Dynamic effects induced by high speed traffic on rail bridges

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IBRACON 2011, 2nd November 2011, Florianópolis, Brasil
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1. Introduction
1. Introduction

Resonance effects on bridges

Due to the regular spacing of the train axles the action is periodic, with

\[ f = \frac{v}{d} \]

\[ d \] – regular distance between axles

\[ v \] – train speed

\[ f \] – load frequency of the train

When the train speed \( v \) is such that

\[ f = n_j \]

\[ n_j \] – natural frequency of the bridge

\[ i = 1, 2, 3, \ldots \]

RESONANCE PHENOMENA
1. Introduction

Resonance effects on bridges

Influence of the speed

TGV (d=18.7m)

\[ n = 5.625 \text{Hz} \]
1. Introduction

Resonance effects on bridges

Influence of the speed

The dynamic amplification factor does not cover the resonance effect.

V_{res} = 5.625 \times 18.7 = 105 \text{ m/s} (375 \text{ km/h})
1. Introduction
Resonance effects on bridges

Influence of the type of train

\[ V_{\text{res}} = 5.625 \times 13.1 = 73.7 \text{ m/s} \] (265 km/h)
\[ V_{\text{res}} = 5.625 \times 18.7 = 105 \text{ m/s} \] (375 km/h)
2. Issues related to design of rail bridges
2. Issues related to design of rail bridges

Static or dynamic analysis?

V – maximum line speed at the site (km/h)
L – span length (m)

\( n_0 \) – first natural bending frequency
\( n_T \) – first natural torsional frequency

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\( v \leq 200 \text{ km/h} \)

Simple structure (1)

Continuous bridge (5)

\( L < 40 \text{ m} \)

\( n_0 \) within limits of Figure 6.10 (6)

\( n_T > 1.2 \, n_0 \)

For the dynamic analysis use the eigenforms for torsion and for bending

Use tables F1 and F2 (2)

Dynamic analysis required. Calculate bridge deck acceleration and \( \phi_{\text{dyn}} \) etc. in accordance with 6.4.6 (4)

Dynamic analysis not required. At resonance acceleration check and fatigue check not required. Use \( \Phi \) with static analysis in accordance with 6.4.3 (1) P (3)

\( v_{\text{lim}} / n_0 \leq (V/n_0)_{\text{lim}} (2) (3) \)

Eigen forms for bending sufficient

--
2. Issues related to design of rail bridges

**Loading**

Real trains specified for the project and speeds over 200 km/h

- **Conventional** (ICE2, ETR-Y, VIRGIN)
- **Articulated** (THALYS, EUROSTAR, TGV)
- **Regular** (TALGO)
2. Issues related to design of rail bridges

Loading

Load Model HSLM for bridges designed for international lines where interoperability criteria are applicable

HSLM-A

HSLM-B

(1) Power car (leading and trailing power cars identical)
(2) End coach (leading and trailing end coaches identical)
(3) Intermediate coach
2. Issues related to design of rail bridges

Speed range

- For each real train or load model HSLM the dynamic calculations should be made for a series of speeds from 40 m/s (≈145 km/h) up to the Maximum Design speed ($v_{DS}$).

- The Maximum Design speed ($v_{DS}$) shall be generally 1.2 x Maximum line speed at the site ($v_{max}$).

- Smaller steps should be made in the vicinity of resonant speeds.
2. Issues related to design of rail bridges

**Bridge damping**

The peak response at resonance is highly dependent upon damping, therefore only lower bound estimates of damping shall be used.
2. Issues related to design of rail bridges

**Bridge damping**

Lower bound estimates of damping

<table>
<thead>
<tr>
<th>Bridge type</th>
<th>Lower limit of percentage of critical damping (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Span $L &lt; 20$ m</td>
</tr>
<tr>
<td>Steel and Composite</td>
<td>$\xi = 0,5 + 0,125 (20-L)$</td>
</tr>
<tr>
<td>Prestressed concrete</td>
<td>$\xi = 1,0 + 0,07 (20-L)$</td>
</tr>
<tr>
<td>Reinforced concrete and Filler Beam</td>
<td>$\xi = 1,5 + 0,07 (20-L)$</td>
</tr>
</tbody>
</table>
For spans up to 30 m dynamic vehicle-bridge mass interaction effects tends to reduce the peak response at resonance. These effects may be taken by:

