15. Design of railway bridges

15.1 Steel for (High-Speed) railway bridges

Caused by special research and development efforts of all the professionals involved in the steel bridges - owners, designers, builders and steel fabricators -, there is a come back of steel in especially the high-speed railway bridges. The steel used for high-speed railway bridges in France illustrates an explanation on this phenomenon.

Statistics of road bridges shows that a single type of structure accounts for the success of steel bridges: the twin girder composite bridge. Because of the economic advantage of the twin girder deck, the French Railways with the help of the steel bridge constructors and steel fabricators started several research programs and finally proved the competitiveness of these decks for new railway bridges; see table 1.

<table>
<thead>
<tr>
<th>High-speed railway line</th>
<th>Steel weight [ton]</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>TGV-Sud Est</td>
<td>-----</td>
<td>1983</td>
</tr>
<tr>
<td>TGV-Atlantique</td>
<td>-----</td>
<td>1990</td>
</tr>
<tr>
<td>TGV-Nord</td>
<td>9573</td>
<td>1993</td>
</tr>
<tr>
<td>Lille</td>
<td>3350</td>
<td>1996</td>
</tr>
<tr>
<td>Interconnection</td>
<td>4220</td>
<td>1996</td>
</tr>
<tr>
<td>Rhone-Alpes</td>
<td>3595</td>
<td>1994</td>
</tr>
</tbody>
</table>
| Total                          | period 1990-1994: 20738
| TGV Mediterrae.               | 42475             | 1999          |
| TGV Est                        | 25000             | 2005          |

Table 1. Steel in TGV-line bridges.

The first high-speed railways in the eighties, South East and Atlantic-lines, had only pre-stressed concrete viaducts while the North TGV line from Paris to Lille included some fifteen steel and composite bridges for a total weight of 20,000 tons. This trend was confirmed in the new Mediterranean TGV with 44,000 tons of steel in 23 bridges. The conceptual design of steel bridges has evolved, caused by increasing span length and especially the aesthetic demands, from the classical twin girder composite deck to more aesthetic bridges (e.g. tied arch bridges). Table 2 shows the structural types with the total length and the steel quantities used.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Twin girder</td>
<td>2697 m 8000 t</td>
<td>1607 m 5115 t</td>
<td>7190 t 24959 t</td>
</tr>
<tr>
<td>Multi girder</td>
<td>234 m 573 t</td>
<td>121 m 700 t</td>
<td>992 t 3553 t</td>
</tr>
<tr>
<td>RAPL</td>
<td>581 m 3350 t</td>
<td>300 m 2000 t</td>
<td>245 t 2040 t</td>
</tr>
<tr>
<td>Warren Truss</td>
<td>91 m 1000 t</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Twin box section</td>
<td>--</td>
<td>--</td>
<td>290 m 1022 t</td>
</tr>
<tr>
<td>Tied arch</td>
<td>--</td>
<td>--</td>
<td>730 m 12900 t</td>
</tr>
<tr>
<td>Special tubular truss</td>
<td>--</td>
<td>--</td>
<td>300 m 1270 t</td>
</tr>
</tbody>
</table>

Table 2. Structural types of steel bridges: total length and steel weight.

Because of the special attention to the dynamic behaviour of the superstructure, the high-speed railway steel bridges systematically incorporate a concrete slab, which takes part in the global and local resistance as part of the composite deck structure. It also carries the ballast, and the concrete brings supplementary mass and damping, thus decreasing noise.

**Structural damping**

The maximum acceleration is dependent upon the rate at which dynamic motions dissipate through damping. The less the damping, the greater the maximum dynamic effects. Therefore, it is important that lower bound values of damping are used in calculations to ensure that safe estimates of peak dynamic effects at resonance are obtained.

**Mass**

The natural frequency of a structure decreases as the mass of the structure increases (providing other parameters such as stiffness do not change). As maximum dynamic load effects are likely to occur at resonant peaks when a multiple of the frequency of loading and a natural frequency of the structure coincide, any underestimation of mass will overestimate the natural frequency of the structure. Therefore, a safe upper bound estimate of bridge mass is required.

Some particular factors concerning the steel material used in the TGV bridges are:
The conceptual design of the steel part is optimised according to the maximum dimensions provided by the rolling mills: lengths of 36 m, widths of 5.2 m, thickness of 150 mm and weights of a single plate reaching 36 tons.

The steel grades and qualities depending on the thickness of the plates are provided without any difficulty according to European standards: from 30 to 150 mm it is the fine grain high strength steel S355N and S355NL. For thickness below 63 mm, the thermo-mechanical S355M and S355ML grades are used. The TM rolled plates with an increased weldability without pre-heating allowed significant cost reduction in fabrication and site works.

The use of LP-plates is significantly increased. This leads to important cost reduction at fabrication, both by decreasing the number of welded connections and the plate thickness at these welds.

Special Z35 steel plates are used for plates subjected to tensile stresses in plate thickness direction.

15.2 Key parameters affecting the dynamics

Bridges have an expected design life that often exceeds 100 years and there are many unknown aspects, especially where the anticipated future development of traffic loads is concerned. Several studies have shown that an increase in the load bearing capacity at the design stage of a bridge is relatively inexpensive. An increase in the basic investment cost of approx. 3-5% results in an approx. increased load-bearing capacity of 20%. However, this conservative philosophy must be abandoned when considering existing bridges, as replacement cost or strengthening cost may be extremely high, especially when traffic disturbances are taken into consideration. From national economics point of view, the possible extra reserves, incorporated at the design stage, might be a point of consideration.

The static stresses and deflection of a railway bridge are increased and decreased under the effects of traffic by the following:

- the rapid rate of loading due to the speed of traffic crossing the structure and the effects of mass, stiffness and damping of the structure
- variations in wheel loads resulting from the track of wheel irregularities
- the passage of successive loadings with approx. uniform spacing which can excite the structure and, in certain circumstances, create resonance

Therefore, a dynamic factor taken into account the dynamics of a railway bridge needs to be included for the design of a railway bridge. The key parameters affecting the dynamics can be classified in four categories:

a. Train characteristics:
   - variation in the magnitude of the axle loads
   - axle spacing
   - spacing of regularly occurring loads
   - the number of regularly occurring loads
   - train speed

b. Structure characteristics:
   - span or the influence length (for simply supported structures these are equivalent)
   - natural frequency (which is itself a function of span, stiffness, mass and support conditions)
   - damping
   - mass per meter of the bridge

c. Track irregularities
   - the profile of the irregularity (shape and size)
   - the presence of regularly spaced defects, e.g. poorly compacted ballast under several sleepers of alternatively the presence of regularly spaced stiff components such as cross-beams
   - the size of the unspring axle masses, an increase in unspring mass causing an increased effective axle force

d. Others
   - out of run wheels, suspension defects, etc.

Mainly from an esthetical point of view, several modern railway bridges have lower damping and higher natural frequencies than the classical railway bridge concepts. For a large number of existing railway bridges, the design
is based on a static analysis only. Especially for the high-speed trains, however, a dynamic analysis is necessary. For example, after introducing the TGV on the line Paris-Lyon, short bridges showed:

- cracks and crumbles of concrete
- high ballast attrition due to high accelerations
- big track irregularities

The dynamic factor, so-called $\phi$ ($\phi_2$ or $\phi_3$) or $\phi$, should be taken into account for the analysis of static stresses and deflection. The factors are also used for fatigue damage calculations.

15.3 Static Load models: ENV 1991-3

Vertical loads

The load models that are used for the design of railway bridges in Europe are shown in fig. 1 - 3. These models represent the static vertical characteristic loads. The loads should be placed at the most unfavourable position for the structural component and load effect in question.

*Representation of the static effect of normal rail traffic loading:*

![Fig. 1. The characteristic vertical loads referred to as UIC 71 (Load Model 71).](image1)

Depending on the span length the Load Model UIC 71 can be simplified as shown in fig. 2.

![Fig. 2. Simplified Load Model UIC 71.](image2)

*Representation of the static effect of heavy rail traffic loading:*

![Fig. 3. Heavy rail traffic loading.](image3)

*Left: The characteristic vertical loads referred to as Load Model SW/0.*

*Right: The characteristic vertical loads referred to as Load Model SW/2.*

The Load Model SW/2 comes from the German *Schwerlast*. This type of load is typical for the transportation of large loads such as turbines, large industrial machines, etc. The loads are often exactly known and the speeds of the trains are often limited to 30 km/h. A typical wagon for such load is shown in fig. 4.

![Fig. 4. A typical wagon used for transportation of a heavy load.](image4)

The characteristic values given in fig. 1/2 and 3 shall be multiplied by a factor $\alpha$ on lines carrying rail traffic which is heavier or lighter than normal traffic. When multiplied by the factor $\alpha$ the loads are called “classified vertical loads”. The relevant authority should specify this factor. All continuous beam bridges designed for Load Model 71 shall be checked additionally for Load Model SW/0. The factor $\alpha$ is applied to Load Model WS/0 and to Load Model 71 (according to NAD-NVN-ENV1991-3: 1.1 times LM 71, this because of expectation near future). For structures carrying one, two or more tracks all relevant actions shall be applied according to table 3.
Table 3. Summary of vertical train loads on bridges depending on number of tracks of a bridge.

For structures carrying more than two tracks the more unfavourable of the two cases a) and b) shall be taken.

LM71"+"SW0 means Load Model 71 and if relevant SW/0 for continuous bridges.

