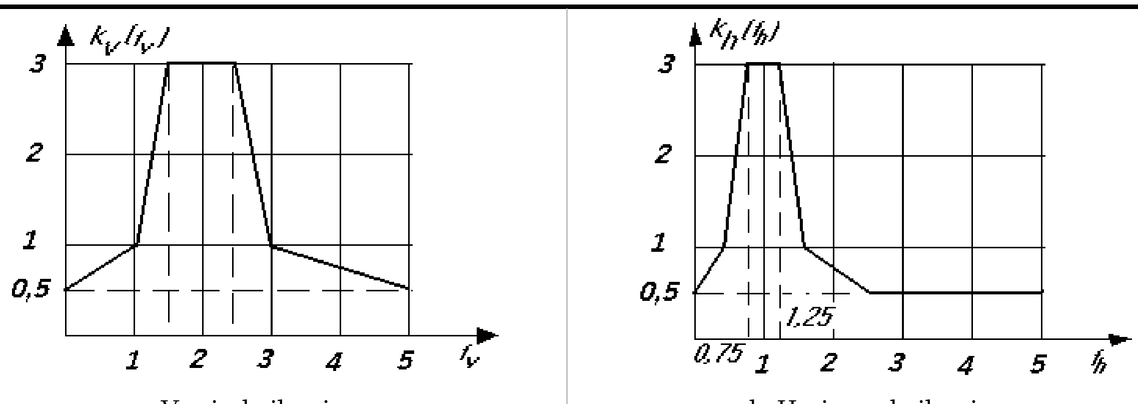


Figure 4.3 Relationships between kv(fv), kh(fh) and frequencies fv, fh

This load model should be systematically used for the verification of the required comfort criteria and should be placed in a fixed position at the most adverse location on the footbridge deck. The background of this load model is as follows:

For a small group of *N* pedestrians, the vertical acceleration derives from the vertical acceleration due to a single pedestrian in accordance with the following formula: $a_{vN} = a_{v1}\sqrt{N}$ (according to the theory of paragraph 2.2).

If $N = 10$, then $\sqrt{10} \approx 3$ which explains the relationships in Figure 4.3 when the natural vertical frequency of the footbridge is not far from 2 Hz. The same conclusions can be drawn for horizontal excitation, but in the vicinity of 1.0 Hz.

For the assessment of inertia effects (for the calculation of f_v *or* f_h), DLM2 should be associated with a static mass equal to 800 kg (if unfavourable), applied at the same location.

4.2.2.3 Dynamic load model of a continuous stream of pedestrians (DLM 3)

The dynamic load model of a continuous stream of pedestrians (DLM3) consists of a uniformly distributed pulsating load (Pa or N/m²) with two components to be taken into account separately:

for vertical vibrations : $q_{s,v} = 12{,}6\,k_v(f_v)\,\sin(2\pi\, f_v\, t)$

for horizontal vibrations : $q_{s,h} = 3{,}2\,k_h(f_h)\sin(2\pi\, f_h\, t)$

where:
f_v is the natural vertical frequency of the bridge, that is the closest to 2 Hz;
f_h is the natural horizontal frequency of the bridge, that is the closest to 1 Hz;
$k_v(f_v)$ is given in Figure 4.3a;
$k_h(f_h)$ is given in Figure 4.3b.

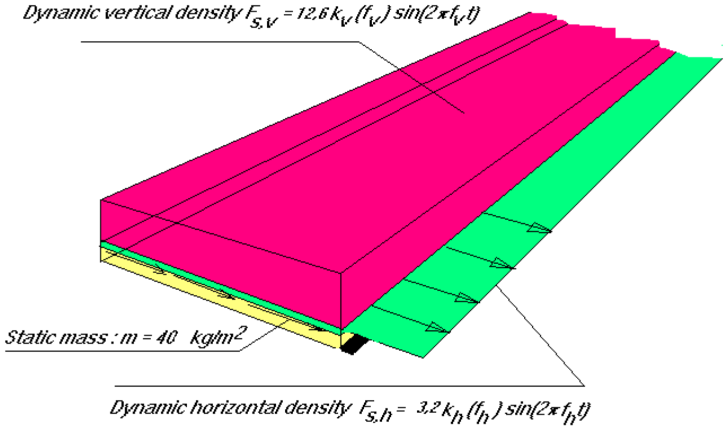


Figure 4.4 Representation of dynamic model of a continuous stream of pedestrians (DLM 3) according to Annex C

This Load model should be used separately from DLM2.
DLM3 should be applied on the relevant areas of the footbridge deck (*e.g.* span by span or on the half-wavelength of the mode of vibration considered), for the verification of the specified comfort criteria as well as for the assessment of inertia effects (in the calculation of f_v *and* f_h) in order to obtain the most unfavourable effect.
For the assessment of inertia effects (for the calculation of f_v *and* f_h), DLM3 should be associated with a static mass equal to 40 kg/m² (if unfavourable), applied at the same location.
This load model is based on the assumption that 0.6 persons/m² crosses simultaneously the bridge, which gives a total number of pedestrians equal to N = 0.6*B*L. DLM3 is relevant for $N < N_{crit.}$.

4.3 British National Annex for Eurocode 1 of EN 1991-2

The British National Annex dealing with dynamic models for pedestrian actions on footbridges is based on the research done by Barker[4] and MacKenzie[5]. The aim of the UK design rules mentioned in the National Annex is to provide sufficient guidance to take

[4] Barker C., DeNeumann S., MacKenzie D., Ko R., "Footbridge Pedestrian Vibration Limits – Part 1: Pedestrian Input", Footbridge 2005 International Conference
[5] MacKenzie D., Barker C., McFadyen N., Allison B., "Footbridge Pedestrian Vibration Limits – Part 2: Human Sensitivity", Footbridge 2005 International Conference

into account the effects of vibration of complicated structures and those in sensitive locations without imposing undue conservatism that might constrain designers in achieving and economic solution[6].

4.3.1 Assessment of natural frequencies and structural damping

The UK National Annex does not mention anything about the assessment of the natural frequencies and the structural damping. However, Barker and MacKenzie advise the following:
The designer is advised to explore the sensibility of the contribution of non-structural elements to investigate potential variation in structural response.
Typical values for the structural damping are given. These are the same values as mentioned for the Proposed Annex C (see Table 4.3).

4.3.2 General provisions

The National Annex assumes an upper limit for the vibration serviceability: if the vertical natural frequency of the unloaded bridge is exceeding 8Hz and if the horizontal frequency of the loaded bridge is exceeding 1.5 Hz, then the vibration serviceability is deemed to be satisfied. If that is not the case, the designer is required to assess the likely dynamic response of the structure.
Two distinct analyses are required:

- the determination of the maximum vertical deck acceleration and its comparison with the comfort criteria, and
- an analysis to determine the likelihood of large synchronized lateral responses.

Like the proposed Annex C, the adopted approach examines the effects of normal operating conditions only and thus, does not consider effects like mass gathering.

4.3.3 Load models

The UK National Annex classifies footbridges in 4 categories and considers the effects of groups both walking and jogging, as well as specific crowd densities (Table 4.4).

[6] Barker C, MacKenzie D, "Design Methodology for Pedestrian induced Footbridge Vibrations", Footbridge 2008 International Conference

Table 4.4 Bridge classification according to UK National Annex to EN1991-2

Bridge Class	Bridge usage	Group size (walking)	Group size (jogging)	Crowd density ρ (pers./m²)
A	Rural locations used and in sparsely populated areas	N = 2	N = 0	0
B	Sub-urban location likely to experience slight variations in pedestrian loading intensity on an occasional basis	N = 4	N = 1	0.4
C	Urban routes subject to significant variation in daily usage (e.g. structures serving access to offices or schools)	N = 8	N = 2	0.8
D	Primary access to major public assembly facilities such as sports stadiums or major public transportation services	N = 16	N = 4	1.5

The UK National Annex describes three main load models. These load models depend on the bridge classification as described before. Two of these load models concern vertical vibrations and one horizontal vibrations.

4.3.3.1 Dynamic actions representing the passage of single pedestrians and pedestrian groups (vertical)

The design maximum vertical accelerations that result from single pedestrians or pedestrians groups should be calculated by assuming that these are represented by the application of a vertical pulsating force F(N), moving across the span of the bridge at a constant speed v_t, as follows:

$$F = F_0 k(f_v)\sqrt{1+\gamma(N-1)}\sin(2\pi f_v t)$$

Where:

N is the number of pedestrians in the group

F_0 is the reference amplitude of the applied fluctuating force (N) given in Table 4.5 (and represents the maximum amplitude of the applied pedestrian force at the most likely pace frequency)

f_v is the natural frequency (Hz) of the vertical mode under consideration

$k(f_v)$ is a combined factor to deal with (a) the effects of a more realistic pedestrian population, (b) harmonic responses and (c) relative weighting of pedestrian sensibility to response, see Figure 4.5.

γ is a factor to allow for the unsynchronised combination of actions in a pedestrian group, is a function of damping and effective span, and is obtained from Figure 4.6.

S_{eff} is an effective span length (m) equal to the area enclosed by the vertical component of the mode shape of interest divided by 0.634 times the maximum of the vertical component of the same mode shape, see Figure 4.7.

Table 4.5 Parameters to be used in the calculation of pedestrian response, according to UK National Annex to EN1991-2

Load Parameter	Walking	Jogging
Reference load F_0 (N)	280	910
Pedestrian crossing speed, v_t (m/s)	1.7	3

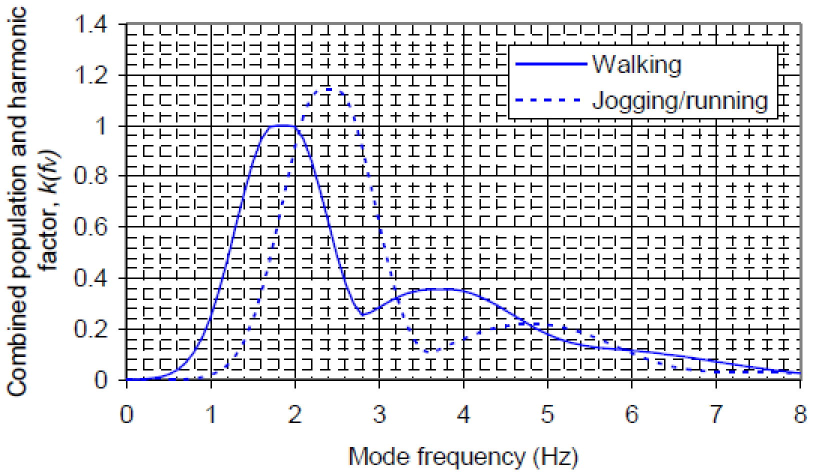


Figure 4.5 Relation between $k(f_v)$ and frequencies f_v, according to UK National Annex to EN1991-2

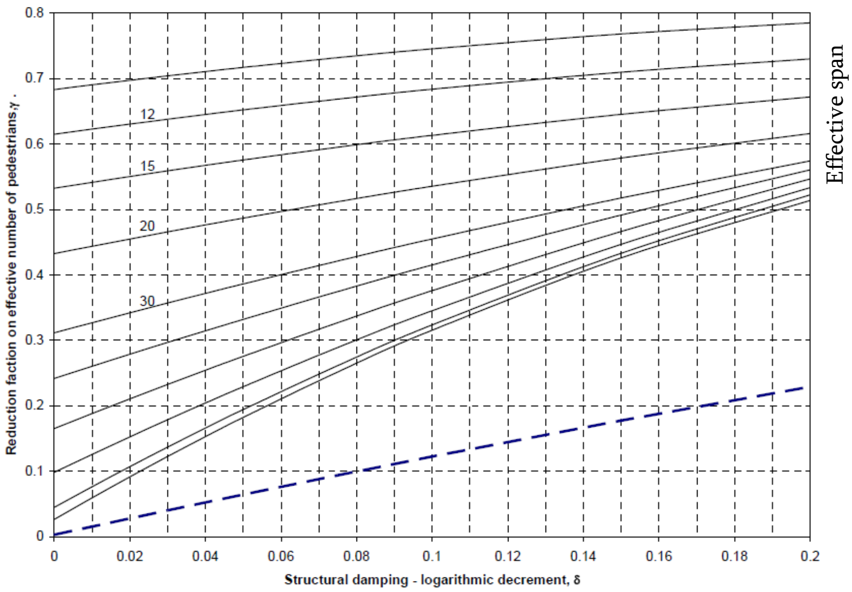


Figure 4.6 Reduction factor, γ, to allow for the unsynchronised combination of pedestrian actions within groups and crowds, according to UK National Annex to EN1991-2

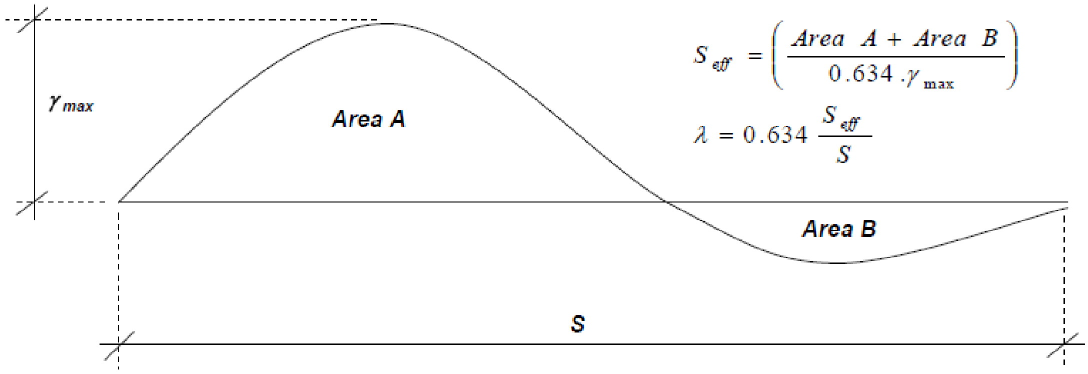


Figure 4.7 Effective span calculation, according to UK National Annex to EN1991-2

The action presented in this paragraph represents a group of N pedestrians where:

- the pedestrians in the group make a single crossing of the bridge together;
- one pedestrian in the group is assumed to walk with a pace frequency that is exactly matched to the frequency of the mode being investigated;
- all other pedestrians in the group (N-1) are assumed to walk with phase and pace rates that are randomly chosen from the pedestrian population model.

One can observe that the code does not describe methods to calculate the responses from the defined action.

4.3.3.2 Steady state modelling of pedestrians in crowded conditions (vertical)

The design maximum vertical accelerations that result from pedestrians in crowded conditions should be calculated by assuming that these are represented by a vertical pulsating distributed load w (N/m² or Pa), applied to the deck for a sufficient time so that steady state conditions are achieved as follows:

$$w = 1.8\left(\frac{F_0}{A}\right)k(f_v)\sqrt{\frac{\gamma N}{\lambda}}\sin(2\pi f_v t)$$

Where:

N is the total number of pedestrians distributed over the span S: N= ρ.A = ρ.S.b

ρ is the required crowd density reference taken from Table 4.4 but with a maximum value of 1.0 pers./m² (This is because crowd densities greater than this value produce less vertical response as the forward motion slows).

S is the span of the bridge.

A is the area of the walking surface of the bridge (m²)

b is the width of the walking surface of the bridge (m)

γ is a factor that reduces for the unsynchronised combination of actions in a crowd and is obtained from Figure 4.6.

λ is a factor that reduces the effective number of pedestrians in proportion to the enclosed area of the mode of interest: $\lambda = 0.634 * (S_{eff} / S)$

In order to obtain the most unfavourable effect this loading should be applied over all relevant areas of the footbridge deck with the direction of the force varied to match the direction of the vertical displacements of the mode for which responses are being calculated.

4.3.3.3 Lateral responses due to crowd loading

The method used to determine the lateral vibration comfort criteria is significantly different than in the vertical direction. The lateral stability condition has a measurable and clearly defined limit that should not be exceeded if large uncontrolled lateral motions are to be avoided.

The UK National Annex proposes to calculate the pedestrian excitation mass damping parameter D:

$$D = \frac{m_{bridge}\xi}{m_{pedestrian}}$$

where:

m_{bridge}	is the mass per length unit of the bridge
$m_{pedestrian}$	is the mass pet unit length of pedestrians obtained from Table 4.4
ξ	is the structural damping when expressed as a damping ration, $\xi = \delta / 2\pi$

The frequency of the lateral mode and parameter D should then determine of the lateral responses are stable, by using Figure 4.8.

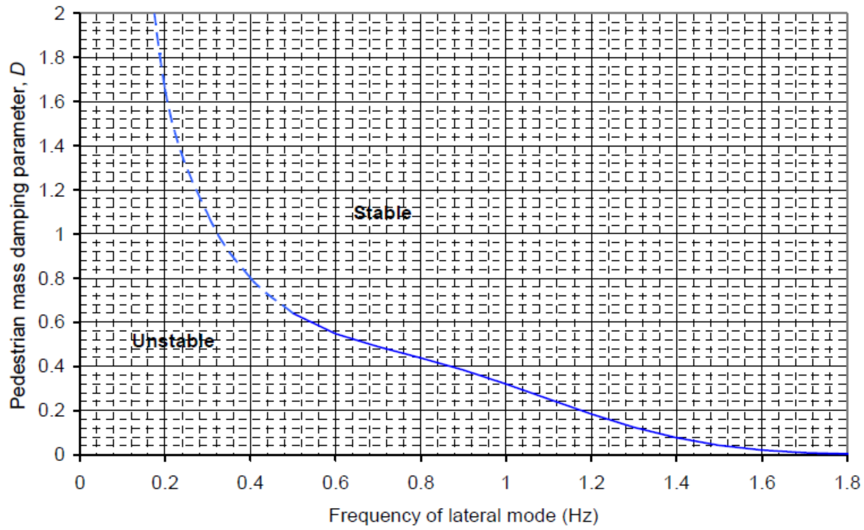


Figure 4.8 Lateral lock-in stability boundary, according to UK National Annex to EN1991-2

4.3.4 Maximum acceleration recommendations

The UK National Annex recommends a vertical design acceleration limit:

$a_{limit} = 1.0\ k_1\ k_2\ k_3\ k_4\ m/s^2$ and $0.5\ m/s^2 \le a_{limit} \le 2.0 m/s^2$

where:

k_1 = site usage factor, according to Table 4.6

k_2 = route redundancy factor, according to Table 4.7

k_3 = height of structure factor, according to Table 4.8

k_4 = exposure factor = 1.0 unless determined otherwise for individual project

Table 4.6 Recommended values for the site usage factor k_1, according to UK National Annex to EN1991-2

Bridge function	**k_1**
Primary route for hospitals or other high sensitivity routes	0.6
Primary route for school	0.8
Primary routes for sports stadia or other high usage routes	0.8
Major urban centres	1.0
Suburban crossings	1.3
Rural environments	1.6

Table 4.7 Recommended values for the route redundancy factor k_2, according to UK National Annex to EN1991-2

Route redundancy	**k_2**
Sole means of access	0.7
Primary route	1.0
Alternative routes readily available	1.3

Table 4.8 Recommended values for the height of structure factor k_3, according to UK National Annex to EN1991-2

Bridge height	**k_3**
Greater than 8 m	0.7
4 to 8 m	1.0
Less than 4 m	1.1

k_4 may be assigned a value of between 0.8 and 1.2 to reflect other conditions that may affect the users' perception towards vibration. These may include consideration of parapet design (such as height, solidity or opacity), quality of the walking surface (such as solidity and opacity) and provision of other comfort-enhancing features.

4.4 Australian Standard

The Australian Standards are issued by Australia Standards which was founded in 1922, originally called Australian Commonwealth Engineering Standards Association. AS 5100.2-2004 is the Australian Standard concerning Bridge Design and Design Loads. Chapter 12 deals about the dynamic behaviour and more in particular, paragraph 12.4 which covers pedestrian bridges.

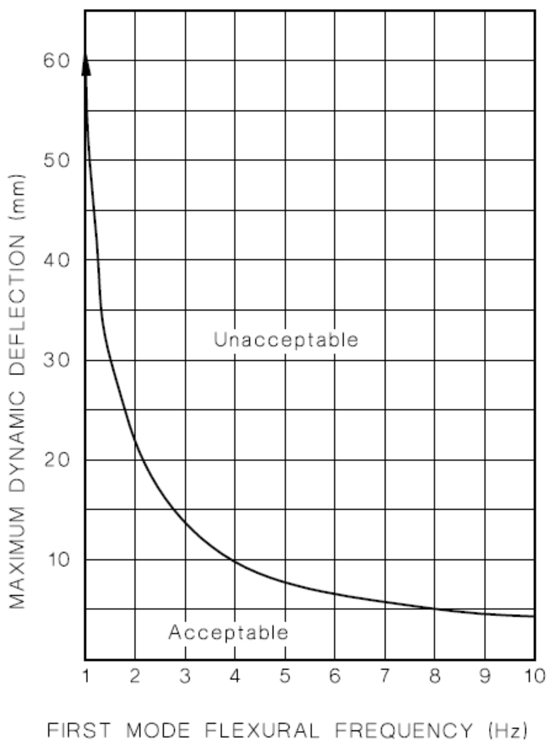


Figure 4.9 Dynamic amplitude limits for pedestrian bridges, according to AS 5100.2

The Australian Standard states that footbridges which have a vertical natural frequency in the range 1.5 Hz and 3.5 Hz should be investigated on dynamic properties: superstructures shall be proportioned such that, with one pedestrian traversing the structure, the maximum dynamic amplitude shall not be greater than the limit shown in Figure 4.9.

The design pedestrian load shall have a weight of 700 N and be assumed to cross the structure at an average walking speed, i.e., 1.75 to 2.5 footfalls per second.
When the fundamental frequency of horizontal vibration is less than 1.5 Hz, special consideration shall be given to the possibility of excitation by pedestrians of lateral movements of unacceptable magnitude.

4.5 Comparison

All codes contain discussed in this chapter contain requirements for the serviceability limit state for vibrations. However the National Annex of the UK (Eurocode) is only one which proposes pedestrian load models, together with the proposal Annex C. The way to determine the responses of the bridge are left over to the designer.

The Eurocode expresses limits in maximum accelerations, the Australian Standard in maximum dynamic deflection. The essential difference between Proposal Annex C and the UK National Annex is that this last one proposes a speed at which the force is moving over the bridge. This corresponds much more to the reality, as pedestrians do not load a bridge dynamically when standing still. Annex C does make this assumption, which could theoretically cause higher vibration amplitudes than in real.

The UK National Annex introduces the Bridge Class. For group of pedestrians, load models change for the different bridge classes. Annex C does not take account of the different situations: all bridges are loaded with the same dynamic force. Figure 4.10 shows the amplitudes which the forces could approximately reach. The green line comes from Annex C and is always the same for vertical vibrations. The two other lines represent the estimation of the minimum and maximum amplitude of the dynamic force: the under bound (blue line) represents a bridge of Bridge Class A with a large effective span. The upper bound (red line) represents a bridge of Bridge Class D with a small effective span.

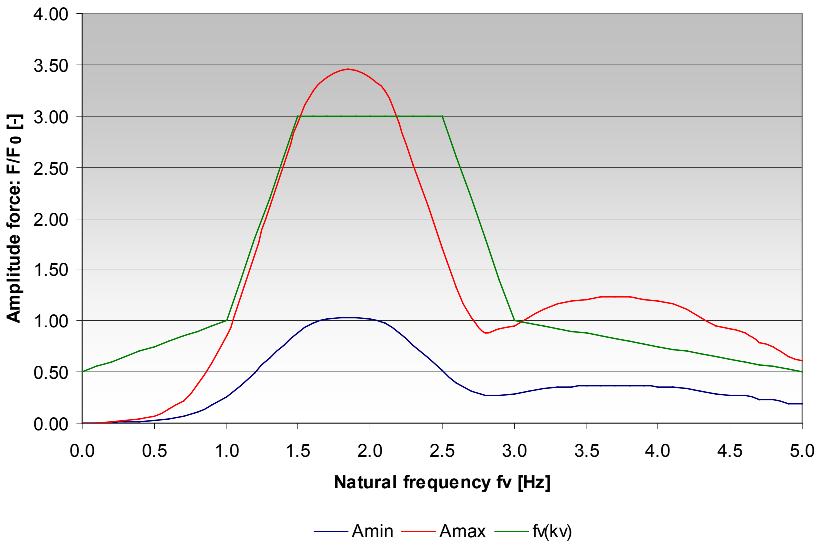


Figure 4.10 Dynamic force amplitudes according to Eurocode Proposal Annex C and UK National Annex

The load models presented in the UK National Annex only deal with vertical vibrations. A method is given to determine if the bridge is laterally stable, but this does not depend on force models. Proposal Annex C however proposes lateral force models.

5 Dynamic Analysis of the Goodwill Bridge

The (proposal) codes described in chapter 4 each have their own way to describe pedestrian loads and how to determine the serviceability limit state for human induced vibrations. To determine the efficiency and the limits of these codes and proposals, the proposed load models are being applied to the bridges that have been described in chapter 3. This chapter describes the analysis of the Goodwill Bridge and discusses the output of the analysis, which is compared to measurements and practice.

The first paragraph deals with the models made in Strand7 and GSA to assess and compare the natural frequencies of the Main Span. The dynamic analyses according to Proposal Annex C, the UK National Annex and the Australian Standard are respectively described in the third, fourth and fifth paragraph. In the last paragraph, the results will be compared with each other and with data that is available from measurements. Note that these paragraphs only mention the essential parts of the analyses: more details are given in appendix 4.1.

5.1 Model

The main span of the Goodwill Bridge is the part that is being modelled and analysed. The models have been made according to the drawings that have been used for the construction of the bridge. The Strand7 model can be seen in Figure 5.1.

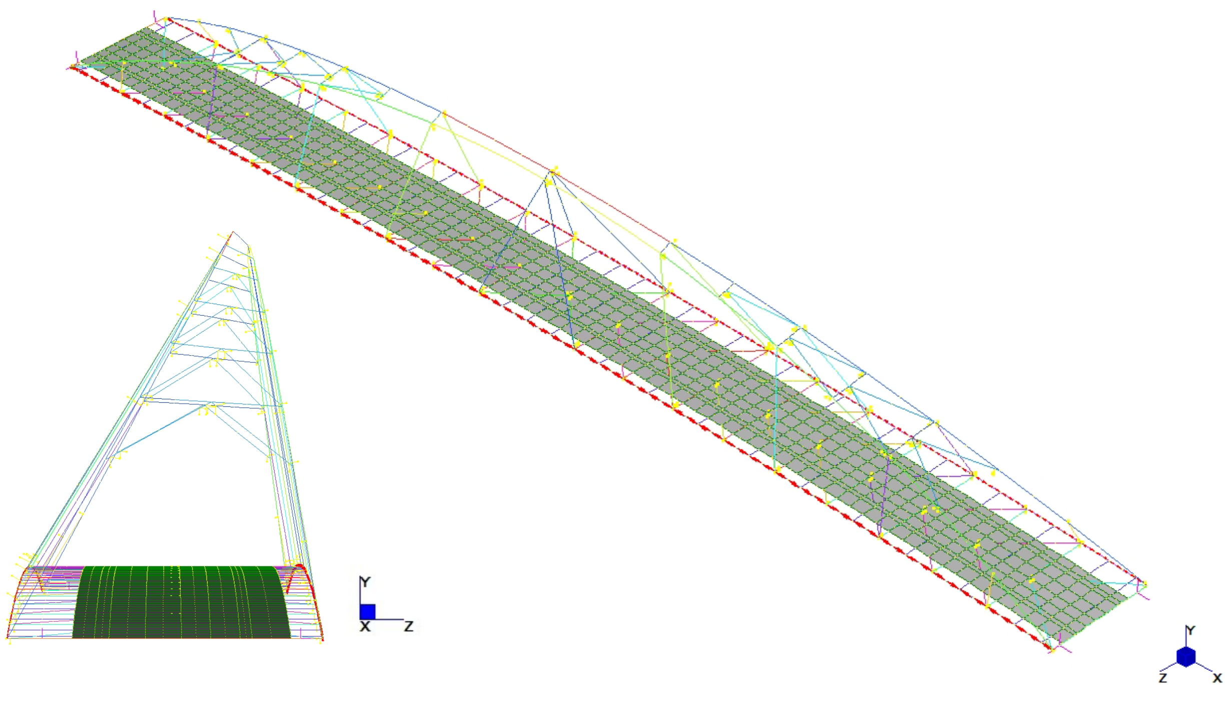


Figure 5.1 Strand7 Model of the Goodwill Bridge (3D View and Side View)

A similar model has been created in GSA to compare the generated Natural Frequencies and modes. This model can be seen in Appendix 4.1 and is not further discussed in this chapter.