1) Carrying out a dynamic vehicle-bridge interactive analysis;

2) Increasing the value of the damping assumed for the structure.

\[ \xi_{TOTAL} = \xi + \Delta \xi \]
2. Issues related to design of rail bridges

**Design checks**

- **Structural safety**

  For the bridge design, the following most unfavorable values should be taken into account:

  \[
  \left( 1 + \varphi_{\text{dyn}} + \frac{\varphi^*}{2} \right) \times \begin{pmatrix} HSLM \\ \text{or} \\ RT \end{pmatrix} \quad \varphi_{\text{dyn}} = \max \left| \frac{y_{\text{dyn}}}{y_{\text{stat}}} \right| - 1
  \]

  or

  \[
  \Phi \times (LM 71"+" SW / 0)
  \]

  a) carefully maintained track

  \[
  \Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82
  \]

  b) track with standard maintenance

  \[
  \Phi_3 = \frac{2.16}{\sqrt{L_\Phi} - 0.2} + 0.73
  \]
2. Issues related to design of rail bridges

**Design checks**

- **Track safety**

  - The deformation and structure vibration limit states, which aim at ensuring railway track safety, refer to (EN1990-AnnexA2):
    
    1) Vertical acceleration of the deck
    2) Torsion of the deck
    3) Vertical deformation of the deck
    4) Transverse deformation and vibration of the deck
2. Issues related to design of rail bridges

**Design checks**

- Vertical acceleration of the deck

Evaluation of the dynamic behaviour of ballasted track under different acceleration levels (ERRI D214/RP9)

Test rig at BAM
2. Issues related to design of rail bridges

**Design checks**

- Vertical acceleration of the deck
2. Issues related to design of rail bridges

**Design checks**

- Vertical acceleration of the deck

  The maximum permitted peak values of bridge deck acceleration calculated along each track shall not exceed:

  i) $3.5 \text{ m/s}^2$ ($\approx 0.35 \text{g}$), for ballasted track

  ![Graph showing vibration levels for different frequencies with max accelerations marked at 0.35 g and 0.70 g]

  Safety factor $= 2$

  ii) $5 \text{ m/s}^2$ ($\approx 0.50 \text{g}$), for direct fastened decks
2. Issues related to design of rail bridges

Design checks

- Passenger comfort (EN1990-Annex A2)

Passenger comfort depends on the vertical acceleration $b_v$ inside the carriages.

<table>
<thead>
<tr>
<th>Level of Comfort</th>
<th>Vertical acceleration $b_v$ (m/s$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very good</td>
<td>1,0</td>
</tr>
<tr>
<td>Good</td>
<td>1,3</td>
</tr>
<tr>
<td>Acceptable</td>
<td>2,0</td>
</tr>
</tbody>
</table>

Generally, for the determination of the acceleration on the carriages a dynamic analysis with the bridge-train interaction could be done.
2. Issues related to design of rail bridges

Design checks

For bridges consisting of simply supported spans or with continuity, which do not exhibit significant variations of span length or stiffness, and for spans up to 120 m, the verification of passengers comfort can be made by a simplified methodology limiting the vertical displacement of the deck.
3. Dynamic analysis of the train-bridge interaction
3. Dynamic analysis of the train-bridge interaction

**Train modelling**

Alfa pendular train

Secondary suspension

Primary suspension
Alfa pendular train – Natural frequencies and mode shapes

Mode 1 - f = 1.05 Hz

Mode 2 - f = 1.36 Hz

Mode 3 - f = 5.39 Hz

Mode 4 - f = 5.41 Hz
3. Dynamic analysis of the train-bridge interaction

**Iterative procedure**

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Vehicle</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Action</strong></td>
<td>$Q^i_{b}(t) = Q^i_{sta} + Q^{i-1}_{dyn}(t)$</td>
</tr>
<tr>
<td><strong>Result</strong></td>
<td>$u^i_{b}(t)$</td>
</tr>
<tr>
<td><strong>Convergence Criterion</strong></td>
<td>$\frac{Q^i_{dyn}(t) - Q^{i-1}<em>{dyn}(t)}{Q^{i-1}</em>{dyn}(t)}$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3. Dynamic analysis of the train-bridge interaction
3. Dynamic analysis of the train-bridge interaction

