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53° CONGRESSO BRASILEIRO DO CONCRETO

IBRACON 2011

Dynamic effects induced by high speed traffic on rail bridges

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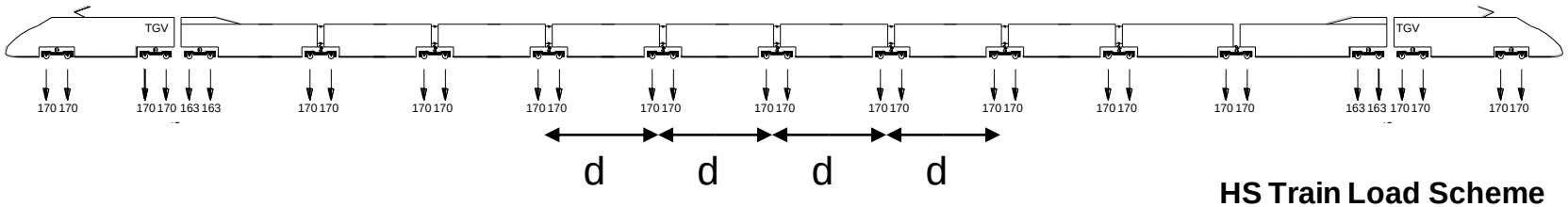
1. Introduction

Resonance effects on bridges



HS Train Load Scheme

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

$$f = v / d$$

d – regular distance between axles

v – train speed

f – load frequency of the train

n_j – natural frequency of the bridge

i = 1, 2, 3, ….

When the train speed v is such that

$$f = n_j$$

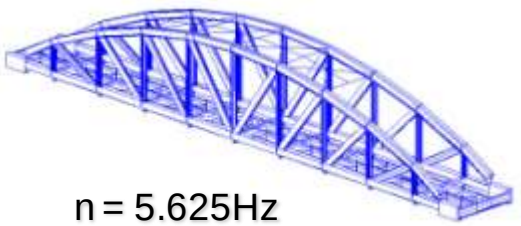
$$f = n_j / i$$

RESONANCE PHENOMENA

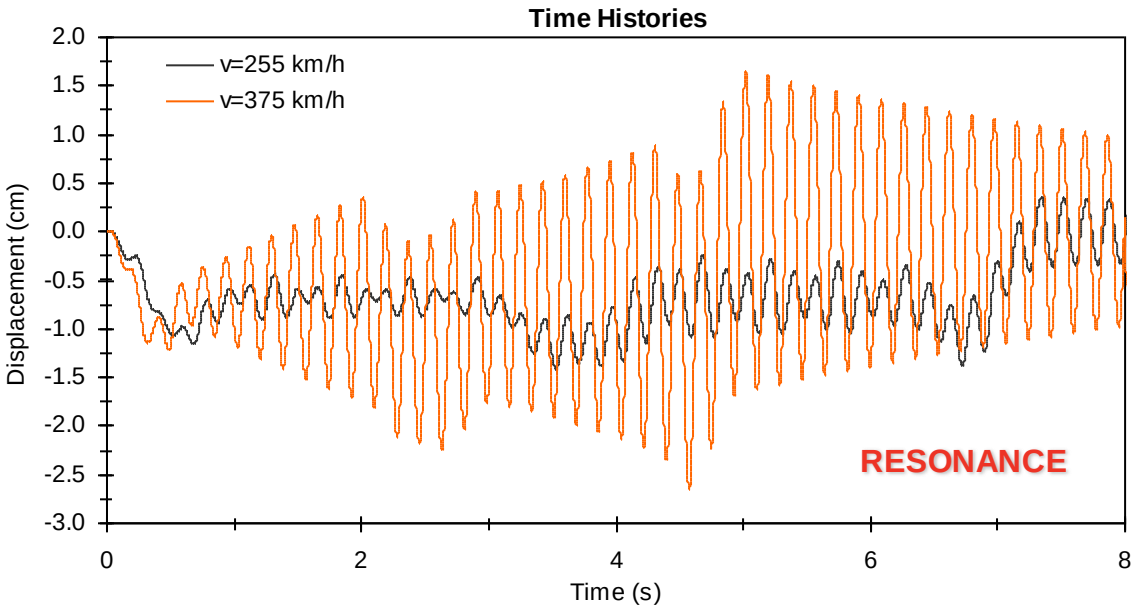
1. Introduction

Resonance effects on bridges

Influence of the speed



n = 5.625Hz

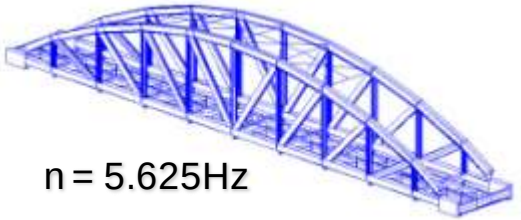


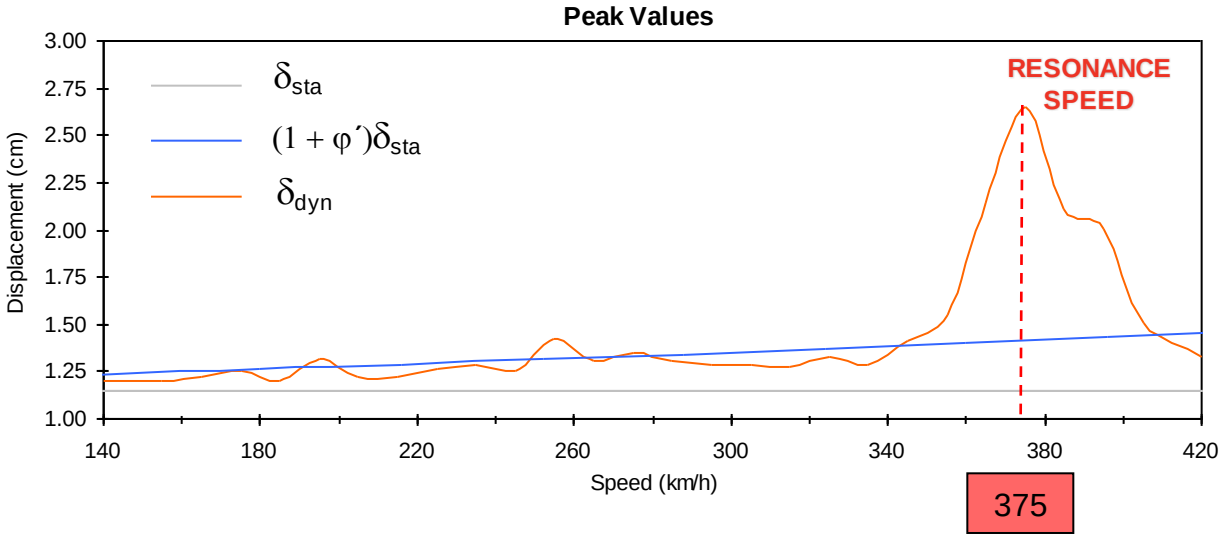
TGV (d=18.7m)

1. Introduction

Resonance effects on bridges

Influence of the speed





TGV (d=18.7m)

The dynamic amplification factor does not cover the RESONANCE effect

V_{res} = 5.625 x 18.7 = 105 m/s (375 km/h)

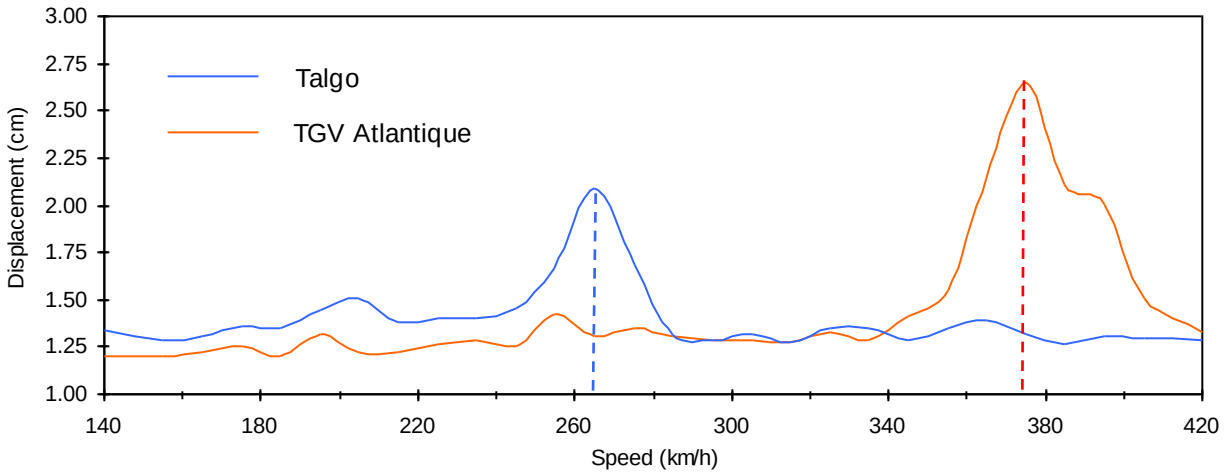
1. Introduction

Resonance effects on bridges

Influence of the type of train

Talgo (d=13.1m)

TGV (d=18.7m)