<table>
<thead>
<tr>
<th>Number of tracks on a bridge</th>
<th>Loaded track</th>
<th>Normal loads</th>
<th>Heavy loads if specified</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Case a)</td>
<td>Case b)</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.0 (LM71+&quot;V SW0)</td>
<td>-</td>
<td>1.0 SW2</td>
</tr>
<tr>
<td>2</td>
<td>1.0 (LM71+&quot;V SW0)</td>
<td>-</td>
<td>1.0 SW2</td>
</tr>
<tr>
<td></td>
<td>1.0 (LM71+&quot;V SW0)</td>
<td>-</td>
<td>1.0 (LM71+&quot;V SW0)</td>
</tr>
<tr>
<td>≥ 3</td>
<td>1.0 (LM71+&quot;V SW0)</td>
<td>0.75 (LM71+&quot;V SW0)</td>
<td>1.0 SW2</td>
</tr>
<tr>
<td></td>
<td>1.0 (LM71+&quot;V SW0)</td>
<td>0.75 (LM71+&quot;V SW0)</td>
<td>1.0 (LM71+&quot;V SW0)</td>
</tr>
<tr>
<td></td>
<td>0.75 (LM71+&quot;V SW0)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

For the ULS verification, the partial factor on the vertical actions is 1.45 (LM71) and 1.2 (SW/2).

For the SLS verification (deformation and vibration), the vertical action to be applied shall be:

- LM71 increased by the dynamic factor $\phi$
- the actual train traffic increased by the relevant dynamic factor when determining the dynamic behaviour in case of resonance or excessive vibrations of the deck

For bridge decks carrying double track the checks for the limits of deflection and vibration shall be made with only one track loaded.

**Horizontal forces**

a. Centrifugal forces

Where the track on a bridge is curved over the whole or part of its length, the centrifugal force and the track cant shall be taken into account. The centrifugal forces shall be taken to act outwards in a horizontal direction at height of 1.80 m above the running surface. The calculations shall be based on the maximum speed compatible with the layout of the line. In the case of LM SW a speed of 80 km/h shall be assumed.

The characteristic value of the centrifugal force shall be determined according to the equation:

$$ Q_{ce} = \frac{v^2}{g \times f} $$

with “$f$” taken as a reduction factor according to:

- LM 71, with its dynamic factor and the centrifugal force for $V = 120$ km/h: $f = 1.0$
- Reduced LM 71, for the maximum speed $V$ specified: $f$ given by figure 5.

The length $L_f$ is the influence length [m] of the loaded part of curved track on the bridge, which is most unfavourable for the design of the structural elements under consideration.

**Fig. 5. Factors of “$f$” for LM 71.**

b. Nosing force

The nosing force $Q_{ns} = 100$ kN, shall be taken as a concentrated force acting horizontally at the top of the rails, perpendicular to the centre-line of track. It shall be applied on both straight track and curved track.

c. Actions due to tracking and braking

Their characteristic values shall be taken as follows:

- Traction force: $Q_{tr} = 30$ [kN] \* [m] \* [1000[kN]] for Load Model 71 and Load Models SW
- Braking force: $Q_{br} = 30$ [kN] \* [m] \* [8000[kN]] for Load Models 71 and SW0
- Braking force: $Q_{br} = 35$ [kN] \* [m] \* [1000[kN]] for Load Model SW2

15. (Comfort related) design railway bridges

Dr. A. Romeijn

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d. Aerodynamic effects and wind loading
The aerodynamic effect, the so-called slipstream effect caused by passing rail traffic, depends mainly on speed of
the train and the aerodynamic shape of the train.
The contribution of this type of loading on static and fatigue strength analyses is in comparison to others small,
although they cannot be neglected. For full description, reference is made to the Eurocode ENV 1991-3.
As an example, for combination of wind and traffic actions the reference area $A_{ref,x}$ should be increased by
adding 4,00 m (unlimited train length) to the deck thickness from the level of the running surface, without
cumulating it with the additional depth of parapets, noise barriers, etc. The combinations to be considered when
traffic and wind act simultaneously are:

- vertical railway loads including dynamic factor together with wind forces. Both actions may occur as
dominant, one at a time
- a uniformly distributed vertical load of 12.5 kN/m, called “unloaded train”, without dynamic factor for
  checking overall stability together with wind forces. The action is to obtain the most unfavourable effect on
  the structure element considered.

Accidental actions
Railway bridges shall be designed in such a way that, in the event of a derailment, the resulting damage to the
bridge is limited to a minimum. In particular, overturning or the collapse of the structure as a whole shall be
prevented. Also, accidental action from road traffic should be considered.
For detailed info, reference is made to the Eurocode ENV 1991-3.
More detailed information on accidental loading (collision, train overturning and derailment) is given at the end
of this chapter.

<table>
<thead>
<tr>
<th>Load type</th>
<th>Vertical loads</th>
<th>Horizontal forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>6.3.2(6.3.3)</td>
<td>6.3.4</td>
</tr>
<tr>
<td>Load system</td>
<td>Load Model 71&lt;sup&gt;th&lt;/sup&gt; and SW/0</td>
<td>Unloaded train&lt;sup&gt;th&lt;/sup&gt;</td>
</tr>
<tr>
<td>gr 11</td>
<td>[1.0]</td>
<td>[0.5]&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>gr 12</td>
<td>-</td>
<td>[1.0]&lt;sup&gt;d&lt;/sup&gt;</td>
</tr>
<tr>
<td>gr 13</td>
<td>[1.0]&lt;sup&gt;g&lt;/sup&gt;</td>
<td>[0.7]&lt;sup&gt;h&lt;/sup&gt;</td>
</tr>
<tr>
<td>gr 14</td>
<td>[1.0]</td>
<td>[0.5]&lt;sup&gt;k&lt;/sup&gt;</td>
</tr>
<tr>
<td>gr 15</td>
<td>[0.5]</td>
<td>[0.5]&lt;sup&gt;n&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

Table 4. Assessment of groups of traffic loads (characteristic values of the multi-component actions).

15.4 Fatigue Load models: ENV 1991-3
For normal traffic, based on characteristic values of Load Model 71, including the (reduced) dynamic factor $\phi$,
the fatigue assessment, depending on whether the structure carries standard traffic mix or predominantly heavy
freight traffic, shall be carried out on the basis of usual traffic (=standard traffic mix) or traffic with 250 kN-
axles (=heavy traffic mix).
Each of the mixes is based on an annual traffic tonnage of 25 x10<sup>6</sup> tonnes passing over the bridge on each track.
For structures carrying multiple tracks, the fatigue loading shall be applied to a maximum of two tracks in the
most unfavourable positions. For example, the standard traffic mix and heavy traffic mix are shown in table 5
and standard traffic type 1 is illustrated by fig. 6.
Table 5. Traffic mix: Left: Standard traffic mix. Right: Heavy traffic mix

As an example, to ensure that the following conditions are satisfied, for a bridge mainly loaded by usual traffic, the following simplified safety verification can be carried out:

\[
\gamma_f \lambda \Phi_2 \Delta_{71} \leq \frac{\Delta \sigma_c}{\gamma_M}
\]

where
- \(\gamma_f\) = partial safety factor for fatigue loading, \(\gamma_f = 1.00\)
- \(\lambda\) = factor which takes account of the service traffic on the bridge and the span of the member
- \(\Phi_2\) = dynamic factor
- \(\Delta_{71}\) = stress range due to the load model 71 being placed in the most unfavourable position for the element under consideration
- \(\Delta \sigma_c\) = the reference value of the fatigue strength element under consideration.

The reference value \(\Delta \sigma_c\) is the fatigue strength at \(N_c = 2 \times 10^6\) cycles

\(\gamma_M\) = partial safety factor for fatigue strength

15.5 Ultimate limit states

For verifications governed by the strength of structural material or of the ground, the partial factors on actions for ultimate limit states in the persistent, transient and accidental design situations are given in table 6. Unless specified otherwise, \(\psi\)-factors for railway bridges are taken as given in table 7.

Table 6. Partial factors on actions – ultimate limit states for railway bridges. (footnotes: see Eurocode 1991-3)

\[
\sum_j \gamma_{G,j} G_{k,j} + \gamma_{Q,j} Q_{k,j} + \sum_{i=1} \gamma_{Q,i} \psi_{Q,i} Q_{k,i}
\]

Table 7. \(\psi\)-factors for railway bridges (footnotes: see Eurocode 1991-3).
15.6 Serviceability limit states

Checks on bridge deformation are required for the following conditions:

- for safety purposes to confirm the stability and continuity of the track and to ensure that rail/wheel contact is maintained:
  - vertical accelerations of the deck
  - twist of the deck
  - rotations at the end of the deck
  - change of horizontal angle
- for passengers’ comfort
  - vertical deflection of the deck
- crack width control

**Vertical acceleration of the deck**

- The check on vertical acceleration is required for speeds \( V > 220 \text{ km/h} \) or when the natural frequency of the structure is not within the limits shown in fig. 9. The check must be based on actual traffic.
- A limiting value of 0.35g for vibrations up to 20Hz should be taken for decks with ballasted tracks unless specified otherwise. For higher values, the stability of the ballast is considered to be at risk (and in extremis a certain loss in wheel/rail contact). In case of non-ballasted tracks a limiting value of the maximum vertical acceleration up to 30Hz of 0.50g can be chosen.
- Where \( V \leq 220 \text{ km/h} \) and the natural frequency of the structure is within the limits shown in fig. 9, the risk of excessive acceleration is covered.

Fig. 7 indicates the degree of non-linear behaviour in the ballast, which commences at above 0.8g corresponding to an observed change in the integrity of ballast. Consideration of the acceleration levels at which the ballast started to exhibit non-linear behaviour and application of a factor if safety of 2.0 results in a permitted acceleration level of 0.35g.