Train modelling

Alfa pendular train
3. Dynamic analysis of the train-bridge interaction

Train modelling

Alfa pendular train – Natural frequencies and mode shapes

Mode 1 – $f = 0.67$ Hz
Rolling mode

Mode 2 – $f = 0.97$ Hz
Bouncing mode

Mode 3 – $f = 1.32$ Hz
Pitching mode

Mode 4 – $f = 7.04$ Hz
1st torsion mode

Mode 5 – $f = 13.99$ Hz
Bending mode
3. Dynamic analysis of the train-bridge interaction

Train modelling

Dynamic test
3. Dynamic analysis of the train-bridge interaction

Train modelling

- Experimental natural frequencies and mode shapes

Mode 1 – $f = 1.01$ Hz
Rolling mode

Mode 2 – $f = 1.33$ Hz
Bouncing mode

Mode 3 – $f = 1.59$ Hz
Pitching mode

Mode 4 – $f = 8.38$ Hz
1st torsion mode

Mode 5 – $f = 12.26$ Hz
Bending mode
3. Dynamic analysis of the train-bridge interaction

Train modelling

- Numerical vs experimental natural frequencies

![Relative error (%) graph for modes 1 to 5 before and after updating.](image)
3. Dynamic analysis of the train-bridge interaction

Train modelling

TGV train
4. Dynamic analysis of a bowstring arch rail bridge
4. Dynamic analysis of a bowstring arch rail bridge

São Lourenço rail bridge

Location and description
4. Dynamic analysis of a bowstring arch rail bridge

Numerical model
4. Dynamic analysis of a bowstring arch rail bridge

Numerical frequencies and mode shapes

Mode 1 – $f = 2.33$ Hz
Transversal bending (arches)

Mode 2 – $f = 4.06$ Hz
Vertical bending

Mode 3 – $f = 5.88$ Hz
Vertical bending

Mode 4 – $f = 6.67$ Hz
Torsion

Mode 5 – $f = 9.01$ Hz
Vertical bending

Mode 6 – $f = 9.67$ Hz
Vertical / transversal bending

Mode 7 – $f = 11.31$ Hz
Transversal bending (arches)

Mode 8 – $f = 13.92$ Hz
Vertical bending

Mode 9 – $f = 14.82$ Hz
Torsion
4. Dynamic analysis of a bowstring arch rail bridge

Ambient vibration test

- Accelerometers located in the axis of the main girders of the deck slab, in the footway cantilever, diagonals, hangers and arches
- Sensor placement restrictions
- 3 fixed reference point (REF) and 59 mobile measurement points
4. Dynamic analysis of a bowstring arch rail bridge

Experimental frequencies and mode shapes

Mode 1 – f = 2.34 Hz
Mode 2 – f = 4.37 Hz
Mode 3 – f = 6.02 Hz

Mode 4 – f = 7.11 Hz
Mode 5 – f = 9.77 Hz
Mode 6 – f = 9.94 Hz

Mode 7 – f = 11.30 Hz
Mode 8 – f = 15.21 Hz
Mode 9 – f = 15.72 Hz
4. Dynamic analysis of a bowstring arch rail bridge

**Model Updating**

- Genetic algorithm (GA)

\[
f = a \sum_{i=1}^{n_{\text{modes}}} \frac{|f_i^{\text{exp}} - f_i^{\text{num}}|}{f_i^{\text{exp}}} + b \sum_{i=1}^{n_{\text{modes}}} \left| \text{MAC}\left(\phi_i^{\text{exp}}, \phi_i^{\text{num}}\right) - 1 \right|
\]
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

- ANSYS
- MATLAB
- OptiSlang

FE Model

k = 0

Values for the unknown variables $\theta_k$

Export matrices $[K_k], [M_k], [C_k]$

Computation of numerical modal data

Eigenvalues $\Lambda_k^{\text{num}}$

Eigenvectors $\Phi_k^{\text{num}}$

Objective function

$\sum_i \lambda_{k,i}^{\text{num}} \sum_i \lambda_{k,i}^{\text{exp}} + \sum_i 1 - \text{MAC}$

Optimization algorithm

- GB
- RSM
- Genetic algorithm

Updated values $\theta_{k+1}$

Minimization step

Convergence?