265

375

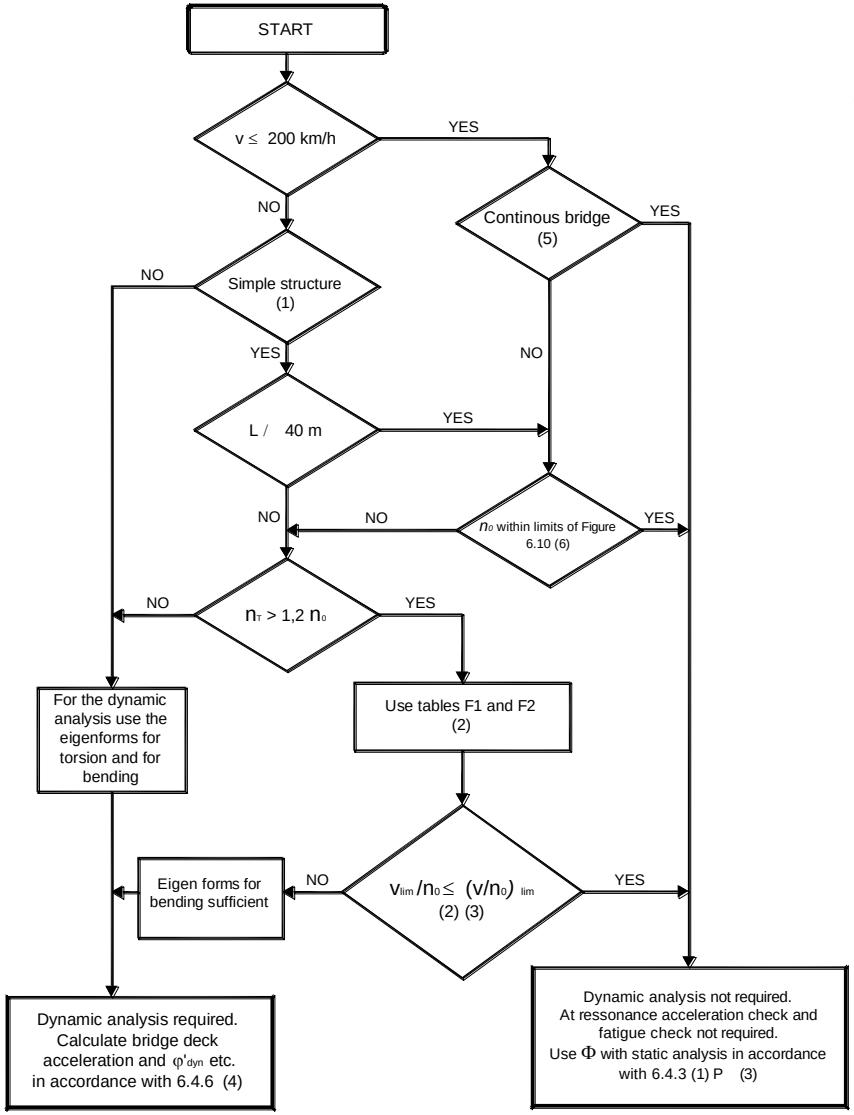
V_{res} = 5.625 x 13.1 = 73.7 m/s (265 km/h)

V_{res} = 5.625 x 18.7 = 105 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)

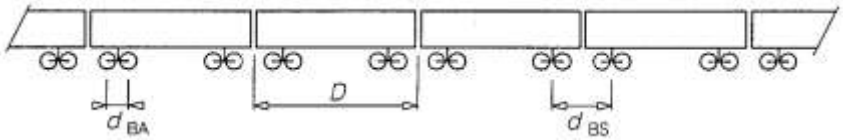
n_o – first natural bending frequency

n_T – first natural torsional frequency

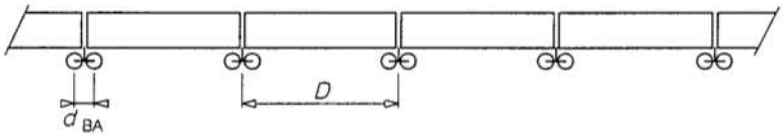
2. Issues related to design of rail bridges

Loading

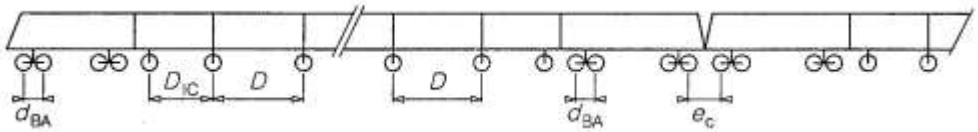
Real trains specified for the project and speeds over 200 km/h



Conventional (ICE2, ETR-Y, VIRGIN)



Articulated (THALYS, EUROSTAR, TGV)



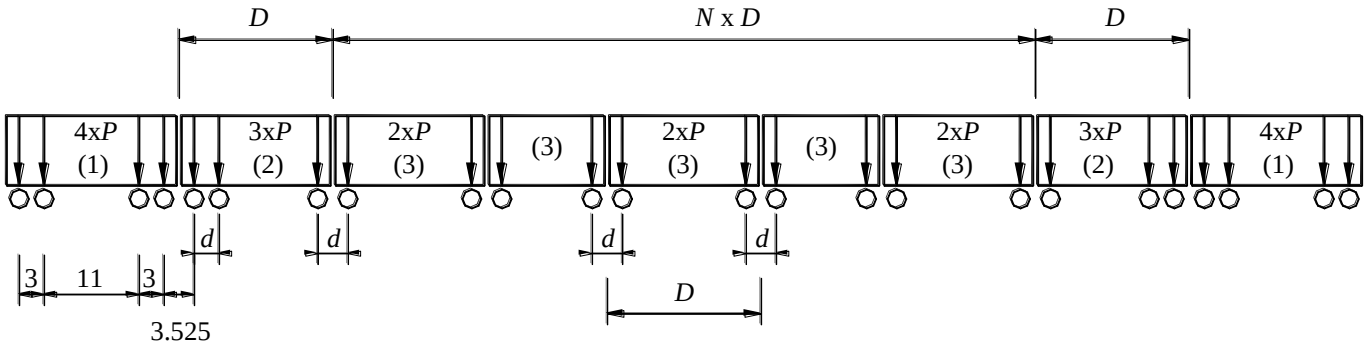
Regular (TALGO)

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

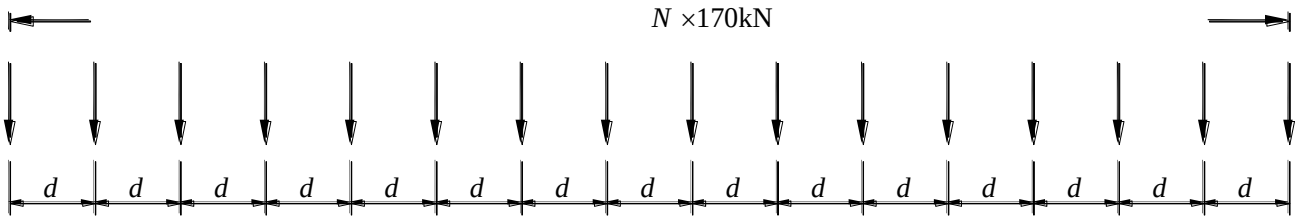


(1) Power car (leading and trailing power cars identical)

(2) End coach (leading and trailing end coaches identical)