**Twist of the deck**

- The twist of the bridge deck is to be calculated taking into account the characteristic values of Load Model 71 multiplied by \( \phi \).
- The maximum twist measured over the length of 3m shall not exceed the following values:
  - \( 120 < V \leq 220 \text{ km/h} \) : \( t \leq 3.0 \text{mm} / 3 \text{m} \)
  - \( V \geq 220 \text{ km/h} \) : \( t \leq 1.5 \text{mm} / 3 \text{m} \)

**Rotations at the end of the deck**

The rotation of the bridge deck is to be calculated taking into account the characteristic values of Load Model 71 multiplied by \( \phi \) and including the consequence of temperature differentials. The maximum angular rotations at the end of the deck on the axis of the track shall not exceed the following:

\[
\theta = \begin{cases} 
5 \times 10^{-7} \text{ radians} & \text{for single track bridges} \\
10 \times 10^{-7} \text{ radians} & \text{for double track bridges} \\
2 \times 10^{-7} \text{ radians} & \text{for transition between the deck and the embankment} \\
4 \times 10^{-7} \text{ radians} & \text{between two consecutive decks} 
\end{cases}
\]

\[
\theta_1 + \theta_2 = \frac{\text{length of the transition}}{\lambda_\text{emb}} \text{ radians} \\
\theta_1 + \theta_2 = \frac{\text{length of the transition}}{\lambda_\text{emb}} \text{ radians} \\
\theta_1 + \theta_2 = \frac{\text{length of the transition}}{\lambda_\text{emb}} \text{ radians} \\
\theta_1 + \theta_2 = \frac{\text{length of the transition}}{\lambda_\text{emb}} \text{ radians}
\]

where:
- \( \lambda_\text{emb} \) is the distance between the rail and the centre of the bridge bearing.

**Angular rotations at the end of the decks.**
Change of horizontal angle

This condition shall be checked for:

- Load Model 71 multiplied by the dynamic factor \( \phi \), wind loads, nosing force, centrifugal forces and the effect of temperature differentials between the two sides of the bridge together.

The horizontal deflection \( \delta_h \) of the deck shall not produce:

- an angular variation greater than the values given in table 8,
- or
- a radius of horizontal curvature less than the values given in table 8.

The horizontal deflection includes the deformation of the superstructure and the substructure (including piers, piles and foundation).

Vertical deflection of the deck

To ensure passenger comfort, limiting values are given for the maximum vertical deflection of railway bridges as a function of the span length \( L \) and the train speed \( V \).

The comfort of the passengers depends on the vertical acceleration \( b_v \) inside the coach during travel. Levels of comfort are classified as shown in table 9.

The vertical deflections shall be determined at the centre line of the track due to traffic loading:

- Load Model 71 increased by the dynamic factor \( \phi \).
- and
- the actual traffic increased by the dynamic factor \( \phi \) when determining the dynamic behaviour in the case of resonance or excessive vibrations of the deck.

For bridge decks carrying double track, the checks for the limits of deflection and vibration shall be made with only one track loaded. Fig. 8 shows the limiting values on deflection.

Where the structure consists of fewer spans, the limiting values shall be increased by the factors given below:

- for 1 span structure: by a factor 2
- for 2 span structure: by a factor 1,5
15.7 Dynamic effects

The dynamic factor takes account of the dynamic magnification of stresses and vibration effect in the structure but does not take account of resonance effects and excessive vibrations of the deck.

For the static stresses and deflections under Load Model 71 (and Load Models SW) the dynamic factor is taken as $\phi_2$ or $\phi_3$, whichever is appropriate, as follows:

a. for carefully maintained track:

$$\phi_2 = \frac{1.44}{\sqrt{V_{\max}} - 0.2} + 0.82$$

with:

$$1.00 \leq \phi_2 \leq 1.67$$

b. for track with standard maintenance

$$\phi_3 = \frac{2.16}{\sqrt{L_0} - 0.2} + 0.73$$

with:

$$1.00 \leq \phi_3 \leq 2.0$$

where $L_0$ is the “determinant” length in [m] as defined by table 10.

The dynamic factor applies only for speeds $V_{\max} = 220$ km/h (NAD: 200 km/h) and where the natural frequency of the structure is within the limits as shown in figure 9.

Fig. 9. Limitation on natural frequency using the dynamic factors.

In a bridge, the natural frequencies of an element are related to the deflected form under permanent action. For a simply supported structure subjected to bending, the natural frequency (first bending frequency) may be obtained from the formula:

$$\omega = \frac{17.75}{\sqrt{\delta_0}}$$

$\delta_0$ is the deflection at mid span due to permanent actions in [mm]. In case of a concrete bridge, a short term modulus must be used in accordance with a loading period appropriate to the passage of the train.

The factor $\phi_0$ can also be expressed by

$$\phi_0 = \max[\frac{y_{\text{dyn}}}{y_{\text{stat}}}]$$

If $\phi_0 > \phi$, than $\phi$ must be used (see NAD).

Table 10. Determinant lengths $L_0$.
(Similar tables exist for deck plate structures, structural components, etc.).
For the static stresses and deflections under actual service trains, considering the greatest speed at which the vehicles can travel, the dynamic factor is taken as

\[
either \quad 1+\varphi = 1+\varphi' + \varphi'' \\
\text{or} \quad 1+\varphi = 1+\varphi' + 0.5\varphi''
\]

with:

For the fatigue assessment, the dynamic enhancement for each service train is reduced to:

\[1+0.5(\varphi' + 0.5\varphi'')\]

Where the characteristics of the structure are such that it is not within the limits specified by fig. 9 and the train speed is greater than 220 km/h, there is a risk that resonance or excessive vibration of the deck may occur. The dynamic effects are not covered by the dynamic factors and further detailed calculations shall be made. The additional calculations shall be made for the maximum speed as well as for speeds of

\[v_{crit} = \frac{n_vL_{vehicle}}{i} \quad i = 1,2,3,4\]

Example:

Bridge on the line Würzburg - Hanover
Steel beams encased in concrete
L = 11.8m, \(n_v = 10,35\) Hz, \(\zeta = 2\%\), \(v_{max} = 280\) km/h.

Trans:
- ICE 1: \(L_{vehicle} = 26.4m\)
- Thalys: \(L_{vehicle} = 16.7m\)
- Talgo: \(L_{vehicle} = 13.14m\)

Critical Speeds [km/h]

<table>
<thead>
<tr>
<th>i</th>
<th>Talgo</th>
<th>Thalys</th>
<th>ICE 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>490</td>
<td>697</td>
<td>984</td>
</tr>
<tr>
<td>2</td>
<td>245</td>
<td>348</td>
<td>462</td>
</tr>
<tr>
<td>3</td>
<td>163</td>
<td>232</td>
<td>328</td>
</tr>
<tr>
<td>4</td>
<td>122</td>
<td>174</td>
<td>246</td>
</tr>
</tbody>
</table>

Fig. 10. Example of results dynamic analysis railway bridge.

In addition to the analyses of the dynamic factor, to ensure passenger comfort (and loss of wheel/rail contact, destabilization of the ballast, etc.) limiting values are given for the maximum vertical deflection of railway bridges as a function of the span length L and the train speed V.

### 15.8 Comfort criterion

To ensure a certain level of passenger’s comfort in a train when passing a bridge, the Eurocode on traffic loads on bridges prescribes maximum levels of vibrations in terms of limiting values for the vertical deflections. Levels of comfort are classified as shown in table 11.
Based on these limits, fig. 8 has been constructed which shows the dependency of the stiffness of the railway bridge and velocity of the train. This figure gives the maximum deflection per length of the bridge: $C=\delta/L$. As shown in fig. 8, the lines given consist of two parts. Namely, a horizontal part for the small span region and a V-shaped part for the region of larger spans. The lowest line $\delta/L = 1/600$ given is considered as a limit situation (practical experience, like risk on safety due to secondary effects) on deflection. This because larger displacements might result in unacceptable aspects like a large non-linear behavior of the bridge, which introduces the so-called secondary stresses. Clearly, it is not possible to give a common answer to the question whether a vibration is acceptable. This because comfort is subjective and entirely depends on the perception of humans. For a classification on perception, three categories of influencing variables can be defined.

**Stimulus variables**

The effect of vibration on a human body can be categorized into:

**Direction of movement:** the excitation can be transversal, longitudinal or vertical

**Intensity:** the energy of the excitation, expressed in maximal amplitude

**Frequency:** the number of cycles per second

**Duration:** the duration of the exposure

**Character:** the form, like harmonic, periodic, repetitive

**External variables**

These parameters deal with the past of the person in question, his culture, his education and his occupation. For example, vibration in a vehicle may become unacceptable because a person becomes aware of a better alternative.

**Human variables**

Relevant parameters are the condition and stamina of the person incl. his length and weight and the way the person stand, sits or lie. Also, personal features like sociological background, character and intelligence are found to have a certain influence.

In the past extensive research has been carried out on a link between vibration and comfort. Variables are observed like the frequency of a vibration, which largely influences comfort. The perception of vibrations with the same amplitude depends on the frequency. Even if vibrations have the same acceleration, their effects on perception differ, depending on the frequency. Some evaluation parameters used when considering comfort criteria are based on acceleration, velocity or displacement. Interpretation of these parameters can be based on RMS values (root mean square), peak values, etc. The maximum displacement, velocity and acceleration are interchangeable with the use of mathematics. The basic formula for a vibration is:

$$x(t) = x \sin(\omega t + \varphi)$$

with

$$\omega = 2\pi f$$

The first derivative is the expression for speed and the second derivative is the expression for acceleration. Since we are interested in the maximum acceleration, the next simplification can be made:

$$v(t) = \frac{dx}{dt} = x \omega \cos(\omega t + \varphi) \quad a(t) = \frac{dv}{dt} = -x \omega^2 \sin(\omega t + \varphi) \quad a = x \omega^2$$

The relation between displacement, speed and acceleration is summarized in table 12.