NO

k = k + 1

YES

Identified variables $\theta_k^{k+1}$

$k = 0$

Model Updating

4. Dynamic analysis of a bowstring arch rail bridge
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

Mode pairing
- MSE / clusters
- EMAC

Experimental modal data
Eigenvalues $\Lambda_k^{\text{exp}}$
Eigenvectors $\Phi_k^{\text{exp}}$

Export matrices $[K]$, $[M]$, $[C]$

Computation of numerical modal data
Eigenvalues $\Lambda_k$
Eigenvectors $\Phi_k$

Export numerical modal data
- Eigenvalues
- MAC values

Objective function
$$\sum_i \left| \frac{\Lambda_{k,i}^{\text{num}}}{\Lambda_{k,i}} - \frac{\Lambda_{k,i}^{\text{exp}}}{\Lambda_{k,i}^{\text{exp}}} \right| + \sum_i \left| - \text{MAC}_{k,i} \right|$$

Optimization algorithm
- GB
- RSM
- Genetic algorithm

Minimization step
Convergence?
NO
$k = k + 1$
YES
$k = 0$

Identified variables $\theta_k$

Minimization step
Convergence?
NO
$k = k + 1$
YES
Identified variables $\theta_{k+1}$

Model Updating

4. Dynamic analysis of a bowstring arch rail bridge
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

Optimization algorithm
- GB
- RSM
- Genetic algorithm

Minimization step

Updated values
$\theta_{k+1}$

Convergence?

YES

Identified variables
$\theta = \theta_{k+1}$

NO

$k = k + 1$

$\sum_i \lambda_{k,i}^{num} \lambda_{k,i}^{exp} + \sum_i (1 - MAC_{k,i})$
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

- Experimental vs numerical natural frequencies

![Graph showing relative error percentages before and after updating for different modes.](image)
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

- Experimental vs numerical mode shapes

Mode 1

Mode 2

Mode 3

Mode 4

Mode 5

Mode 6

Mode 7

Mode 8

Mode 9
4. Dynamic analysis of a bowstring arch rail bridge

Dynamic tests under railway traffic

- Displacements and accelerations
- Deformations

Neutral axis
4. Dynamic analysis of a bowstring arch rail bridge

**Experimental tests under railway traffic**

- Measuring displacements using an advanced non-contact video system
4. Dynamic analysis of a bowstring arch rail bridge

Characterization of the track irregularities

- Track inspection vehicle (EM120)
4. Dynamic analysis of a bowstring arch rail bridge

**Experimental validation of the numerical model**

- Alfa pendular train $v=185$ km/h
4. Dynamic analysis of a bowstring arch rail bridge

Simulation results

\[ v_{res}(i, j) = \frac{dn_j}{i} \]

\[ n_0 = 4.37 \text{ Hz} \]
\[ d = 25.9 \text{ m} \]

\[ V_{res}(1,2) = 25.9 \times 4.37 = 113 \frac{m}{s} \approx 405 \text{ km/h} \]
4. Dynamic analysis of a bowstring arch rail bridge

Simulation results

![Graph of Displacement (cm) vs Speed (km/h) for a VIRGIN train. The graph shows multiple lines representing different modes of vibration (15 Hz, 30 Hz, 45 Hz, 60 Hz). The peak displacement is at 376 km/h.](image)

\[ v_{\text{res}}(i, j) = \frac{dn_j}{i} \]

\( n_0 = 4.37 \text{ Hz} \)

\( d = 23.9 \text{ m} \)
4. Dynamic analysis of a bowstring arch rail bridge

Simulation results

\[ v_{\text{res}}(i, j) = \frac{dn_j}{i} \]

\[ n_0 = 4.37 \text{ Hz} \]
\[ d = 18.7 \text{ m} \]
4. Dynamic analysis of a bowstring arch rail bridge

Simulation results

TGV train

TGV (D=18.7m)

Mode shape

Modal force

4. Dynamic analysis of a bowstring arch rail bridge

Simulation results

TGV train

TGV (D=18.7m)