The elements of the main span that have been modelled are:

- The chords
- The secondary beams between the chords
- The wind braces (under the concrete deck and in between the arches)
- The arches
- The hangers
- The concrete deck
- The beams on which the bridge is supported.

The rest area in the middle of the arch has been omitted as the area is not susceptible to be loaded dynamically and has not much influence on the dynamic behaviour. The same applies for the parapets and the canopy, as it is not believed to have much influence on the general dynamic behaviour of the bridge. Note that elements like parapets could have some effects on the lateral movement of a bridge, but in this case the triangular cross section of the bridge seems to give the bridge enough lateral stiffness.

Figure 5.2 Cable stayed part of the Goodwill Bridge supports the main span.

The purple stripes at each end of the bridge model (in Figure 5.1) represent the supports of the bridge. All supports have been modelled as fully restrained, even though the support is different on both sides of the bridge. On the South Bank side, the main span is directly supported by the concrete foundation which is founded on rocks. On the CBD side, it is supported by a cable stayed part of the bridge, as shown in Figure 5.2. This cable stayed part however has such stiffness in vertical direction that it can be considered as infinitely stiff for the purpose of this analysis. The stiffness in lateral direction however is probably not that stiff, because of the long piles in the water to the rocks under the cable stayed part. However, this has been omitted in the calculations. The responses in lateral direction should therefore be used with precaution.

More images and explanation about both models can be found in Appendix 4.1.

5.2 Assessment of the Natural Frequencies

The load models described in Chapter 4 refer to the natural frequencies of the bridge. The codes stipulate to load the bridge in the natural frequencies that lie within the walking and/or running frequency range and that are the most unfavourable and most likely to occur. This paragraph describes the assessment of the natural frequencies of the main span of the Goodwill Bridge with Strand7, using the model described in the former paragraph. Chapter 2 of Appendix 4.1 gives a full overview of the assessment, with a comparison with the natural frequencies assessed in GSA.

Table 5.1 Natural Frequencies of the Main Span Goodwill Bridge according to Strand7 (1)

Mode #	f_v [Hz]	DZ / DY [%] *(2)*	Type	Remarks
1	0.627218	0.34	Bending mode	Frequency out of walking / running frequency range
2	0.803591	1.84	Bending mode	Frequency out of walking / running frequency range
3	1.152210	12.29	Torsional mode	Frequency out of walking / running frequency range
4	1.929980	1.79	Bending mode	Within Walking frequency Range, near 2 Hz, so most likely to occur. No significant lateral displacements.
5	2.265080	5.65	Torsional mode	Within Walking and Running frequency Range, likely to occur. No significant lateral displacements.
6	2.494950	13.05	Bending mode	Within Running frequency Range. Lateral displacement more significant.
7	2.703090	55.96	Torsional mode	Within Running frequency Range. Lateral displacement very significant.
8	2.716540	79.90	Bending mode	Within Running frequency Range. Lateral displacement very significant.
9	3.077530	49.76	Bending mode	Within Running frequency Range. Lateral displacement very significant.
10	3.599360	4.97	Torsional mode	Frequency out of walking / running frequency range
11	3.860980	2.09	Bending mode	Frequency out of walking / running frequency range
12	4.312130	0.46	Bending mode	Frequency out of walking / running frequency range
13	4.613830	4.74	Torsional mode	Frequency out of walking / running frequency range
14	4.813970	2.54	Torsional mode	Frequency out of walking / running frequency range

(1) The natural frequencies regarding bending mode of local elements have been omitted in this table.
(2) Ratio between the maximum lateral displacement DZ and the maximum vertical displacement DY in the concrete deck. This value gives an idea about the chance on a severe lateral movement due to a vertical load. Note that these lateral displacements occur due to the asymmetrical cross section of the bridge, and not due to lateral loads.

The natural frequencies assessed in Strand7 are mentioned in the second column of Table 5.1, with their corresponding mode shape number in the first column. As the main span of the bridge is asymmetrical, lateral movements due to vertical movements are likely to occur. To compare the mode shapes on lateral movements, the ratio between maximum vertical displacement and the maximum lateral displacement has been calculated for each of the modes. The values of this ratio are given in the third column. This should give an idea about the likelihood of severe lateral accelerations. Note that only the frequencies under 5 Hz are mentioned in the table, as they are the most likely to occur with pedestrian loads.

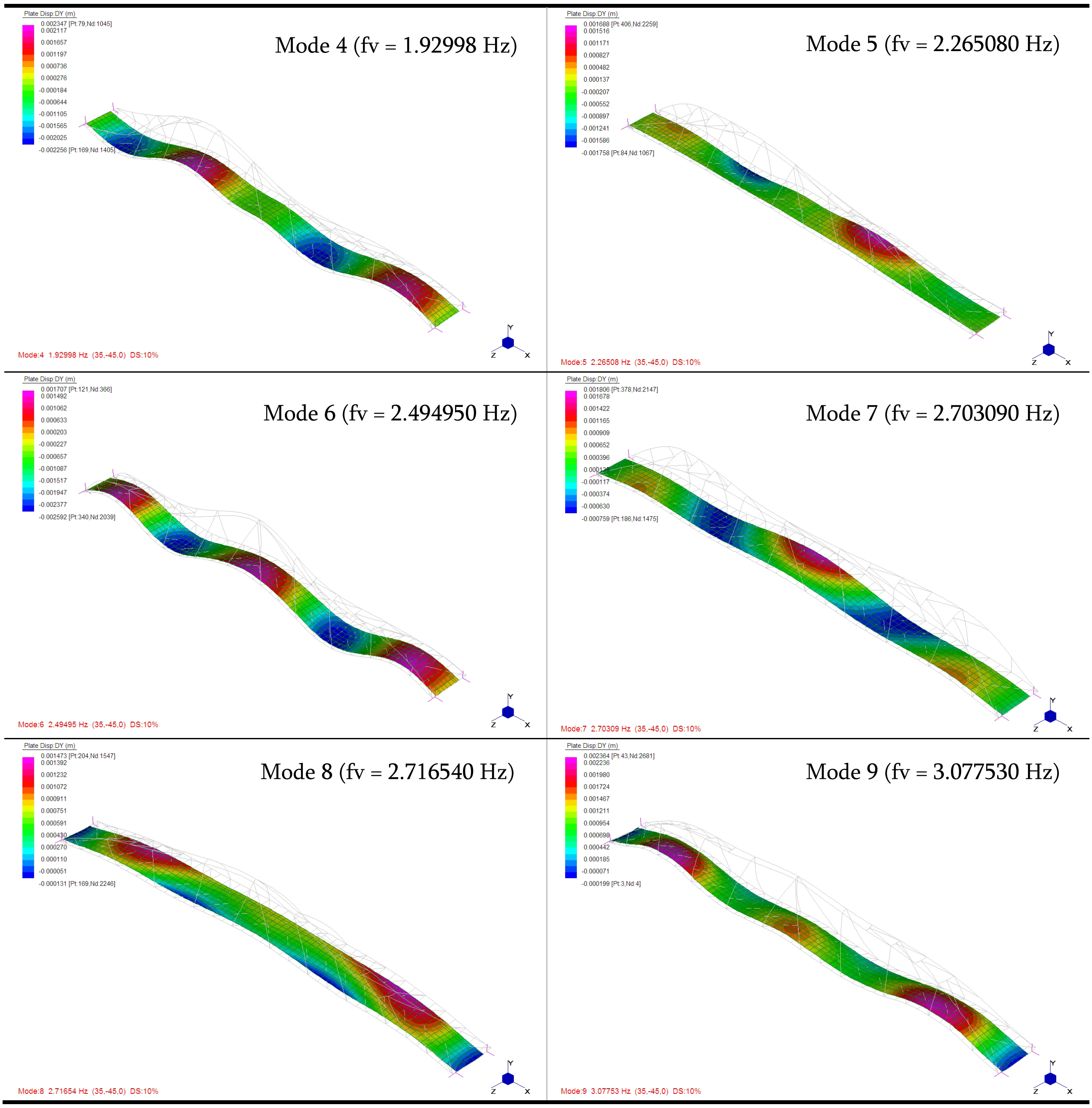


Figure 5.3 Mode shapes of modes 4 to 9 (Strand7 Output)

In Chapter 2, it has been shown that the walking frequencies lie between 1.3 and 2.4 Hz. The running frequencies lie between 2 and 3.5 Hz. According to the results presented in Table 5.1, only modes 4 to 9 are therefore likely to occur. The mode shapes are shown in Figure 5.3.

Modes 4 and 5 are the two modes that have to be considered when loading the bridge with walking pedestrians. These modes are higher degree bending and torsional modes. Mode 5 to 9 are the most likely to occur when the bridge is loaded with joggers. Like mode 4, these are higher degree bending and torsional modes.

5.3 Dynamic Analysis according to Proposal Annex C

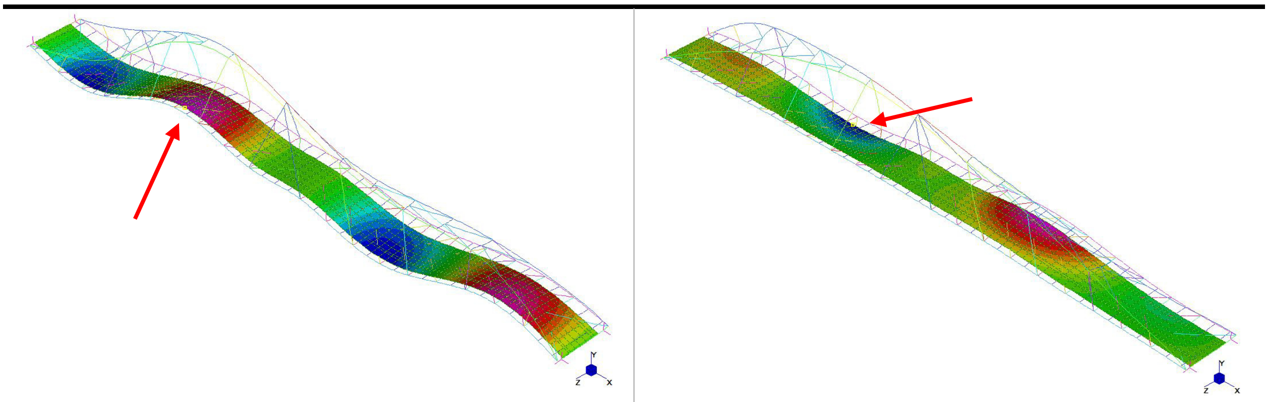
5.3.1 Considered mode shapes

Proposal Annex C only considers loads derived from walking pedestrians and thus not from joggers. Note that these loads are modelled as non moving loads. Only mode 4 and 5 are therefore relevant to this section. The dynamic load specified in the code has two components: one vertical and one lateral.

The code states that the dynamic load should be placed at the most unfavourable point of the bridge. This point is the node which has the largest displacement in the mode shapes and thus should have the largest acceleration when loaded dynamically. This analysis concerns a Serviceability Limit State of the bridge, therefore only the vibrations in the concrete deck are relevant. Table 5.2 gives an overview of the nodes on the concrete deck with the largest displacement in each of the relevant modes. Figure 5.4 shows the position of these nodes.

Table 5.2 Nodes with largest vertical displacement for modes 4 and 5

Mode #	Frequency [Hz]	Node #
4	1.92998	1045
5	2.26508	1067



Node 1045 (Mode 4) | Node 1067 (Mode 5)

Figure 5.4 Position of normative nodes in mode 4 and mode 5

The lateral dynamic load should have a frequency that is half the vertical frequency. However, the code specifies to take the natural horizontal frequency that is the nearest of 1 Hz. One should note that there is no natural horizontal frequency of the bridge near 1 Hz, however due to the asymmetrical cross section of the bridge horizontal displacements for each mode shape can be expected. One can see in Table 5.1 that two modes lie in the neighbourhood of 1 Hz: mode 2 and 3. Mode 3 however has relatively larger lateral displacements than mode 2. It is thus logical to use the frequency of mode 3 (f = 1.15221 Hz) for the purpose of this analysis. The maximum horizontal displacement in mode 3 is in node 372, as shown in Figure 5.5.

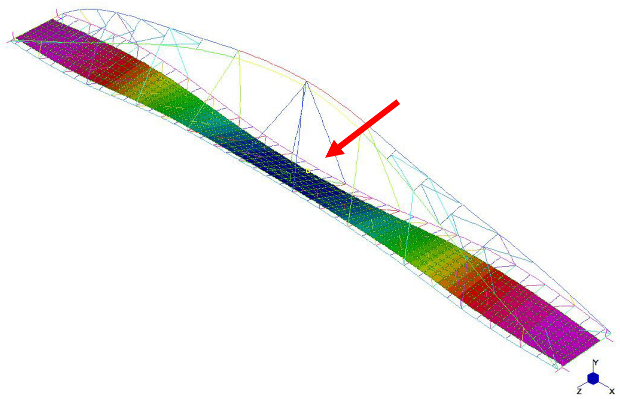


Figure 5.5 Position of normative nodes in mode 3 for the lateral displacement

Placing the load in the critical node of mode shapes 4 and 5 however will always be normative: as mentioned before, the triangular shape of the cross section gives a relative large lateral stiffness. It can be concluded that mode 3 can not be decisive in combination with a vertical harmonic load corresponding to the natural frequency of mode 4 or 5.

This does not count for DLM3, as the dynamic load in this situation is equally distributed over the bridge.

Proposal Annex C requires adding an extra vertical static load to the dynamic loads from load cases DLM2 and DLM3. A mass of 800 kg should be added in the case of DLM2 and 40 kg/m² should be added in the case of DLM3. This is however not possible in Strand7. Practice has shown that the Dynamic Transient Solver of Strand7 cannot handle static load very well, giving responses that are incorrect. The question remains how much these static loads influence the dynamic behaviour of the bridge. Theoretically extra mass changes the Natural Frequency of the modes and its damping. However, one can state that the Natural Frequencies only change slightly. That would mean that the frequency of the load should be changed into that same frequency for the purpose of the dynamic analysis. One can state that this small change would not have much effect on the magnitude of the response. Regarding the damping a mass of 800 kg would not change this dramatically either, especially when considering the total mass of the bridge. The static mass for DLM3 could have some more effect on the damping and could cause the response of the bridge being smaller.

5.3.2 Dynamic Loads

For Load Cases DLM1 and DLM2, the dynamic forces that have to be applied are in the form of:

- Vertical: $Q_{pv} = 280 k_v (f_v) \sin(2\pi f_v t)$ [N]
- Lateral: $Q_{ph} = 70 k_h (f_h) \sin(2\pi \times 1.15221 \times t)$ [N]

For Load Case DLM3, the following dynamic force should be applied:

- Vertical: $q_{sv} = 12.6 k_v (f_v) \sin(2\pi f_v t)$ [N/m²]
- Horizontal: $q_{sh} = 3.2 k_h (f_h) \sin(2\pi \times 1.15221 \times t)$ [N/m²]

Two situations for f_v can be distinguished: fv = 1.92998 Hz (mode 4) and fv = 2.26508 Hz (mode 5).

Table 5.3 gives an overview of the load cases and the forces that should be applied.

Table 5.3 Load Situations to be applied to the main span of the Goodwill Bridge (Pr. Annex C)

Load Case	Applied on Node#	Vertical amplitude [N] or [N/m²]	Horizontal amplitude [N] or [N/m²]
DLM1 A	1045	280	70
DLM1 B	1067	280	70
DLM2 A	1045	280 * 3 = 840	70 * 3 = 210
DLM2 B	1067	280 * 3 = 840	70 * 3 = 210
DLM3 A	Relevant nodes mode shape 4 (1)	12.6 * 3 = 37.8	3.2 * 3 = 9.6
DLM3 B	Relevant nodes mode shape 5 (1)	12.6 * 3 = 37.8	3.2 * 3 = 9.6

(1) Proposal Annex C states that in the case of DLM3 only relevant areas of the footbridge deck should be loaded, so that is as most unfavourable. In this case, only the parts with a positive (negative would have had the same effect) displacement in mode shape of Mode 4 and 5 have been loaded, as can be seen in Figure 5.6. As the vertical and horizontal should be coupled, the vertical displacements are considered being normative.

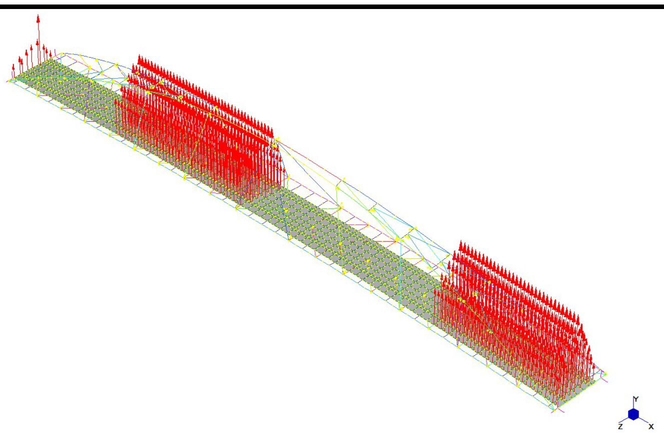


Figure 5.6 Parts of the bridge that are loaded in DLM3, considering mode shape 4 (The horizontal loads have been placed on the same nodes, see Appendix 4.1)

Representations of all load cases can be found in Appendix 4.1.

Each load is being applied for 120 seconds, a time that should be enough to have a steady state situation. No load time is mentioned in Proposal Annex C.

5.3.3 Estimation damping

An important input parameter for the bridge is the damping ratio ζ. Generally, this value can only be estimated. General values for the damping ratio have been given and discussed in chapter 3 of this report (paragraph 3.1.2). The Goodwill Bridge is a composite bridge, as both steel and concrete participate to the stiffness, the mass and the structural strength. The concrete deck represents about 50% of the 550 tons main span of the bridge. Proposal Annex C proposes using a logarithmic decrement of 0.04 for steel and concrete composite, which represents a damping ratio of 0.006 (See table 4.3). This value is equal to the mean value of damping ratio for composite bridges according to the research of Bachmann (See table 2.1). Bachman also gives a minimum damping ratio of 0.003. In the purpose not to overestimate the damping ratio of the Main Span of the Goodwill Bridge, a damping ratio of 0.004 is being used for these analyses.

5.3.4 Output

Further input for the analysis can be found in Appendix 4.1. This paragraph gives an overview of the output for each load situation discussed earlier. According to Proposal Annex C, the vertical acceleration should be limited to the smallest of these values:

- 0.7 m/s²
- $0.5\sqrt{f_v} = 0.5 \times \sqrt{1.92998} = 0.69$ m/s²

For the lateral (horizontal) direction, the acceleration should be limited to the smallest of these values:

- 0.15 m/s²
- $0.14\sqrt{f_h} = 0.14 \times \sqrt{1.15221} = 0.1503$ m/s²

Note that the values for the maximum lateral accelerations are lower than the ones mentioned in Eurocode 0 (see chapter 4).

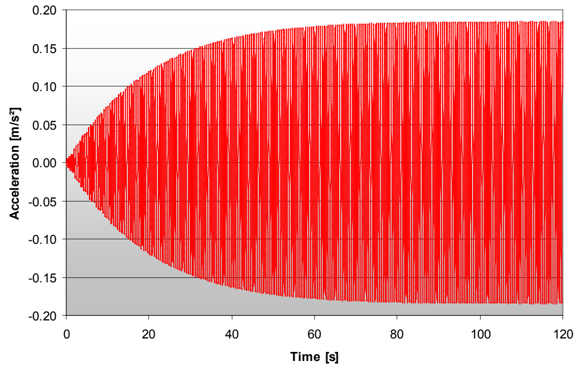
The results of the analyses are summarized in Table 5.4.

Table 5.4 Results Strand7 analyses for Pr. Annex C

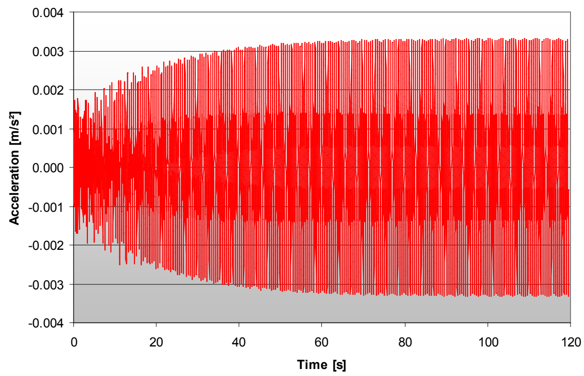
Situation	Vertical acceleration	in node #	Criteria vertical acceleration	Lateral acceleration	in node #	Criteria lateral acceleration
DLM1 A	0.184 m/s² ✓	1045	≤ 0.69 m/s²	0.003 m/s² ✓	1045	≤ 0.15 m/s²
DLM1 B	0.099 m/s² ✓	1067	≤ 0.69 m/s²	0.004 m/s² ✓	1067	≤ 0.15 m/s²
DLM2 A	0.552 m/s² ✓	1045	≤ 0.69 m/s²	0.010 m/s² ✓	1045	≤ 0.15 m/s²
DLM2 B	0.298 m/s² ✓	1067	≤ 0.69 m/s²	0.011 m/s² ✓	1067	≤ 0.15 m/s²
DLM3 A	4.116 m/s² ✗	1045	≤ 0.69 m/s²	0.051 m/s² ✓	1045	≤ 0.15 m/s²
DLM3 B	1.182 m/s² ✗	1067	≤ 0.69 m/s²	0.069 m/s² ✓	1067	≤ 0.15 m/s²

✓ Acceleration is under the tolerated acceleration, ✗ Acceleration is above the tolerated acceleration

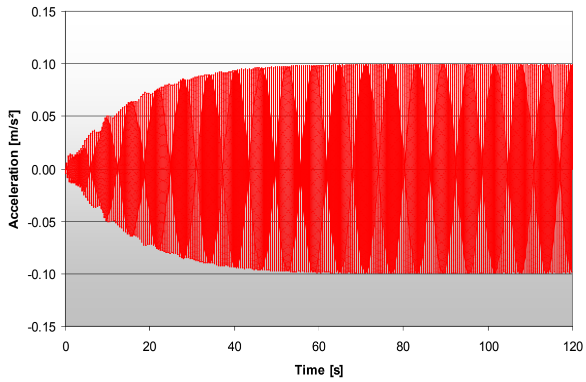
The response of each load case is represented in Figure 5.7.



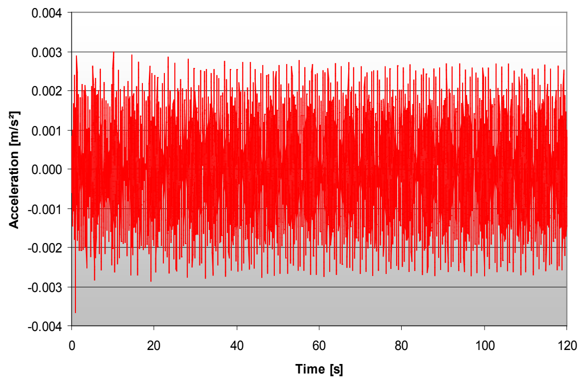
A. Vertical acceleration of node 1045 (DLM1 A)



B. Lateral acceleration of node 1045 (DLM1 A)

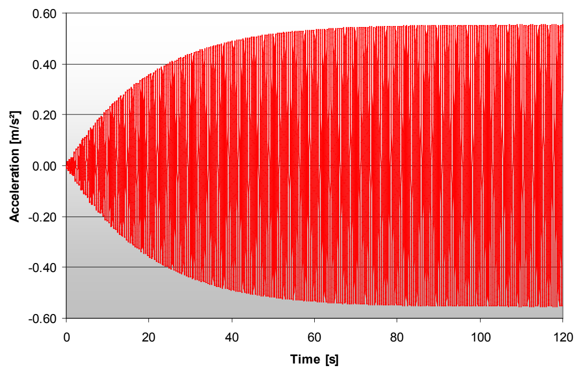


C. Vertical acceleration of node 1067 (DLM1 B)

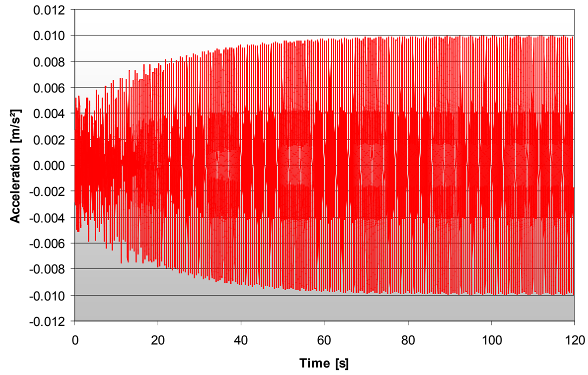


D. Lateral acceleration of node 1067 (DLM1 B)

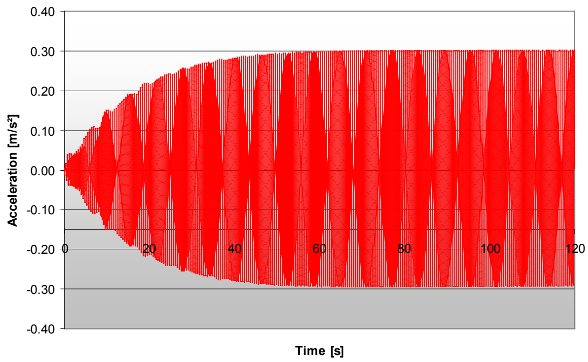
Figure 5.7 continues on next page ➔



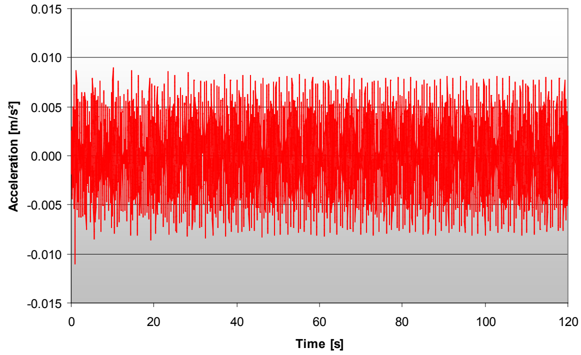
E. Vertical acceleration of node 1045 (DLM2 A)



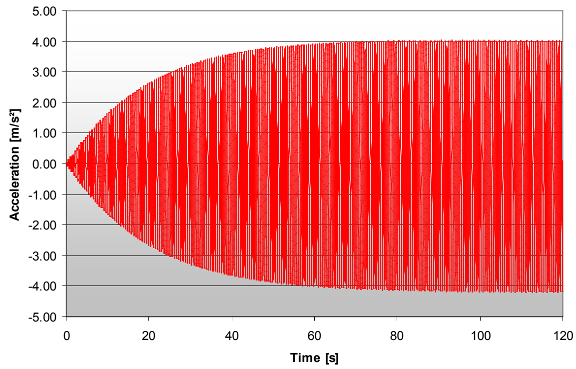
F. Lateral acceleration of node 1045 (DLM2 A)



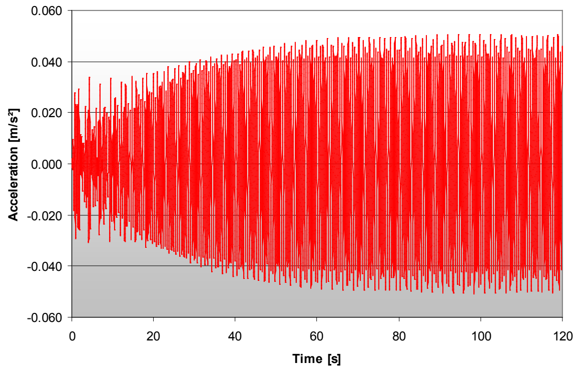
G. Vertical acceleration of node 1067 (DLM2 B)



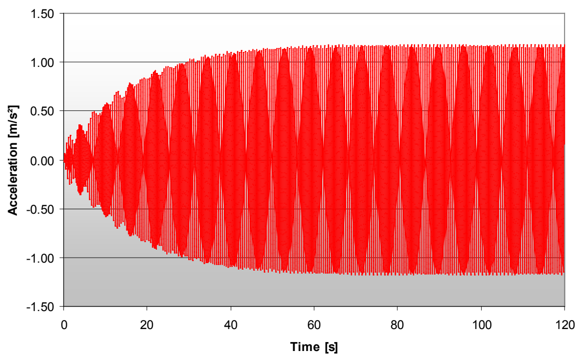
H. Lateral acceleration of node 1067 (DLM2 B)



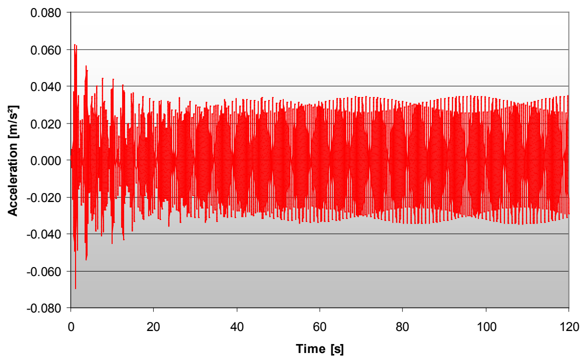
I. Vertical acceleration of node 1045 (DLM3 A)



J. Lateral acceleration of node 1045 (DLM3 A)



K. Vertical acceleration of node 1067 (DLM3 B)



L. Lateral acceleration of node 1067 (DLM3 B)

Figure 5.7 Vertical and horizontal accelerations of critical nodes for DLM1, DLM2 and DLM3

5.3.5 Analysis of Output

This paragraph aims to clarify the response of the bridge for each load case. Considering Figure 5.7, the vertical accelerations seem to be quite evident: the response of the loaded (and critical) node is growing until steady state is reached. This is a behaviour that is logical and expected. For the lateral response however, the behaviour of the critical node is not always clear: especially the first seconds of the responses are often disorganized.