(3) Intermediate coach

HSLM-B



2. Issues related to design of rail bridges

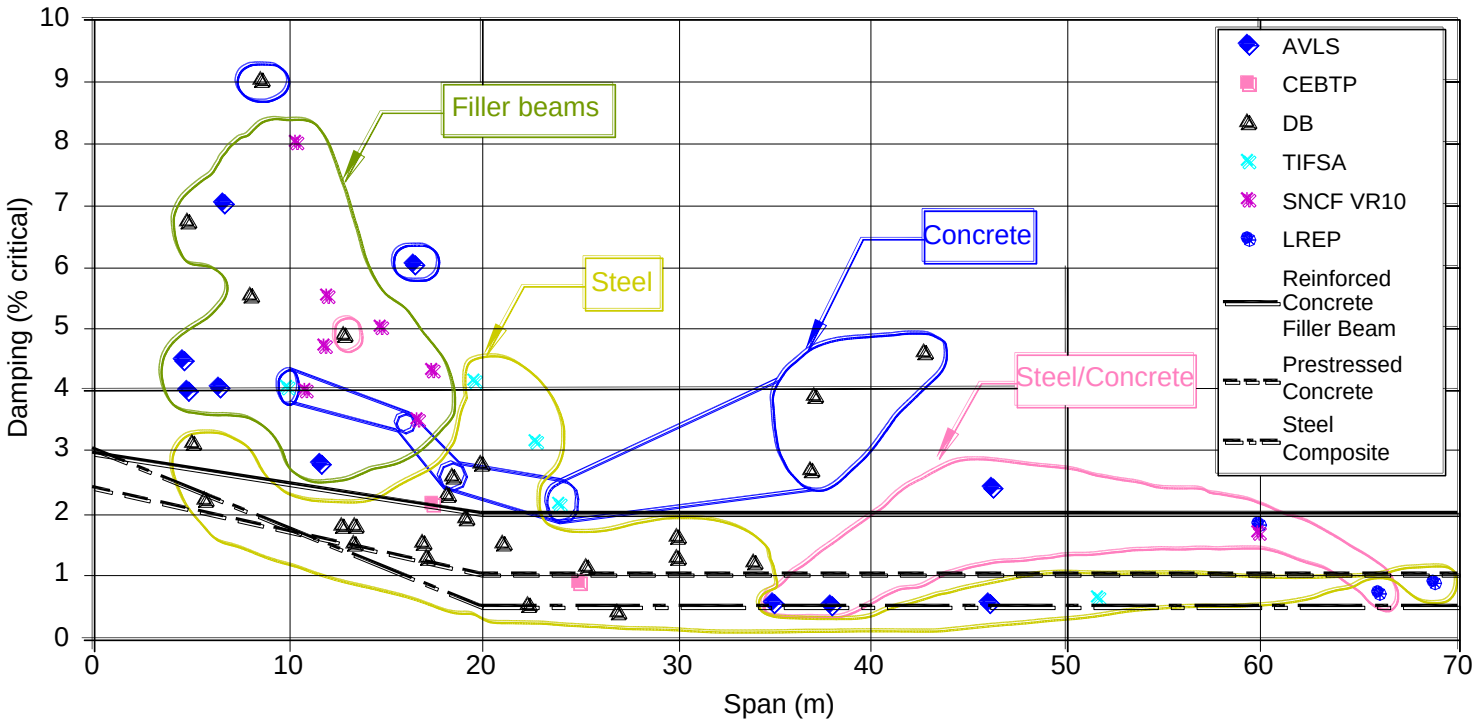
Speed range

- 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

Bridge type	Lower limit of percentage of critical damping (%)	
	Span $L < 20$ m	Span $L \geq 20$ m
Steel and Composite	$\xi = 0{,}5 + 0{,}125\,(20\text{-}L)$	$\xi = 0{,}5$
Prestressed concrete	$\xi = 1{,}0 + 0{,}07\,(20\text{-}L)$	$\xi = 1{,}0$
Reinforced concrete and Filler Beam	$\xi = 1{,}5 + 0{,}07\,(20\text{-}L)$	$\xi = 1{,}5$

2. Issues related to design of rail bridges

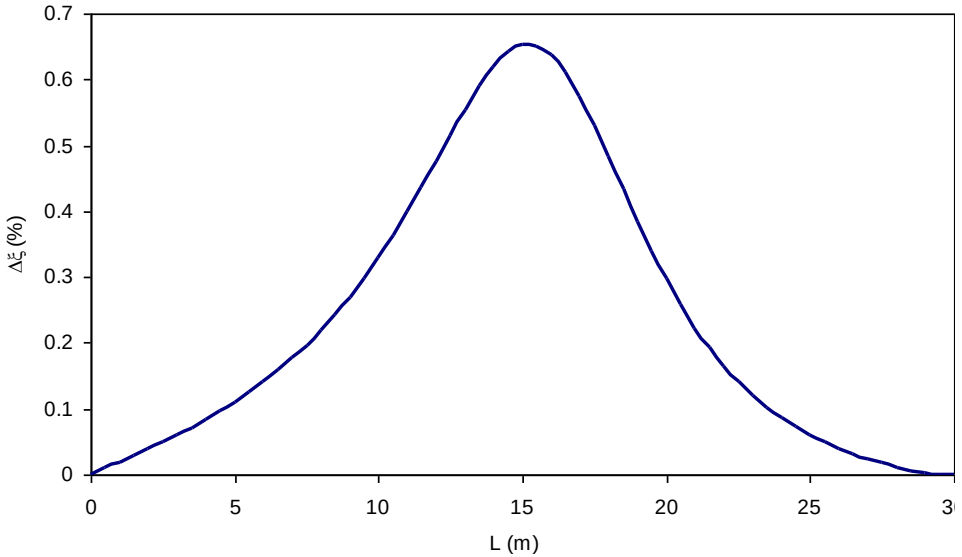
Bridge damping

- 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

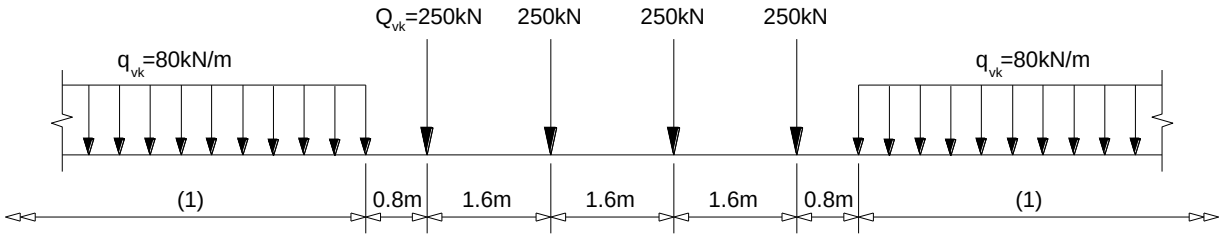
- Structural safety

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

$$\left(1+\varphi'_{dyn}+\frac{\varphi''}{2}\right)\times\begin{pmatrix} HSLM \\ \text{or} \\ RT \end{pmatrix} \qquad \varphi'_{dyn}=\max\left|\frac{y_{dyn}}{y_{stat}}\right|-1$$

or

$$\Phi\times\left(LM71"+"SW/0\right)$$



(1) No limitation

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

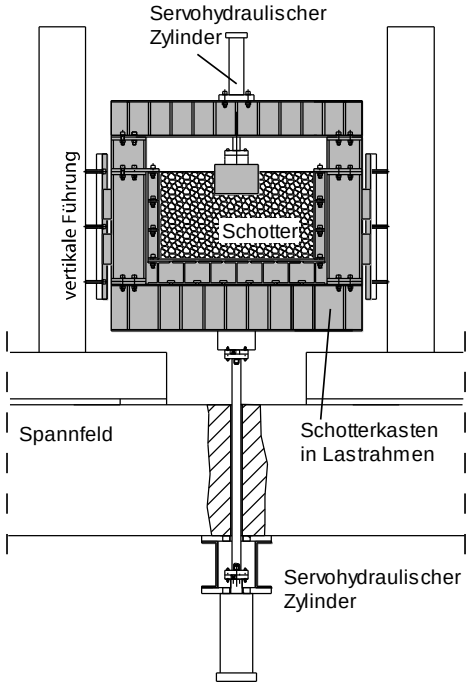
2. Issues related to design of rail bridges

Design checks

- Vertical acceleration of the deck

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

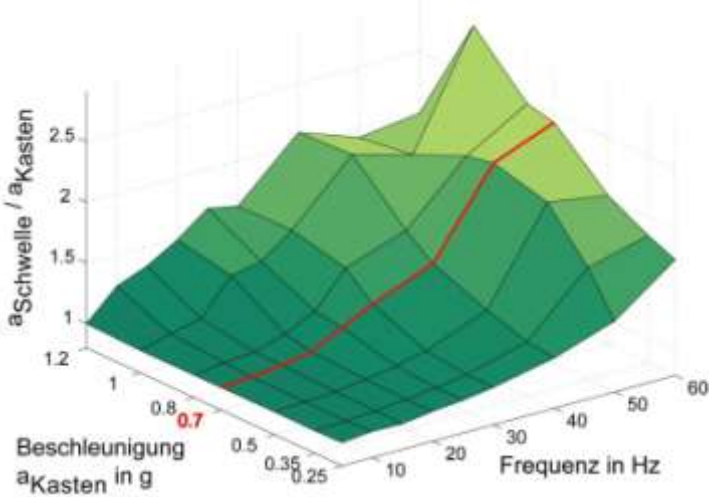
Test rig at BAM

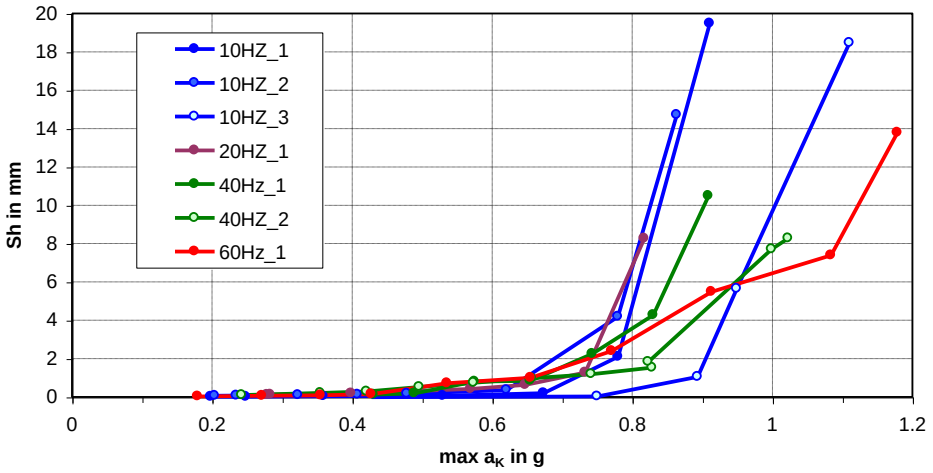


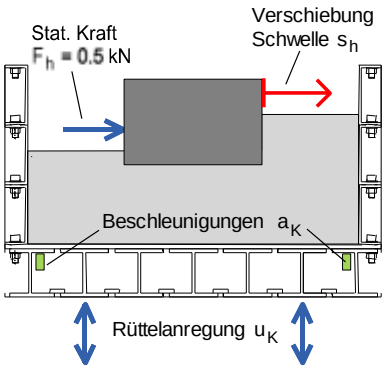
2. Issues related to design of rail bridges