Mode shape

Modal force
4. Dynamic analysis of a bowstring arch rail bridge

Traffic safety

- Vertical acceleration of the deck

\[ a_{\text{max}} = 3.5 \text{ m/s}^2 \]
4. Dynamic analysis of a bowstring arch rail bridge

**Passengers comfort**

- Alfa pendular train $v = 250 \text{ km/h}$

\[ a_{\text{max}} = 1.0 \text{ m/s}^2 \]
4. Dynamic analysis of a bowstring arch rail bridge

Passengers comfort

• Alfa pendular train $v = 405$ km/h

$\text{a}_{\text{max}} = 1.0 \text{ m/s}^2$
5. Dynamic analysis of a precast rail viaduct
5. Dynamic analysis of a precast rail bridge

Alverca viaduct

Location and description
5. Dynamic analysis of a precast rail bridge

Alverca viaduct

Location and description
5. Dynamic analysis of a precast rail bridge

Alverca viaduct

Location and description
5. Dynamic analysis of a precast rail bridge

Alverca viaduct

Cross section of the deck
5. Dynamic analysis of a precast rail bridge

Numerical model
5. Dynamic analysis of a precast rail bridge

Numerical model
5. Dynamic analysis of a precast rail bridge

Numerical frequencies and mode shapes

1G – f=6.73 Hz
1L – f=26.68 Hz
2G – f=6.78 Hz
2L – f=53.01 Hz
3G – f=9.79 Hz
3L – f=60.28 Hz
5. Dynamic analysis of a precast rail bridge

Ambient vibration test – identification of global modes

Norte

Sul

[m]

● POSIÇÃO FIXA
○ POSIÇÃO MÓVEL
5. Dynamic analysis of a precast rail bridge

Ambient vibration test – identification of local modes
5. Dynamic analysis of a precast rail bridge

Ambient vibration test

Application of Stochastic Subspace Identification (SSI) method
5. Dynamic analysis of a precast rail bridge

Experimental frequencies and mode shapes

1G – f=6.76Hz; $\xi=1.53\%$

1L – f=25.48Hz; $\xi=2.03\%$

2G – f=6.95Hz; $\xi=3.72\%$

2L – f=53.18Hz; $\xi=2.27\%$

3G – f=9.65Hz; $\xi=2.21\%$

3L – f=60.18Hz; $\xi=3.16\%$
5. Dynamic analysis of a precast rail bridge

Numerical vs experimental results

<table>
<thead>
<tr>
<th>Mode</th>
<th>Experimental (Hz)</th>
<th>Numerical (Hz)</th>
<th>Error(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1G</td>
<td>6,76</td>
<td>6,73</td>
<td>0,44</td>
</tr>
<tr>
<td>2G</td>
<td>6,95</td>
<td>6,78</td>
<td>2,45</td>
</tr>
<tr>
<td>3G</td>
<td>9,65</td>
<td>9,79</td>
<td>1,45</td>
</tr>
</tbody>
</table>

Local modes

<table>
<thead>
<tr>
<th>Mode</th>
<th>Experimental (Hz)</th>
<th>Numerical (Hz)</th>
<th>Error(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1L</td>
<td>25,48</td>
<td>26,68</td>
<td>4,71</td>
</tr>
<tr>
<td>2L</td>
<td>53,18</td>
<td>53,01</td>
<td>0,32</td>
</tr>
<tr>
<td>3L</td>
<td>60,18</td>
<td>60,28</td>
<td>0,17</td>
</tr>
</tbody>
</table>
5. Dynamic analysis of a precast rail bridge

Results

Influence of frequency range

![Graph showing acceleration vs. speed for TALGO train at 30 Hz and 60 Hz.]
5. Dynamic analysis of a precast rail bridge

Results

Influence of structural damping

![Graph showing acceleration vs speed for TALGO train at 60 Hz and 60 Hz (ζ=1.22%)](image)

- Orange line: 60 Hz
- Dashed line: 60 Hz (ζ=1.22%)
5. Dynamic analysis of a precast rail bridge

Traffic safety

\[ a_{\text{max}} = 3.5 \text{ m/s}^2 \]
6. Dynamic analysis of a short span filler beam bridge
6. Dynamic analysis of a short span filler beam bridge