With a Fourier analysis, one can determine which frequencies influence the response and thus which modes are activated by the harmonic load. Note that the Fourier analysis has been done on the displacement of the node as this delivers more accurate results. Appendix 3.2 gives more information about this.

Response DLM1A

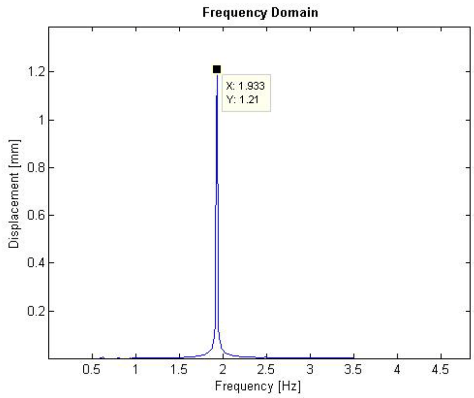


Figure 5.8 Frequency Domain of vertical response DLM1A

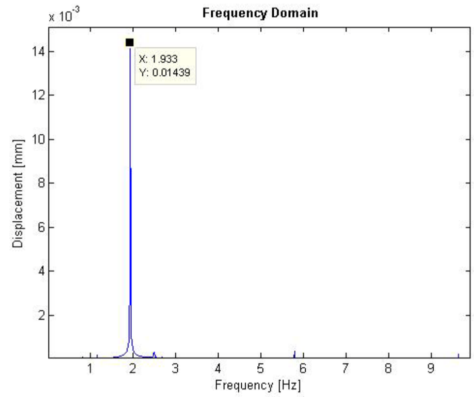


Figure 5.9 Frequency Domain of horizontal response DLM1A

<u>Figure 5.8: Vertical response</u>

This graph clearly shows that the only frequency that occurs is 1.933 Hz. This corresponds to the frequency of the harmonic load (f_v = 1.92998 Hz). Mode shape 4 is thus the only mode that is activated in vertical direction.

<u>Figure 5.9: Horizontal response</u>

The frequency of the vertical load is dominating the response. This is caused by the asymmetrical cross section of the bridge, which causes the bridge to vibrate in lateral direction. The frequency of the lateral harmonic load has nearly no influence on the response. This is caused by the fact that the lateral load is not placed at a strategic point.

Response DLM1B

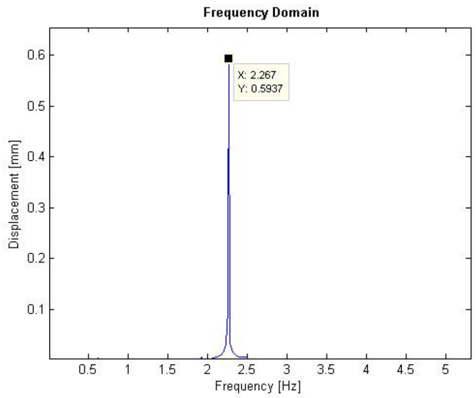


Figure 5.10 Frequency Domain of vertical response DLM1B

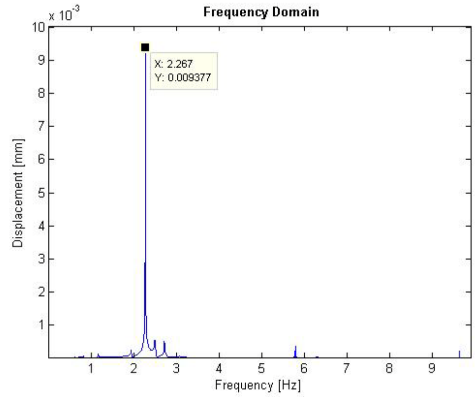


Figure 5.11 Frequency Domain of horizontal response DLM1B

Figure 5.10: Vertical response

As for DLM1A, only the frequency of the harmonic load occurs, in this case f_v = 2.267 Hz.

Figure 5.11: Horizontal response

The frequency of the vertical load is dominating the response, for the same reason as in the former load case. The frequency of the lateral harmonic load has nearly no influence on the response. This is caused by the fact that the lateral load is not placed at a strategic point. The other activated frequencies correspond to higher degree bending and torsional modes, but have not much influence on the total behaviour of the node.

Response DLM2A & DLM2B

These responses are exactly the same as load cases DLM1A and DLM1B, except for the magnitude. The same frequencies as in DLM1 are thus activated.

Response DLM3A

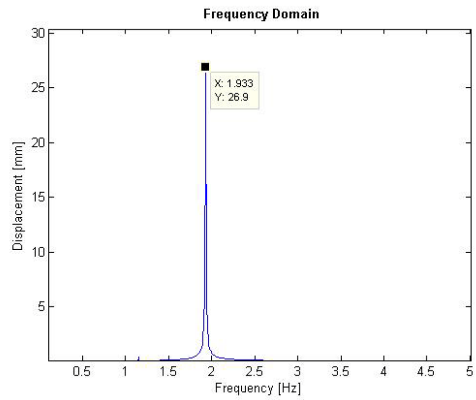


Figure 5.12 Frequency Domain of vertical response DLM3A

Figure 5.12: Vertical response

The only vertical frequency that is activated is f_v = 1.933 Hz, which represents mode shape 4.

Figure 5.13: Horizontal response

The response is dominated by the frequency of the vertical load. However, the frequency of the horizontal load (f = 1.15 Hz) has more influence in this case than in former load cases. The frequencies

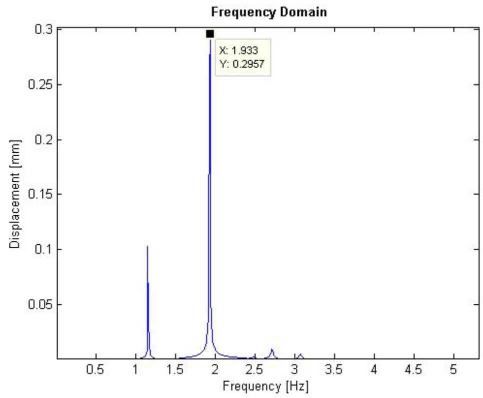


Figure 5.13 Frequency Domain of horizontal response DLM3A

corresponding to mode shape 7 and 9 (respectively f = 2.708 Hz and f = 3.067 Hz) are also activated, but have much less influence than the dominating frequency.

It can be concluded that the vertical harmonic load mainly dominates the lateral motion of the bridge.

Response DLM3B

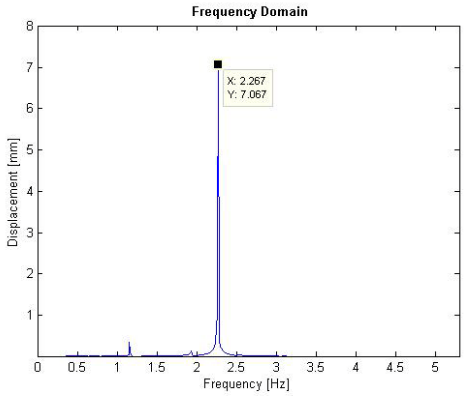


Figure 5.14 Frequency Domain of vertical response DLM3B

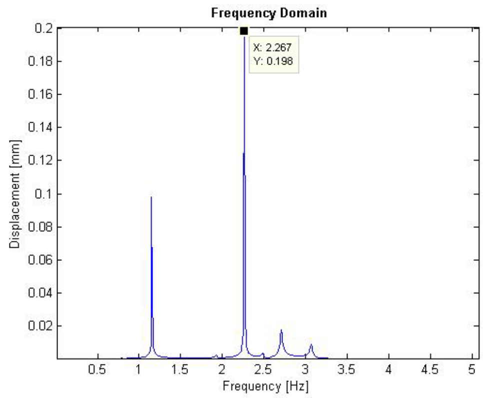


Figure 5.15 Frequency Domain of horizontal response DLM3B

<u>Figure 5.14: Vertical response</u>

The dominating mode is the one with a frequency f_v = 2.267 Hz (mode 5). Notice that the horizontal load has somewhat more effect on the vertical motion in this load case, even though it remains very small compared to the dominating mode.

<u>Figure 5.15: Horizontal response</u>

The response is dominated by the frequency of the vertical load (f = 2.267 Hz), but also strongly by the frequency of the horizontal load (f = 1.15 Hz). The frequencies corresponding to mode shape 7 and 9 (respectively f = 2.708 Hz and f = 3.067 Hz) are also activated, but have much less influence.

It can be concluded that the lateral motion is dominate by both the vertical harmonic load and the horizontal harmonic load.

5.3.6 Conclusions

At the application of DLM1 (Single pedestrian) and DLM2 (Group of about 10 pedestrians), the response of the bridge stays within the limits proposed in Proposal Annex C. However, when the crowd load case (DLM3) is being applied, one can notice very high vertical accelerations, up to 4 m/s². This is mainly due to the unfavourable place of the load and its high amplitude. The load is present on about 50% of the deck. The rest of the deck is unloaded. In increase in damping has however not been taken into account. Note that the load cases based on the frequency of mode shape 4 are always decisive.

An important fact about the lateral motion of the bridge is that is largely dominated by the frequency of the vertical harmonic load. This is due to the asymmetrical cross section of the bridge: whenever a vertical load is applied, the displacement can be divided in a vertical and horizontal component. Thus, next to vertical accelerations, also horizontal accelerations can be observed. All horizontal accelerations however have such amplitude that they meet the requirements stated in Proposal Annex C.

The results are discussed more deeply in paragraph 5.6 in which they will be able to be compared to results of the analyses according to the UK National Annex.

5.4 Dynamic Analysis according to UK National Annex

The UK National Annex proposes moving loads (vertical only) for pedestrians which are moving along the most unfavourable line across the bridge. Both walking persons and joggers are considered. The velocity the dynamic load is moving is dependant on the type of pedestrian. Like in Proposal Annex C, a crowded situation has to be considered.

The Dynamic Load Factors which are applied to the loads vary according to the type of pedestrian (walker, jogger), the number of pedestrians, the effective span of the bridge (which is dependant of the mode shape) and the frequency of the considered mode. This has been described earlier in chapter 4.

5.4.1 Considered mode shapes

For walking pedestrians only mode 4 is considered, because it has been proved in former dynamic analyses that this load case is always decisive. The moving load should in this case move along the line that crosses node 1045, as shown in Figure 5.16.

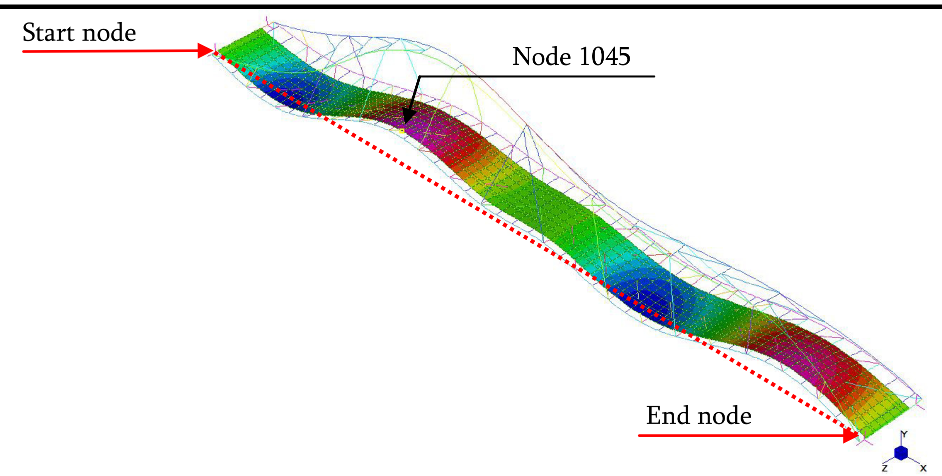


Figure 5.16 Mode 4: Red line represents the path of the dynamic load for pedestrians (walking)

Regarding the joggers, it has been shown in table 5.1 that five mode shapes have to be considered: Mode 5 to Mode 9. All these mode frequencies lie within the frequency range of joggers (2 – 3.5 Hz). For each of these modes, an analysis has thus to be performed. For each of the mode, the most unfavourable node along which the dynamic load should be moving has been represented in Figure 5.17 to Figure 5.21.

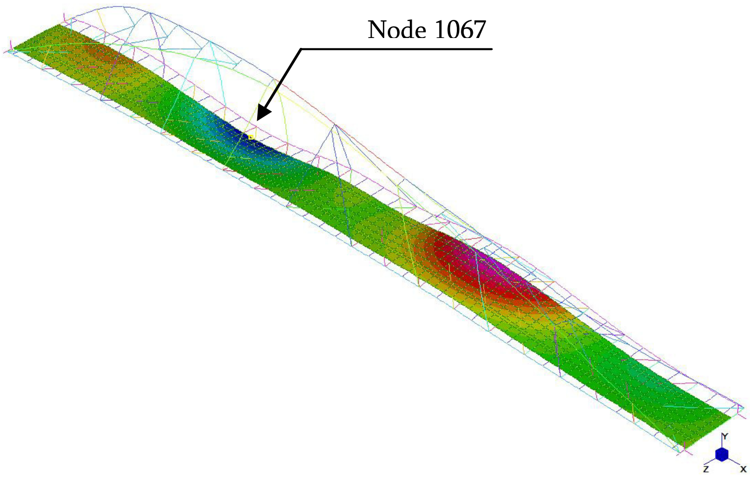


Figure 5.17 Mode 5: Path through node 1067 is the most unfavourable (f = 2.26508 Hz)

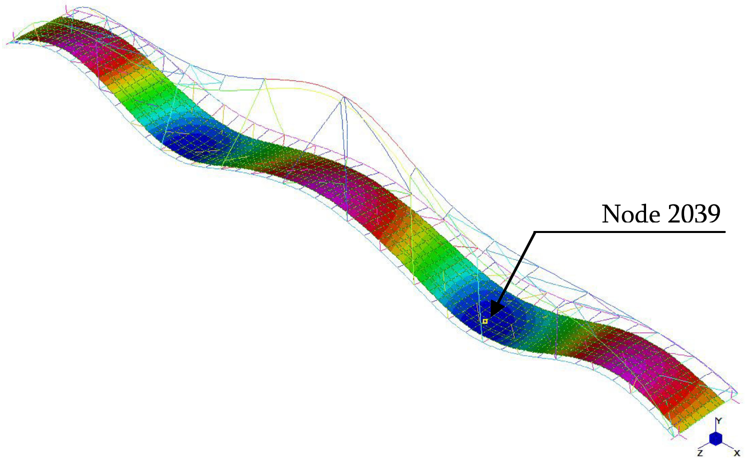


Figure 5.18 Mode 6: Path through node 2039 is the most unfavourable (f = 2.49495 Hz)

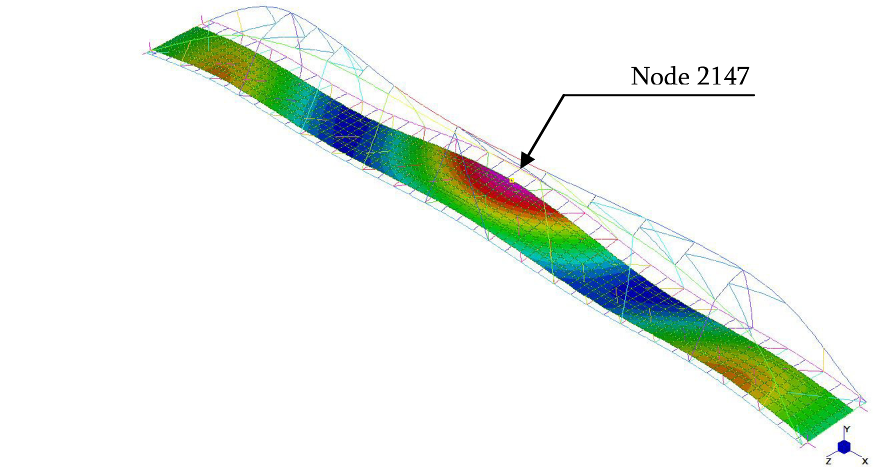


Figure 5.19 Mode 7: Path through node 2147 is the most unfavourable (f = 2.70309 Hz)

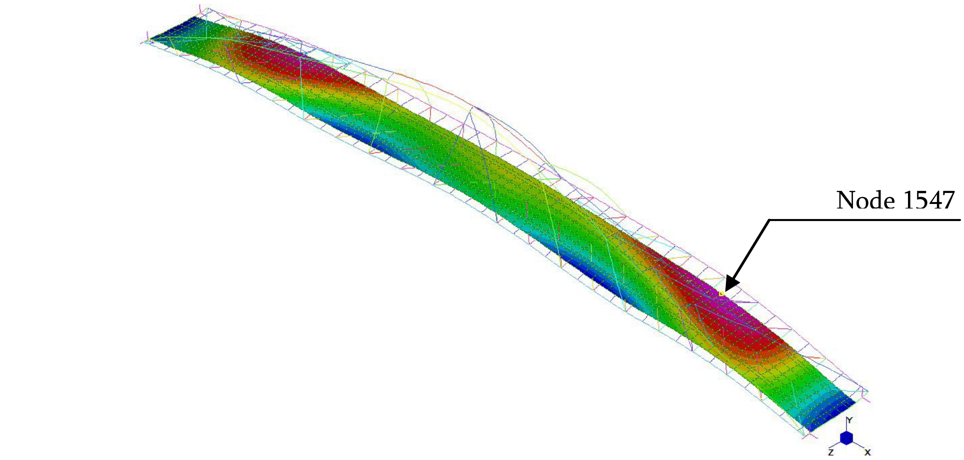


Figure 5.20 Mode 8: Path through node 1547 is the most unfavourable (f = 2.71654 Hz)

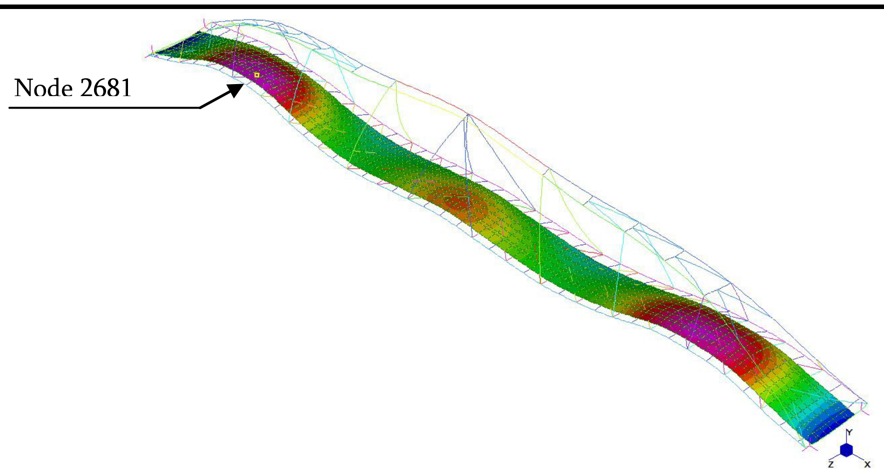


Figure 5.21 Mode 9: Path through node 2681 is the most unfavourable (f = 3.07753 Hz)

For the crowd situation, only Mode 4 is being considered.

5.4.2 Bridge Class

According to the UK National Annex, the Goodwill Bridge can best be categorized in Bridge Class C, as it is built in an urban area and is used intensively. Table NA.7 recommends the following values to be considered in the analyses:

- Group walking: N = 8
- Group jogging: N = 2
- Crowd density: ρ = 0.8 persons/m²

However, to be able to compare the results with Proposal Annex C, the following cases will also be considered:

- Group walking: N = 1 (single pedestrian)
- Group walking: N = 16

5.4.3 Load Cases to be considered

Table 5.5 summarizes the load cases which are being analysed in Strand7.

Table 5.5 Load cases considered for the dynamic analysis according the UK National Annex

Load Case #	Mode #	Frequency [Hz]	Path through Node #	Type	N [pers.] / ρ [pers./m²]
UKNA W1	4	1.92998	1045	Walking	1
UKNA W2	4	1.92998	1045	Walking	8
UKNA W3	4	1.92998	1045	Walking	16
UKNA J1	5	2.26508	1067	Jogging	2
UKNA J2	6	2.49495	1438 (1)	Jogging	2
UKNA J3	7	2.70309	2147	Jogging	2
UKNA J4	8	2.71654	1547	Jogging	2
UKNA J5	9	3.07753	2681	Jogging	2
UKNA C	4	1.92998	All (2)	Crowd	0.8

(1) Node 1438, which lies next to node 2039, has been chosen for practical reasons. The deflection at this point is nearly exactly the same as at node 2039.

(2) All nodes needs to be as most unfavourable possible. This means that all forces on the nodes with a positive vertical displacement in mode shape 4 should be the opposite sign of the forces placed on nodes with a negative displacement, as shown in Figure 5.22.

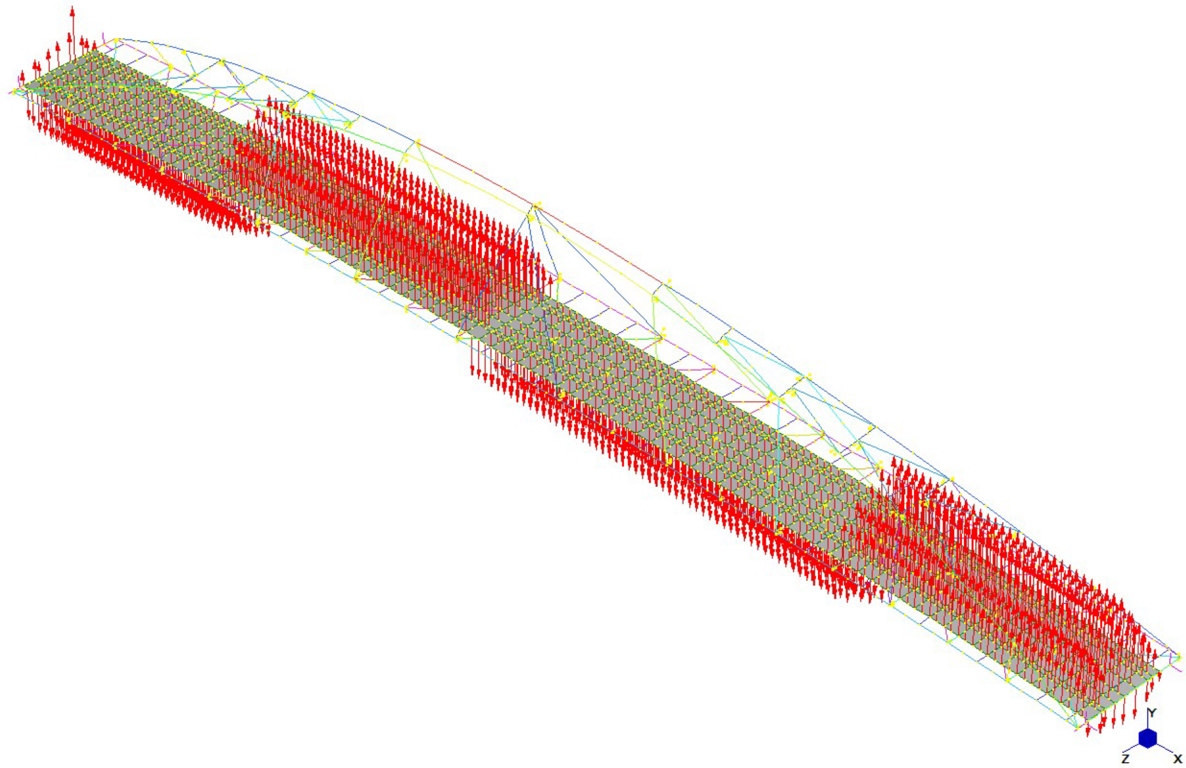


Figure 5.22 Parts of the bridge that are loaded in Load Case UKNA 6

5.4.4 Modelling of moving dynamic loads

A moving dynamic load means that the amplitude of the load is fluctuating in time with a certain frequency and is moving over the bridge with a certain speed. In this case, the load is moving from node to node, as this is the most practical. A vertical point load, with an amplitude of the Reference Load F_0 multiplied by the correct Dynamic Load factor, is placed in the same direction on each node of the path. Each of the loads is created in its own 'load case', so that a 'Factor vs Time' Table can be coupled to each one of them. The 'Factor vs Time' Tables are built up in such a way that each load is applied on the correct time and is varying according to the correct frequency. Each load is applied for the time that is needed to cover a distance beginning half way between the current node and the last node and ending half way between the current and the next node.
As there are 165 nodes on each path (which means 165 loads and 165 Tables), the Input File for Strand7 has been generated with the help of a program created in Excel. This can be found on the CD at the end of the report.