Design checks

- Vertical acceleration of the deck







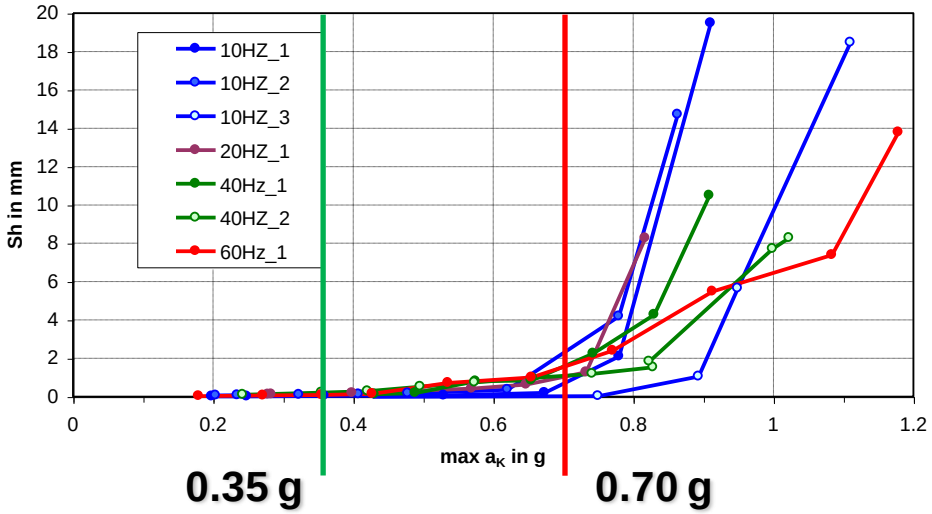
2. Issues related to design of rail bridges

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 m/s^2 (≈0.35g), for ballasted track



Safety factor = 2

ii) 5 m/s^2 (≈0.50g), 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

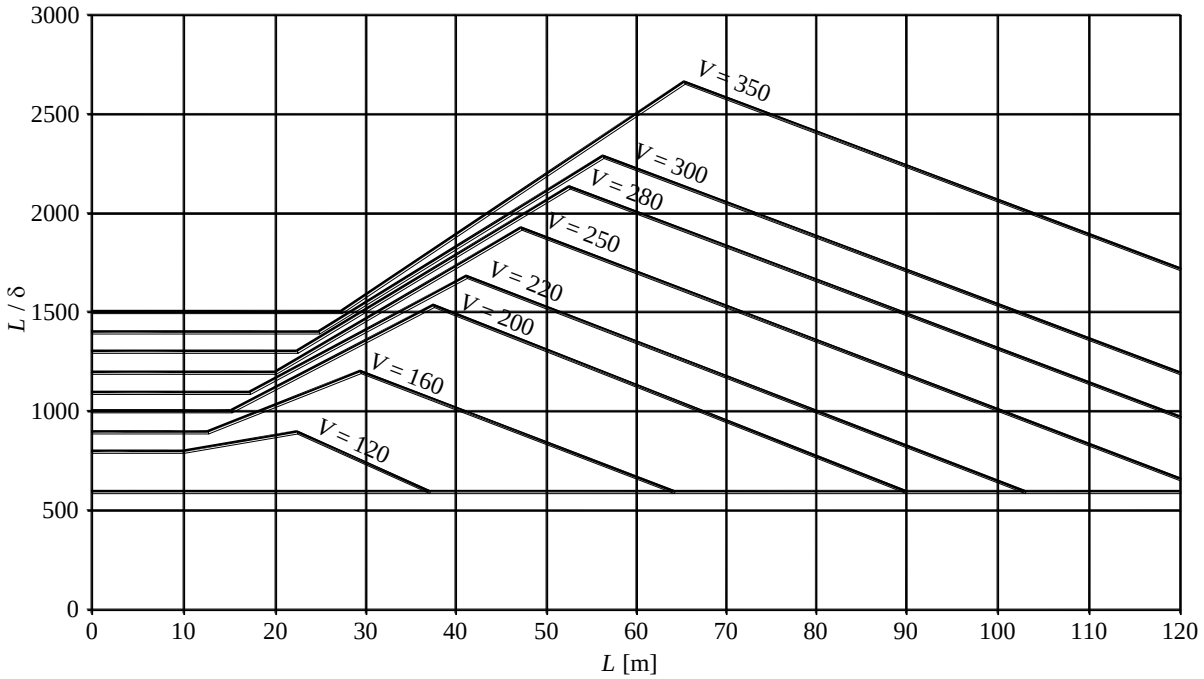
Level of Comfort	Vertical acceleration b_v (m/s²)
Very good	1,0
Good	1,3
Acceptable	2,0

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.

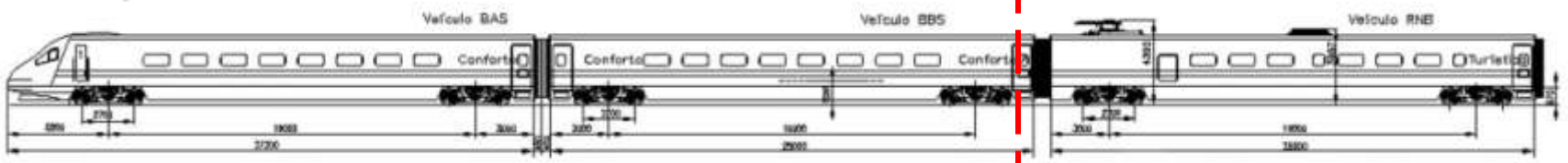


3. Dynamic analysis of the train-bridge interaction

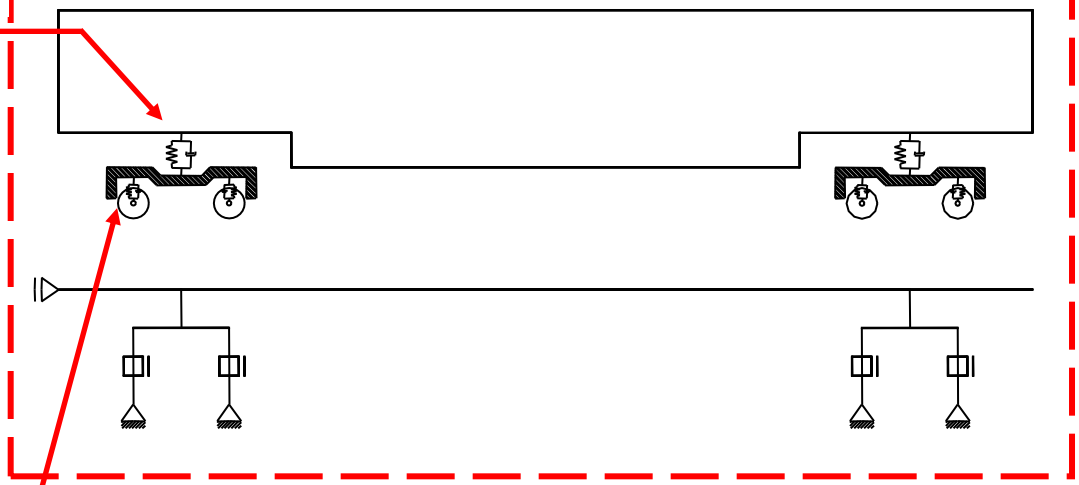
3. Dynamic analysis of the train-bridge interaction