<table>
<thead>
<tr>
<th>Displacement [x]</th>
<th>Velocity [v]</th>
<th>Acceleration [a]</th>
</tr>
</thead>
<tbody>
<tr>
<td>x = x</td>
<td>x = v / 2\pi</td>
<td>x = a / (2\pi)^2</td>
</tr>
<tr>
<td>\nu = 2\pi x</td>
<td>\nu = v</td>
<td>\nu = a / 2\pi</td>
</tr>
<tr>
<td>a = (2\pi)^2 x</td>
<td>a = 2\pi \nu</td>
<td>a = a</td>
</tr>
</tbody>
</table>

**Table 12. Transfer functions on relation between displacement, velocity and acceleration.**
For an acceleration of 1.0 m/s² some general observations about the effect of the frequency of the vibration on the peak to peak displacement (up and down) are as follows:

- \( f = 0.2 \text{ Hz} \) 1.3 m similar to the vertical oscillation of a ship
- \( f = 1.0 \text{ Hz} \) 50 mm similar to the movement of a pendulum
- \( f = 5.0 \text{ Hz} \) 2 mm similar to movement on the seat of an old car on a rough road
- \( f = 20 \text{ Hz} \) 0.13 mm similar to the passengers comfort inside a helicopter

An indication about the effect of frequency on human is as follows:

- \( f < 0.5 \text{Hz} \) the motion may cause symptoms of motion sickness like sweating and vomiting
- \( f > 8 \text{Hz} \) the effects of motion on the body rapidly diminish
- \( 10 < f < 20 \text{ Hz} \) the voice may be caused to warble
- \( 15 < f < 60 \text{ Hz} \) the vision may be blurred

As shown, the perception of vibrations with the same amplitude depends largely on the frequency. For an answer on what frequencies have greatest effects on comfort, a frequency-weighting filter is defined. These curves define the multiplying factor for each frequency, according to the effects on the body. The weighting has high values at frequencies of great importance and low values at frequencies that have little effects. Figure 11 shows an example of ISO and NASA frequency weighting filters.

**Fig. 11. ISO and NASA frequency weighting filters for vertical vibrations on sitting humans.**

The acceleration of vibration can be expressed in terms to the peak acceleration. With complex motions this may result in the severity of the vibration being determined by one unrepresentative peak, thus it is preferred to express severity in terms of an average measure. The measure, used with signal processing techniques, is the root mean square value RMS:

\[
a_{\text{RMS}} = \sqrt{\frac{1}{T} \int_0^T a^2(t) \, dt}
\]

The solution of the integral is

\[
a_{\text{RMS}} = \frac{a_0}{\sqrt{2}}
\]

That means that the RMS of sinusoidal vibrations can be obtained by dividing the maximum amplitude by 1.4. For the existing evaluated formulas and values used on comfort limits, figures 12 and 13 describe the minimum threshold and the comfort boundaries for the different comfort models, for sinusoidal vibrations expressed in the RMS vertical acceleration and RMS vertical displacement.

Standards limit the maximum for a certain comfort describing parameter. This parameter doesn’t always depend on the duration of the vibration, like the Eurocode, while some studies show a trade-off between time and maximum magnitude of the vibration. Discomfort increases with duration of the exposure. This causes the maximum amplitudes to diminish in time for the same level of comfort.
Fig. 12. Limits for RMS values of vertical acceleration.

Fig. 13. Limits for RMS values of vertical displacement.
As shown in figure 14, three lines have been plotted, starting in the upper left corner, which show the direction of diverse time effects. One representing fourth power time dependence (like the VDV), one representing the third power time dependence (LI<sub>h</sub>) and one representing the second power time dependence (MSDV).

**VDV** = the vibration dose value, used by BS and ISO. The VDV is a robust method of assessing the severity of all motions (deterministic or random, stationary or non-stationary, transient or block) but has some disadvantages. The major disadvantage is that it cannot be used for the assessment for vibrations with short or long duration.

Very short duration results in too high values for the maximum acceleration and by lengthening the duration, the acceleration limit can lay beneath the threshold.

**LI<sub>h</sub>** = the level of discomfort harmonic, used by ERRI (European Rail Research Institute). This method is capable to take the frequency and duration effects into account and is based on little experiments with short time exposure.

**MSDV** = motion sickness dose value, used by BS and ISO. The most important frequency for nausea is 0.2 Hz, in contrary to the 5 Hz for comfort. Therefore, nausea has to be described by another frequency weighting filter than comfort.

The method considering LI<sub>h</sub> is most commonly used and will be explained in more detail. The LI<sub>h</sub> is an indicator for the percentage of people experiencing discomfort by a harmonic vibration.

\[
LI_h = 107.77 \sqrt{\frac{1}{t_0} \int_0^{t_0} |\phi(t)|^3 \, dt}
\]

The signal must be weighted for the vertical vibrations by weighting filter W<sub>b</sub>, which is shown in figure 15.

As shown in figure 16, an increase of LI<sub>h</sub> results to an increase of the percentage of people with a negative perception of the vibrations. Conform the ERRI 190, a LI<sub>h</sub> of 45 expresses good comfort. In that case the perception of the vibrations is negative for 10.4% of the subjects versus 82.7% positive.

**Fig. 15. Frequency weighting filter W<sub>b</sub> from British Standards.**
Recently, investigations on comfort criteria have been carried out based on the so-called RMQ-model (root mean quad model). This new comfort model assesses the occurring vibrations on duration and frequency and is not only able to predict the level of comfort for short time events, like passing a bridge, but also for long duration vibrations, like complete rides. The model can be used in combination with a probabilistic dynamic approach.

15.9 Effect of different trains

| Thalys (Thalys PBKA) | Configuration:  
|  | Basic:  
|  | 26 axles, length 200 m  
|  | Loc - subloc - 8 * coach - subloc - loc  
|  | Extended: 2 * basic  
|  | First natural frequency passenger coach is 0.8 Hz  
|  | Average weight in use: 1.7 ton/m  

| Intercity Express (ICE-2) | Configuration:  
|  | 56 axles, length 350 m  
|  | Loc - 12 * coach - loc  
|  | First natural frequency passenger coach is 0.9 Hz  
|  | Average weight in use: 1.6 ton/m  

| Double Decker (DDM) | Configuration:  
|  | Basic:  
|  | 16 axles, length 95 m  
|  | loc1700 - 2 * coach - endcoach  
|  | Extended: 2 * basic  
|  | First natural frequency passenger coach is 1.5 Hz  
|  | Average weight in use: 1.9 ton/m  

According to train types specified in table 13 as illustrated in figure 17, most trains have the first dominant frequency around 1.0 Hz and their second around 10 Hz.

For the dynamic behaviour (interaction train – bridge) it is important to know whether the differences in train types have effect on the vibrations and comfort. The main characteristics for most common type of trains are given in figure 17.
15.10 Reconstruction Eurocode, comfort and dynamic model

By evaluating different models of trains passing bridges and different comfort criteria, a rational background is given for the stiffness limits as given in figure 8. Information considering the following approaches (models) will be given:

- **Approach one, basic model**
- **Approach two, model with transfer function**
- **Approach three, single mass spring system**
- **Approach four, coupled two mass spring system**

The approaches differ in the way they describe the bridge, the train, the interaction and the comfort limit, as shown in table 14.

---

<table>
<thead>
<tr>
<th>Run</th>
<th>Type of train</th>
<th>First dominant frequency</th>
<th>Peak value</th>
<th>Second dominant frequency</th>
<th>Peak value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DDM</td>
<td>1.30</td>
<td>0.16</td>
<td>9.0</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>Thalys PKBA</td>
<td>0.90</td>
<td>0.13</td>
<td>8.0</td>
<td>0.015</td>
</tr>
<tr>
<td>3</td>
<td>DD-IRM</td>
<td>1.35</td>
<td>0.31</td>
<td>9.0</td>
<td>0.03</td>
</tr>
<tr>
<td>4</td>
<td>ICE-35M</td>
<td>0.75</td>
<td>0.15</td>
<td>12.0</td>
<td>0.15</td>
</tr>
<tr>
<td>5</td>
<td>ICM</td>
<td>1.18</td>
<td>0.30</td>
<td>11.9</td>
<td>0.07</td>
</tr>
<tr>
<td>6</td>
<td>DD-IRM</td>
<td>1.20</td>
<td>0.22</td>
<td>12.0</td>
<td>0.07</td>
</tr>
<tr>
<td>7</td>
<td>ICM</td>
<td>1.15</td>
<td>0.14</td>
<td>9.5</td>
<td>0.07</td>
</tr>
<tr>
<td>8</td>
<td>DDM</td>
<td>1.25</td>
<td>0.15</td>
<td>12.0</td>
<td>0.02</td>
</tr>
<tr>
<td>9</td>
<td>DDM</td>
<td>1.4</td>
<td>0.10</td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

‘X’ – no clear peak found.

Table 13
Average peak values for each trip.

Fig. 18. Average amplitude spectra of trains trips, starting with trip number 1 in the left upper corner until trip number 9 left below.

Table 14. Differences between approaches.
Approach one, basic model.
For this model, the bridge is infinite stiff and already deformed. The load of the train has no effect on the shape of the deflection of the bridge. The train is simulated as a moving mass, without mass or damping, and moves along the bridge like it is infinite stiff joined together with the bridge.

Summary:

**Bridge:** continuous and single span, infinite stiff, preshaped
**Trains:** modelled by a stiff mass
**Comfort criterion:** maximum peak acceleration of 1.0 m/s²
**Train – bridge interaction:** no interaction

The deformation of the bridge can have different shapes, depending on the modeling of the supports, as shown in figure 19.