Canelas bridge

Location
6. Dynamic analysis of a short span filler beam bridge

Canelas bridge

Description
6. Dynamic analysis of a short span filler beam bridge

Numerical model
6. Dynamic analysis of a short span filler beam bridge

Experimental frequencies and mode shapes

Mode 1 – $f = 8,7 \text{ Hz}$

Mode 2 – $f = 9,8 \text{ Hz}$

Mode 3 – $f = 14,9 \text{ Hz}$

Mode 4 – $f = 16,6 \text{ Hz}$

Mode 5 – $f = 28,3 \text{ Hz}$
6. Dynamic analysis of a short span filler beam bridge

**Model updating**

- Experimental vs numerical frequencies

<table>
<thead>
<tr>
<th>Mode</th>
<th>Experimental (Hz)</th>
<th>Numerical Initial model (Hz)</th>
<th>Error (%)</th>
<th>Numerical Updated model (Hz)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8,7</td>
<td>9,43</td>
<td>8,38</td>
<td>9,07</td>
<td>4,23</td>
</tr>
<tr>
<td>2</td>
<td>9,8</td>
<td>11,01</td>
<td>12,39</td>
<td>9,78</td>
<td>-0,21</td>
</tr>
<tr>
<td>3</td>
<td>14,9</td>
<td>16,62</td>
<td>11,57</td>
<td>14,89</td>
<td>-0,04</td>
</tr>
<tr>
<td>4</td>
<td>16,6</td>
<td>30,37</td>
<td>82,97</td>
<td>16,60</td>
<td>0,00</td>
</tr>
<tr>
<td>5</td>
<td>28,3</td>
<td>27,38</td>
<td>-3,26</td>
<td>27,25</td>
<td>-3,73</td>
</tr>
</tbody>
</table>
6. Dynamic analysis of a short span filler beam bridge

Model updating

- Experimental vs numerical mode shapes

![Mode 1](image1.png)

![Mode 2](image2.png)

![Mode 3](image3.png)

![Mode 4](image4.png)

![Mode 5](image5.png)

Numerical

Experimental
6. Dynamic analysis of a short span filler beam bridge

Simulation scenario

- TGV train
6. Dynamic analysis of a short span filler beam bridge

**Deterministic response**

- TGV train

![Graph showing acceleration and speed](image-url)
6. Dynamic analysis of a short span filler beam bridge

Deterministic response

- TGV train $v = 250$ km/h
6. Dynamic analysis of a short span filler beam bridge

Deterministic response

• TGV train \( v = 300 \text{ km/h} \)
6. Dynamic analysis of a short span filler beam bridge

**Stochastic simulation of the dynamic response**

- **Random variables**

<table>
<thead>
<tr>
<th>Variable [simulation]</th>
<th>Distribution</th>
<th>Mean (gaussian) or Min. (uniform)</th>
<th>Std. Deviation (gaussian) or Max. (uniform)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete density</td>
<td>Gaussian</td>
<td>2.5 t/m³</td>
<td>0.1 (CV = 4%)</td>
</tr>
<tr>
<td>Ballast density</td>
<td>Uniform</td>
<td>17 kN/m³</td>
<td>21 kN/m³</td>
</tr>
<tr>
<td>Ballast area</td>
<td>Uniform</td>
<td>1,48659 m²</td>
<td>2,76081 m²</td>
</tr>
<tr>
<td>HEB 500 area</td>
<td>Gaussian</td>
<td>Nominal area</td>
<td>0.04 x nominal area</td>
</tr>
<tr>
<td>Elasticity modulus concrete</td>
<td>Gaussian</td>
<td>36.1 GPa</td>
<td>2,888 (CV = 8%)</td>
</tr>
<tr>
<td>Concrete height</td>
<td>Gaussian</td>
<td>Nominal value</td>
<td>10 mm</td>
</tr>
<tr>
<td>Concrete width</td>
<td>Gaussian</td>
<td>Nominal value</td>
<td>5 mm</td>
</tr>
<tr>
<td>Distortion modulus neoprene</td>
<td>Uniform</td>
<td>0.75 MPa</td>
<td>1.18 MPa</td>
</tr>
</tbody>
</table>
6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- First natural frequency for the simulated bridges
6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