Train modelling

Alfa pendular train



Secondary suspension



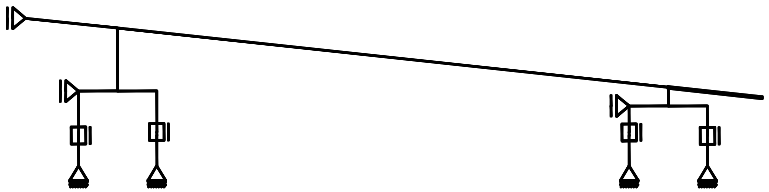
Primary suspension

3. Dynamic analysis of the train-bridge interaction

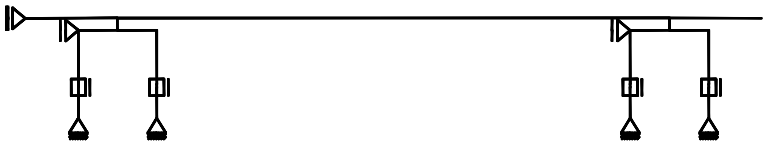
Train modelling

Alfa pendular train – Natural frequencies and mode shapes

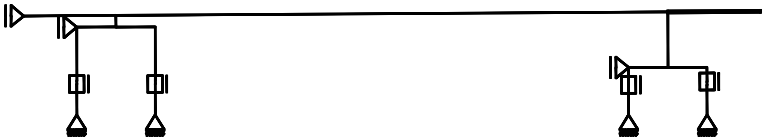
Mode 1 - f = 1.05 Hz



Mode 2 - f = 1.36 Hz



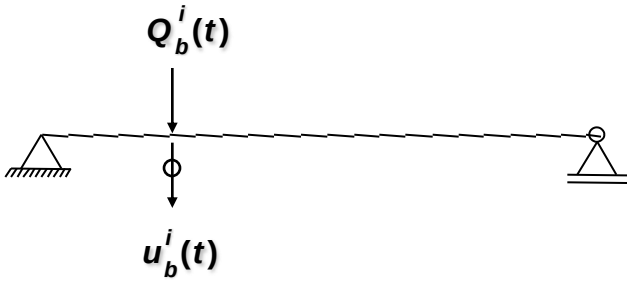
Mode 3 - f = 5.39 Hz



Mode 4 - f = 5.41 Hz

3. Dynamic analysis of the train-bridge interaction

Iterative procedure



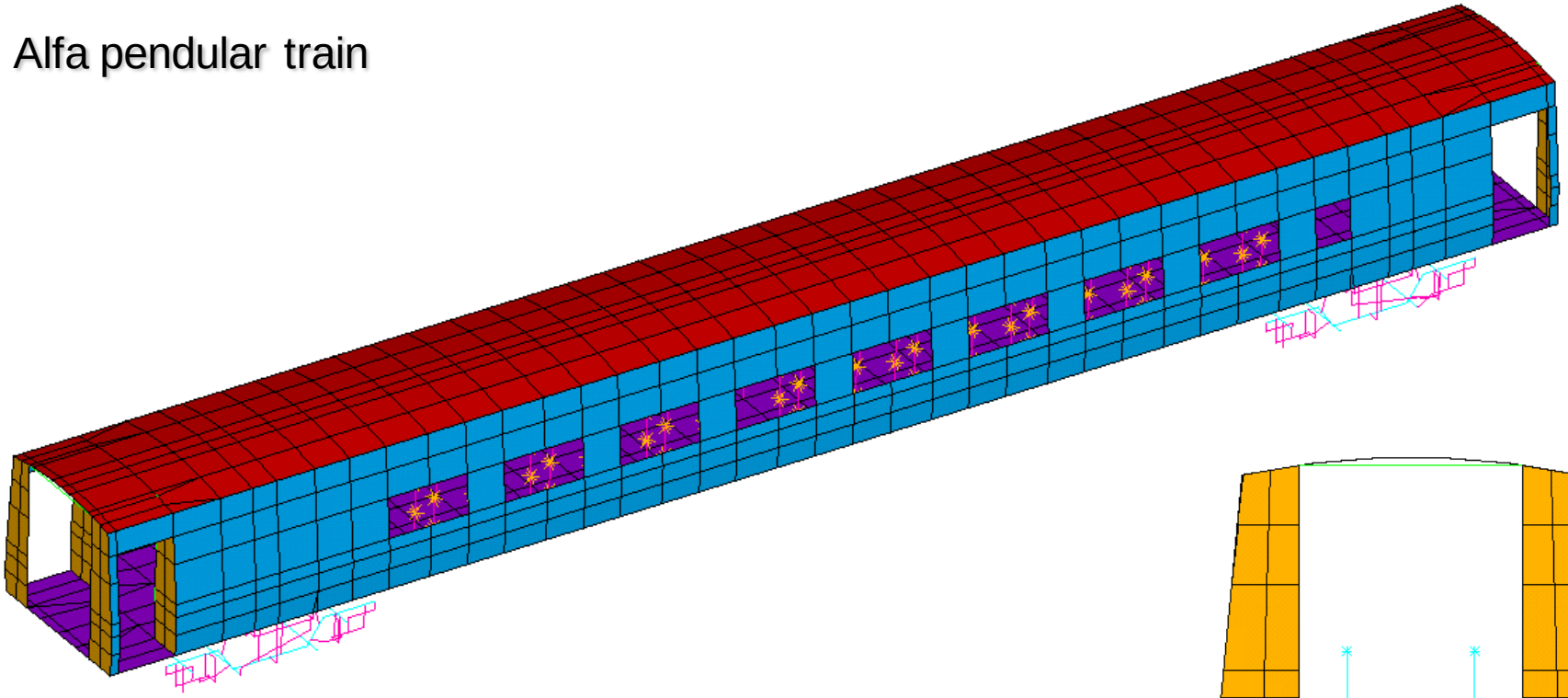
	Bridge	Vehicle
Action	$Q_b^i(t) = Q_{sta} + Q_{dyn}^{i-1}(t)$	$u_v^i(t) = u_b^{i-1}(t)$
Result	$u_b^i(t)$	$Q_{dyn}^i(t) = Q_v^i(t)$
Convergence Criterion	$\dfrac{Q_{dyn}^i(t) - Q_{dyn}^{i-1}(t)}{Q_{dyn}^{i-1}(t)}$	If > tolerance $\rightarrow i+1$ If < tolerance $\rightarrow t + \Delta t$

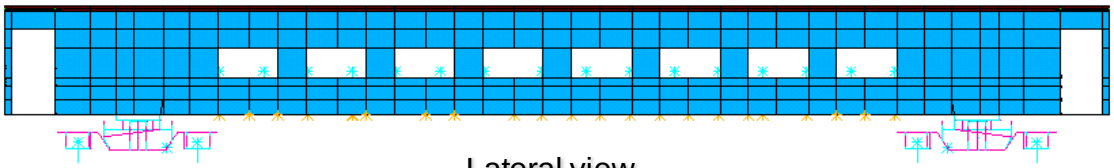
3. Dynamic analysis of the train-bridge interaction

3. Dynamic analysis of the train-bridge interaction

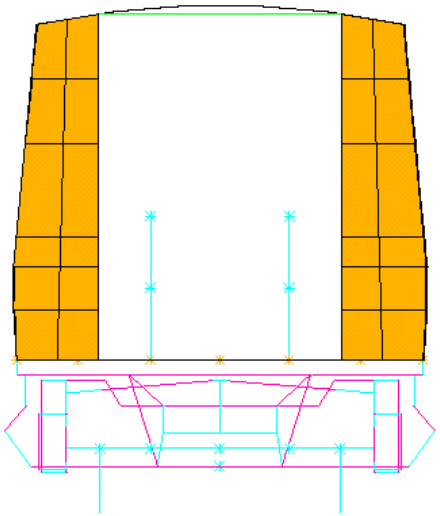
Train modelling

Alfa pendular train





Lateral view

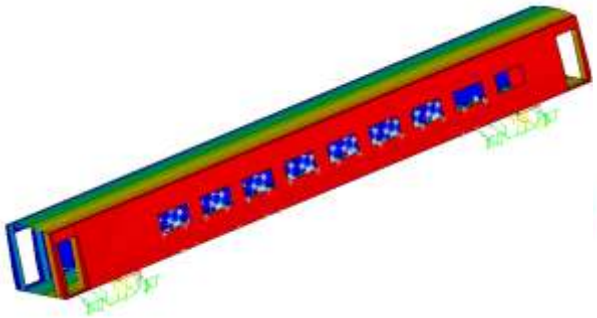


Top view

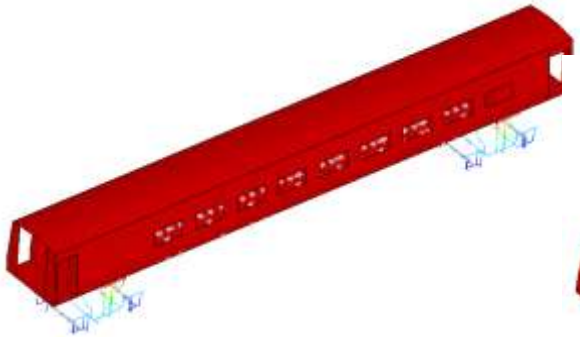
3. Dynamic analysis of the train-bridge interaction

Train modelling

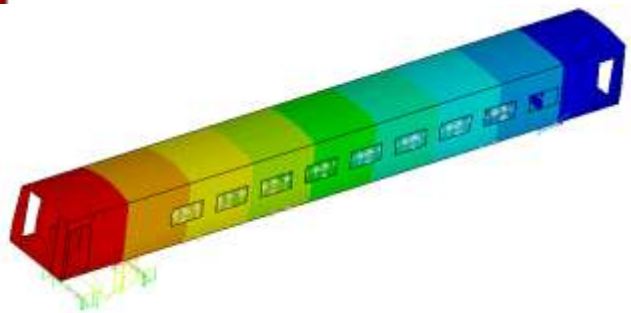
Alfa pendular train – Natural frequencies and mode shapes