In case of a single span bridge, the following set of equations are used:

- the shape of the deflection is described by:
  \[ z_b = -\delta \sin \left( \frac{\pi x}{L} \right) \]

- the horizontal position of the train can be described by:
  \[ x = v_x t \]

- resulting in the vertical position of the wheel of the train
  \[ z_b = -\delta \sin \left( \frac{\pi L}{L} \right) \]

- the vertical velocity is
  \[ v_z = \frac{dz}{dt} = -\pi \delta \cos \left( \frac{\pi x}{L} - \frac{\pi}{2} \right) \]

- the vertical acceleration is
  \[ a_z = \frac{d^2 z}{dt^2} = \pi^2 \delta \sin \left( \frac{\pi x}{L} - \frac{\pi}{2} \right) \]

In case of a continuous span bridge, the following set of equations are used:

- the shape of the deflection is described by:
  \[ z_b = \frac{\delta}{2} \cos \left( \frac{\pi x}{L} \right) \]

- resulting in the vertical position of the wheel of the train
  \[ z_b = \frac{\delta}{2} \cos \left( \frac{\pi L}{L} \right) \]

- the vertical velocity is
  \[ v_z = \frac{dz}{dt} = \pi \delta \sin \left( \frac{\pi x}{L} - \frac{\pi}{2} \right) \]

- the vertical acceleration is
  \[ a_z = \frac{d^2 z}{dt^2} = 2\pi^2 \delta \cos \left( \frac{\pi x}{L} - \frac{\pi}{2} \right) \]

For a continuous bridge, the maximum acceleration occurs at the supports and maximum negative acceleration halfway the span. For a single span, the maximum acceleration occurs in the middle of the span. The absolute value of the maximum vertical acceleration is the amplitude in next equation

\[ a_z, \text{continuous} = 2\pi^2 \delta \]

Taken into consideration that the fast passenger trains weights 60% less than heavy transport trains, for which the maximum deflection is represented in figure 8, an extra factor has been introduced.

\[ \lambda \delta, \text{cr} = \delta \quad \text{with} \quad \lambda = 0.4 \]

The target for description of the basic model is to obtain a definition about minimal stiffness of the bridge, based on the maximal acceleration. The stiffness of a bridge is given by

\[ C = \frac{L}{\delta} \]

Resulting into the following equations:

\[ C_{\text{single}} = \frac{\lambda \sqrt{2} \pi^2}{a_z L} \quad C_{\text{continuous}} = 2\lambda \frac{\sqrt{2} \pi^2}{a_z L} \]

15. (Comfort related) design railway bridges
Dr. A. Romeijn
The Eurocode limits the vertical acceleration to 1 m/s², indifferent of the frequency and the duration. By plotting the velocity versus the length for two values of C, the minimum value and a higher one, fig. 20 is obtained.

**Fig. 20. Stiffness for continuous span for maximum acceleration 1 m/s².**

**Approach two, model with transfer function**

Adjustments to basic model (approach one). The acceleration of the bridge is not the same as the acceleration of the coach. This difference can be taken into account with the use of a transfer function. Another improvement on the basic model is the frequency dependence of the limit for vibrations. Summary:

- **Bridge:** continuous and single span, infinite stiff, preshaped
- **Trains:** modelled by a system with a transfer function
- **Comfort criterion:** maximum frequency weighted Peak acceleration of 1.0 m/s²
- **Train – bridge interaction:** semi dynamic (movement bridge effects movements train, but not in the other direction)

The transfer function describes the relation between the magnitudes of the acceleration of the train and the acceleration of the bridge. As shown in fig. 21, the majority of the trains have their first natural frequency between 0.7 and 1.7 Hz, which must be emphasized by the transfer function.

**Fig. 21 Acceleration transfer function from bridge to coach.**

Taken the transfer function into account results into the following equation:

\[
C_{\text{single}} = \frac{L}{\delta} = \lambda \frac{\nu^2 \pi^2}{a L} H \frac{\nu_p}{\nu_f} \quad C_{\text{continuous}} = \frac{L}{\delta} = 2\lambda \frac{\nu^2 \pi^2}{a L} H \frac{\nu_p}{\nu_f} \quad f = \frac{\nu_p}{L}
\]

As shown in fig. 22, the ISO 2613 specifies for a duration of 1.0 minutes the values on maximal peak acceleration of 1.0 m/s².

**Fig. 22. ISO 2613: maximal peak acceleration of 1.0 m/s².**

The limit for the maximum acceleration is a function of the frequency. This has to be added to the equation resulting into
When considering this transfer function and comfort limits, the limits for the stiffness are as shown in figure 23.

**Approach three, single mass spring system**

Adjustments to approach two. The train is simulated as a damped one-mass-spring system, and the maximal vertical acceleration is dependent on the frequency like in the British Standards and ISO 2613. The bridge is modeled in the same way as the first two approaches.

Summary:

- **Bridge:** continuous and single span, infinite stiff, preshaped
- **Trains:** modelled by a single mass spring system
- **Comfort criterion:** maximum frequency weighted Peak acceleration of $1.0 \text{ m/s}^2$
- **Train – bridge interaction:** semi dynamic (movement bridge effects movements train, but no in the other direction)

The transfer function for the acceleration of the bridge to the acceleration of the mass for a single mass-spring system is equal to

\[
H_{\text{seg}} \approx \sqrt{\frac{H_{\text{seg}}}{2k\Omega^2}}
\]

**Fig. 24. Single mass spring system.**

The recommendations from the HSL600 prescribe the next characteristics for several high velocity trains:

<table>
<thead>
<tr>
<th>Train</th>
<th>Part</th>
<th>Mass</th>
<th>Natural Frequency</th>
<th>Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thalys-1</td>
<td>Locomotive</td>
<td>44680</td>
<td>1.46</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>Coach</td>
<td>27140</td>
<td>0.704</td>
<td>5.5</td>
</tr>
<tr>
<td>ICE-2</td>
<td>Locomotive</td>
<td>60768</td>
<td>1.12</td>
<td>10.4</td>
</tr>
<tr>
<td></td>
<td>Coach</td>
<td>33930</td>
<td>0.64</td>
<td>3.8</td>
</tr>
</tbody>
</table>

**Table 15. Train characteristics.**

Based on these values, a representative value for stiffness $c$ and $k$ can be calculated using the following set of equations:

\[
\omega = \sqrt{\frac{k}{m}} = 2\pi f
\]

\[
k = 4\pi^2 f^2 m
\]
\[ C_r = 2\sqrt{k/m} \quad \text{where} \quad C_r = \text{critical damping} \]

This results into the following data:

<table>
<thead>
<tr>
<th>Train</th>
<th>Part</th>
<th>Damping</th>
<th>k</th>
<th>( C_r )</th>
<th>c</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thalys-1</td>
<td>Locomotive</td>
<td>4.9</td>
<td>3759920</td>
<td>819740</td>
<td>40167</td>
</tr>
<tr>
<td></td>
<td>Coach</td>
<td>5.5</td>
<td>531025</td>
<td>240100</td>
<td>13206</td>
</tr>
<tr>
<td>ICE-2</td>
<td>Locomotive</td>
<td>10.4</td>
<td>300936</td>
<td>855269</td>
<td>88948</td>
</tr>
<tr>
<td></td>
<td>Coach</td>
<td>3.8</td>
<td>548660</td>
<td>272881</td>
<td>10369</td>
</tr>
</tbody>
</table>

*Table 16. Train characteristics using approach 3.*

Fig. 25 illustrates the transfer function of a Thalys passenger coach.

A high peak value exists at its natural frequency and the response to a static displacement can be found at a frequency of 0 Hz and is as expected exactly 1.

The maximum acceleration of the mass, representing the maximum vertical acceleration of the coach on a continuous span, can be calculated by:

\[
\dot{a}_{z, \text{continuous}} = 2\pi^2 f^2 \delta \cdot H_{aw} (f)
\]

*Fig. 25. Transfer function of a Thalys passenger coach.*

As mentioned in ISO 2631 and illustrated by figure 26, the value for the maximum vertical vibration in the coach is limited.

For the design of a bridge, the analyzed peak acceleration is not allowed to exceed the values given by the red line of figure 26, so:

\[
\dot{a}_{z, \text{ISO}} \geq 2\pi^2 f^2 \delta \cdot H_{aw} (f)
\]

*Fig. 26. Maximal acceleration from the ISO 2613, peak and RMS.*

This result into the condition

\[
\delta \leq \frac{\dot{a}_{z, \text{ISO}}}{2\pi^2 f^2 \cdot H_{aw} (f)}
\]

A multiplication factor \( \lambda = 0.40 \) representing the ratio between fast passengers trains weight and heavy train transport weight result into the equation:

\[
\delta_{\text{UIC}} \leq \frac{1}{\lambda} \frac{\dot{a}_{z, \text{ISO}}}{2\pi^2 f^2 \cdot H_{aw} (f)}
\]
With these formulae, for each combination of length, train type, velocity, etc. the stiffness limits can be analysed. Some results are given in fig. 27. If more trains would be used, the enclosed line could be rather similar to the lines given by Eurocode.

![Fig. 27. Combined plot for stiffness limits for four different types of trains.](image)

Although a complex system of modelling is considered, there are still some shortcomings in the modelling, namely:

- the bridge is static, so the mass of the train has no effect on the deflection of the bridge
- the passenger bridge interaction has been modeled by a single mass-spring system, while in reality a complicated mass-spring systems exist. The second natural frequency of a train is somewhere around 10 Hz. If the train would be modelled by a two mass spring system, the second natural frequency will give lower C values for these high frequencies.
- the model gives no information on the influence of the effect of roughness of the rail, and the change in stiffness when the train moves form soil to the bridge.

**Approach four, two mass spring system**

Because more natural frequencies are present in a train, the model from approach three is extended considering a two mass spring system.