• Maximum deck acceleration for the simulated bridges

\[
\begin{align*}
\text{v} &= 250 \text{ km/h} \\
\text{v} &= 285 \text{ km/h}
\end{align*}
\]
6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- Maximum deck acceleration for the simulated bridges

\[ v = 250 \text{ km/h} \]

\[ v = 285 \text{ km/h} \]
6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- Cumulative probability

\[ v = 285 \text{ km/h} \]

[Diagram showing cumulative probability curve with 100,000 simulations]
6. Dynamic analysis of a short span filler beam bridge

**Stochastic simulation of the dynamic response**

- Probability of failure
7. Dynamic effects of the train passage on transition zones
7. Dynamic effects on transition zones

**Introduction**

- Balasted track to slab track transition

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Aditional rails l=20m

Sleepers with variable distances 60/63/65 cm and different fasteners

Soil-cement

Homogeneous platform

Limit of slab track
7. Dynamic effects on transition zones

Introduction

• Embankment to bridge transition

Bridge

Embankment

4 Height alternative $\geq 20$ m backfill built with embankment

4 Height alternative $\geq 50$ m the embankment is existing
7. Dynamic effects on transition zones

Introduction

• Maintenance interventions (Madrid - Seville high speed line)

<table>
<thead>
<tr>
<th>TRACK SECTION CONSIDERED</th>
<th>EXCEEDANCE LEVEL(*) DENSITY(EXCEEDANCE Nº/KM OF TRACK/INSPECTION)</th>
<th>RELATIVE RATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Track section without any switches, expansion devices, bridges or embankments</td>
<td>0.075</td>
<td>100</td>
</tr>
<tr>
<td>Track section on embankment, without any switches, expansion devices or bridges</td>
<td>0.094</td>
<td>125</td>
</tr>
<tr>
<td>Track section on bridge</td>
<td>0.159</td>
<td>194</td>
</tr>
<tr>
<td>Track section at beginning of bridge-natural infrastructure transition</td>
<td>0.259</td>
<td>315</td>
</tr>
<tr>
<td>Track section over culvert</td>
<td>0.487</td>
<td>594</td>
</tr>
<tr>
<td>Track section (on ballast) in tunnel</td>
<td>0.026</td>
<td>32</td>
</tr>
</tbody>
</table>

*Pita, A. Lopez - Deterioration of track geometric quality on high speed lines: the experience of the Madrid - Seville line, Workshop Track for high-speed railways, Portugal, 2006

(*) número de vezes por quilómetro de via que a aceleração vertical do eixo do comboio AVE ultrapassou 30m/s².
7. Dynamic effects on transition zones

**Introduction**

- Vertical track stiffness variation

![Diagram of track stiffness variation](image)
7. Dynamic effects on transition zones
Hydraulic underpass PH126A

• Location and description

Linha do Norte
km 40+250
7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Track receptance tests
7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Dynamic monitoring of track displacement
7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Dynamic monitoring of track displacement
7. Dynamic effects on transition zones
Hydraulic underpass PH126A

- Dynamic monitoring of axle loads
7. Dynamic effects on transition zones

Case studies

• ADIF solution (Spain)
7. Dynamic effects on transition zones

Case studies

• SNCF solution (France)
7. Dynamic effects on transition zones

Numerical modelling

• ADIF solution (Spain)

A = Enbankment soil
MG = Gravel
MT = Gravel-cement
TN = Natural ground
T = Deck
E = Abutment
7. Dynamic effects on transition zones

**Numerical modelling**

- SNCF solution (France)

Diagram:

- A = Enbankment soil
- MG = Gravel
- MT = Gravel-cement
- TN = Natural ground
- T = Deck
- E = Abutment
- VT = Soil-cement under the track
7. Dynamic effects on transition zones

Numerical modelling

• Comparison of vertical track stiffness
7. Dynamic effects on transition zones

Numerical modelling

• Eurostar train
7. Dynamic effects on transition zones

Numerical modelling

• Train-track interaction
7. Dynamic effects on transition zones

Short term behaviour

- Wheel rail interaction force

ADIF

SNCF
7. Dynamic effects on transition zones

Short term behaviour

- Wheel rail interaction force
7. Dynamic effects on transition zones

Long term behaviour

Train-track model

Static analysis
Permanent track settlement

Dynamic analysis
Train-track dynamic interaction analysis
Dynamic forces
Stresses