5.4.5 Damping

The same damping ratio as in the analyses of Proposal Annex C is used to run these analyses: $\zeta = 0.004$.

5.4.6 Applied Loads

As written earlier, the amplitude of the loads is dependant of several parameters. Table 5.6 gives an overview of the loads and the Dynamic Load Factors which should be applied to each Load Case. More details about the calculations can be found in Appendix 4.1.

Table 5.6 Amplitudes of Loads for each Load Case

Load case	Reference Load F_0 [N]	DLF [-] (1)	Amplitude Load [N] (2)	Frequency Load [Hz]
UKNA W1	280	1.000	280.0	1.92998
UKNA W2	280	1.442	403.8	1.92998
UKNA W3	280	1.819	509.3	1.92998
UKNA J1	910	1.1895	1082.4	2.26508
UKNA J2	910	1.238	1126.6	2.49495
UKNA J3	910	1.105	1005.6	2.70309
UKNA J4	910	1.105	1005.6	2.71654
UKNA J5	910	0.496	451.4	3.07753
UKNA C	See Appendix 4.1			1.92998

(1) DLF stands for Dynamic Load Factor
(2) Amplitude Load = Reference Load * DLF

5.4.7 Output

The results of the analyses are summarized in Table 5.7.

Table 5.7 Results Strand7 analyses for UK National Annex

Load Case	Vertical acceleration	in node #	Criteria vertical acceleration
UKNA W1	0.015 m/s² ✓	1045	≤ 0.7 m/s²
UKNA W2	0.022 m/s² ✓	1045	≤ 0.7 m/s²
UKNA W3	0.027 m/s² ✓	1045	≤ 0.7 m/s²
UKNA J1	0.101 m/s² ✓	1067	≤ 0.7 m/s²
UKNA J2	0.182 m/s² ✓	1438	≤ 0.7 m/s²
UKNA J3	0.113 m/s² ✓	2147	≤ 0.7 m/s²
UKNA J4	0.104 m/s² ✓	1547	≤ 0.7 m/s²
UKNA J5	0.137 m/s² ✓	2681	≤ 0.7 m/s²
UKNA C	2.087 m/s² ✗	1045	≤ 0.7 m/s²

✓ Acceleration is under the tolerated acceleration, ✗ Acceleration is above the tolerated acceleration

Note that the dynamic analyses have been done with the so called 'Superposition' method, which means that the natural frequencies of the bridges have been generated before the analyses and have been used as input for these analyses. It has been observed that the amount of natural frequencies that should be taken into account should be carefully chosen. Too less or too much natural frequencies generate responses of the bridge that are not correct. This is only the case for moving dynamic loads. The reason remains unclear. However, taking into account natural frequencies up to 20 Hz seem to give correct and logical responses. More information can be found in Appendix 3.2.

But before going further in detail to the analysis of the results, it would be interesting to know which parameters influence the dynamic behaviour of the bridge under influence of a moving harmonic load. There is not much literature available on this subject. That is why two case studies have been considered in Appendix 5. The first study case concerns a simply supported beam on which a moving harmonic load has been placed, in the same way as has been done for the Goodwill Bridge. The responses of different load velocities have been compared: it seems that the faster the load is crossing the bridge the later the maximum displacement occurs in the critical node. The slower the load, the more the maximum displacement tends to occur at the moment the load is passing the critical node. The second case study concerns a simplified arch bridge. In this case study, the effect of the structural elements has been studied. It seems that the stiffness of the arch has much effect on the dynamic behaviour of the deck, more than the hangers have. The larger the stiffness of the arch, the more the dynamic response seems to correspond to the considered mode shape. In other words, less mode shapes seem to be activated when increasing the stiffness of the arch. These study cases show that the dynamic responses of the Goodwill Bridge are dependent on many factors.

Load Cases UKNA W1, UKNA W2 and UKNA W3

These three load cases have been analysed separately in Strand7. Only the amplitude of the load is different from case to case. All other parameters are the same (path, velocity, frequency of the load). Figure 5.23 shows the response in node 1045 for each these load cases. The x-axes represent the quotient of the time t and the total time needed to cross the bridge T.

The responses of walking pedestrians are more complicated than the ones from Proposal Annex C. One can see that the magnitude of the acceleration increases and decreases over time. This change in magnitude seems to correspond to the considerate mode shape. The maximum acceleration occurs around 0.3 (t = 20 s), just next to critical node 1045. Note that accelerations remain under the limit stated in the code (see also Table 5.7).

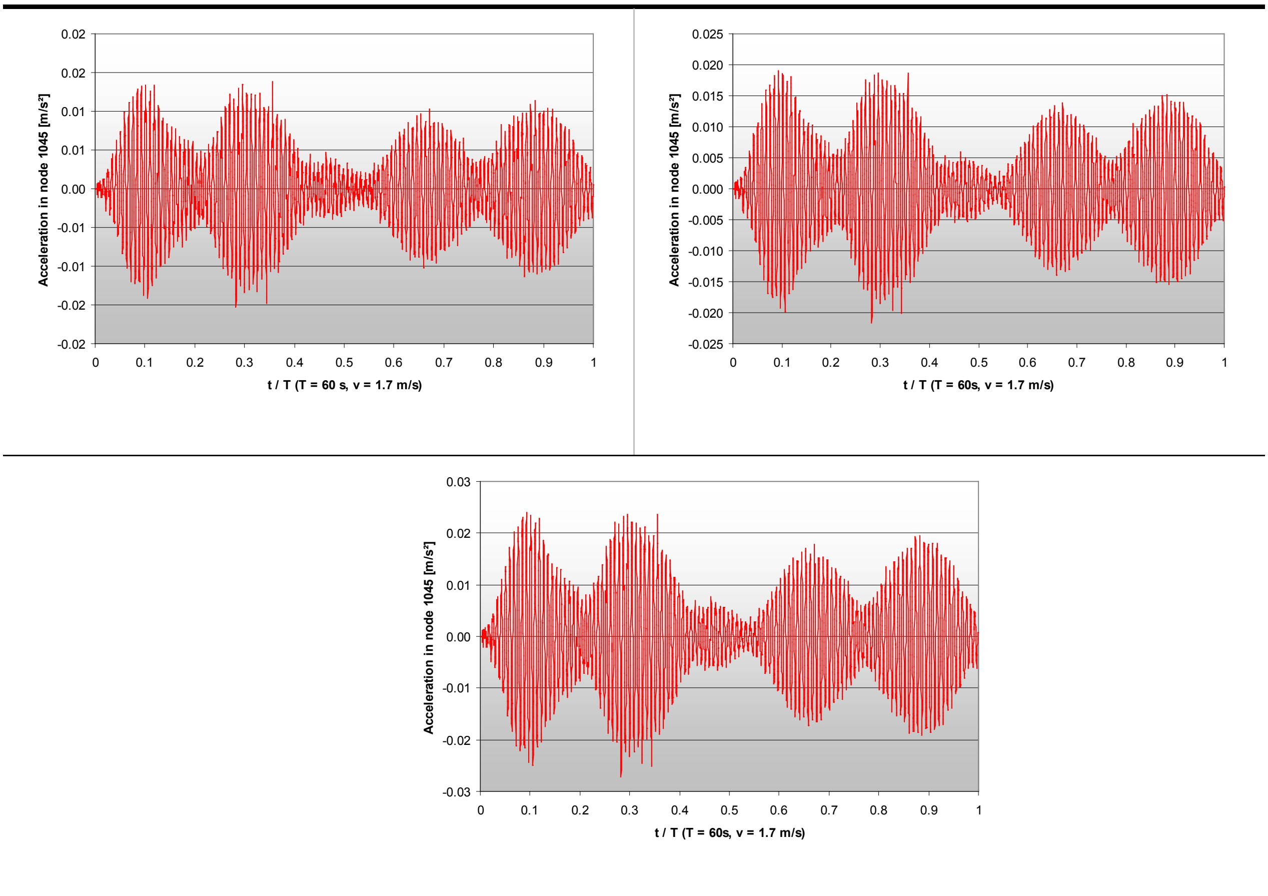


Figure 5.23 Response of the bridge in node 1045 for Load Cases UKNA W1, UKNA W2 and UKNA W3 (walking pedestrians)

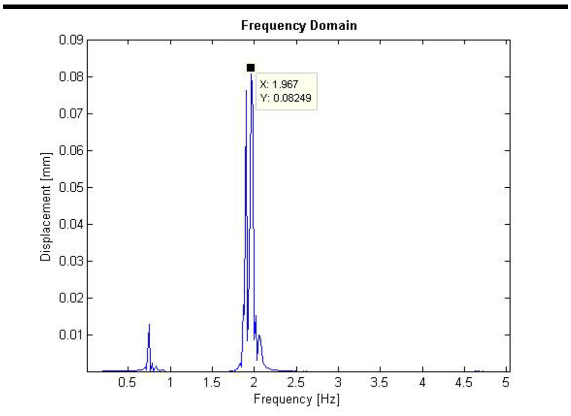


Figure 5.24 Frequency Domain of the response in node 1045 (for Load Cases UKNA W1, UKNA W2 and UKNA W3)

To have a better understanding of the responses of the bridge under moving dynamic loads, the frequency domain of each of the displacement of the critical node is being assessed. This will show which mode shapes are being activated by the moving load. Figure 5.24 shows the Frequency Domain of the responses in node 1045, which is the same for all three load cases.

The Domain Frequency of all three load cases is identical because only the load amplitude changes.

One can see that the frequency f = 1.967 Hz is dominating the behaviour of the response of the bridge. This corresponds approximately to the natural frequency of mode shape 4 (f = 1.92998 Hz), which has been used as input for the frequency of the moving harmonic load. The mode shape corresponding to a frequency of 0.75 Hz is also activated, but in a much smaller extend. This is mode 2 (f = 0.803591 Hz), the first bending mode. No other frequencies are clearly being activated. Both modes are represented in Figure 5.25.

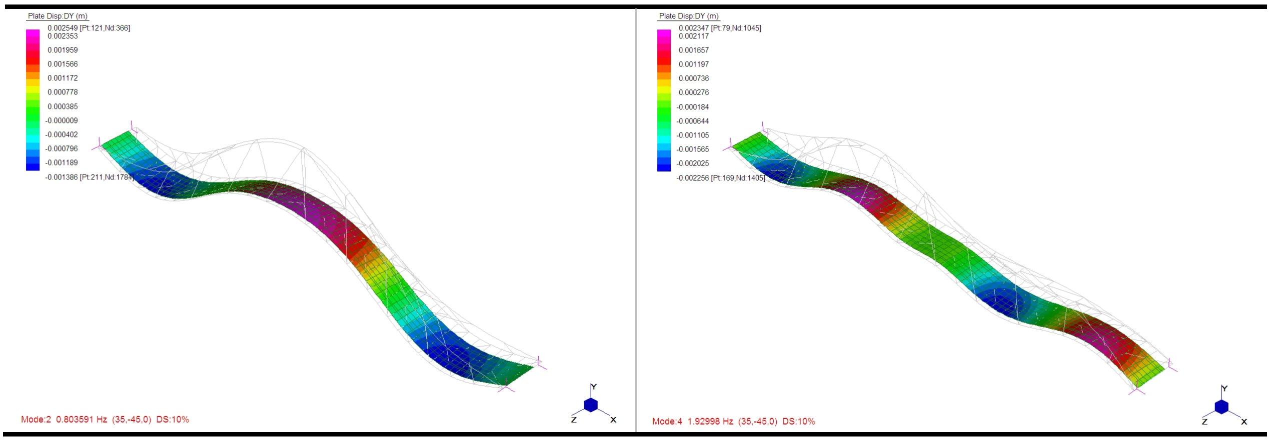
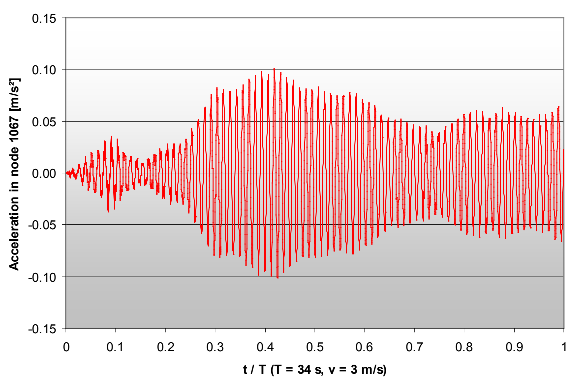


Figure 5.25 Mode 2 (left) and Mode 4 (Right) of the Goodwill Bridge

This explains also the evolution of the acceleration in node 1045 which follows the curvatures of mode shape 4. The maximum acceleration in node 1045 occurs when the moving dynamic load is in the neighbourhood of this node, which means that the velocity of the load has not much influence in this case.

Load Case UKNA J1

Both the response of the critical node (Node 1067) and its Frequency Domain have been represented in Figure 5.26 and Figure 5.27 respectively.



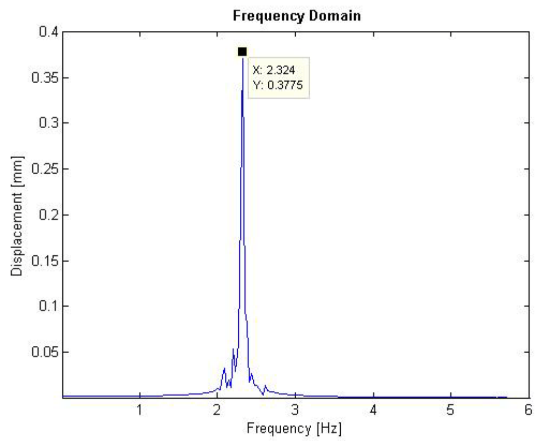


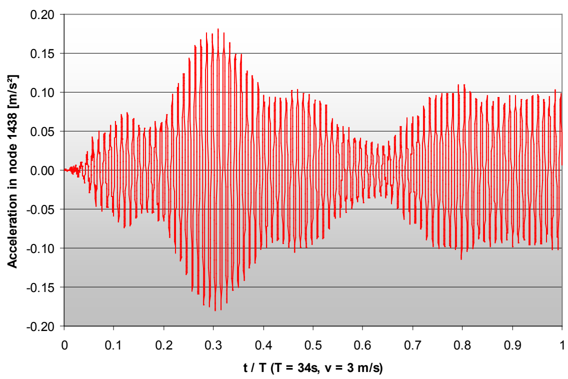
Figure 5.27 shows that the frequency f = 2.324 Hz dominates the behaviour of the bridge. This approaches the frequency of the load (f = 2.26508 Hz) and thus it can be considered that mode shape 5 dominates the behaviour of the bridge. Other mode shapes are also activated, like mode shape 6 (f = 2.49495 Hz), but in a much smaller extend.

One can see in Figure 5.26 that the maximum acceleration in node 1067 does not occur at the moment the load is passing through that node (t = 10.7 s), but somewhat later at 0.42

(t = 14.3 s). As shown in Appendix 5, the higher velocity of the load could be the reason of this effect.

Load Case UKNA J2

Both the response of the critical node (Node 1438) and its Frequency Domain have been represented in Figure 5.28 and Figure 5.29 respectively.



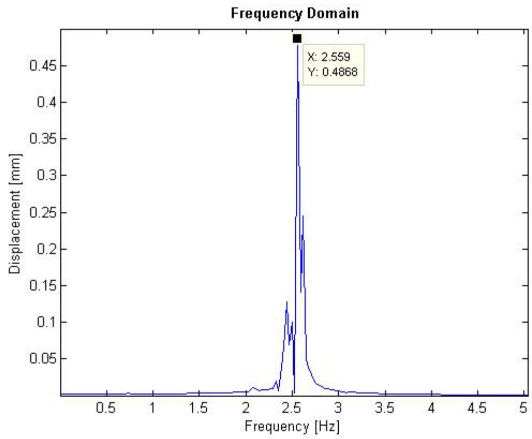
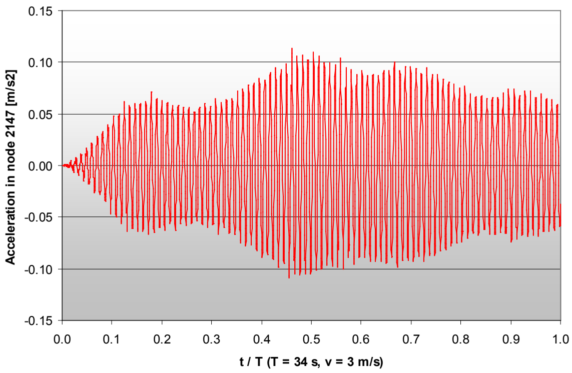


Figure 5.29 shows that the frequency f = 2.559 Hz dominates the behaviour of the bridge. This approaches the frequency of the load (f = 2.49495 Hz) and thus it can be considered that mode shape 6 dominates the behaviour of the bridge. Other mode shapes are also activated, like mode shape 5 (f = 2.26508 Hz), but in a smaller extend.

One can see in Figure 5.28 that the maximum acceleration in node 1438 does not occur at the moment the load is passing through that node (t = 24.5 s), but already much earlier at 0.3 (t = 10.2 s). When considering mode shape 6 (Figure 5.18), it becomes clear that at about one third of the length of the bridge, the displacement is nearly as large as in node 1438. A large acceleration of node 1438 can thus be expected when passing this area. The combination of mode shape 5 and 6 could result in lower acceleration in node 1438.

Load Case UKNA J3

Both the response of the critical node (Node 2147) and its Frequency Domain have been represented in Figure 5.30 and Figure 5.31 respectively.



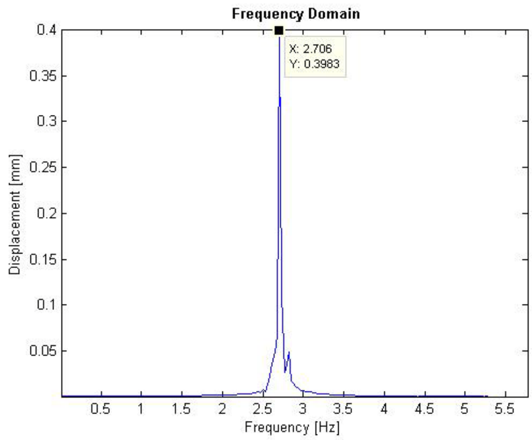
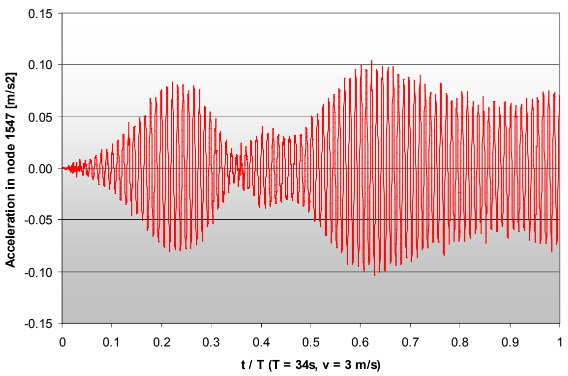


Figure 5.31 shows that the frequency f = 2.706 Hz dominates the behaviour of the bridge. This approaches the frequency of the load (f = 2.70309 Hz) and thus it can be considered that mode shape 7 dominates the behaviour of the bridge. Mode shape 8 (f = 2.71654 Hz) is also activated but in a less extend.
One can see in Figure 5.30 that the maximum acceleration in node 2147 does occur when the load is in the neighbourhood of the critical node.

Load Case UKNA J4
Both the response of the critical node (Node 1547) and its Frequency Domain have been represented in Figure 5.32 and Figure 5.33 respectively.



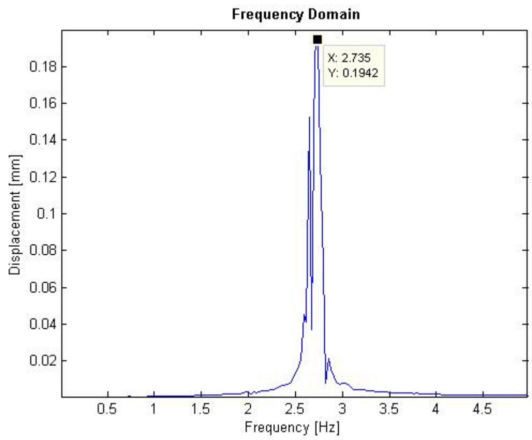
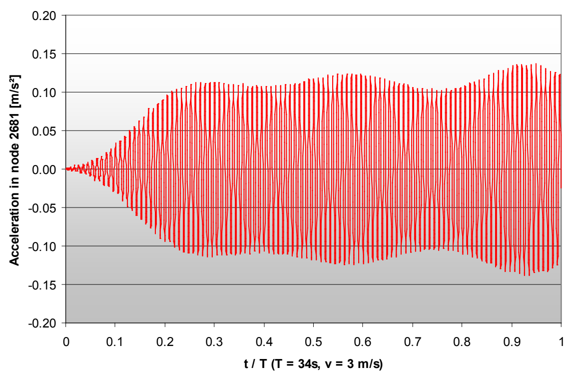


Figure 5.33 shows that the frequency f = 2.735 Hz dominates the behaviour of the bridge. This approaches the frequency of the load (f = 2.71654 Hz) and thus it can be considered that mode shape 8 dominates the behaviour of the bridge. Mode shape 7 (f = 2.70309 Hz) is also activated but in a less extend.

One can see in Figure 5.32 that the maximum acceleration in node 1547 does occur when the load is at 0.63 (t = 21.4 s), before the load reaches the critical node. This is probably due to the superposition of mode shapes 7 en 8.

Load Case UKNA J5

Both the response of the critical node (Node 2681) and its Frequency Domain have been represented in Figure 5.34 and Figure 5.35 respectively.



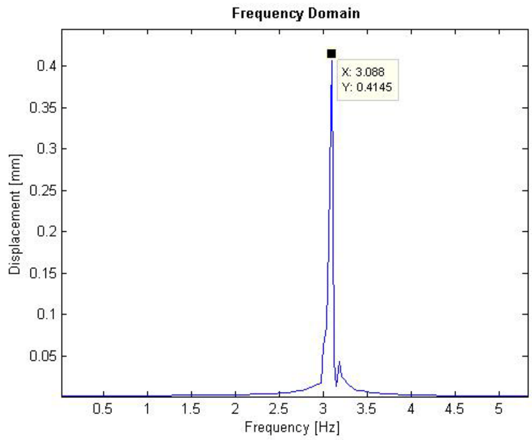


Figure 5.35 shows that the frequency f = 3.088 Hz dominates the behaviour of the bridge. This approaches the frequency of the load (f = 3.07753 Hz) and thus it can be considered that mode shape 9 dominates the behaviour of the bridge.

One can see in Figure 5.34 that the maximum acceleration in node 2681 does occur when the load is at 0.93 (t = 31.6 s), long after the load has passed the critical node.

Load Case UKNA C

The particularity of this load model is that the load is not moving over the bridge. Both the response of the critical node (Node 2681) and its Frequency Domain have been represented in Figure 5.36 and Figure 5.37 respectively.

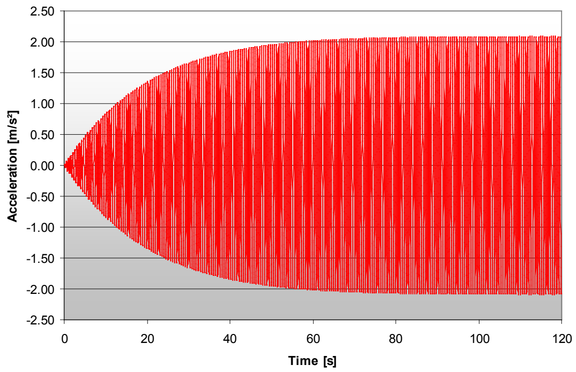


Figure 5.36 Response of node 1045 for Load Case UKNA C (Critical node 1045 at 0.316)

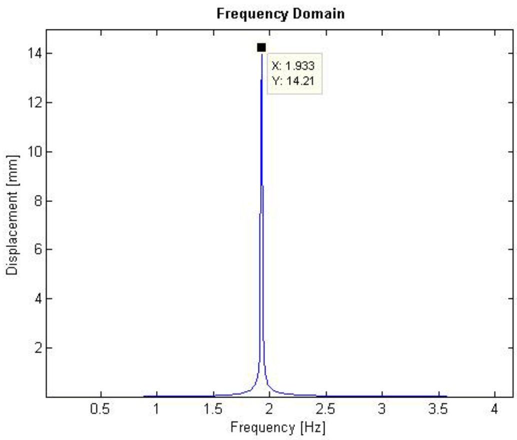


Figure 5.37 Frequency Domain of response node 1045 (Load Case UKNA C)

Figure 5.37 shows that the frequency f = 1.933 Hz dominates the behaviour of the bridge. This approaches the frequency of the load (f = 1.92998 Hz) and thus it can be considered that mode shape 4 dominates the behaviour of the bridge. No other mode shape is activated.

5.4.8 Conclusions

One can notice that the response of the bridge stays under the limits stated in the UK National Annex for both pedestrians walking and running. The crowd load here again shows much higher accelerations, up to 2 m/s², which is above the limit stated in the code.

The way the Goodwill Bridge behaves under a moving dynamic load is dependent on many factors, as is explained in Appendix 5. It has been shown that often more mode shapes are being activated, especially those whose frequency lies in the neighbourhood of the frequency of the dynamic load. However, the frequency of the load is mostly dominating the behaviour of the bridge. Considering all these facts it is not always clear why the bridge is behaving in a certain way. However, it has been shown that these responses can be expected from Arch bridges and thus can be considered as being true.

5.5 Dynamic Analysis according to the Australian Code

The Australian Standard limits the dynamic deflection caused by one pedestrian crossing the bridge at step frequency between 1.75 and 2.5 Hz.

5.5.1 Input Analysis

The load model UKNA W1 which has been used for the UK National Annex satisfies the requirements stated in the Australian Code:

- the frequency is 1.92998 Hz;
- the design pedestrian is based on a weight of 700 N;
- The load is moving over the bridge (however 1.7 m/s² could be considered on the high side).

5.5.2 Output Analysis

Unlike the Eurocode, the Australian Standard has set up criteria based on the dynamic deflection. More information can be found in paragraph 4.4 of this report. The maximum deflection caused in this situation is 0.37 mm in node 1045. The Australian Standard limits the deflection to 22 mm at a mode frequency of 2 Hz.

5.5.3 Conclusions

According to the Australian Standard, the main span of the Goodwill Bridge satisfies the requirements regarding the vibrations. One can notice that the amplitude of the deflection is very small compared to the acceptable deflection; a difference of a factor 60 can be noticed. This is 30% more than the difference between the generated maximum acceleration with the UK National Annex and its limit: a factor 45 appears to be the case. This would suggest that the criteria in the Australian Code are somewhat less conservative than the criteria from the Eurocode. The Australian Standard does not give any specification how to control the lateral excitation of the bridge.