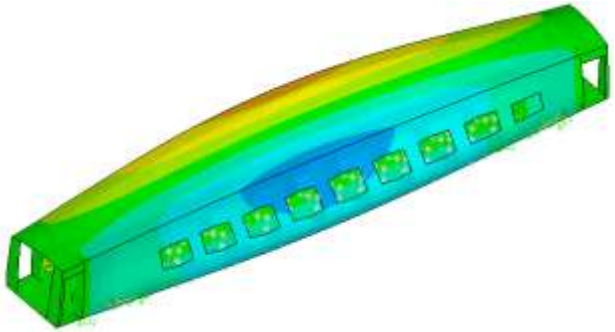
Mode 1 – f = 0.67 Hz
Rolling mode



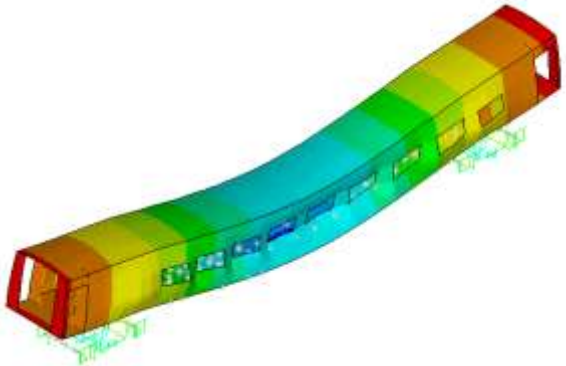
Mode 2 – f = 0.97 Hz
Bouncing mode



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

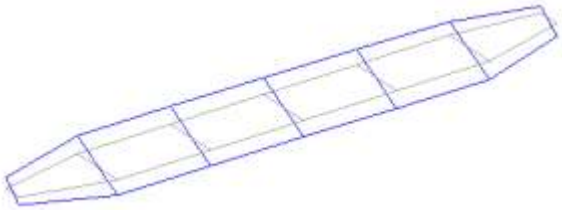
Train modelling

Dynamic test

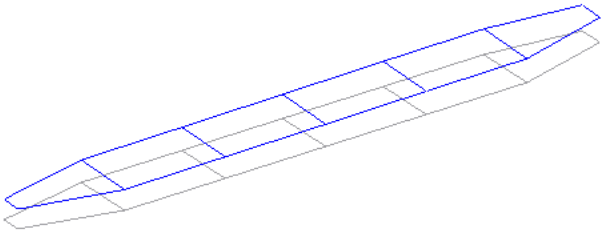
3. Dynamic analysis of the train-bridge interaction

Train modelling

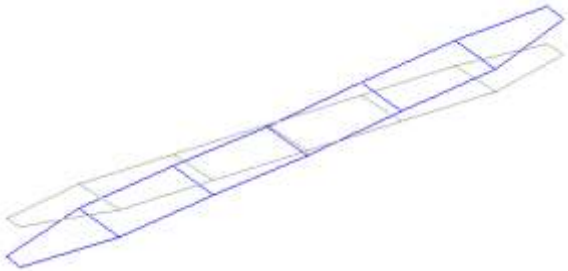
- Experimental natural frequencies and mode shapes