The transfer function can be derived in a similar way as described for approach three and an example is illustrated in figure 28.

\[ m_1 = \text{mass of the train} \]
\[ m_2 = \text{mass of the wheel set} \]
\[ k = \text{spring stiffness} \]
\[ d = \text{damping constant} \]

![Fig. 28. Two mass spring system.](image)

Assumptions made for the example given in figure 29 are:

- the first two natural frequencies are 1.0 and 10 Hz
- the mass of the complete train is 50 tons, with the majority of the weights in the wheel set.

![Fig. 29. Transfer function for 2 mass-spring system.](image)
The results of the two mass spring system can be found in figure 30. 
The results shown in fig. 30 are close to the results obtained considering approach three. 
The transfer function of approach four is for high frequencies higher than the one for approach three, resulting in higher stiffness limits for high frequent bridge passages. (Small span with high velocities).

**Fig. 30. Stiffness limits from a couple two mass spring system.**

### Comparison of results for the approaches analyzed

The prescribed limit from the first approach of excellent comfort of an unweighted peak value of 1.0 m/s² is not realistic. Especially for long bridges, with low frequent vibrations is this limit too severe. This frequency dependency can be evaluated with a frequency weighting filter, like the one used in approach two. The second approach results in the same stiffness limits like the ones in the Eurocode. This approach uses a sort of circumscribed transfer function for all types of trains and a weighted peak value of 10 m/s². The transfer function takes only the first natural frequency into account. The third model approaches the Eurocode only for long spans, because small spans result in high frequent vibrations in order of 10 Hz, and the transfer function is approx. zero at 10 Hz. This, and the fact that trains contain several natural frequency leads to an extension of the model to a two mass spring system: the fourth approach. This approach, with arbitrary constants, can reach the stiffness limits from the Eurocode. Depending on type of bridge and railway traffic considered, the convoy mechanical properties, its mass and its dampers, can sometimes be neglected and also the interaction between the convoy and the bridge structure needs not always to be taken into account. This hypothesis is only justified by the fact that the convoy mass is negligible versus the structure mass and the structure deformations stay very small.

### 15.11 Software for describing the interaction vehicle-track-bridge-soil

For a description of the vibration effects caused by the interaction of:
- natural frequency of the bridge
- wheel imperfection
- regularly spaced supports of the bridge
- spacing of train axles
- natural frequency of the train
- roughness of the track

specific software has been developed like TRINT (Train Rail INTeraction). Some information is given about the possibilities using TRINT.

**natural frequency of the bridge**

The approaches from previous paragraph used a static model for the bridge instead of a dynamic model. TRINT uses a dynamic model by considering an Euler beam for the bridge. The first natural angular frequency depends on the mass per length, the bending stiffness and the length of the bridge and can be calculated with the following equation:

\[
\omega_b = C \sqrt{\frac{EI}{\rho AL^4}}
\]

where

- \( EL \) = bending stiffness [Nm²]
- \( \rho \) = density [kg/m³]
- \( A \) = area [m²]
- \( L \) = span length [m]
C = constant, depending on boundary condition
Hinged: \( C = \pi^2 \)
Fixed: \( C = 22.4 \)

wheel imperfection
Wheel imperfection generates additional vibrations in the bridge-train system. The effect of wheel imperfection depends on the shape of the deformation.

\[ d = + \]

\[ = \]

\[ = \]

\[ = \]

\[ = \]

\[ = \]

\[ = \]

\[ = \]

\[ = \]

The displacements of the axle as a result of these imperfections can be calculated according to:

\[ z_{ax} = dR \sin \left( \frac{2\pi f t}{K} \right) + \phi \]

where

\[ dR = \text{imperfection [m]} \]
\[ t = \text{time [s]} \]
\[ v = \text{velocity of the trains [m/s]} \]
\[ R = \text{radius [m]} \]
\[ \phi = \text{phase [rad]} \]
\[ n = \text{number of periods per rotation} \]

which can be illustrated as shown in fig. 32.

\[ \]

\[ \]

\[ \]

\[ \]

\[ \]

train modeling
The software package is able to describe trains with more than two wheel sets per wagon, or two wagons sharing the same wheel set, or even a combination of these within a single train, etc.
An example on different configurations of trains is given in fig. 33.

\[ \]

\[ \]

\[ \]

roughness
In practice every rail has a certain roughness. According to ISO specification, the roughness is described through its power spectral density (PSD). The roughness is can be translated as input data from which a RMQ histogram can be analysed.

15.12 Bridge parameter: damping
The logarithmic decrement of damping \( d \) [%] is defined as

\[ \delta = \frac{\ln(a_{n+1})}{\ln(a_n)} \]

With \( a_{n+1} \) and \( a_n \) are two consecutive amplitudes of the vibration of as system left free to vibrate.
The logarithmic decrement of damping \( d \) is calculated by:

\[
d = d_s + d_a + d_d \quad \text{where}
\]

\[
d_s = \text{logarithmic decrement of structural damping}
\]

\[
d_a = \text{logarithmic decrement of aerodynamic damping for the fundamental mode}
\]

\[
d_d = \text{logarithmic decrement of damping due to special devices}
\]

The damping ratio factor \( \beta \) is equal to logarithmic decrement divided by \( 2\pi \). In more practical sense, the damping ratio factor \( \beta \) is defined as the ratio between actual damping and critical damping:

\[
\beta = \frac{\delta}{2\pi}
\]

As an indication table 17 summarises \( \beta \)-values, which can be used in case of movements caused by wind loading.

The critical damping value can be expressed in terms of system mass and stiffness.

<table>
<thead>
<tr>
<th>Type of structure</th>
<th>( \beta )-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stay cables</td>
<td>0.001</td>
</tr>
<tr>
<td>Bars and hangers</td>
<td>0.001</td>
</tr>
<tr>
<td>Welded steel structure</td>
<td>0.002</td>
</tr>
<tr>
<td>Bolted steel structure</td>
<td>0.005</td>
</tr>
<tr>
<td>Prestresses concrete deck</td>
<td>0.005</td>
</tr>
<tr>
<td>Concrete structure</td>
<td>0.01</td>
</tr>
<tr>
<td>Composite steel-concrete structure</td>
<td>0.007</td>
</tr>
</tbody>
</table>

Table 17. Design value for damping ratio factor \( \beta \).

As part of the ERRI D214 studies, a database of the results of previous damping tests and measurements was set up to facilitate analysis of damping by the type of structure. The studies determined that it is appropriate to divide railway bridges into four categories: steel, concrete, composite and filler beams as shown in fig. 34. Analysis shows that there is little correlation of damping with eigen frequency. As shown in fig. 34, there is some correlation between damping and span length. Generally higher damping factors are indicated on short spans. However, in view of the divergent results obtained it seems to be necessary to adopt a pragmatic approach to the development of guidance on damping values to be used for design. The values of critical damping, proposed by [4], are shown in table 18.

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>Span ( L \leq 20 \text{ m} )</th>
<th>Span ( L \geq 20 \text{ m} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>( \zeta = 0.5 + 0.2(20-L) )</td>
<td>( \zeta = 0.5 )</td>
</tr>
<tr>
<td>Composite</td>
<td>( \zeta = 0.5 + 0.2(20-L) )</td>
<td>( \zeta = 0.5 )</td>
</tr>
<tr>
<td>Filler beam</td>
<td>( \zeta = 2.0 + 0.2(20-L) )</td>
<td>( \zeta = 2.0 )</td>
</tr>
<tr>
<td>Reinforced concrete</td>
<td>( \zeta = 2.0 + 0.2(20-L) )</td>
<td>( \zeta = 2.0 )</td>
</tr>
<tr>
<td>Prestressed concrete</td>
<td>( \zeta = 1.0 + 0.2(20-L) )</td>
<td>( \zeta = 1.0 )</td>
</tr>
</tbody>
</table>

Table 18. Values of \( \zeta \) critical damping (%) to be assumed for design purposes.
15.13 Design guide Railway bridges: some interesting articles specified in document number OVS00030 of Prorail B&I Information centre

Part I, art. 2.5: Sag
Under loaded condition (traffic loading), the bridge should still have a positive sag. Normally 0.1% of the span length is used. The vertical alignment is based on this criterion.

Part I, art. 3.2 Cross-section
For a span length larger than 100 m, a free getting off for the train passengers must exist. Therefore, in case of a trough- or plate girder bridge with a low positioned deck and a single track, the pathway must be positioned at a level of maximum 0.84 m above top of rail.

Part I, art. 4.2 Drain system
- The system should be dimensioned for a shower of 150 l/s/ha during 15 min. The whole must be drained within 15 min.
- For the combination of a ballast bed and span length larger than 10 m, a minimum slope of 1 % in transverse direction must exist. The maximum level of rainwater should not exceed the level of 250 mm from bottom side of the cross beam. At the expansion joints beneath the ballast bed the rainwater may not flow from one bridge span to the other.

Part I, art. 7.5 Jacking up
Each bridge should be provided with points for jacking up, which creates the possibility of lifting the bridge 100% free from its bearing system (supports).

Part II, art. 1.1 Walkway for maintenance
The width of the walkway should be minimal 1500 mm and located at a minimum distance from the centre of the track of (explained by fig. below):

---

Appendix 1 Type of bridge: criteria
Parameters of influence are:

- **construction depth**
The depth is defined as the distance between top of rail and lowest past of the bridge (superstructure). For the total costs, the construction depth might be an important factor. Normally, using steel only results into a minimum of construction depth. In comparison to a single bridge having two tracks, two separate railway bridges both having a single track results into a lower construction depth and higher total costs of the superstructure themselves. The use of a macadam results into an increase of construction depth (and increase of the weight of the supporting steel structure). On the contrary, the track maintenance is less expensive. This advantage disappears for a bridge span larger than 20 m.