5.6 Comparison of the Results

5.6.1 Vertical Response

Single pedestrian

The results from both Proposal Annex C and the UK National Annex for a single pedestrian are shown in Table 5.8. One can notice that the maximum acceleration caused by load model UKNA W1 is more than three times lower than the one caused by load model DLM1, which is expected.

Table 5.8 Comparison Load Models for a Single pedestrian (walking)

	Proposal Annex C	UK National Annex
Load Case	DLM1	UKNA W1
Loading time	120 s	60 s
Reference Load	280 N	280 N
DLF	1.0	1.0
Amplitude Load	280 N	280 N
Maximum acceleration	0.184 m/s²	0.015 m/s²
Max. acceleration between 0 and 60 s.	0.178 m/s²	0.015 m/s²
Criteria	≤ 0.69 m/s²	≤ 0.70 m/s²

A major difference between both load models is the Loading Time. As there is no requirement about this issue in Proposal Annex C, the bridge has been loaded until Steady State occurs. Steady State has occurred when the natural vibration of the bridge has been fully damped and when the vibration of the bridge fully follows the harmonic load. In the case of DLM1, steady state occurs at about 80 seconds, as can be seen in Figure 5.7A. In the steady state behaviour, the maximum acceleration is limited to 0.184 m/s². At 60 seconds, the maximum acceleration is 0.178 m/s², which is not much lower than at t = 80 seconds. In the case of UKNA W1, no steady state situation occurs. The Loading Time is limited by the time the load needs to cross the bridge. In this case, the load moves at a speed of 1.7 m/s. It takes then 60 seconds to cross the main span of the bridge which has a length of 102 meter.

The criteria for the maximum acceleration are the nearly the same in both cases, as is the case for the Reference Load and the Dynamic Load Factor.

Group of pedestrians

The results from both Proposal Annex C and the UK National Annex for a group of pedestrians are shown in Table 5.9. Two different situations of the UK National Annex have been compared to DLM2.

Table 5.9 Comparison Load Models for a Group of pedestrians (walking)

	Proposal Annex C	UK National Annex	
Load Case	DLM2	UKNA W2	UKNA W3
Number of pedestrians	10 ≤ N ≤ 15	N = 8	N = 16
Loading time	120 s	60 s	60 s
Reference Load	280 N	280 N	280 N
DLF	3.0	1.442	1.819
Amplitude Load	840 N	403.8 N	509.3 N
Maximum acceleration	0.552 m/s²	0.022 m/s²	0.027 m/s²
Max. acceleration between 0 and 60 s.	0.533 m/s²	0.022 m/s²	0.027 m/s²
Criteria	≤ 0.69 m/s²	≤ 0.70 m/s²	≤ 0.70 m/s²

The criteria for the maximum tolerated acceleration remain the same as in the case of a single pedestrian, as is the case for the Loading Time. Whereas the Reference Loads remain the same, the Dynamic Load Factor is much lower in the load case of the UK National Annex than in DLM2. This is mainly due to the fact that the UK National Annex takes into account a certain unsynchronisation within the group, which is expressed with parameter γ in the Dynamic Load Factor. The load model according to Proposal Annex C does not take account of this affect and thus assumes all pedestrians walk synchronised in a natural frequency of the bridge.

This situation results in a lower maximum acceleration for Load Cases UKNA2 and UKNA3 than for Load Case DLM2. The acceleration generated for DLM2 is not far from the criteria acceleration, but remains under it. The responses due to the load cases from the UK National Annex are at least ten times lower. As was the case for the Single Pedestrian load models, a lower acceleration for the load models from the UK National

Annex is expected to happen because of the moving loads and the lower dynamic load factors.

Note that there is a linear relationship between the load and the acceleration within the same code for pedestrians.

Joggers

Proposal Annex C does not propose any load model for joggers. Two cases for the load model proposed by the UK National Annex are thus being compared. The results are shown in Table 5.10.

Table 5.10 Comparison Load Models for Joggers

	UK National Annex				
Load Case	UKNA J1	UKNA J2	UKNA J3	UKNA J4	UKNA J5
Number of pedestrians	N = 2	N = 2	N = 2	N = 2	N = 2
Loading time	34 s	34 s	34 s	34 s	34 s
Reference Load	910 N	910 N	910 N	910 N	910 N
DLF	1.1895	1.238	1.105	1.105	0.496
Amplitude Load	1082.4 N	1126.6 N	1005.4 N	1005.4 N	451.4 N
Maximum acceleration	0.101 m/s²	0.182 m/s²	0.113 m/s²	0.104 m/s²	0.137 m/s²
Criteria	≤ 0.70 m/s²	≤ 0.70 m/s²	≤ 0.70 m/s²	≤ 0.70 m/s²	≤ 0.70 m/s²

Following the recommendations of the UK National Annex, only two joggers have been considered to run over the bridge. As they move with a velocity of 3 m/s, it only takes 34 seconds to cross the main span. Whereas the Reference Loads are identical, the Dynamic Load Factors highly differ. This is mainly due the difference in the factor $k(f_v)$, which has been explained in Chapter 4.

Unlike the former discussed load cases, the load amplitude and the acceleration have no direct relationship. This is due to the fact that the loads follow a different path over the bridge and that the loads do not have the same frequency.

The criteria for the acceleration remain the same as former load cases. Both generated accelerations remain under these limits.

Crowd

Both Proposal Annex C and the UK National Annex consider a crowded load on the bridge. The results of these load models are shown in Table 5.11. In both cases, the dynamic load is a non moving load. The load cases however are based on different densities.

Table 5.11 Comparison Load Models for Crowds

	Proposal Annex C	UK National Annex
Load Case	DLM3	UKNA C
Density ρ	(based on) 0.6 pers./m²	0.8 pers./m²
Loading time	120 s	60 s
Reference Load	12.6 N/m²	0.40 N/m²
DLF	3.0	23.85
Amplitude Load	37.8 N/m²	9.54 N/m²
Maximum acceleration	4.116 m/s²	2.087 m/s²
Criteria	≤ 0.69 m/s²	≤ 0.70 m/s²

The amplitude of the load is nearly four times higher in load case DLM3 than in load case UKNA C. However, the load is not applied the same way: in the case of DLM3 only the parts with a positive deflection in the mode shape have been loaded, which represent about 50% of the bridge. In the case of UKNA C, the whole main span has been loaded such that the load works positively on the deflection of the mode shape.

The criteria for the acceleration remain the same as for smaller groups of walkers and joggers. The accelerations generated in both load cases are far above these limits, up to 0.5G in the case of DLM3. UK National Annex generates lower accelerations. This is due to the lower amplitude of the load.

Comparisons of the results with measurements

These load models representing groups of pedestrians and small groups of joggers reflect the most a normal use of the bridge and can therefore be compared to the data from the measurements. More information about the measurement data can be found in Appendix 4.1.

The average acceleration over a typical use of the bridge is around 0.15 m/s², with peaks to 0.26 m/s². During the time of measurement, both joggers and walkers were making use of the bridge.

It can therefore be assumed that small groups of pedestrians cause vertical accelerations varying from about 0.1 to 0.2 m/s². In normal use, one or two joggers are regularly present on the bridge. It can be assumed that joggers cause a vertical acceleration that lies in the range 0.2 – 0.3 m/s². Note that these values are just estimations and should be used with precaution.

In Table 5.12, these values are being compared to the generated maximum accelerations from Strand7.

Table 5.12 Comparison of accelerations from load models with measurements

Load Case	Type	Generated maximum acceleration [m/s²]	Estimation from measurements [m/s²] (1)
DLM2	Group of pedestrians	0.552	0.1 – 0.2
UKNA W2	Group of pedestrians	0.022	0.1 – 0.2
UKNA W3	Group of pedestrians	0.027	0.1 – 0.2
UKNA J1	Joggers	0.101	0.2 – 0.3
UKNA J2	Joggers	0.182	0.2 – 0.3
UKNA J3	Joggers	0.113	0.2 – 0.3
UKNA J4	Joggers	0.104	0.2 – 0.3
UKNA J5	Joggers	0.137	0.2 – 0.3

(1) These values are just estimations and should be used with precaution.

Regarding the group of pedestrians, the responses from the load models from the UK National Annex seem to be low compared to measurements. The responses from Proposal Annex C deliver high accelerations compared to the measurements. Proposal Annex C tends therefore to be conservative whereas the UK National annex is the opposite. The same applies for the load models for joggers: however the generated accelerations approach more the measured data, even if they are on the low side.

There are measurements available for large groups of joggers, but none of the codes described any load models for situation. The data presented in Appendix 4.1 shows acceleration peaks up to 1.56 m/s².

5.6.2 Horizontal component

As seen in Chapter 4, the way the horizontal response is checked in both codes is fundamentally different. As in Proposal Annex C, all loads in the load cases contain a horizontal component, the UK National Annex only proposes a method to control the lateral responses due to crowd loading.

Proposal Annex C

The horizontal accelerations generated according to the load models described in Proposal Annex C are shown in Table 5.13. One can notice that no severe lateral accelerations occur, even in the case of crowded situation. Experience has proved that no severe lateral accelerations have ever occurred on the main span of the Goodwill Bridge. It can be concluded that the models give a constant prediction.

Table 5.13 Overview of the lateral accelerations for the load cases from Proposal Annex C

	DLM 1	DLM 2	DLM 3
Loading time	120 s	120 s	120 s
Reference Load	70 N	70 N	3.2 N/m²
DLF	1.0	3.0	3.0
Amplitude Load	70 N	210 N	9.6 N/m²
Maximum acceleration	0.003 m/s²	0.010 m/s²	0.051 m/s²
Criteria	0.15 m/s²	0.15 m/s²	0.15 m/s²

Note that these values should be used carefully, as the bridge has been modelled as fully restrained in lateral direction. In reality, this is probably not the case.

UK National Annex

According to Table 5.1, none of the modes below 1.5 Hz (modes 1, 2 and 3) have significant lateral displacements. The UK National Annex stipulates in this case that it can be assumed that the main span is not susceptible to have unstable lateral responses. Parameter D thus has not to be calculated in this case.

Conclusion

Both methods show that no excessive lateral acceleration is due to happen on the main span of the Goodwill Bridge. As mentioned earlier, experience has proven this situation. No data is available to control the generated accelerations with the load models from Proposal Annex C.

6 Dynamic Analysis of the Milton Road Bridge

This chapter deals about the second bridge presented in chapter 3. The Milton Road Bridge is located next to the Suncorp Stadium in Brisbane. It links the entrance of the building with the closest train station in Milton. It is generally only used in relatively crowded situations, before and after events in the stadium. Only few persons use this bridge during the rest of the time.

The model made in Strand7 is described in the first paragraph. The Natural frequency assessment is presented in the second paragraph. In the third and fourth paragraph, the dynamic analyses according to the Proposal Annex C and the UK National Annex are respectively described. The results are being compared in the last paragraph. Appendix 4.2 gives more details about this chapter.

6.1 Model

The model has been made according to the drawings that have been used for the construction of the bridge. The representation of the Strand7 model can be found in Figure 6.1.

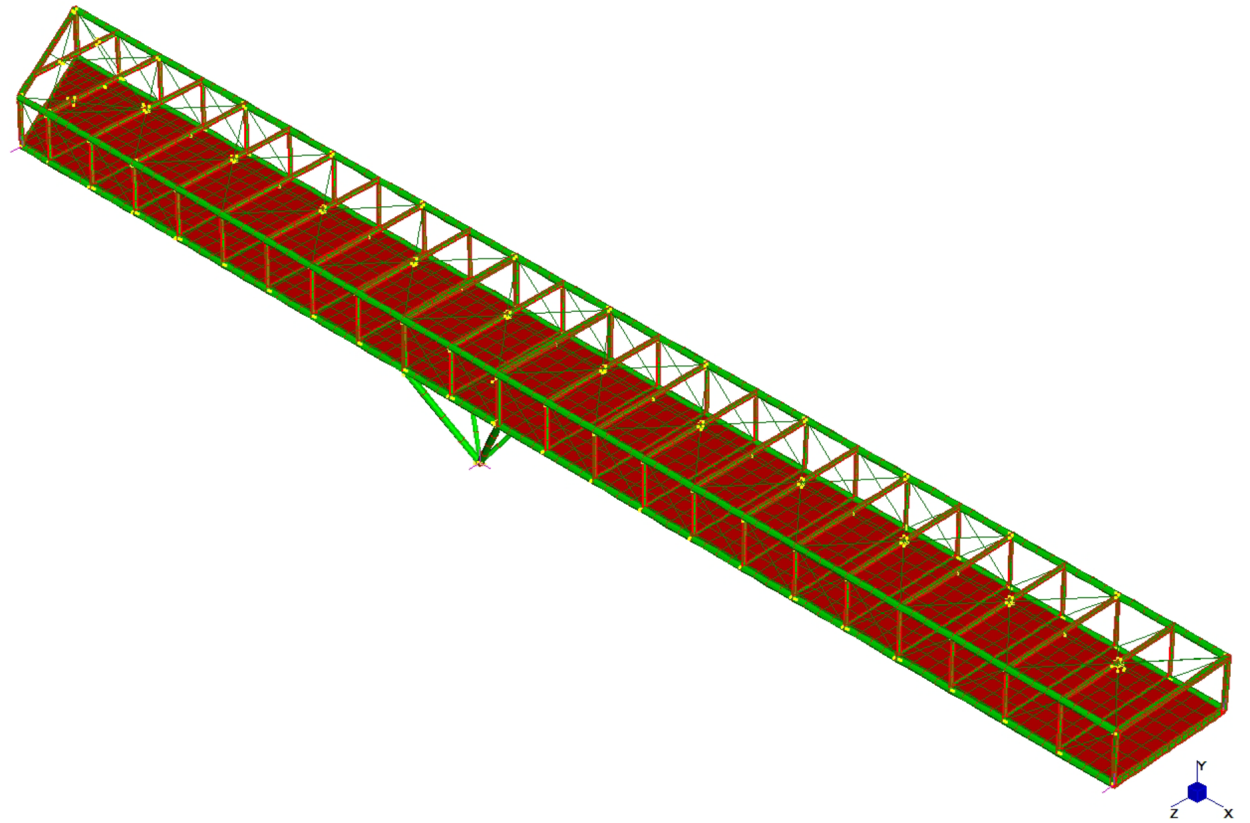


Figure 6.1 Strand7 Model of the Milton Road Bridge

The elements of the footbridge that have been modelled are:

- The top and bottom chords;
- The bottom and top cross beams (struts);
- The top wind bracings and additional bracings at both ends of the bridge;
- The longitudinal beams between the struts;
- The vertical elements between the top and bottom chords;
- The concrete deck;
- The columns at the mid-support.

The vertical elements, the wind bracings and the longitudinal beams have pin connections with their adjacent members, represent in yellow stripes in Figure 6.1. The supports have been placed and modelled as is shown in the technical drawing in Appendix 4.2.

As the concrete deck varies in thickness over the width of the bridge (from 205 to 255 mm), a mean value of 230 mm has been used to model it. Quad8 and Tri6 elements have been used for this purpose.

The supports are all fully retrained in vertical direction. Only the mid-supports are restrained in the longitudinal direction (x-direction).

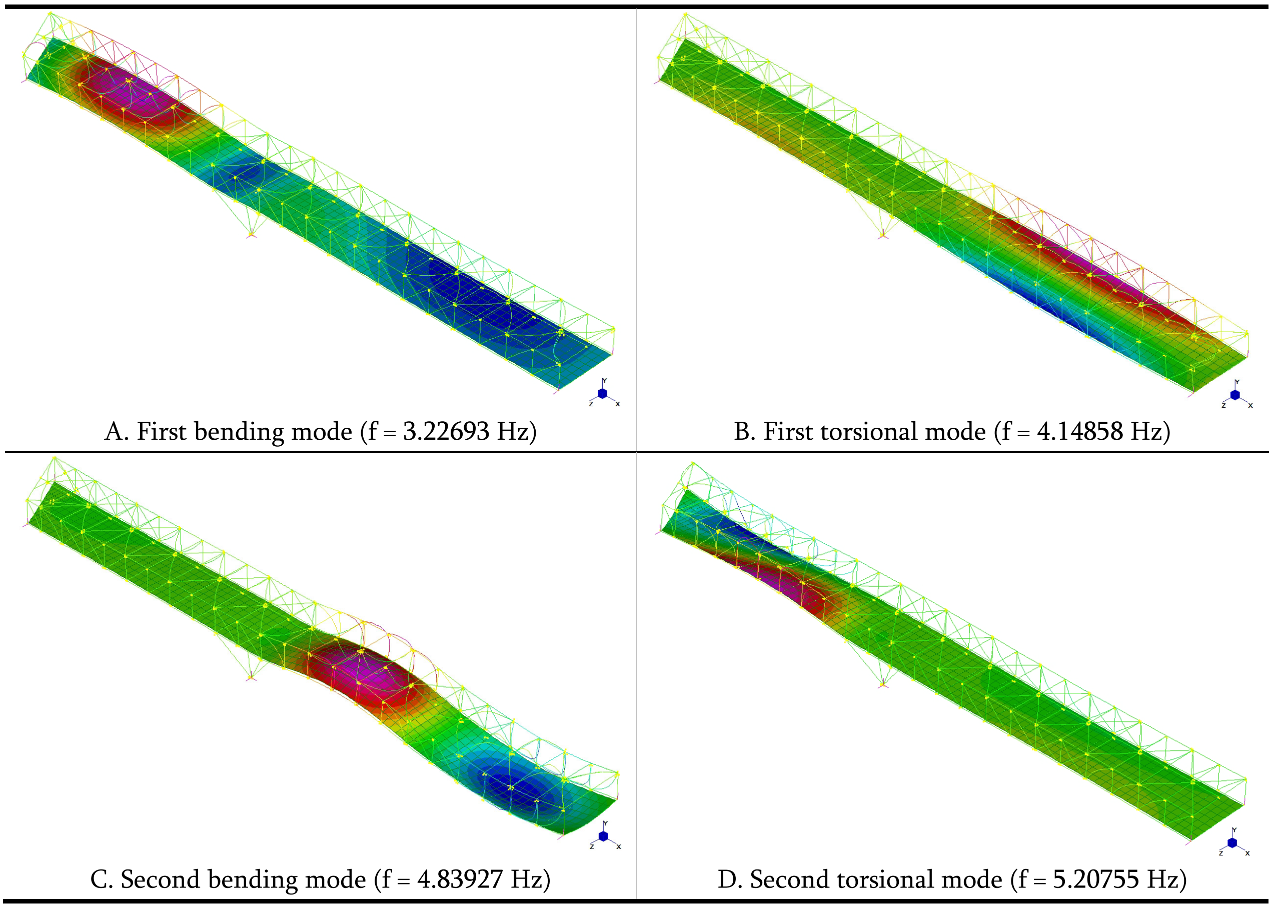
6.2 Assessment of the natural frequencies

The load models described in chapter 4 refer to the natural frequencies of the bridge. The codes stipulate to load the bridge in the natural frequencies that lie within the walking and/or running frequency range and that are the most unfavourable and most likely to occur. This paragraph describes the assessment of the natural frequencies of the Milton Road Bridge with Strand7, using the model described in the former paragraph. Chapter 2 of Appendix 4.2 gives a full overview of the assessment, with a comparison with the natural frequencies assessed in GSA.

The relevant modes are shown in Table 6.1. All other modes generated by Strand7 are local vibrations which occur within the elements and are thus not relevant for the general behaviour of the bridge.

Table 6.1 Natural Frequencies of the Main Span Goodwill Bridge according to Strand7

Mode #	f_v [Hz]	Remarks	Refers to
147	3.22693	First bending mode	Figure 6.2A
152	4.14858	First torsional mode	Figure 6.2B
153	4.83927	Second bending mode	Figure 6.2C
154	5.20755	Second torsional mode	Figure 6.2D



A. First bending mode (f = 3.22693 Hz)

B. First torsional mode (f = 4.14858 Hz)

C. Second bending mode (f = 4.83927 Hz)

D. Second torsional mode (f = 5.20755 Hz)

Figure 6.2 Relevant mode shapes

In contrast to the Goodwill Bridge, these modes can distinctly be distinguished in bending and torsional modes. This is mainly due to the symmetrical cross section of the bridge. The lateral displacements are near to zero in all of these cases. The wind bracings on the top and the large concrete deck on the bottom cause the structure to be very stiff in this direction.

One can note that the mode frequencies are relatively high, especially compared to the ones of the Goodwill Bridge. The mode shapes shown in Figure 6.2 are the ones that are the most relevant for this study, as their frequency lie under or near 5 Hz. Theoretically, these values are not within the walking frequency range which lies between 1.3 and 2.4 Hz. However the first bending mode lies within the running frequency range (2 – 3.5 Hz).

The first torsional mode is particularly interesting because its most relevant node (the node with the largest displacement) is exactly situated on the place where the highest vibrations have been noticed in practice. Even though its mode frequency is outside any walking or running frequency range, this mode shape should be considered in the analyses.

The first bending mode and the first torsional mode are therefore the modes that should be considered in the analyses.

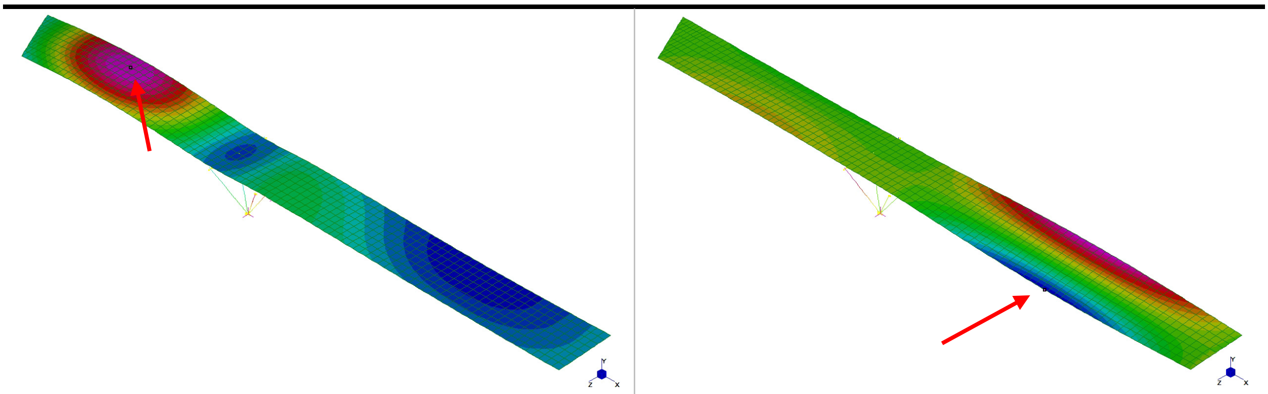
6.3 Dynamic Analysis according to Proposal Annex C

6.3.1 Considered mode shapes

According to Proposal Annex C, only the mode shape with a mode frequency that lies the nearest to 2 Hz should be considered in the analysis. This would in this case be the first bending mode, which has a natural frequency of 3.22693 Hz. As mentioned in the former paragraph, the first torsional mode also seems to be an interesting mode and is therefore considered in the analysis. Table 6.2 and Figure 6.3 give an overview of the critical nodes for each of the modes.

Table 6.2 Nodes with largest vertical displacement for modes 147 and 152

Mode #	Frequency [Hz]	Node #	Vertical displacement [mm]	Horizontal (lateral) displacement [mm]
147	3.22693	1508	3.1798	0.0682
152	4.14858	2617	4.1687	0.2560



Node 1508 (Mode 147) | Node 2617 (Mode 152)

Figure 6.3 Position of critical nodes in mode 147 and mode 152

For the horizontal component of the dynamic load, the horizontal mode frequency that lies the nearest to 1 Hz should be considered. The Milton Road Bridge has a relatively high lateral stiffness and therefore there is no lateral mode shape near 1 Hz. However, small lateral displacements can be noticed in the first bending and first torsional modes, which are in an order of 6 to 15 smaller than the maximum vertical displacements of these modes. Therefore, for the horizontal component, the frequency of the considered mode shape will be used in the analysis.

6.3.2 Dynamic Loads

For Load Cases DLM1 and DLM2, the dynamic forces that have to be applied are in the form of:

- Vertical: $Q_{pv} = 280\, k_v(f_v)\sin(2\pi\, f\, t)$ [N]
- Lateral: $Q_{ph} = 70\, k_h(f_h)\sin(2\pi\, f\, t)$ [N]

For Load Case DLM3, the following dynamic force should be applied:

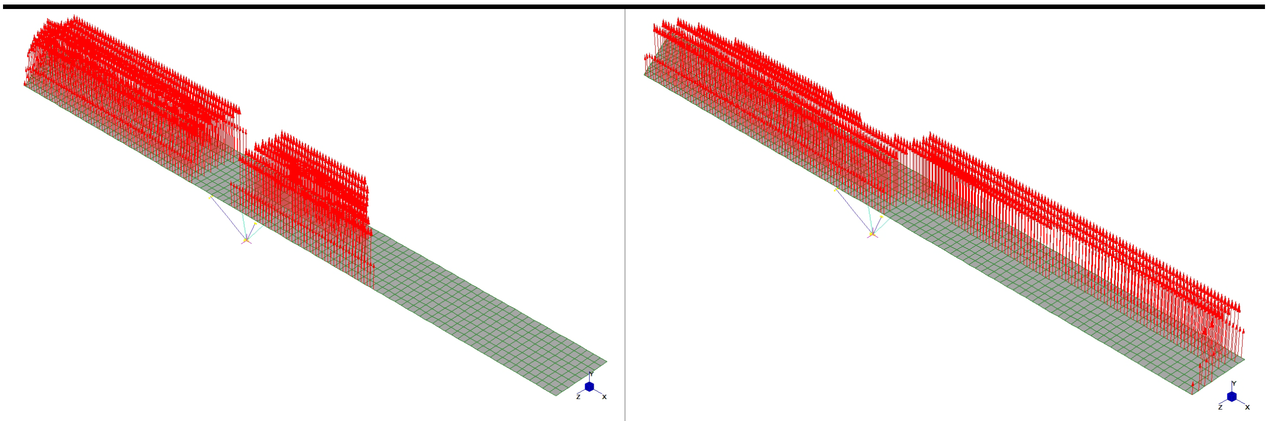
- Vertical: $q_{sv} = 12.6\, k_v(f_v)\sin(2\pi\, f\, t)$ [N/m²]
- Horizontal: $q_{sh} = 3.2\, k_h(f_h)\sin(2\pi\, f\, t)$ [N/m²]

Table 6.3 gives an overview of the load cases and the forces that should be applied.

Table 6.3 Load Situations to be applied to the Milton Road Bridge (Pr. Annex C)

Load Case	Applied on Node#	Frequency f [Hz]	Vertical amplitude [N] or [N/m²]	Horizontal amplitude [N] or [N/m²]
DLM1A	1508	3.22693	280	70
DLM1B	2617	4.14858	280	70
DLM2A	1508	3.22693	280 * 0.943 = 264	70 * 0.5 = 35
DLM2B	2617	4.14858	280 * 0.713 = 200	70 * 0.5 = 35
DLM3A	Relevant nodes (1)	3.22693	12.6 * 0.943 = 11.9	3.2 * 0.5 = 1.6
DLM3B	Relevant nodes (1)	4.14858	12.6 * 0.713 = 9.0	3.2 * 0.5 = 1.6

(2) Proposal Annex C states that in the case of DLM3 only relevant areas of the footbridge deck should be loaded, so that is as most unfavourable. In this case, only the parts with a positive (negative would have had the same effect) displacement in the mode shape of Modes 147 and 152 have been loaded, as can be seen in Figure 6.4. As the vertical and horizontal should be coupled, the vertical displacements have been considered as normative.