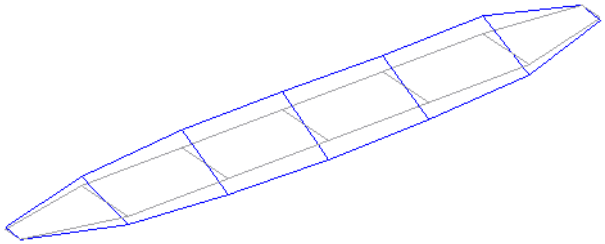
Mode 1 – f = 1.01 Hz
Rolling mode



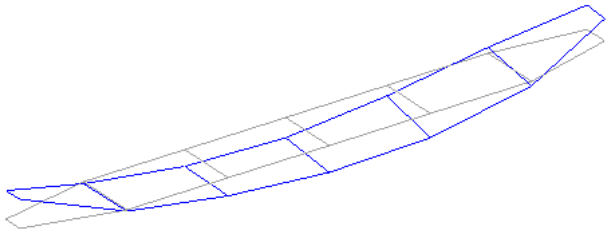
Mode 2 – f = 1.33 Hz
Bouncing mode



Mode 3 – f = 1.59 Hz
Pitching mode



Mode 4 – f = 8.38 Hz
1st torsion mode

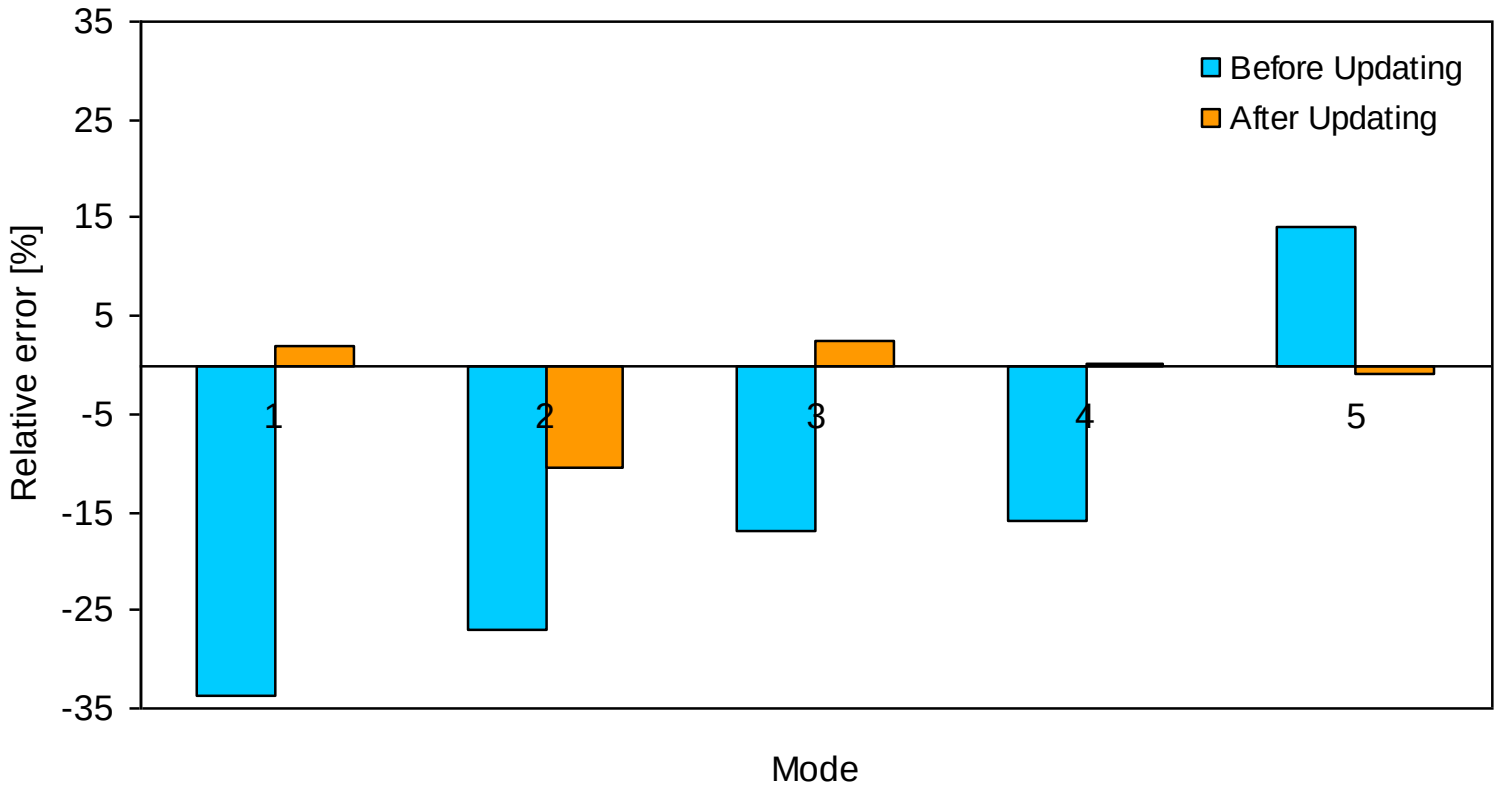


Mode 5 – f = 12.26 Hz
Bending mode

3. Dynamic analysis of the train-bridge interaction

Train modelling

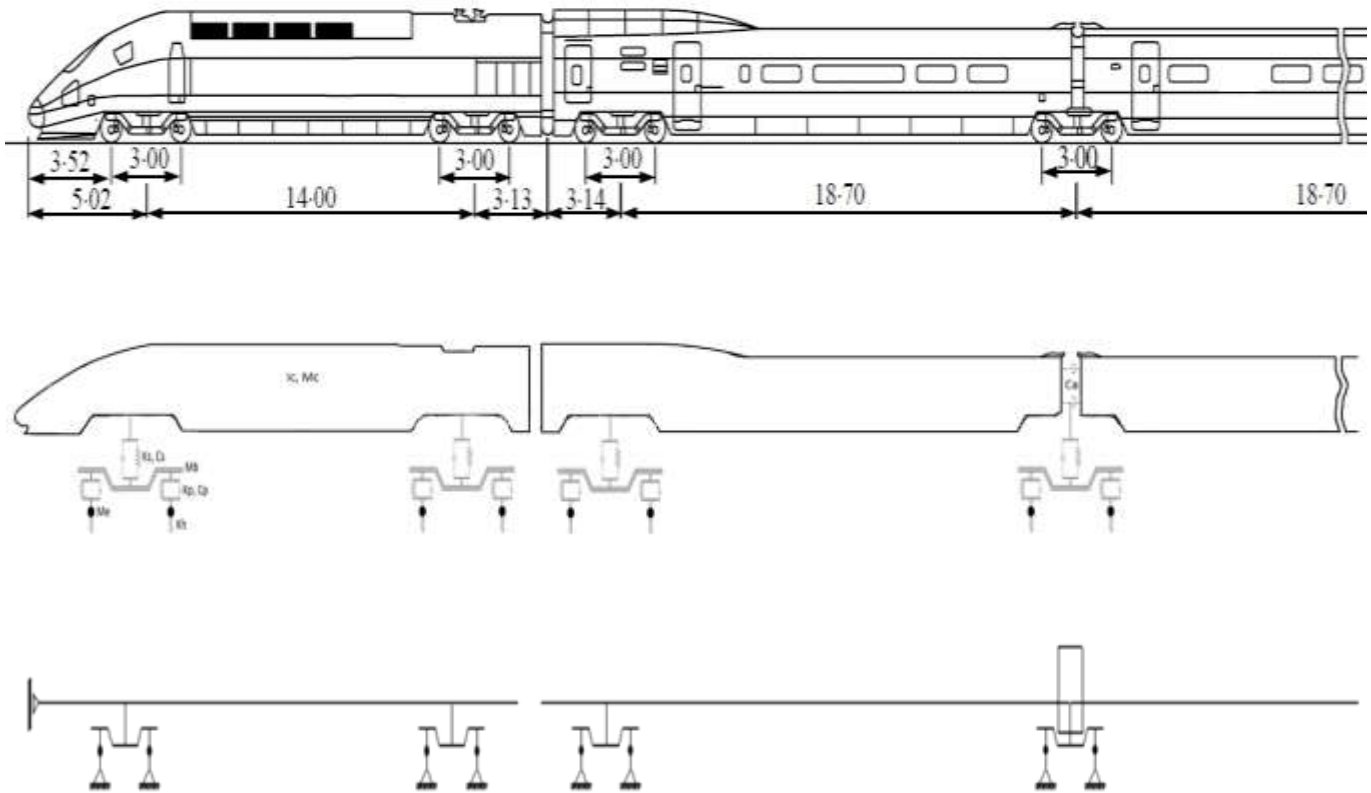
- Numerical vs experimental natural frequencies



3. Dynamic analysis of the train-bridge interaction

Train modelling

TGV train

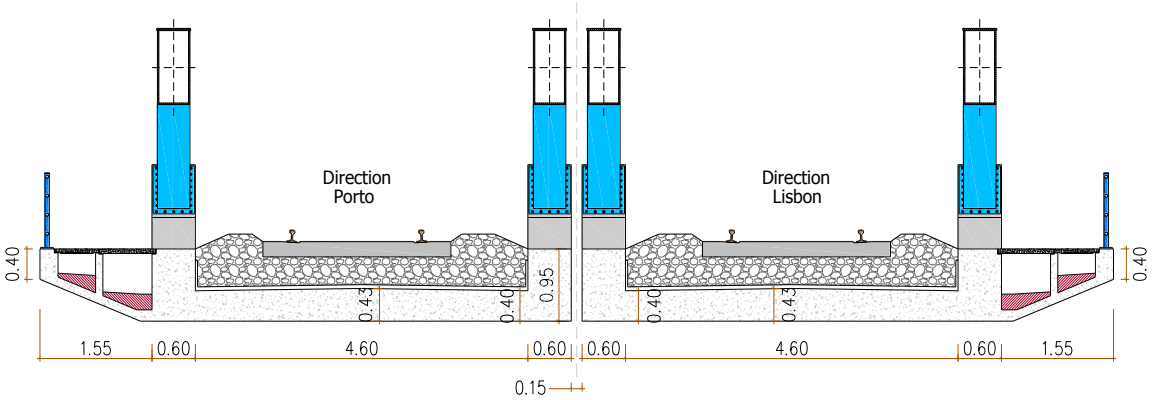


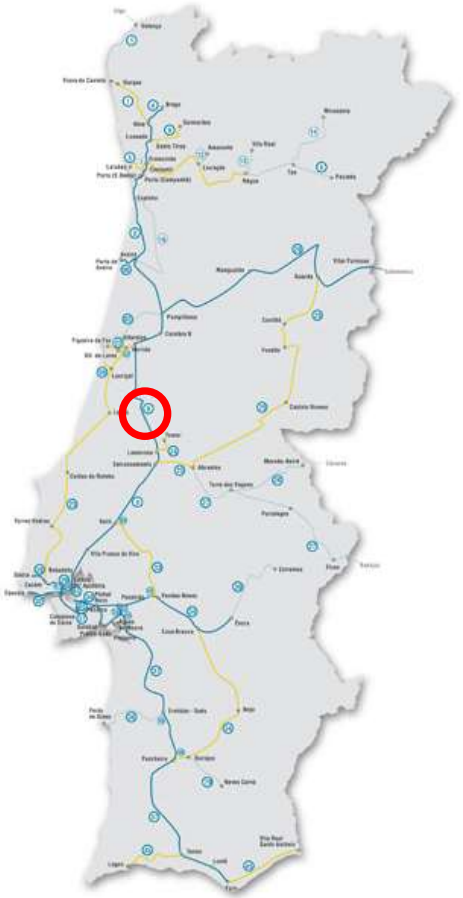
4. Dynamic analysis of a bowstring arch rail bridge

4. Dynamic analysis of a bowstring arch rail bridge

São Lourenço rail bridge

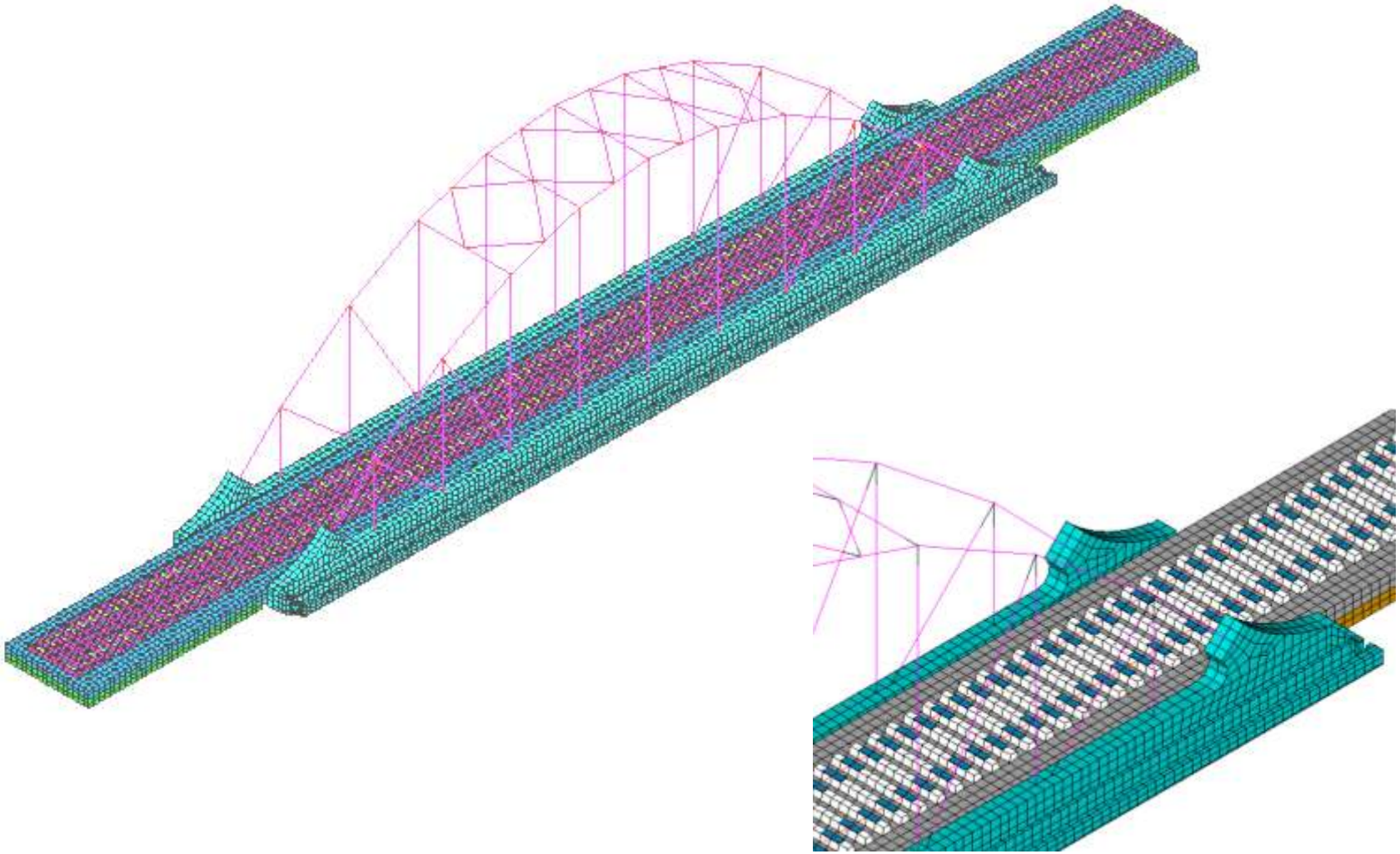
Location and description





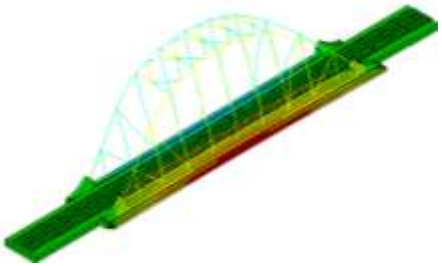
4. Dynamic analysis of a bowstring arch rail bridge