Material degradation laws
Permanent strains

N – number of cycles

Track geometry

Permanent track settlement

N – number of cycles
7. Dynamic effects on transition zones

Long term behaviour

- Initial stresses
7. Dynamic effects on transition zones

**Long term behaviour**

- Initial stresses – section S1
7. Dynamic effects on transition zones

**Long term behaviour**

- Initial stresses – section S1

![Graph showing initial stresses](image)

**A - top**

**A - 4 m depth**
7. Dynamic effects on transition zones

Long term behaviour

• Material degradation laws

\[ \varepsilon_p = f(N) \cdot g(p_{\text{max}}, q_{\text{max}}) \]

Number of cycles \hspace{1cm} Material stresses

\[ f(N) = A \left[ 1 - \left( \frac{N}{100} \right)^{-B} \right] \]

\[ g(p_{\text{max}}, q_{\text{max}}) = \varepsilon_1 \cdot \left( \frac{l_{\text{max}}}{p_a} \right)^n \cdot \frac{1}{m + \frac{s}{p_{\text{max}}} - \frac{q_{\text{max}}}{p_{\text{max}}}} \]

\[ q = \sigma_1 - \sigma_3 \]

\[ p = \frac{\sigma_1 + 2.\sigma_3}{3} \]

\[ l_{\text{max}} = \sqrt{p_{\text{max}}^2 + q_{\text{max}}^2} \]

\[ p_a = 100 \text{kPa} \]

Material dependent parameters: \( \varepsilon_1, m, n, s \)
7. Dynamic effects on transition zones
Long term behaviour

- Permanent strain evolution

\[ \varepsilon_P \times 10^{-3} \]

Number of cycles (x10^4)
7. Dynamic effects on transition zones

Long term behaviour

• Wheel displacement
7. Dynamic effects on transition zones

Long term behaviour

- Wheel-rail interaction force
7. Dynamic effects on transition zones

Long term behaviour

- Wheel-rail interaction force

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Maximum dynamic load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V = 250</td>
<td>180</td>
</tr>
<tr>
<td>250 &lt; V ≤ 300</td>
<td>170</td>
</tr>
<tr>
<td>V &gt; 300</td>
<td>160</td>
</tr>
</tbody>
</table>
7. Dynamic effects on transition zones

Long term behaviour

• Axle acceleration

<table>
<thead>
<tr>
<th>Axle acceleration (m/s²)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>a &lt; 30</td>
<td></td>
</tr>
<tr>
<td>30 &lt; a &lt; 50</td>
<td></td>
</tr>
<tr>
<td>50 &lt; a &lt; 70</td>
<td></td>
</tr>
<tr>
<td>a &gt; 70</td>
<td></td>
</tr>
</tbody>
</table>
7. Dynamic effects on transition zones

Long term behaviour

• Introduction of Under Sleeper Pads (USP’s)
7. Dynamic effects on transition zones

**Long term behaviour**

- Introduction of Under Sleeper Pads (USP’s)

![Graph showing displacement over position with and without USP](image-url)
8. Dissemination and training activities
8. Dissemination and training activities

Workshops / advanced courses

3-4 June 2004, FEUP, Porto

12-13 October 2006, FEUP, Porto

20-23 September 2005, FEUP, Porto

15-16 October 2007, FEUP, Porto
8. Dissemination and training activities

*Workshops / advanced courses*

- **2-3 October 2008, FEUP, Porto**
- **1-2 October 2009, FEUP, Porto**
8. Dissemination and training activities

Workshops / advanced courses
9. Research team
9. Research team

PhD Researchers: 4
PhD Students: 14
Research Assistants: 6
10. International links
10. International links

- MIT, USA
- University of São Paulo, Brasil
- Ghent University, Belgium
- KU Leuven, Belgium
- UPM, Spain
- Pisa University, Italy
- Linköping University, Sweden
- Royal Institute of Technology, Sweden
- Bauhaus-University Weimar, Germany
- Beijing Jiaotong University, China
Acknowledgments