DLM3A DLM3B

Figure 6.4 Parts of the bridge that are loaded in DLM3 (The horizontal loads have been placed on the same nodes, see Appendix 4.2)

It becomes clear from Table 6.3 that load case DLM1 is always decisive compared to load case DLM2. This is to the high natural frequencies which cause low Dynamic Load Factors (below 1). Therefore, load case DLM2 will not be considered for the rest of this analysis. The representation of all other load cases can be found in Appendix 4.2.

Each load is being applied for 120 seconds, a time that should be enough to have a steady state situation. No load time is mentioned in Proposal Annex C.

6.3.3 Estimation damping

As for the Goodwill Bridge, the exact damping ratio ζ of the Milton Road Bridge is unknown and has therefore to be estimated. This bridge can be considered as a composite bridge, as the steel structure and the concrete deck are working together for the stiffness as well as for the distribution of forces. Proposal Annex C proposes to use a logarithmic decrement of 0.04, which corresponds to a damping ratio of 0.006 (see table 4.3). This is the mean value of the damping ratio found by Bachmann (see table 2.1). Bachmann also gives a minimum value for the damping ratio of composite bridges which is 0.003. Considering the fact that practice has shown that only small sensible vibrations occur and the fact that the bridge is partly welded, a damping ratio of 0.005 can be considered as reasonable for this bridge and is therefore used in the analyses.

6.3.4 Output

Further input for the analysis can be found in Appendix 4.2. This paragraph gives an overview of the output for each load situation discussed earlier. According to Proposal Annex C, the vertical acceleration should be limited to the smallest of these values:

- 0.7 m/s²
- $0.5 \times \sqrt{f_v} = 0.5 \times \sqrt{3.22693} = 0.90$ m/s²

For the lateral (horizontal) direction, the acceleration should be limited to the smallest of these values:

- 0.15 m/s²
- $0.14 \times \sqrt{f_h} = 0.14 \times \sqrt{3.22693} = 0.25$ m/s²

Note that the values for the maximum lateral accelerations are lower than the ones mentioned in Eurocode 0 (see chapter 4).

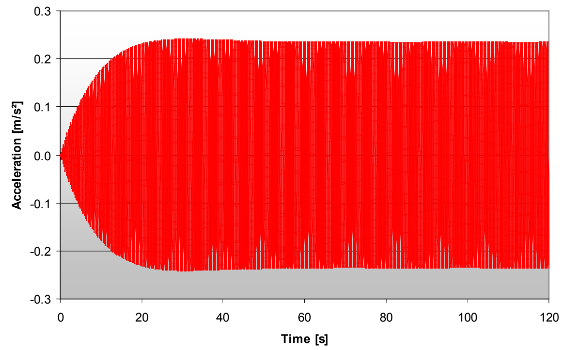
The results of the analyses are summarized in Table 6.4.

Table 6.4 Results Strand7 analyses for Pr. Annex C

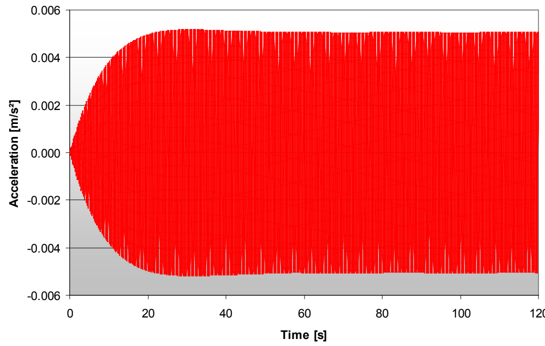
Situation	Vertical acceleration	in node #	Criteria vertical acceleration	Lateral acceleration	in node #	Criteria lateral acceleration
DLM1A	0.24 m/s² ✓	1508	≤ 0.70 m/s²	0.005 m/s² ✓	1508	≤ 0.15 m/s²
DLM1B	0.36 m/s² ✓	2617	≤ 0.70 m/s²	0.022 m/s² ✓	2617	≤ 0.15 m/s²
DLM3A	1.52 m/s² ×	1508	≤ 0.70 m/s²	0.033 m/s² ✓	1508	≤ 0.15 m/s²
DLM3B	0.98 m/s² ×	2617	≤ 0.70 m/s²	0.062 m/s² ✓	2617	≤ 0.15 m/s²

✓ Acceleration is under the tolerated acceleration, × Acceleration is above the tolerated acceleration

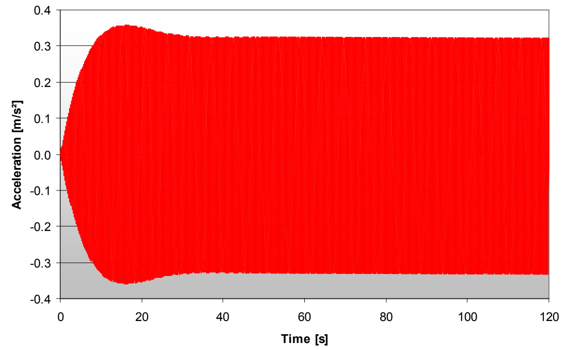
Graphics of the responses generated by Strand7 can be found in Figure 6.5.



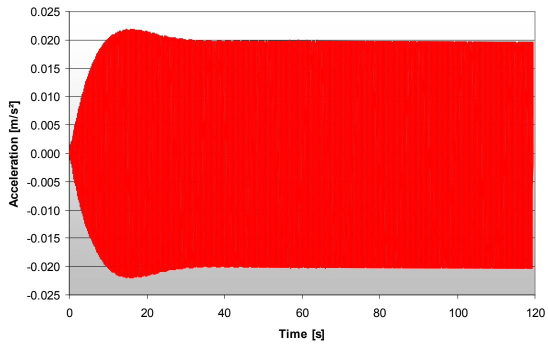
A. Vertical acceleration of node 1508 (DLM1A)



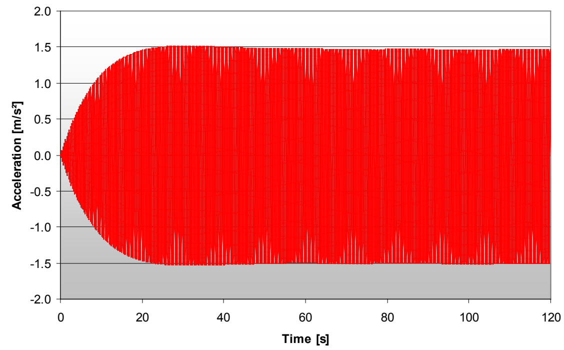
B. Lateral acceleration of node 1508 (DLM1A)



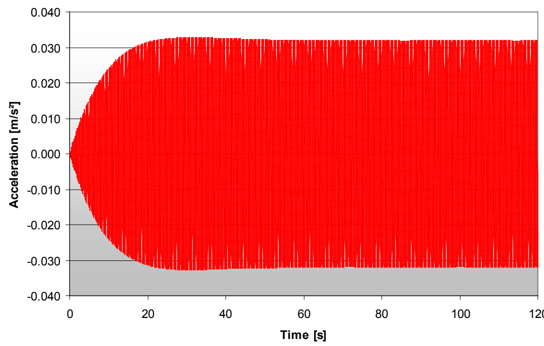
C. Vertical acceleration of node 2617 (DLM1B)



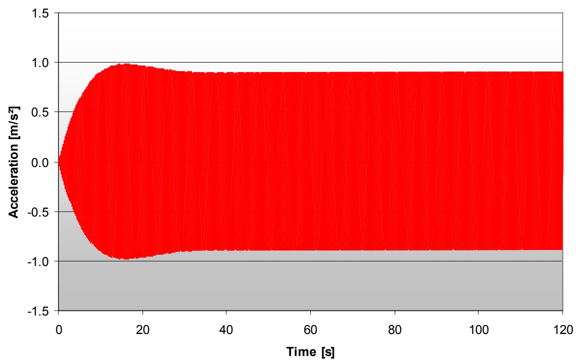
D. Lateral acceleration of node 2617 (DLM1B)



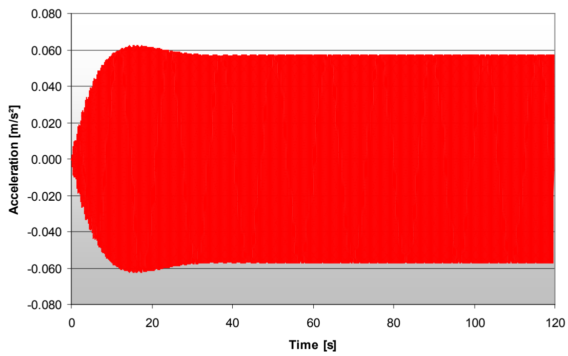
E. Vertical acceleration of node 1045 (DLM3A)



F. Lateral acceleration of node 1045 (DLM3A)



G. Vertical acceleration of node 2617 (DLM3B)



H. Lateral acceleration of node 2617 (DLM3B)

Figure 6.5 Vertical and horizontal accelerations of nodes 1045 and 2617 for DLM1 and DLM3

6.3.5 Analysis of Output

This paragraph aims to clarify the response of the bridge for each load case. Considering Figure 6.5, the vertical and horizontal accelerations seem to be quite evident: the response of the critical node is growing until steady state is reached. This is a behaviour that is logical and expected.

With a Fourier analysis, one can determine which frequencies influence the response and thus which modes are activated by the harmonic load. Note that the Fourier analysis has been done on the displacement of the node as this delivers more accurate results. Appendix 3.2 gives more information about this.

Response DLM1A

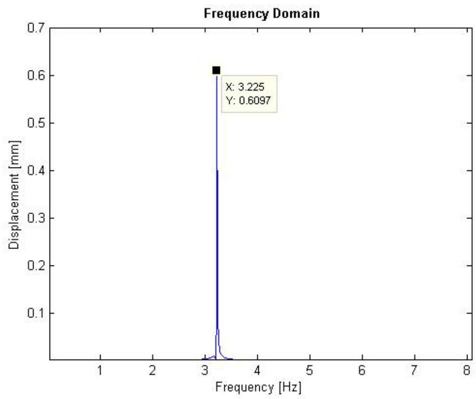


Figure 6.6 Frequency Domain of vertical response DLM1A

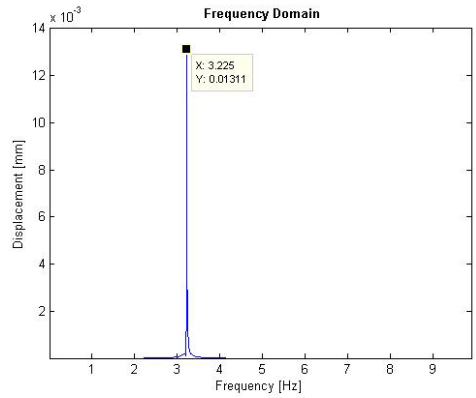


Figure 6.7 Frequency Domain of horizontal response DLM1A

Figure 6.6 and Figure 5.9:

These graphs clearly show that the only frequency that occurs is 3.225 Hz. This corresponds to the frequency of the harmonic load in vertical and horizontal direction ($f_v = f_h = 3.22693$ Hz). Mode shape 147 is thus the only mode that is activated by these two loads.

Response DLM1B

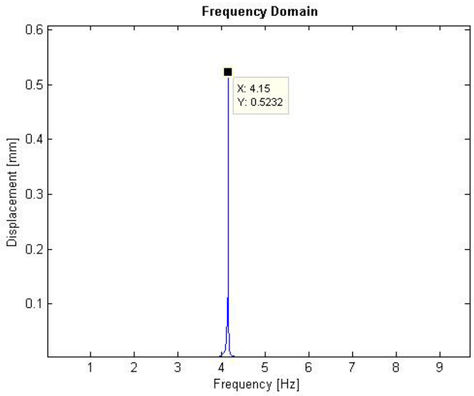


Figure 6.8 Frequency Domain of vertical response DLM1B

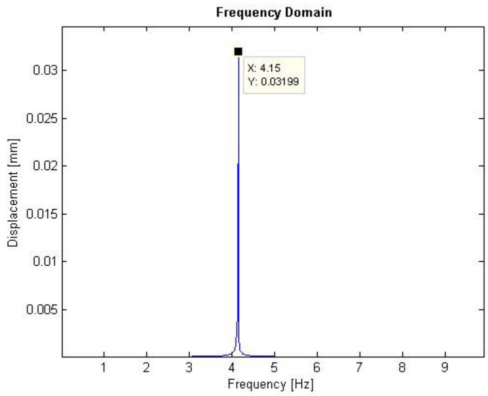


Figure 6.9 Frequency Domain of horizontal response DLM1B

Figure 5.10 and Figure 5.11:

These graphs clearly show that the only frequency that occurs is 4.15 Hz. This corresponds to the frequency of the harmonic load in vertical and horizontal direction ($f_v = f_h = 4.14858$ Hz). Mode shape 152 is thus the only mode that is activated by these two loads.

Response DLM2A & DLM2B

These responses are exactly the same as load cases DLM1A and DLM1B, except for the magnitude. The same frequencies as in DLM1 are thus activated.

Response DLM3A

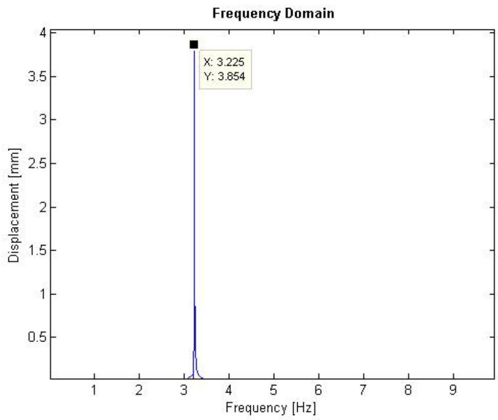


Figure 6.10 Frequency Domain of vertical response DLM3A

Figure 5.12 and Figure 5.13:

These graphs clearly show that the only frequency that occurs is 3.225 Hz. This corresponds to the frequency of the harmonic load in vertical and horizontal direction ($f_v = f_h = 3.22693$ Hz). Mode shape 147 is thus the only mode that is activated by these two loads.

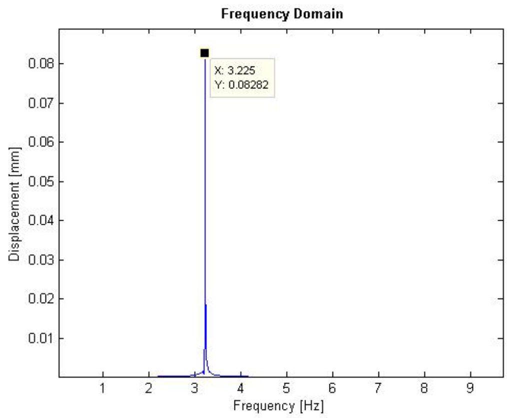


Figure 6.11 Frequency Domain of horizontal response DLM3A

Response DLM3B

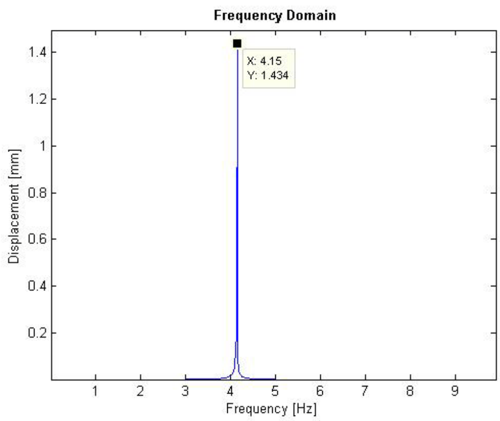


Figure 6.12 Frequency Domain of vertical response DLM3B

<u>Figure 5.14 and Figure 5.15:</u>
These graphs clearly show that the only frequency that occurs is 4.15 Hz. This corresponds to the frequency of the harmonic load in vertical and horizontal direction ($f_v = f_h = 4.14858$ Hz). Mode shape 152 is thus the only mode that is activated by these two loads.

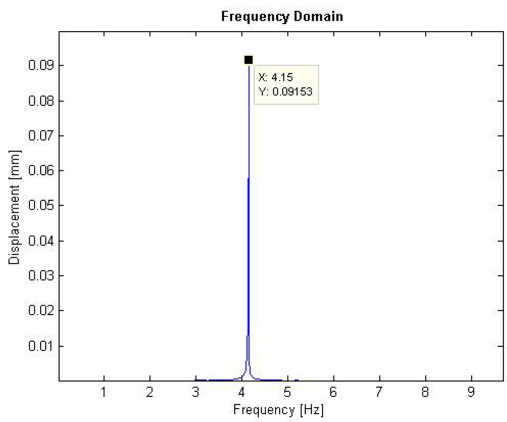


Figure 6.13 Frequency Domain of horizontal response DLM3B

6.3.6 Conclusions

The load case generated by a single pedestrian (DLM1) is always decisive in relation to load case DLM2. This is due to the fact that the natural frequencies of the bridge are relatively high, which cause the Dynamic Load Factor being rather low: pedestrians walking or jogging in pace with these frequencies is small, especially groups walking in pace. Steady-state occurs at about 30 seconds after the beginning of the loading.

In both load cases DLM1A and DLM1B, the accelerations remain below the limits stated in the code, vertically as well as horizontally. In the case of DLM1B, the acceleration of 0.36 m/s² could confirm the practice: no extreme but sensible vibrations. Load Cases DLM3A and DLM3B however show high vertical accelerations up to 1.52 m/s². As for the Goodwill Bridge, these accelerations are above the limit stated in code. This would suggest that intolerable accelerations would occur when the bridge is used by crowds. Practice has shown this is not the case. Horizontally, the accelerations remain below the limits.

6.4 Dynamic Analysis according to UK National Annex

The UK National Annex proposes moving loads (vertical only) for pedestrians which are moving along the most unfavourable line across the bridge. Both walking persons and joggers are considered. The velocity the dynamic load is moving is dependant on the type of pedestrian. Like in Proposal Annex C, a crowded situation has to be considered.

The Dynamic Load Factors which are applied to the loads vary according to the type of pedestrian (walker, jogger), the number of pedestrians, the effective span of the bridge (which is dependant of the mode shape) and the frequency of the considered mode. This has been described earlier in Chapter 4.

6.4.1 Considered mode shapes

As for Proposal Annex C, the two main modes (the first bending mode and the first torsion mode) are being considered for all load cases (walking, jogging and crowd), even though both lie outside the walking frequency range. The most unfavourable path for each mode has been represented in Figure 6.14. As is the case for Proposal Annex C, the load case for crowds is a non moving load and the load is equally spread over the bridge. Only the load cases from walking pedestrians and joggers are moving loads which use the most unfavourable paths.

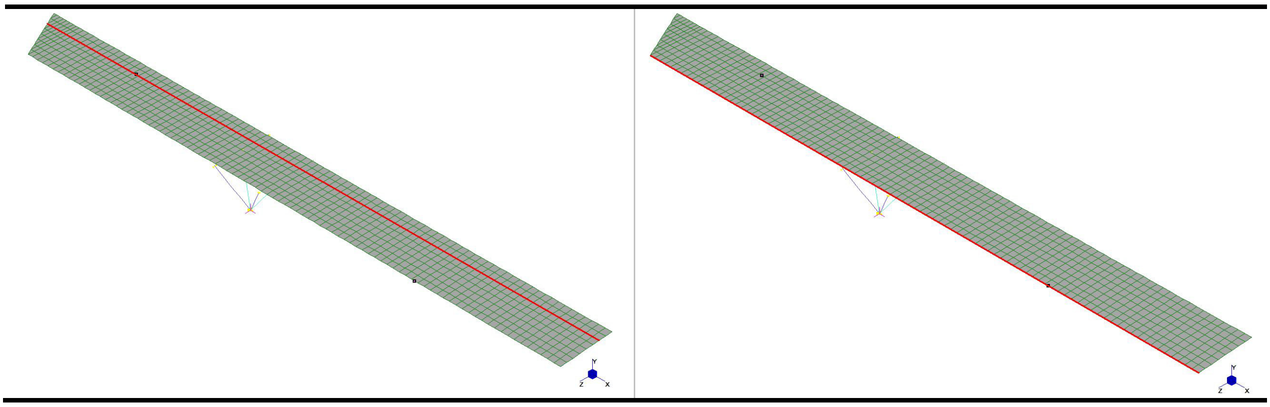


Figure 6.14 Most unfavourable path for the First bending Mode (left) and the First torsional Mode (right)

6.4.2 Bridge Class

The bridge has been designed to link the local Milton Train Station with the Brisbane Suncorp Stadium. Even though it has been built in an urban area, it is usually only used before and after events in the stadium (sport events, concerts etc.). The rest of the time, the bridge is only used by few pedestrians or joggers. According to the UK National Annex, the Goodwill Bridge can therefore best be categorized in Bridge Class D. Table NA.7 recommends the following values to be considered in the analyses:

- Group walking: N = 16
- Group jogging: N = 4
- Crowd density: ρ = 1.5 persons/m²

However, to be able to compare the results with Proposal Annex C, the following case will also be considered:

- Group walking: N = 1 (single pedestrian)

6.4.3 Load Cases to be considered

Table 6.5 summarizes the load cases which are being analysed in Strand7.

Table 6.5 Load cases considered for the dynamic analysis according the UK National Annex

Load Case #	Mode #	Frequency [Hz]	Through Node #	Type	N [pers.] / ρ [pers./m²]
UKNA W1	147	3.22693	1508	Walking	1
UKNA W2	147	3.22693	1508	Walking	16
UKNA W3	152	4.14858	2617	Walking	1
UKNA W4	152	4.14858	2617	Walking	16
UKNA J1	147	3.22693	1508	Jogging	4
UKNA J2	152	4.14858	2617	Jogging	4
UKNA C1	147	3.22693	All (1)	Crowd	1.5
UKNA C2	152	4.14858	All (1)	Crowd	1.5

(1) All nodes needs to be as most unfavourable possible. This means that all forces on the nodes with a positive vertical displacement in mode shape 147 or 152 should be the opposite sign of the forces placed on nodes with a negative displacement, as shown in Figure 6.15.

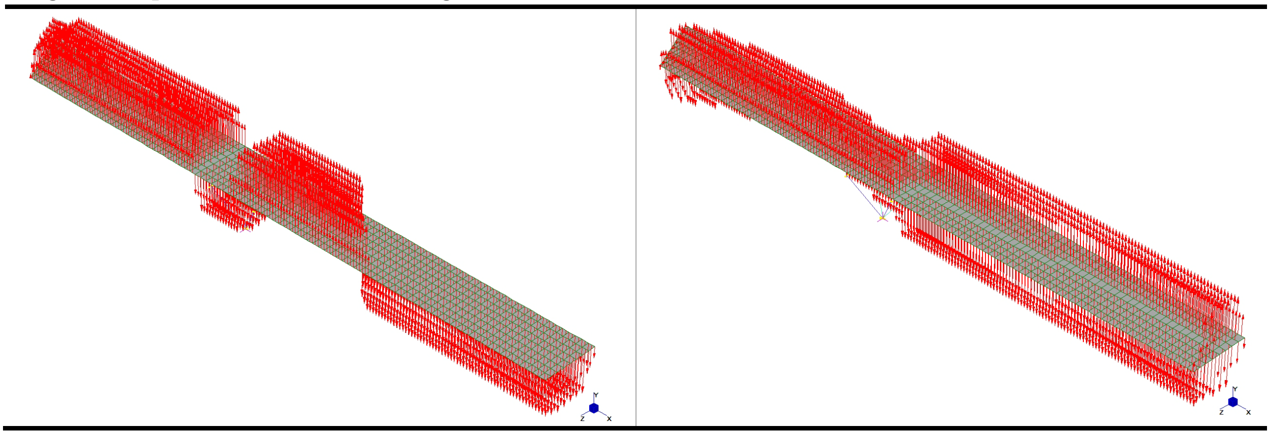


Figure 6.15 Parts of the bridge that are loaded in Load Cases UKNA C1 & UKNA C2

6.4.4 Damping

The same damping ratio as in the analyses of Proposal Annex C is used to run these analyses: $\zeta = 0.005$.

6.4.5 Applied Loads

As written earlier, the amplitude of the loads is dependant of several parameters. Table 6.6 gives an overview of the loads and the Dynamic Load Factors which should be applied to each Load Case. More details about the calculations can be found in Appendix 4.1.

Table 6.6 Amplitudes of Loads for each Load Case

Load case	Reference Load F_0 [N]	DLF [-] (1)	Amplitude Load [N] (2)	Frequency Load [Hz]
UKNA W1	280	0.320	89.6	3.22693
UKNA W2	280	0.638	178.7	3.22693
UKNA W3	280	0.340	95.2	4.14858
UKNA W4	280	0.668	187.1	4.14858
UKNA J1	910	0.455	414.1	3.22693
UKNA J2	910	0.226	205.7	4.14858
UKNA C1	See Appendix 4.2			3.22693
UKNA C2	See Appendix 4.2			4.14858

(1) DLF stands for Dynamic Load Factor
(2) Amplitude Load = Reference Load * DLF

6.4.6 Output

The results of the analyses are summarized in Table 6.7.

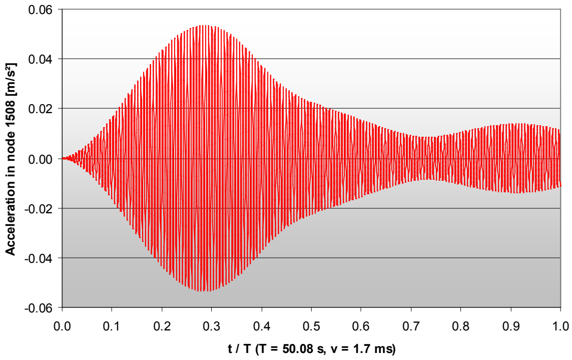
Table 6.7 Results Strand7 analyses for UK National Annex

Load Case	Vertical acceleration	in node #	Criteria vertical acceleration
UKNA W1	0.053 m/s² ✓	1508	≤ 0.8 m/s²
UKNA W2	0.107 m/s² ✓	1508	≤ 0.8 m/s²
UKNA W3	0.110 m/s² ✓	2617	≤ 0.8 m/s²
UKNA W4	0.216 m/s² ✓	2617	≤ 0.8 m/s²
UKNA J1	0.182 m/s² ✓	1508	≤ 0.8 m/s²
UKNA J2	0.202 m/s² ✓	2617	≤ 0.8 m/s²
UKNA C1	0.428 m/s² ✓	1508	≤ 0.8 m/s²
UKNA C2	0.456 m/s² ✓	2617	≤ 0.8 m/s²

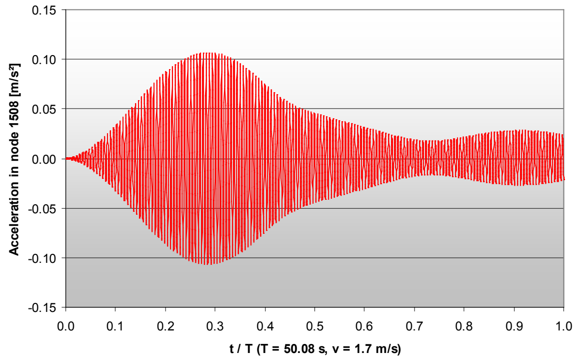
✓ Acceleration is under the tolerated acceleration, ✗ Acceleration is above the tolerated acceleration

Load Cases UKNA W1, UKNA W2, UKNA W3 and UKNA W4

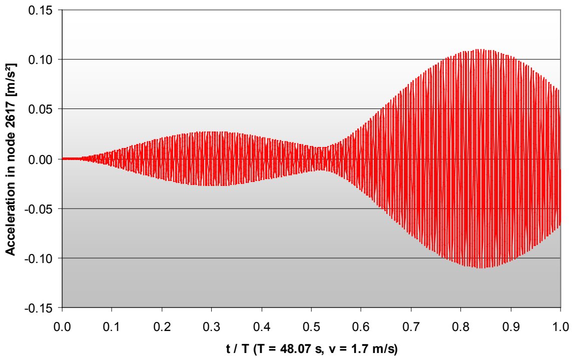
The responses generated by Strand7 for these load cases in the corresponding critical nodes can be found in Figure 6.16.