- **span length**
For the total bridge length, an increase of span length means a decrease of piers and so costs. For the number of piers to be chosen, an optimum on cost (superstructure + substructure) exists. When using concrete railway bridges instead of steel railway bridges, this optimum exist for a smaller span length.
- soil conditions – bearing capacity
For cost optimisation, bad soil conditions mean an increase of span length. In case of bridge replacement, when still using the existing substructure, because of low self-weight the use of steel instead of concrete can be very advantageous. However, when a macadam is used the advantage of using steel is negligible.

- safety
A railway bridge should have a cross section on such a way that safety exist not only for the passing traffic but also for people responsible for maintenance, traffic passing below the bridge, etc.
This means providing the bridge with guidance in case of derailment, side walkway for maintenance, fast access of area for escape, etc. Therefore, trough girder bridges and plate girder bridges with low positioned deck might be unsuitable.
As an example, the cross section of the HSL railway Moerdijk bridge is shown.
Regarding safety of traffic passing below the bridge, the positioning of piers are an important issue. Barriers can be used.
Internationally spoken, a clearance of 8 m from tl considered to be “acceptable” safe. This means regarding extension of traffic capacity and chang:

- stagnation of existing traffic during constructing
The type of bridge chosen might be strongly relationship with another railway line or an existing railway line.
If the project concerns a replacement of a steel bridge with ballast bed, the self-weight is increased by a factor 2 – 3. For such a situation, the substructure can mostly not be used without additional bearing capacity.
If the project concerns a new built railway line crossing an existing highway or railway line, because of stagnation, steel is preferred above concrete. For a span smaller than 20 m the bridge can be constructed using prefab concrete.

- noise emission
Not considering the consequence of a bridge, as a standard the noise emission amounts to approx. 65 dB(A) for the embankments. Depending on many factors the noise due to the bridge themselves amounts to 0 – 20 dB(A). Most important factors are type of rail fastener, ballast bed, material and type of connections used (steel – concrete).
For concrete bridges with direct rail fasteners, the noise emission of the bridge is not per definition below the noise emission of steel bridges using direct rail fasteners.
As an indication only, the increase of noise caused by the bridge themselves amounts to:

Using the silent bridge technology: 0 dB(A)
Concrete bridge or composite bridge with ballastbed 1 – 5 dB(A)
Steel bridge with ballastbed 1 – 7 dB(A)
Concrete bridge or composite bridge with direct rail fastener 5 – 15 dB(A)
Steel bridge with direct rail fastener 5 – 15 dB(A)

- Static system
In comparison to a static determined system, a static undetermined system will result into a less number of bridge bearings and less use of material. However, the static undetermined system is more sensitive to support settlement which might be an important issue for the decision about the system used.

- Bridge curvature
Caused by curvature in horizontal plane, centrifugal forces occur. This force is a function of
\[
\frac{v^2}{r} \quad [v = \text{train speed, } r = \text{radius of curvature}]
\]

In general, in case of bridge curvature a box shaped instead of open shaped section can best be used.

- **Cable-stayed bridge**
  The stiffness behaviour in general (vertically as well as horizontally) and the condition of permanent tensile forces in all cable under all load conditions (the ratio on self-weight-variable loading of a railway bridge is more unfavourable than in case of a highway bridge) makes the use of a cable-stayed bridge for railway traffic very uneconomical.

- **Maintenance costs**
  As an indication only, the costs of maintenance / year is:
  
<table>
<thead>
<tr>
<th>Type</th>
<th>Maintenance Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed steel bridges</td>
<td>0.5% of initial costs</td>
</tr>
<tr>
<td>Concrete bridges</td>
<td>0.4% of initial costs</td>
</tr>
<tr>
<td>Movable bridges</td>
<td>1.3% of initial costs</td>
</tr>
</tbody>
</table>

Appendix 2  **Checklist new built railway traffic bridges**

A minimum distance between two tracks of 4 m is required. However, the Prorail-policy is the use of a distance of 4.25 m.

Appendix 6  **In addition to the NEN-codes**

Art. 2.3.2: permanent loading – settlement of pier / foundation

The bridge should be dimensioned considering the following differences caused by settlement:

- a difference of 0.01 m per support in longitudinal direction
- a rotation of 0.05° in transverse direction of the support

Art. 2.3.: lifting

Caused by lifting a difference in vertical displacement of 10 mm per support should be taken into account

Art. 3.1.2: dynamic factor so called “stootcoefficient”

In case of passengers railway traffic with \(v = 200 \text{ km/h}\) or cargo railway traffic with \(v = 120 \text{ km/h}\), the dynamic factor can be determined by NEN 6788 / NEN 6723 appendix A.2.10 and A.2.11. For a larger speed, in agreement with ProRail, the factor can be determined using prEN 1991-2.

For the design of the foundation, pier walls, etc. this factor needs not to be included.

Art. 5: accidental loading: collision

- **Caused by road traffic**: load specified by VBB 1995 (NEN 6723), art. 4.4.2.2 and art. 4.4.2.3.

  **Art. 4.4.2.2**

In case of absence of adequate protection by a barrier, the supporting substructure of the railway bridge beside the road should be designed for an impact loading with a surface area of 0.25 m x 1.00 m (as a maximum width, else column width) at a level of 1.20 m + road:

<table>
<thead>
<tr>
<th>Traffic Type</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highway</td>
<td>(F_{a,rep} = 2000 \text{ kN}, \text{ or } F_{a,rep} = 1000 \text{ kN})</td>
</tr>
<tr>
<td>Other road</td>
<td>(F_{a,rep} = 1500 \text{ kN}, \text{ or } F_{a,rep} = 750 \text{ kN})</td>
</tr>
</tbody>
</table>

  **Art. 4.4.2.3**

In case of absence of adequate protection, the superstructure of the railway bridge crossing a
road bridge should be designed for an impact loading with a surface of 0.25 x 2.00 m² parallel
to the road at most severe position:

- highway traffic: \( F_{\text{a,prim}} = 2000 \text{ kN} \)
- other road traffic: \( F_{\text{a,prim}} = 1500 \text{ kN} \)

Design means, avoiding bridge instability and avoiding parts of the bridge drops down.

And also in vertical plane on lowest position of the superstructure:

- highway traffic: parallel to the road \( F_{\text{a,\text{ß,prim}}} = 600 \text{ kN} \)
- other road traffic: parallel to the road \( F_{\text{a,\text{ß,prim}}} = 450 \text{ kN} \)

Design means: avoiding serious damage of the bridge

- **Caused by railway traffic:**

  If an adequate protection by a barrier exists (the barrier must be dimensioned by values given
  in table 18), the consequence of collision may not to be taken into account when designing a
  substructures located beside the track.

  If no adequate protection by a barrier exists, the supporting substructure beside a track should
  be designed for an impact loading with a surface area of 0.25x0.25 m² as given in table 1.

  If such a design is ignored, the whole structure above the track should stay stable for the
  situation that a single support is 100% missing.

  **Table 18. Horizontal loading caused by train collision: valid only for a speed \( v = 120 \text{ km/h} \) (for a
  speed larger than 120 km/h adequate protection much exist).**

<table>
<thead>
<tr>
<th>Distance “S” between</th>
<th>Loading parallel to track</th>
<th>Loading perpendicular to track</th>
</tr>
</thead>
<tbody>
<tr>
<td>support – track center [m]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Support: continuous wall</td>
<td>10.000 kN*</td>
<td>3.500 kN</td>
</tr>
<tr>
<td>S &lt; 3.0 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Support: column row</td>
<td>End columns: 10.000 kN*</td>
<td>End columns: 3.500 kN</td>
</tr>
<tr>
<td>S &lt; 3.0 m</td>
<td>Inbetween columns: 4.000 kN</td>
<td>Inbetween columns: 1.500 kN</td>
</tr>
<tr>
<td>S = 3.0 m</td>
<td>End columns:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>14280,6-1428,6xS = 0 kN*</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Inbetween columns:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5174-5174xS = 0 kN</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>*</td>
<td>at a level of 1.8 m+ B.S. (top of rail)</td>
<td>all other loadings at a level of 1.0 m + B.S.</td>
</tr>
<tr>
<td></td>
<td>loading parallel and perpendicular to the track need not to be combined.</td>
<td></td>
</tr>
</tbody>
</table>

Art. 5: accidental loading: Train overturning: remaining balanced on its edge

\( q_{\text{hor.}} = 18 \text{ kN/m} \)

(example, column c.t.c. 5 m means \( F_{\text{hor.}} 5 \times 18 = 90 \text{ kN} \))

Art. 5: accidental loading: Derailment

Two design situations shall be considered:

Design situation I: derailment of locomotive or heavy freight wagons, with the derailed vehicles
remaining in the track area of the bridge.

Design situation II: derailment of locomotive of heavy wagons, with the derailed vehicles remaining in
the track area without falling off the bridge, but remaining balanced on its edge.
CT5125: Steel bridges – file <railway-bridges-design>

15. (Comfort related) design railway bridges
Dr. A. Romeijn

For the overall stability, an equivalent load shall be taken as a vertical line load with a design value of
\( q_{A2d} = 80 \text{ kN/m} \), over a total length of 20,00 m at a maximum distance from the track centre-line 1,5 times the
gauge.

In addition to the two design situations, for reposition of the train the following load situation must be taken into account:

\( F_{\text{lifting}} = 500 \text{ kN} \)
\( A_{\text{lifting}} = 0.7 \times 1.6 \text{ m}^2 \)

Art. 6.5 Deflection
The vertical displacement caused by characteristic traffic loading (without dynamic factor) should not be larger than L/1000 (for a speed up to 200 km/h). This because of criteria on angular rotations at the end of the decks, track stability, passengers comfort, etc. The code ENV 1991 specifies in more detail the criteria on deflection. Especially in case of very good passengers comfort, the criteria on vertical displacement are more stringent and therefore can be very decisive for the whole design of the bridge.