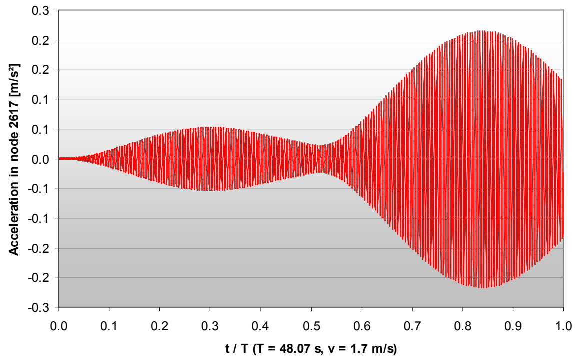
A. UKNA W1 (Critical node 1508 at 0.196)



B. UKNA W2 (Critical node 1508 at 0.196)



C. UKNA W3 (Critical node 2617 at 0.809)



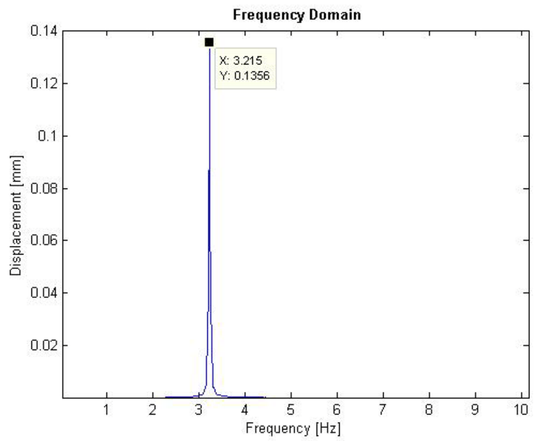
D. UKNA W4 (Critical node 2617 at 0.809)

Figure 6.16 Response of the bridge in the critical nodes for Load Cases UKNA W1 to UKNA W4 (Pedestrians)

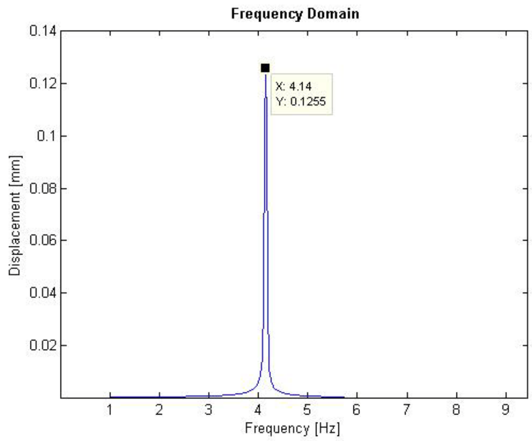
To have a better understanding of the responses of the bridge under moving dynamic loads, the frequency domain of each of the displacement of the critical nodes are being assessed. This shows which mode shapes are being activated by the moving load. Figure 6.17 shows the Frequency Domain of the responses for Load Cases UKNA W1 and UKNA W3. Note that the Frequency Domain of UKNA W2 is similar to UKNA W1, only the amplitude changes. The same applies to UKNA W4 which is similar to UKNA W3.

One can clearly notice that the modes that are activated correspond to the modes which frequency is nearby the frequency of the load. No other mode is being activated.

The maximum acceleration in each load case occurs when the load is in the neighbourhood of the critical node, as would be expected. The fact that it is not occurring at the exact moment the load is passing the critical node can be the result of the velocity of the load, as explained in Appendix 3.2.



A. Frequency Domain UKNA W1

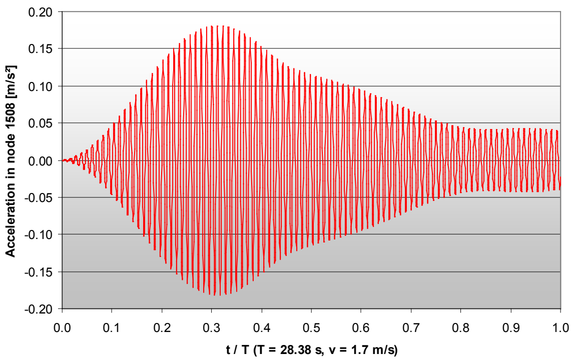


B. Frequency Domain UKNA W3

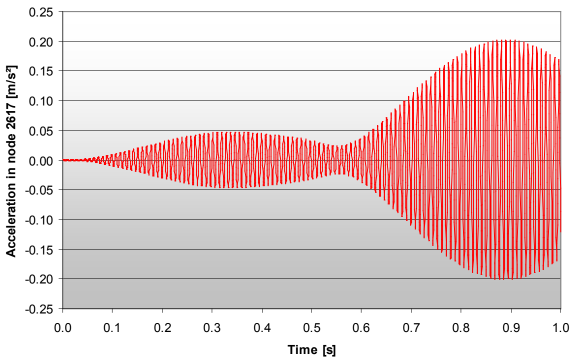
Figure 6.17 Frequency Domains of the responses of UKNA W1 and UKNA W3

Load Cases UKNA J1 and UKNA J2

The responses generated by Strand7 for these load cases in the corresponding critical nodes can be found in Figure 6.18.



A. UKNA J1 (Critical node 1508 at 0.196)



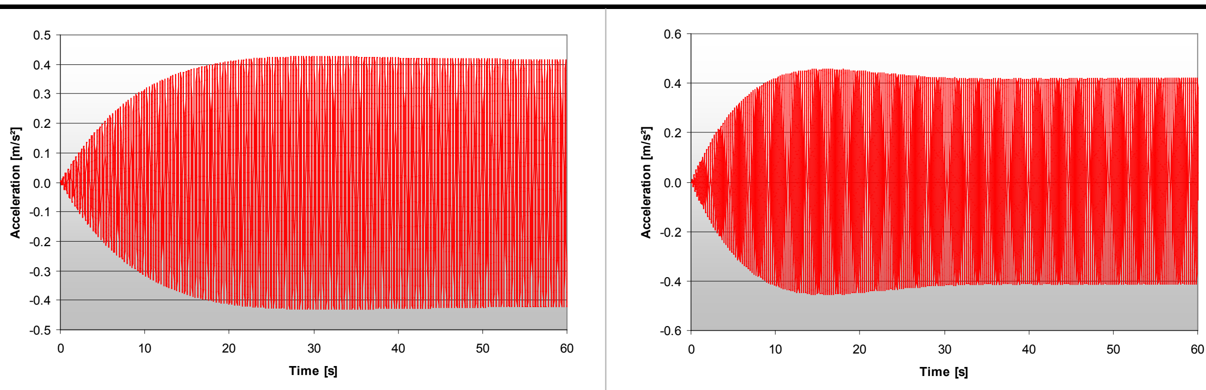
B. UKNA J2 (Critical node 2617 at 0.809)

Figure 6.18 Response of the bridge in the critical nodes for Load Cases UKNA J1 and UKNA J2 (Joggers)

These generated accelerations look like the ones generated by the load cases for walking pedestrians. Only the amplitude changes and the moment the largest acceleration in the critical node occurs change. The fact that the maximum acceleration occurs somewhat later confirms the fact that it is probably due the velocity of the speed. The higher magnitude is logical and expected with larger load amplitudes. For these responses, it is therefore not needed to assess the frequency domain. It can be stated that the frequency domains are similar to the ones from Figure 6.17.

Load Cases UKNA C1 and UKNA C2

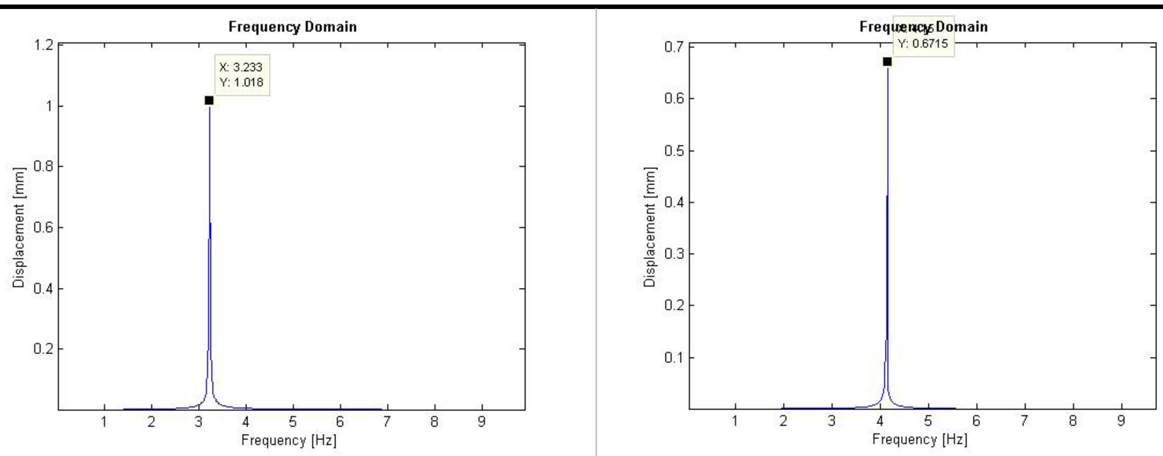
The responses generated by Strand7 for these load cases in the corresponding critical nodes can be found in Figure 6.19. The particularity of this load model is that the load is not moving over the bridge.



A. UKNA C1 (in decision node 1508) | *B. UKNA C2 (in critical node 2617)*

Figure 6.19 Response of the bridge in the critical nodes for Load Case UKNA C1 and UKNA C2 (Crowd)

The Frequency Domain of each of these responses is represented in Figure 6.20.



A. Frequency Domain UKNA C1 | *B. Frequency Domain UKNA C2*

Figure 6.20 Frequency Domains of the responses of UKNA C1 and UKNA C2

It can be concluded from Figure 6.20 that only the mode corresponding to the frequency of the load is activated, as is the case in the former load cases.

6.4.7 Conclusions

In all cases, the response of the bridge stays under the limits stated in the UK National Annex. The largest accelerations are generated by Load Case UKNA C2, which represents a crowd situation. However, one should consider that the natural frequencies are relatively high. These accelerations are therefore not likely to occur during normal use of the bridge.

Unlike the Goodwill Bridge, in this situation only the modes are activated which have a frequency similar to the frequency of the load applied. The maximum acceleration in the cases of moving loads occurs when the load is in the neighbourhood of the critical node. It generally occurs somewhat later: this is probably due to the velocity of the speed.

6.5 Dynamic Analysis according to the Australian Code

The Australian Standard limits the dynamic deflection caused by one pedestrian crossing the bridge at step frequency between 1.75 and 2.5 Hz.

6.5.1 Input Analysis

None of the load models from the UK National Annex or Proposal Annex C corresponds to this load situation. The bridge is not loaded in its natural frequency in this situation. The most unfavourable frequency between 1.75 and 2.5 Hz should therefore be considered.

Considering the First bending mode and the First torsional mode, the most obvious would be to load the bridge with a frequency which is as near as possible of one of the natural frequencies. That means that the bridge should be loaded at a frequency of 2.5 Hz, which is the nearest to the frequency of the First bending Mode (f_v = 3.22693 Hz). The critical node should then be node 1508.

The Australian Code does not mention any specific Dynamic Load Factor. That is why a DLF of 1 is being used (as is the case for DLM1A). The amplitude of the dynamic load is 280 N, based on a pedestrian of 700 N. As was the case for the Goodwill Bridge, the load is moving with a velocity of 1.7 m/s.

6.5.2 Output Analysis

Figure 6.21 shows the displacements caused by the dynamic load. Unlike the Eurocode, the Australian Standard has set up criteria based on the dynamic deflection.

The maximum deflection caused in this situation is 0.02 mm in node 1508, which is far beneath the limit stated in the Australian Code (22 mm at a frequency of 2 Hz).

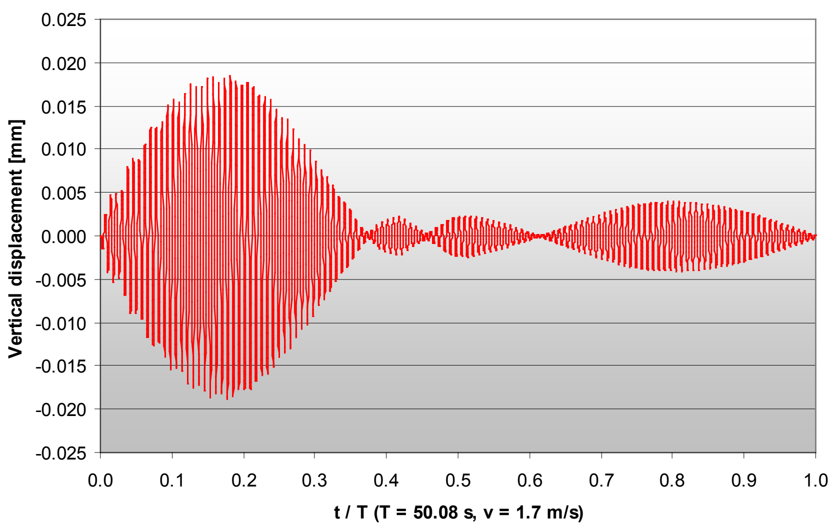


Figure 6.21 Displacement node 1508

6.5.3 Conclusions

According to the Australian Standard, the Milton Road Bridge satisfies the requirements regarding vibrations. The Australian Standard does not give any specification how to control the lateral excitation of the bridge. A deeper analysis of the code will be done in Chapter 7, as the difference between the generated and the tolerated displacement seems to be very large.

6.6 Comparison of the Load Models and the Results

6.6.1 Vertical Response

Single pedestrian

The results from both Proposal Annex C and the UK National Annex for a single pedestrian are shown in Table 6.8.

Table 6.8 Comparison Load Models for a Single pedestrian (walking)

	Proposal Annex C	UK National Annex	Proposal Annex C	UK National Annex
Relevant node #	1508	1508	2617	2617
Load Case	DLM 1A	UKNA W1	DLM 1B	UKNA W3
Loading time	120 seconds	50.08 seconds	120 seconds	48.07 seconds
Reference Load	280 N	280 N	280 N	280 N
DLF	1.0	0.32	1.0	0.34
Amplitude Load	280 N	89.6 N	280 N	95.2 N
Maximum acceleration	0.24 m/s²	0.053 m/s²	0.36 m/s²	0.110 m/s²
Criteria	≤ 0.70 m/s²	≤ 0.80 m/s²	≤ 0.70 m/s²	≤ 0.80 m/s²

The accelerations generated by Proposal Annex C are 3 to 5 larger than the ones generated by the UK National Annex. This is mainly due to the Dynamic Load Factors, which are substantially lower in the cases of the UK National Annex, and in less extend to the fact that the load is moving in these load cases. For Proposal Annex C, the maximum accelerations are the ones that occur at Steady State, after about 30 seconds of loading time. Steady State does not occur for the load cases of the UK National Annex. In these cases, the maximum acceleration occurs at the time that the load is passing the most relevant node of the considered mode shape.

One should note that only DLM 1A should be considered according to Proposal Annex C, as the frequency is the one that approaches the most the average walking frequency of 2 Hz.

The analysis according to the Australian Standard, which only considers one pedestrian walking over the bridge and which is not mentioned in Table 6.8, shows displacements that are far below the limits. In this Load Case, the bridge has been loaded with a frequency that is half of the natural frequency of the First bending Mode. This is thus a frequency at which no resonance occurs: the displacements (as well as the accelerations) remain relatively low.

Group of pedestrians

The results from both Proposal Annex C and the UK National Annex for a group of pedestrians are shown in Table 6.9.

Table 6.9 Comparison Load Models for a Group of pedestrians (walking)

	Proposal Annex C	UK National Annex	Proposal Annex C	UK National Annex
Relevant Node #	1508	1508	2617	2617
Load Case	DLM 2A	UKNA W2	DLM 2B	UKNA W4
Number of pedestrians	10 ≤ N ≤ 15	N = 16	10 ≤ N ≤ 15	N = 16
Loading time	120 seconds	50.08 seconds	120 seconds	48.07 seconds
Reference Load	280 N	280 N	280 N	280 N
DLF	0.943	0.638	0.713	0.668
Amplitude Load	264 N	178.7 N	200 N	187.1 N
Maximum acceleration	(1) 0.226 m/s²	0.107 m/s²	(1) 0.257 m/s²	0.216 m/s²
Criteria	≤ 0.70 m/s²	≤ 0.70 m/s²	≤ 0.70 m/s²	≤ 0.70 m/s²

(1) Not generated with Strand7, but calculated thanks to the linearity between the load and the acceleration.

A remarkable effect for the Load Cases of Proposal Annex C is that the applied loads are smaller in the case of group of pedestrians than in the case of a single pedestrian. This is due to the relative high natural frequency which lower the chance to have large groups all walking in pace. The load cases from the UK National Annex do not show such a phenomenon: the loads are about twice as high than the ones representing a single pedestrian.

The accelerations generated by the load cases from the UK National Annex are smaller than the ones generated by the load cases from Proposal Annex C. This is in accordance with former analyses. However, the differences between the accelerations from Proposal Annex C and the ones from the UK National Annex are smaller than other cases: in the worse case, the accelerations from the UK National Annex are twice as small as the one from Proposal Annex C.

Joggers

Proposal Annex C does not propose any load model for joggers. The two cases for the load model proposed by the UK National Annex are thus being compared. The results are shown in Table 6.10.

Table 6.10 Comparison Load Models for Joggers

	UK National Annex	
Load Case	UKNA J1	UKNA J2
Number of pedestrians	N = 4	N = 4
Loading time	28.38 seconds	27.24 seconds
Reference Load	910 N	910 N
DLF	0.455	0.226
Amplitude Load	414.1 N	205.7 N
Maximum acceleration	0.182 m/s²	0.202 m/s²
Criteria	≤ 0.80 m/s²	≤ 0.80 m/s²

Groups of four joggers have been analysed which is recommended for such bridges in the UK National Annex. As is the case for walking pedestrians, the Dynamic Load Factors are low because of the high natural frequencies of the bridge. UKNA J1 would be the most likely to occur, as its frequency lies within the running frequency range.

Both generated accelerations remain below the limit stated in the code: they would probably only be sensible to other users of the bridge that are standing still, which is not likely to occur. These accelerations are not clearly sensible to walking and jogging pedestrians.

Crowd

Both Proposal Annex C and the UK National Annex consider a crowded load on the bridge. The results of these load models are shown in Table 6.11. In all cases, the dynamic load is a non moving load. The load cases however are based on different densities.

Table 6.11 Comparison Load Models for Crowds

	Proposal Annex C	UK National Annex	Proposal Annex C	UK National Annex
Relevant Node #	1508	1508	2617	2617
Load Case	DLM 3A	UKNA C1	DLM 3B	UKNA C2
Density ρ	(based on) 0.6 pers./m²	1.5 pers./m²	(based on) 0.6 pers./m²	1.5 pers./m²
Loading time	120 seconds	60 seconds	120 seconds	60 seconds
Reference Load	12.6 N/m²	0.416 N/m²	12.6 N/m²	0.416 N/m²
DLF	0.943	6.490	0.713	6.584
Amplitude Load	11.9 N/m²	2.70 N/m²	9.0 N/m²	2.73 N/m²
Maximum acceleration	1.52 m/s²	0.428 m/s²	0.98 m/s²	0.456 m/s²
Criteria	≤ 0.70 m/s²	≤ 0.80 m/s²	≤ 0.70 m/s²	≤ 0.80 m/s²

As was the case for the Goodwill Bridge, the amplitudes of the load largely differ between the UK National Annex and Proposal Annex C. Accelerations up to 1.5 m/s² are being generated in the case of Proposal Annex C, which is more than twice as large as the limit stated in that code. The load model for crowds in Proposal Annex C generates accelerations which would not be tolerable in practice.

The UK National Annex however generates accelerations which are within the limits stated by the code.

Comparison of the results with the practice

There are no measurements available for this bridge. However, the generated accelerations can be compared to the practice. Practice has shown that small accelerations occur, but never excessive ones. The particularity of the Milton Road Bridge is that one pedestrian usually generates more sensible vibrations than groups of pedestrian. This would confirm the load model from Proposal Annex C, which generates higher accelerations in the case of a single pedestrian than in the case of a group of pedestrians. However, one should note that pedestrians usually walk at a much lower frequency than the ones which have been used for the analyses: resonance should be less likely to occur in practice, according to the found natural frequencies.

Crowded situations regularly occur on the bridge. Vibrations can clearly be felt on different places on the bridge deck. The accelerations however are not excessively large. This confirms the fact that the accelerations generated by Proposal Annex C cannot be correct and that the ones generated by the UK National Annex are more likely to occur.

6.6.2 Horizontal component

As seen in chapter 4, the way the horizontal response is checked in both codes is fundamentally different. As in Proposal Annex C, all loads in the load cases contain a horizontal component, the UK National Annex only proposes a method to control the lateral responses due to crowd loading.

Proposal Annex C

The horizontal accelerations generated according to the load models described in Proposal Annex C are shown in Table 6.12. One can notice that no severe lateral accelerations occur, even in the case of crowded situation. Experience has proved that no severe lateral accelerations have ever occurred on the Milton Road Bridge. Note that the bridge has a large lateral stiffness. It can therefore be concluded that the values of the acceleration are in the right range.

Table 6.12 Overview of the lateral accelerations for the load cases from Proposal Annex C

	DLM 1A	DLM 1A	DLM 2A	DLM 2B	DLM 3A	DLM 3B
Loading time	120 sec.	120 sec.	120 sec.	120 sec.	120 sec.	120 sec.
Reference Load	70 N	70 N	70 N	70 N	3.2 N/m²	3.2 N/m²
DLF	1.0	1.0	0.5	0.5	0.5	0.5
Amplitude Load	70 N	70 N	35 N	35 N	1.6 N/m²	1.6 N/m²
Max. acceleration	0.005 m/s²	0.022 m/s²	0.003 m/s²	0.011 m/s²	0.033 m/s²	0.062 m/s²
Criteria	≤ 0.15 m/s²	≤ 0.15 m/s²	≤ 0.15 m/s²	≤ 0.15 m/s²	≤ 0.15 m/s²	≤ 0.15 m/s²

UK National Annex

According to Table 6.1, there are no lateral mode shapes with a frequency lower than 1.5 Hz. The UK National Annex stipulates in this case that it can be assumed that the bridge is not susceptible to have unstable lateral responses. Parameter D thus has not to be calculated in this case.

6.6.3 Conclusion

Both methods show that no excessive lateral acceleration is due to happen on the main span of the Goodwill Bridge. As mentioned earlier, experience has proven this situation. No data is available to control the generated accelerations with the load models from Proposal Annex C.

7 Evaluation

In Chapter 5 and 6 the Goodwill Bridge and the Milton Road Bridge have been dynamically analysed according to Proposal Annex C (of the Eurocode), the British National Annex (of the Eurocode) and the Australian Standard. This chapter aims to analyse the results and evaluate the load models presented in the Codes.

7.1 Evaluation Responses

The load models show scattered results. Proposal Annex C tends to generate accelerations which are on the high side compared to reality, whereas the accelerations generated with the UK National Annex always seem on the low side. The displacements generated for the Australian Standard seem to be on the low side, both for the Goodwill Bridge and Milton Road Bridge. This paragraph tends to give more meaning to the responses of the analyses, according to the theory of Chapter 2, in terms of sensitivity of the bridge users. As a clear distinction between vertical and lateral sensitivity can be made, these two aspects are being dealt separately.

7.1.1 Vertical responses

The maximum values of the responses of the relevant analyses have been summarized in Table 7.1. Note that in each case both the maximum acceleration and the maximum displacement have been displayed. This has been done to be able to compare the response of the Eurocode and the Australian Standard, but also to be able to compare these values to theoretical limits stated in Chapter 2.

Table 7.1 Overview maximum value of responses of each analysis

		Proposal Annex C	UK National Annex	Australian Standard
Single Pedestrian	Goodwill Bridge	**a_{max} = 0.184 m/s²** u_{max} = 1.25 mm	**a_{max} = 0.015 m/s²** u_{max} = 0.09 mm	a_{max} = 0.015 m/s² **u_{max} = 0.09 mm**
	Milton Road Bridge	**a_{max} = 0.360 m/s²** u_{max} = 0.52 mm	**a_{max} = 0.110 m/s²** u_{max} = 0.16 mm	a_{max} = 0.005 m/s² **u_{max} = 0.02 mm**
Group of Pedestrians	Goodwill Bridge	**a_{max} = 0.552 m/s²** u_{max} = 3.76 mm	**a_{max} = 0.027 m/s²** u_{max} = 0.16 mm	-
	Milton Road Bridge	**a_{max} = 0.257 m/s²** u_{max} = 0.37 mm	**a_{max} = 0.216 m/s²** u_{max} = 0.32 mm	-
Joggers	Goodwill Bridge	-	**a_{max} = 0.182 m/s²** u_{max} = 0.70 mm	-
	Milton Road Bridge	-	**a_{max} = 0.202 m/s²** u_{max} = 0.29 mm	-
Crowd	Goodwill Bridge	**a_{max} = 4.116 m/s²** u_{max} = 28.16 mm	**a_{max} = 2.087 m/s²** u_{max} = 14.16 mm	-
	Milton Road Bridge	**a_{max} = 1.520 m/s²** u_{max} = 3.68 mm	**a_{max} = 0.456 m/s²** u_{max} = 0.66 mm	-

It can be noticed that in most of the time the responses generated with the UK National Annex are smaller than the ones generated with Proposal Annex C. This is mainly caused by the fact that moving loads are used in the first case. However this does not explain the magnitude of the differences which vary with a factor between 1.2 and 12. The reason behind this must be found in the Dynamic Load factor. Let's for example have a closer look at the load cases representing a single pedestrian and a group of pedestrians on the Milton Road Bridge. Whereas in the case of a single pedestrian the responses generated with Proposal Annex C are three times higher than the ones generated with the UK National Annex, there is nearly no difference in the case of a group of pedestrians. Looking closer to the amplitude of the applied loads (see Table 7.2) it becomes clear that in Proposal Annex C the Dynamic Load Factor is smaller in the case of a group of pedestrians than in the case of a single pedestrian and that that is the other way around with the UK National Annex. The Milton Road Bridge has a relative high Natural Frequency. This influences the Dynamic Load Factor of Proposal Annex C considerably, as it is the only parameter considered. The way the Dynamic Load Factor is set up in the UK National Annex is somewhat more complicated: different parameters assess the Dynamic Load factor, such as the frequency, the degree of synchronisation and the number of pedestrians. Especially the number of pedestrians has a considerable influence. That's the reason why the Dynamic Load Factor is higher in the case of group of pedestrian than in the case of a single pedestrian.