For the end cross beam, the deflection must be limited to L/2000, with a maximum of 4 mm at track rail position.
15.14 Railway bridge versus Highway bridge
A short summary on main differences is given.

1. Design
   a. live load: vertical loading, braking forces, tracking forces, collision forces
   b. safety margins (train accident means potentially more victims)
   c. ratio live load / dead load (consequences on deflection, fatigue, hanger configuration, etc.)
   d. comfort standards (like stiffness criteria)
   e. daily temperature effects (because of presence of ballast track)
   f. impact factor for dynamic loading (train speed versus truck speed)
   g. transverse wind loading
   h. constructability and maintainability without interruption to traffic
   i. noise emission
   j. truck loading is more severe and has more impact on the deck than the passing of a train.

2. Appearance
   a. cable stayed bridge and suspension bridge in general not suitable as a railway bridge.
      due to stiffness requirements, truss and arch bridges are frequently used as a railway bridge.
   b. railway bridges are constructed more frequently as a single span bridge instead of continuous bridge
   c. approaching railway line having a small gradient
   d. construction depth for a railway bridge is general higher
   e. bridge cross section for both type of bridges entirely differs

REFERENCES

APPENDIX:
List of Symbols
Case HSL railway bridge-Moerdijk
Railway bridge versus Highway bridge
List of symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS</td>
<td>British Standards</td>
<td></td>
</tr>
<tr>
<td>DDM</td>
<td>Double-decker material</td>
<td></td>
</tr>
<tr>
<td>DIN</td>
<td>Deutsches Institut für Normung</td>
<td></td>
</tr>
<tr>
<td>ERRI</td>
<td>European Rail Research institute</td>
<td></td>
</tr>
<tr>
<td>ICE</td>
<td>Intercity Express</td>
<td></td>
</tr>
<tr>
<td>ISO</td>
<td>International organisation of Standardisation</td>
<td></td>
</tr>
<tr>
<td>ORE</td>
<td>Office for Research and Experiments</td>
<td></td>
</tr>
<tr>
<td>PDF</td>
<td>Probability density function</td>
<td></td>
</tr>
<tr>
<td>TGV</td>
<td>Train à grande vitesse (high velocity train)</td>
<td></td>
</tr>
<tr>
<td>W</td>
<td>Weighting filter</td>
<td></td>
</tr>
<tr>
<td>RMQ</td>
<td>Root mean quad</td>
<td></td>
</tr>
<tr>
<td>RMS</td>
<td>Root mean square</td>
<td></td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate limit state</td>
<td></td>
</tr>
<tr>
<td>SLS</td>
<td>Serviceability limit state</td>
<td></td>
</tr>
<tr>
<td>a</td>
<td>acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>a_{peak}</td>
<td>Peak acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>a</td>
<td>RMQ acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>a</td>
<td>RMS acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>a</td>
<td>Weighted acceleration</td>
<td>[m/s²]</td>
</tr>
<tr>
<td>C</td>
<td>Stiffness parameter</td>
<td>[-]</td>
</tr>
<tr>
<td>c</td>
<td>Critical damping</td>
<td>[Ns/m]</td>
</tr>
<tr>
<td>d</td>
<td>Damping</td>
<td>[Ns/m]</td>
</tr>
<tr>
<td>EI</td>
<td>Bending stiffness</td>
<td>[Nm²]</td>
</tr>
<tr>
<td>f</td>
<td>Frequency</td>
<td>[Hz]</td>
</tr>
<tr>
<td>f_{u,b}</td>
<td>Natural frequency of bridge</td>
<td>[Hz]</td>
</tr>
<tr>
<td>f_{u,t}</td>
<td>Natural frequency of train</td>
<td>[Hz]</td>
</tr>
<tr>
<td>G_d</td>
<td>Spectral quality</td>
<td>[m²]</td>
</tr>
<tr>
<td>h</td>
<td>Hour</td>
<td>[h]</td>
</tr>
<tr>
<td>k</td>
<td>Stiffness</td>
<td>N/m</td>
</tr>
<tr>
<td>M</td>
<td>Mass</td>
<td>[kg]</td>
</tr>
<tr>
<td>m</td>
<td>Meter</td>
<td>[m]</td>
</tr>
<tr>
<td>s</td>
<td>Second</td>
<td>[s]</td>
</tr>
<tr>
<td>T</td>
<td>Duration of vibration</td>
<td>[s]</td>
</tr>
<tr>
<td>t</td>
<td>Time</td>
<td>[s]</td>
</tr>
<tr>
<td>v</td>
<td>Velocity</td>
<td>[m/s]</td>
</tr>
<tr>
<td>w</td>
<td>Displacement</td>
<td>[m]</td>
</tr>
<tr>
<td>d</td>
<td>Deflection</td>
<td>[m]</td>
</tr>
<tr>
<td>?</td>
<td>% of critical damping</td>
<td>[-]</td>
</tr>
<tr>
<td>pA</td>
<td>Weight per length</td>
<td>[kg/m]</td>
</tr>
<tr>
<td>f</td>
<td>Phase</td>
<td>[rad]</td>
</tr>
<tr>
<td>?</td>
<td>Angular frequency</td>
<td>[rad/s]</td>
</tr>
<tr>
<td>?_0</td>
<td>Natural angular frequency</td>
<td>[rad/s]</td>
</tr>
<tr>
<td>?</td>
<td>Spatioal frequency</td>
<td>[rad/m]</td>
</tr>
</tbody>
</table>
Case HSL railway bridge-Moerdijk

The bridge over the Holland’s Diep, with a total length of 1190 m, consists of 11 main spans of 105 m and 2 side spans of 70 m. In cross section the superstructure consists of a 6 m wide and 4.5 m high open steel box girder, supporting a 14.2 m wide concrete deck slab with a maximum thickness of 0.5 m above the web. At the piers, the height of the composite girder is increased to 10.5 m. The outer webs of the steel girder are 1:20 skew upwards, reducing the girder width by 1 m at the bearing position. The bridge is supported by four rubber bearings at each pier. This support scheme added bending stiffness of the superstructure because of the semi-rigid connection to the pier. Because of longitudinal displacements, like those caused by thermal actions, sliding bearings are installed at the outer piers and on the abutments. In lateral direction the bridge is supported at all piers. This support transfers part of the ship collision impact force to other piers (by high torsional rigidity of the box girder).

As shown in fig. 1, each main span is divided by two different sections, so-called the 59 m long field-section and the 46 m hammer-section. The reason for this distinction is because of weight limitation: transportation of the sections from the yard to the site and accessibility of lifting equipment at the site. The units are transported to site on barges and placed in position by lifting devices in the form of floating cranes and jack-up arrangements.

**Fig 1. Side view and cross-section HSL-bridge.**

The hammer-sections are transported to the site without concrete deck, and in one action the 500-ton sections are rotated to vertical position and placed on the piers by floating crane. Temporary support frames are used to provide stability during construction. The field sections are prefabricated incl. concrete deck at yard. On site, the 1200-ton sections are lifted from the barge with strand-lifts that are placed on the hammer-sections. After installing and welding the connection hammer – field, the concrete deck of the hammer-sections is casted in situ. By using such a construction sequence, the concrete tensile stress above the pier is limited.

**Fig. 2. Erection of the hammer-section and field-section [6].**
During erection, in order to be able to adjust the position of the hammer-section on the pier, a temporary-supporting frame is used. The hammer-section is set on hydraulic jacks; two on top of the pier and two on the supporting frame. For the field-section, the number of buckling stiffeners is limited to 1 through-shape stiffener at the 40 mm bottom plate and one at the 22 mm web plate. Every 5 m, the cross section of the girder is braced by transverse T-shaped stiffeners 0.5 m high. The B55 concrete deck is poured upon formwork made from permanent hd-galvanized Comflor-70 steel sheet plates. The deck slab varies in thickness from 400 mm in the middle to 500 mm at the web-location and 200 mm at the outer edge. With the configurations of span it appeared that the lowest natural frequency of the bridge is about 1 Hz. In combination of this lowest natural frequency, a train speed of 399 km/h (83 m/s) resulting in a passage frequency of the spans of about 1.26 Hz, a systematic deflection due to creep etc., (giving the deflection line of the bridge a dominant wave length of 105 m), and a natural frequency of the train coaches in he range of 0.6 – 1.1 Hz made the bridge sensitive to dynamic behaviour.

For comfort reasons the maximum vertical acceleration of the train coaches have to meet $L_{th} < 45$, which results in $a_{max} < 0.7 \text{ m/s}^2$ for all speeds between 160 km/h (44 m/s) and 330 km/h (92 m/s).

Figure 4 [2], gives an example of the simulation results of Thalys1. This train is modelled as a series of one degree of freedom systems that have a damping ratio of 0.10 and a natural frequency shown in the legend: 0.6 – 1.1 Hz. For a great number of speeds, the maximum vertical acceleration is plotted.

A maximum of 0.5 m/s$^2$ is obtained for a train speed of 92 m/s. The maximum vertical acceleration of the bridge itself was about 0.3 m/s$^2$, far below the limit of 5.0 m/s$^2$. Totally five trains were modelled by a load system with constant loads that moved over the bridges in the centre of the loaded track. All train speeds from 144 km/h up to 360 km/h (1.2 x design speed) were considered with a step of 3.6 km/h. Therefore, the bridge was calculated for at least 300 train passages per design variant. Is was found that the response of the bridge in terms of maximum acceleration and vertical displacement during a train-passage appeared strongly dependent on the speed V of the moving loads, the lowest natural frequency of the bridge and the dämpingsratio.

Passenger comfort appeared a decisive factor in determining the stiffness of the bridge over Holland’s Diep.