Table 7.2 Load amplitudes (and Dynamic Load Factors) used on the Milton Road Bridge

	Proposal Annex C	**UK National Annex**
Single Pedestrian	F = 280 N *DLF = 1.0*	F = 95.2 N *DLF = 0.34*
Group of Pedestrians	F = 264 N *DLF = 0.943*	F = 187.1 N *DLF = 0.668*

In Chapter 2, it has been concluded that vibrations can be felt differently, mostly depending on the velocity of the bridge user: a person standing still tends to feel more than someone walking. The values in Table 7.1 only give indication on the sensibility of pedestrians if the values are placed within the usage context of the bridge.

Regarding the use of the bridge, the Goodwill Bridge and the Milton Road Bridge are totally different. The Goodwill Bridge is an important link between the Central Business District of Brisbane and the southern suburbs and is crossing the Brisbane River. The bridge is appreciated by tourists for the view over the city and the river. It is also appreciated by joggers. The bridge is therefore used for different purposes. Joggers, people walking at different speed and people standing still should all be considered. The Milton Road Bridge on the other hand is a bridge that is nearly only used for larger groups of people going from the Suncorp Stadium to the Milton Train Station. It is therefore only used during certain periods and there is no reason to stand still.

Figure 7.1 is a figure that has already been shown and explained in Chapter 2. This graph can give more information about the values given in Table 7.1 and considering the way the bridges are used.

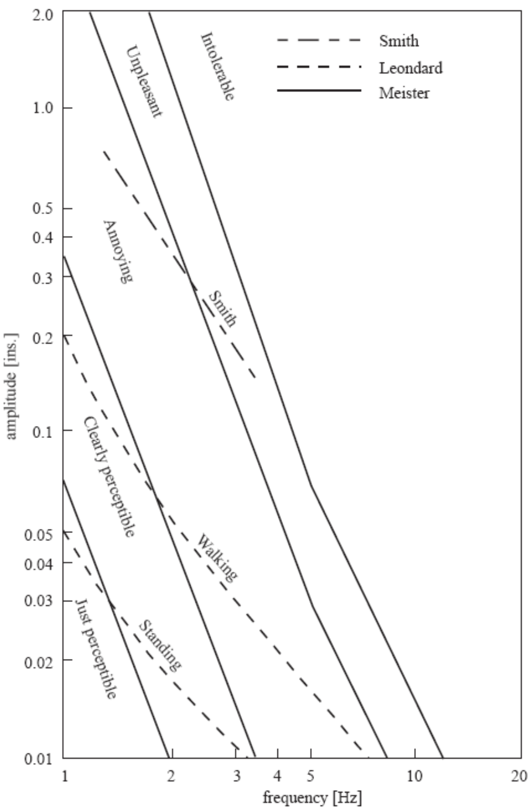


Figure 7.1 Human perception to vertical vibrations (Note: 1 ins = 25.4 mm)

The x-axis represents the frequency of the loaded bridge and the y-axis the maximum amplitude of the critical node. Smith, Leonard and Meister have independently stated limits. The limit stated by Smith is the upper bound of pedestrian tolerance. Meister defined different areas which represent the sensitivity of pedestrians. Finally, Leonard defined lower bounds for the comfort threshold of walking and standing users. As these limits are all based on experiments, they do not always correspond to each other.

Nonetheless this can be used to determine what the values from Table 7.1 mean and can therefore better be compared to practice. The meaning of these values is shown in Table 7.3. Note that the displacement of the critical node has been used for this purpose.

The Smith upper bound limit has only been mentioned when the response is above it.

Note that the last column contains the noticed vibrations in practice. These vibrations have been noticed and categorized while standing still on the bridge.

Table 7.3 Meaning of the values given in Table 7.1 according to Smith, Leonard and Meister

		Proposal Annex C	UK National Annex	Australian Standard	Practice
Single Pedestrian	Goodwill Bridge	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	**Leonard:** Under standing and walking threshold **Meister:** Just perceptible	**Leonard:** Under standing and walking threshold **Meister:** Just perceptible	Just perceptible
	Milton Road Bridge	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	**Leonard:** Under standing and walking threshold **Meister:** Just perceptible	**Leonard:** Under standing and walking threshold **Meister:** Just perceptible	Clearly perceptible
Group of Pedestrians	Goodwill Bridge	**Leonard:** Above standing and walking threshold **Meister:** Annoying	**Leonard:** Under standing and walking threshold **Meister:** Just perceptible	-	Just perceptible
	Milton Road Bridge	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	-	Just perceptible
Joggers	Goodwill Bridge	-	**Leonard:** Above standing threshold Under walking threshold **Meister:** Clearly perceptible	-	Clearly perceptible
	Milton Road Bridge	-	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	-	-
Crowd	Goodwill Bridge	**Leonard:** Above standing and walking threshold **Meister:** Intolerable **Smith:** Above upper bound pedestrian tolerance	**Leonard:** Above standing and walking threshold **Meister:** Unpleasant **Smith:** Above upper bound pedestrian tolerance	-	Clearly perceptible
	Milton Road Bridge	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	**Leonard:** Above standing threshold Under walking threshold **Meister:** Annoying	-	Clearly perceptible

In practice, most of the vibrations on the Goodwill Bridge and the Milton Road Bridge can only be felt when standing still. These vibrations are usually in the area as Meister described as 'Just perceptible' or 'Clearly perceptible'. It is known that in some cases joggers can clearly perceive vibrations when running over the Goodwill Bridge. However, this has never lead to annoying vibrations so far. Crowded situations have occurred on both bridges but no excessive vibrations have ever been noticed.

The load models from Proposal Annex C tends to generate responses which are considered as 'Annoying' by Meister and even 'Intolerable' in case of the Crowd situation on the Goodwill Bridge. This last situation is confirmed by Smith. In most situations, according to the lower bound limits stated by Leonard, most vibrations should be felt by the bridge users that are standing still only. The only exception is the response generated on the Goodwill Bridge with the load case representing a group of pedestrians: according to Leonard, both standing and walking people should feel the vibrations.
These results are not fully in accordance with the practice. This confirms that what has been concluded earlier: Proposal Annex C tends to generate higher responses than the ones that could occur in practice.

The responses generated with the load models from the UK National Annex are more scattered. For the load models representing walking pedestrians, the responses are such that they can mostly be categorized as 'Just perceptible', which corresponds to the assumptions made earlier. However, according to lower bound limits of Leonard, none of the vibrations should be felt in these situations. The load cases for Joggers and crowded situations show different results. The generated responses are such that they can be categorized as 'Clearly Perceptible', 'Annoying' or 'Unpleasant' according to Meister. According to Leonard, most of these vibrations should only be felt by users that stand still on the bridge. The response of the Goodwill Bridge for the crowded situation is such that all users should feel it. According to Smith, the response is above the upper bound pedestrian tolerance.
It seems that the analyses of the load models representing a single pedestrian, a group of pedestrians and joggers correspond the most to the reality according to Meister. However this is not the case for the crowd situation.

Concerning the Australian Standard, the responses are such that they are categorized as 'Just perceptible' according to Meister. None of the vibrations should be felt by any user according to Leonard. One should note that the Australian Standard does not suggest any load model, but only mentions certain conditions. The load model used in this analysis is based on the UK National Annex. The same conclusions regarding a single pedestrian for the UK National Annex therefore remain.

This paragraph has confirmed the fact that the load models do not always seem to be appropriate without being compared to the comfort criteria stated in the codes. However it can stated that the vibrations induced by the load models described in the UK National

Annex correspond the most to the sensibility of pedestrians, despite the fact that the magnitude of the responses do not always correspond to reality. The comfort criteria stated in the codes should therefore be further discussed. This is being discussed in paragraph 7.2.

7.1.2 Horizontal responses

Only the load models from Proposal Annex C propose to generate horizontal accelerations. The results can be seen in Table 7.4.

Table 7.4 Lateral accelerations

		Proposal Annex C
Single Pedestrian	Goodwill Bridge	a_{max} = 0.004 m/s²
	Milton Road Bridge	a_{max} = 0.022 m/s²
Group of Pedestrians	Goodwill Bridge	a_{max} = 0.011 m/s²
	Milton Road Bridge	a_{max} = 0.008 m/s²
Crowd	Goodwill Bridge	a_{max} = 0.051 m/s²
	Milton Road Bridge	a_{max} = 0.062 m/s²

As mentioned in Chapter 2, there have not been many experiments for the lateral motion of bridges and data is therefore scarce. But it has been shown that lateral accelerations of about 0.3 m/s² are clearly perceptible by pedestrians and can influence their walking behaviour. The values shown in Table 7.4 are far beneath this value. It can thus be concluded that these vibrations are not being felt by the users of the bridge. This confirms the practice: on both bridges, lateral movements have never been noticed by users.

7.2 Evaluation Codes

7.2.1 Load Models

Table 7.4 presents an overview of the main characteristics of the load models from the three considered codes. The Load models are being evaluated considering the results from the analyses done on the Goodwill Bridge and the Milton Road Bridge.

The most important conclusion from the analyses presented in former chapters is that the responses from the load models in the British National Annex always seem to be on the low side, especially for walking pedestrians. The responses from the Load Cases representing joggers seem to be closer to the real behaviour. However, the generated responses with the load models from Proposal Annex C are always on the high side.

The parameters influencing the load models are being discussed separately.

Table 7.5 Overview considered Load models

	Proposal Annex C	UK National Annex	Australian Standard
Takes into account:			
- Single pedestrian (walking)	✓	✓	✓
- Group of Pedestrians (walking)	✓	✓	×
- Joggers	×	✓	×
- Crowd	✓	✓	×
Load Model Characteristics:			
Load	Non moving harmonic load	Moving and non moving harmonic loads	Moving harmonic load
Loading Time	Until Steady State occurs	Depending on velocity of the load	Depending on velocity of the load
Load Frequency	Natural Frequency of the Bridge	Natural Frequency of the Bridge	Between 1.75 Hz and 2.5 Hz
Dynamic Load Factor dependent on:			
- Group size	✓	✓	*Not mentioned*
- Natural Frequency	✓	✓	*Not mentioned*
- Degree of synchronisation between pedestrians	✓	✓	*Not mentioned*
Application conditions:			
- Vertical	$f_v < 5$ Hz	$f_v < 8$ Hz	1.5 Hz $< f_v <$ 3.5 Hz
- Horizontal (lateral)	$f_h < 2.5$ Hz	$f_h < 1.5$ Hz	$f_h < 1.5$ Hz

7.2.1.1 Load Frequency and Application Conditions

A major difference between the Annexes of the Eurocode and the Australian Standard is the way the load frequency is chosen. Proposal Annex C for example proposes to check vertical vibrations in the case that at least one of the Natural Frequency lies below 5 Hz; the frequency that is the most nearby to 2 Hz should be considered for the analysis. For the Milton Road Bridge, this frequency (f = 3.2 Hz) lies outside the walking frequency range but has been used to model walking pedestrians. The chance that this walking frequency occurs in reality however is very small, especially in the case that lock-in is not likely to occur. It is much more likely to occur with joggers. The UK National Annex seems therefore to deal somewhat better with such a situation, even though the loads for the walking pedestrians still use this frequency.

The Australian Standard acknowledges this situation and therefore states limits to the frequency of the load representing a walking pedestrian. The application conditions are

however somewhat broader because some it is assumed that some resonance is likely to occur if the Natural Frequency is in the neighbourhood of the load frequency.
Stating limits to the Load Frequency would therefore be a realistic approach, both for walking pedestrians and for joggers. In combination with appropriate application conditions designed for each load type, this would avoid to load a bridge unrealistically. Table 7.6 gives a suggestion for these values.

Table 7.6 Suggestion of Load Frequency Ranges and Application conditions

Load Type	Load Frequency Range	Application conditions
Walking	1.0 – 2.5 Hz	f_v < 3.5 Hz
Jogging	2.0 – 3.5 Hz	1.0 Hz < f_v < 4.5 Hz

One can see that in such a way bridges are always loaded with realistic load frequencies. The application conditions have been set up somewhat broader to permit some resonance if the natural frequency of the bridge is just outside the load frequency range.

Concerning the application conditions in lateral direction, a limit of 1.5 Hz should be sufficient. Higher values are unlikely to occur because pedestrians' lateral stability increases as their step frequency increases. Note that this does not take account the fact that somewhat larger lateral frequencies could be started by other sources (like wind) which could lead to lock in.

7.2.1.2 Dynamic load amplitude in crowded situations

It has been shown earlier that there is a relation between the pedestrian velocity, his step frequency and his dynamic load amplitude. In general, velocity and step frequency are directly connected: the slower the pedestrian walks, the lower the step frequency. The UK National Annex does not seem to take account of this effect. Considering the fact that the Natural Frequency of the bridge is being used as step frequency, an average walking velocity could be considered. However, the question remains in which extend this would have influence on the response. It has been proven that the load velocity can have influence on the moment the highest acceleration occurs and in certain extend also on the amplitude of the response.

Both Proposal Annex C and the UK National Annex are based on Bachmann's coefficients: 40% of the body weight is accounted as dynamic load in the case of a walking pedestrian. According to Blanchard, this should correspond to the dynamic load of a brisk or fast walking pedestrian. Normally walking pedestrians seem to create lower dynamic forces, up to 20% of the body weight. In general the amplitude of the dynamic load is increasing with increasing step frequency. This phenomenon is not taken into account within the Codes. Considering the fact that the load models for crowd situations tend to generate out of proportion responses, especially in the case of the Goodwill Bridge, this effect could have a positive influence on the responses. It is known that in crowded situations, pedestrians tend to walk slower and thus the step frequency decreases.

It can be concluded that the density of crowded situations may have serious influence on the amplitude of the dynamic load and thus on the response of the bridge.

7.2.1.3 Dynamic Load Factor

Proposal Annex C and the UK National Annex propose different methods to assess the Dynamic Load Factor. This leads to scattered results as can be seen in Table 7.7.

Table 7.7 Overview of used Dynamic Load Factors

	Goodwill Bridge		Milton Road Bridge	
	Proposal Annex C	UK National Annex	Proposal Annex C	UK National Annex
Single Pedestrian	-	1.000	-	0.320
Group of Pedestrians	3.000	1.819	0.713	0.340
Joggers	-	1.238	-	0.226
Crowd	3.000	23.850	0.713	6.584

It is difficult in this stage to determine which of these methods the best describes the reality. However one can state that the Dynamic Load Factor in the UK National Annex is more flexible to individual cases. However it is also based on different graphs which may not always be clear and thus may lead to errors.

Is has also been shown that the UK National Annex tends to generate responses which are on the low side and that Proposal Annex C tends to generate responses which are too high. The Dynamic Load Factor can be one of the factors that contribute to this situation.

7.2.1.4 Densities for Crowd load cases

One should notice that there is a difference between the density used for the check of the structural strength and the one used for the vibrations. A characteristic value of 5 kN/m² should be taken into account for the structural strength. Considering an average weight of 70 kg, this corresponds to a density of more than 7 pers./m². A maximum density of 1.5 pers./m² is used for the dynamic behaviour of the bridge.

It is clear that with a density of 7 pers./m² (as far as that is feasible) not much dynamic load should be considered in vertical direction: pedestrians are not able to move. However, this may not be the case in the lateral direction. Such high densities could amplify existing lateral vibrations.

A density of 7 pers./m² may be an unrealistic high density, but one can wonder if a density 1.5 pers./m² may be on the low side, especially for bridges that are susceptible to move laterally. This is a question that should be answered separately for each bridge, but should need attention.

7.2.1.5 Representation of groups (pedestrians and joggers)

A surprising fact is that loads representing groups (except crowds) are all concentrated in one point. It is assumed that all pedestrians are walking at the same place at the same moment. This assumption could be considered as conservative and thus lead to conservative responses of the bridge, as has been shown in paragraph 7.1.

This could be avoided by considering a certain density (low enough to walk freely) and the number of pedestrians and assess the corresponding area. The loads would therefore be distributed over a few nodes.

7.2.1.6 Moving vs. non moving loads

One of the essential differences between the UK National Annex and Proposal Annex C is the fact the first one proposes moving loads (except for the load cases representing crowds) and the second one non moving loads. It is clear that moving loads better represent the reality than the non moving loads do. It can however been stated that in practice moving dynamic loads are somewhat more difficult to implement. Only advanced Finite Element software packages are now available to model such loads. Modelling moving dynamic loads can be a time consuming task, depending on which software package is being used and the experience of the engineer. Furthermore, interpreting the results of moving dynamic loads is much more difficult than interpreting the results of non moving loads. Errors can therefore easily occur.

Representing pedestrians (walking, jogging) by non moving loads as is done by Proposal Annex C could therefore be a suitable solution to avoid these difficulties. Proposal Annex C however shows responses which are too high. More attention should therefore be paid to the used Dynamic Load Factors. These should take account to the fact that the dynamic load is standing still but is representing a moving pedestrian.

7.2.2 Comfort Criteria

It has been shown in paragraph 7.1 that responses can be interpreted differently. It does not really matter if the response is expressed in terms of acceleration or displacement, as both are correlated.

The way each annex and code deals with the comfort criteria is described in Table 7.8. One can notice that the comfort criteria are mostly only dependent on the frequency of the bridge. The upper limit is greatly scattered especially in vertical direction. The UK National Annex is somewhat more flexible in assessing the maximum tolerable acceleration, but does not take account the frequency.

Table 7.8 Overview comfort criteria stated in the considered codes

	Proposal Annex C	**UK National Annex**	**Australian Standard**
Vertical Comfort Criteria:			
- expressed in terms of	Maximum accepted acceleration [m/s²]	Maximum accepted acceleration [m/s²]	Maximum accepted displacement [mm]
- Dependent on	Natural Frequency	- Bridge function - Route redundancy - Bridge height - Users' perception on vibration	Natural Frequency
- Upper limit	0.7 m/s²	2.0 m/s²	-
Horizontal Comfort Criteria:			
- expressed in terms of	Maximum accepted acceleration [m/s²]	Parameter D [-]	*Left over to Engineer*
- Dependent on	Natural Frequency	- Mass of the bridge - Mass of the pedestrians - Structural damping	*Not mentioned*
- Upper limit	0.2 or 0.4 m/s²	Dependent on the considered horizontal frequency	*Not mentioned*

Meister, Leonard and Smith have however proved that the sensitivity of pedestrians is also based on the frequency as shown in Figure 7.1. Meister defined different areas which describe the sensitivity of the bridge users. Leonard showed with lower bound limits that these areas should be dependent on the type of user (standing still, walking, jogging and crowds). The theoretical background of sensitivity of vibrations is a complex topic based on physical and psychological aspects.

Considering these facts it does not make sense to define only one limit to the bridge responses. It should also be related to how the bridge is used. Let's for example consider the limits for the Goodwill Bridge and the Milton Road Bridge. As mentioned earlier these bridges are used in different ways. However the comfort criteria are for both bridges the same. There is no particular reason why a standing pedestrian should not perceive clear vibrations on the Milton Road Bridge, as for the Goodwill Bridge these should be limited as much as possible.

A method to separate these criteria is to use a graph as shown in Figure 7.1. Similar graphs could also be set up in terms of acceleration and for each type of pedestrian (standing still, walking/jogging and within a crowd). Depending on the expecting use of the bridge, one can then determine if the responses are acceptable in a more appropriate way than is done now.

8 Conclusions

The load models representing dynamic pedestrian loads and described in Proposal Annex C (to EN 1991-2:2003), in the British National Annex (to EN 1991-2:2003) and in the Australian Standard (AS 5100.2-2004) have been applied to two bridges: the Goodwill Bridge and the Milton Road Bridge, both situated in Brisbane, Australia. The Goodwill Bridge is an arch bridge over the Brisbane River and is intensively used by walking pedestrians and joggers who regularly stop to enjoy the city view. The Milton Road Bridge is a bridge that links an important stadium to a train station. It is hence generally only used by large groups of pedestrians transferring to the train station.

The Codes have different approaches to represent pedestrians. Proposal annex C proposes load models for a single pedestrian, a group of pedestrians and a crowd and represents them as non moving harmonic loads. The British National Annex (also called UK National Annex) proposes models for walking pedestrians, joggers and crowds and represents the first two as harmonic loads that cross the bridge at a certain speed and the last one as a non moving load. The Australian Standard only proposes a load model for a single pedestrian moving over the bridge. Whereas the annexes based on the Eurocode give detailed information about the assessment of the Dynamic Load Factor (and thus the amplitude of the load), the frequency of the load and the speed of the load, the Australian Standard only gives some basic conditions (load frequency range) the harmonic load should satisfy. The rest is thus left to the Engineer.

The load frequency is generally chosen such that it corresponds to a natural frequency of the bridge which lies below 5 Hz which represents a wide walking and jogging frequency range. Resonance can therefore occur which causes the largest vibrations. The Australian Standard however limits the load frequency range between 1.75 and 2.5 Hz which corresponds to the walking frequency range only.

Each code also states its own comfort criteria which are expressed in term of maximum acceptable acceleration in the annexes based on the Eurocode and in terms of maximum acceptable displacement in the Australian Standard. The displacement and the acceleration are strongly correlated to each other. The criteria are based on the sensitivity of pedestrians for vibrations. The sensitivity is a complex topic which is mainly influenced by psychological and physical factors. The comfort criteria stated in the codes are based on researches done in this domain and are set up in such a way that one limit value determines the requirements for all pedestrians on the bridge.

A Finite Element software package has been used to model both bridges mentioned earlier and the different load models described in the codes. It was found that both bridges have natural frequencies that lie within the walking and jogging frequency range and are thus susceptible to vibrate considerably.

The generated responses showed scattered results. Most of the responses satisfy the requirements stated in the Codes, except those generated with the crowd load cases. Compared to the real behaviour (and measurements on the Goodwill Bridge), it has been found that Proposal Annex C always generates accelerations that are too high. The UK National Annex and the Australian Standard (in the case of a single pedestrian) however generate accelerations and displacements that are somewhat on the low side, except for the load case representing a crowd. This load case generates responses that are too high but in less extend than the ones generated by Proposal Annex C. However it has been shown that according to researches of Meister regarding the sensibility of humans to vibrations, the load models from the UK National Annex describe the best the behaviour of the bridges. The load models proposed by the UK National Annex can therefore be considered as the most accurate and complete.

Moving harmonic load models hence are the most appropriate to represent pedestrians and joggers. However the load models described in the UK National Annex do not always generate correct responses, especially the one representing a crowd. This latter situation could be explained by the fact that the pedestrian density is not taken into account in the amplitude of the dynamic forces of pedestrians. Pedestrians tend to walk slower in case of high densities and as a consequence produce smaller dynamic forces (hence smaller Dynamic Load Factors). The codes propose different methods to assess the Dynamic Load Factors. The one proposed by Proposal Annex C is simplified and can lead to too large amplitudes. The one described in the UK National Annex is more complex and is largely dependent on graphs that can lead to mistakes and which may not always be understood. It can thus lead to mistakes. However the produced Dynamic Load Factors seem to be more realistic.

Mistakes are usually not that easily made with the load models from Proposal Annex C. This annex describes non moving harmonic loads to represent walking pedestrians. However, as mentioned earlier, the generated responses with these load models have too large amplitude.

A discrepancy in the annexes of the Eurocode has been noticed concerning the application conditions of the load models. Whereas researches have proved that walking and jogging lie within a closed frequency range, the annexes require analysing bridges with natural frequencies that lie far outside of this frequency range. It can therefore happen that pedestrians are represented with load frequencies that are not realistic. The Australian Standard limits the walking frequency range within more realistic values.

Responses generated with moving harmonic loads can be difficult to interpret in comparison to the ones generated with non moving harmonic loads. Dynamic analyses with moving harmonic loads can be sensitive and can easily lead to errors. Especially the time step of the analysis and the bridge model can have a significant influence on the response of the bridge. Other factors may also play a role but that depends per bridge. One should therefore carefully pay attention to this situation when doing some dynamic analyses.

As mentioned earlier the comfort criteria are based on the sensitivity of pedestrians to vibrations. An important fact is that the sensitivity of pedestrians is influenced by a combination of psychological and physical factors. People standing still are more sensitive to vibrations than people walking. However, walking in crowds reduces the sensitivity of pedestrians. Limiting the comfort criteria with one value is thus a conservative method, especially if certain type of pedestrians should not be considered. Think about people standing still on a bridge that is most of the time only used by large crowds. This situation is not likely to occur often and thus higher vibrations could be permitted. As a consequence this confirms the fact that higher accelerations can be generated with the crowd load models.

This report clearly shows that moving harmonic loads the best represent pedestrian loads in the existing codes of practice. The load models described in the UK National Annex are the most accurate and complete, even though careful attention should be paid to the analyses. Non moving harmonic loads are easier to use in practice. The load models are susceptible to some improvements. Interpretation of the responses could be done in a more sensible way to determine the sensitivity of the pedestrians.

9 Recommendations

1. Representing pedestrians by moving harmonic loads are difficult to use and can easily lead to errors. It would therefore be preferable to represent them by non moving harmonic loads, such as in Proposal Annex C. However one should reconsider the Dynamic Load Factors used. This should take into account the fact that the load is actually moving and thus should generate lower responses.

2. If moving harmonic loads are chosen to represent pedestrian loads, one should do research in the parameters that are influencing the responses. This could lead to a general guideline which describes how to model structures and loads to avoid errors.

3. One should assess appropriate Dynamic Load Factors for the crowd load models. These should take into account the fact that pedestrians walking more slowly generate smaller dynamic forces on the deck.

4. The application conditions should be restricted to frequencies that are more realistic considering the type of pedestrians that are expected to use the bridge. The frequencies used for the load frequencies should be restricted to the frequency range of the considered pedestrians. Somewhat wider application conditions should then be set up, because larger displacements can also occur when load frequency is nearby one of the natural frequencies.

5. The comfort criteria should be redefined. It is recommended to split up the criteria in such a way that different types of users are considered: people standing still, pedestrians walking, joggers and crowds. In all these cases vibrations are perceived differently. It is therefore logical to define different comfort criteria for these users. One can then also choose which users should be taken into account.

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Appendices

*NOTE THAT ONLY SHORT INTRODUCTIONS
AND THE TABLES OF CONTENT
ARE PRESENTED ON THE NEXT PAGES*

*THE CONTENT CAN BE FOUND ON THE DVD
AT THE END OF THE REPORT*

Appendix 1 Overview Bridges

This appendix gives more information about the analysed bridges in this report and complements chapter 3 of the report which only gives a short overview of the footbridges. The information presented in this appendix comes essentially from literature and drawings (as constructed) of the bridges. Note that one can refer to Appendix 2 for more information about the steel sections mentioned.

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Appendix 2 Steel Sections Details

The Australian steel sections differ from the ones used in the Netherlands. This appendix presents details about the steel sections used in the different analysed bridges. Note that both standard and project specific sections are clarified.

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Appendix 3 Strand7 & Calculation methods

Strand7 is the Finite Element Analysis software package used during this research. Appendix 3.1 gives a general overview of this software package and an overview of the possibilities related to the modelling and the dynamic analysis in order to get familiar with the software package. Appendix 3.2 compares two different methods to generate dynamic responses within Strand7.

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Appendix 4 Analyses: Modelling & Results

The load models presented in the Proposal Annex C and the UK National Annex are being applied to the two bridges described in this report. The bridges are being dealt separately.

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Appendix 5 Dynamic Analysis of Simplified Structures

The aim of this appendix is to give an overview of the parameters that influence the dynamic behaviour of a bridge on which a harmonic load is crossing. The first chapter deals about a simply supported beam and aims to analyse the influence of the speed of the load. The second chapter deals about a simplified Arch bridge and aims to analyse the influence of the structural elements on the dynamic behaviour of the bridge.

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