Numerical model



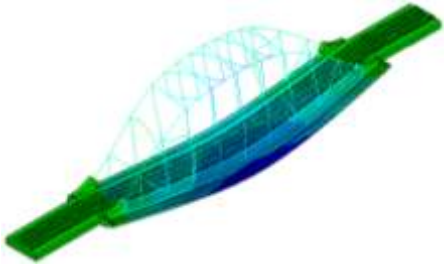
4. Dynamic analysis of a bowstring arch rail bridge

Numerical frequencies and mode shapes

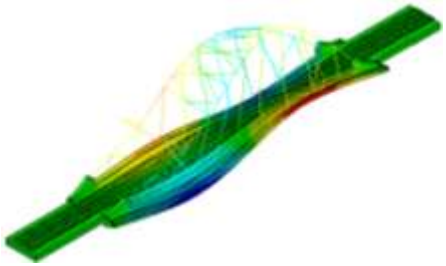


Mode 1 – f = 2.33 Hz
Transversal bending (arches)

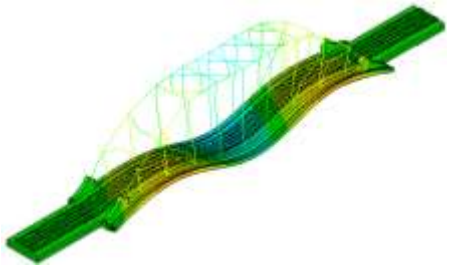
Mode 2 – f = 4.06 Hz
Vertical bending



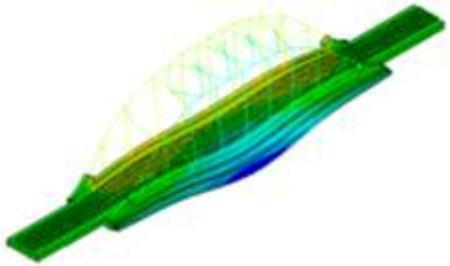
Mode 3 – f = 5.88 Hz
Vertical bending



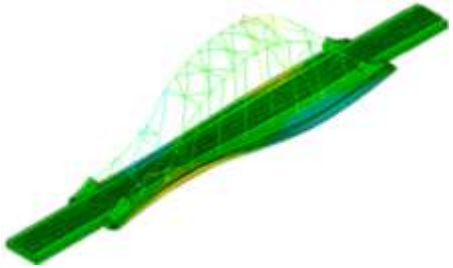
Mode 4 – f = 6.67 Hz
Torsion



Mode 5 – f = 9.01 Hz
Vertical bending

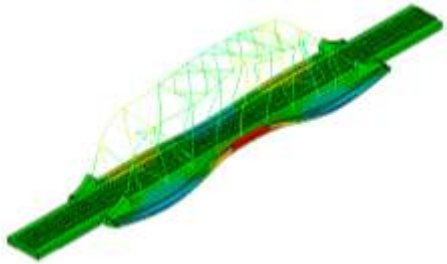


Mode 6 – f = 9.67 Hz
Vertical / tranversal bending



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

Mode 8 – f = 13.92 Hz
Vertical bending

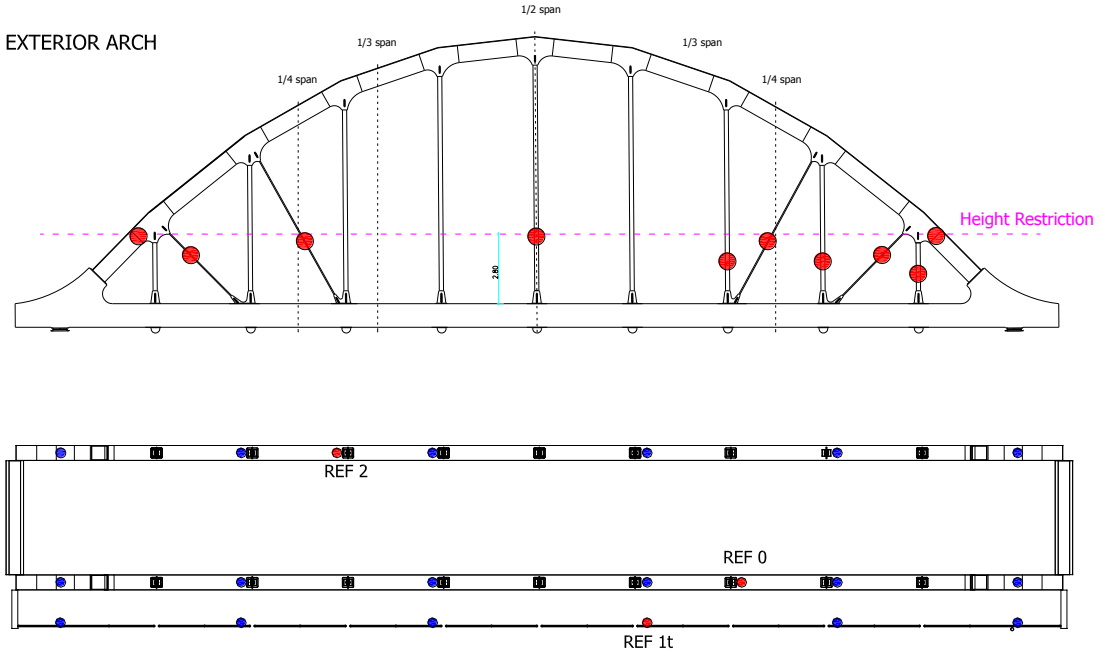


Mode 9 – f = 14.82 Hz
Torsion

4. Dynamic analysis of a bowstring arch rail bridge

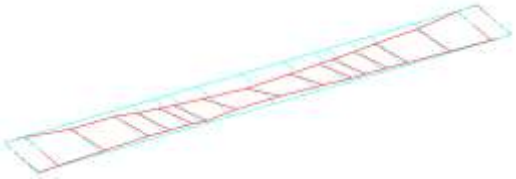
Ambient vibration test

- 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

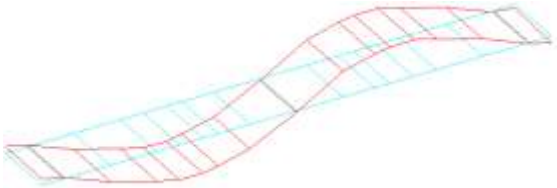


4. Dynamic analysis of a bowstring arch rail bridge

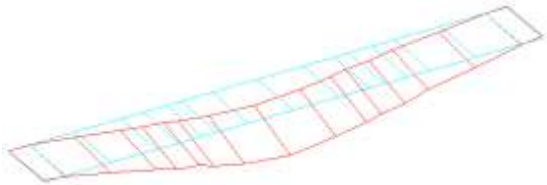
Experimental frequencies and mode shapes



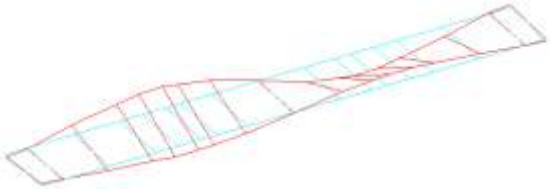
Mode 1 – f = 2.34 Hz



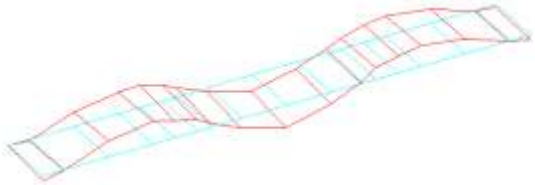
Mode 2 – f = 4.37 Hz



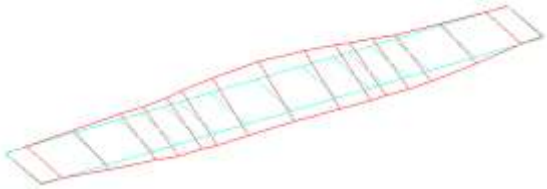
Mode 3 – f = 6.02 Hz



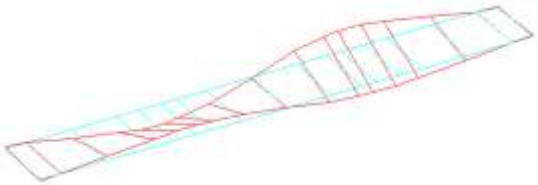
Mode 4 – f = 7.11 Hz



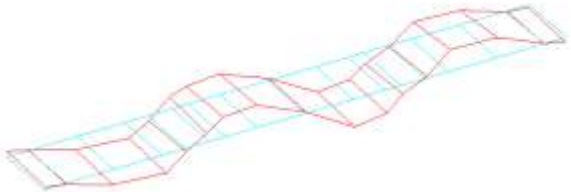
Mode 5 – f = 9.77 Hz



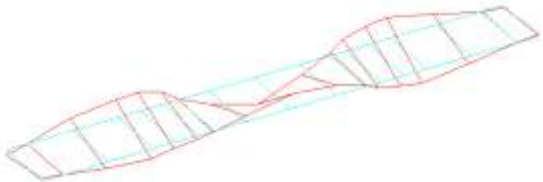
Mode 6 – f = 9.94 Hz



Mode 7 – f = 11.30 Hz



Mode 8 – f = 15.21 Hz



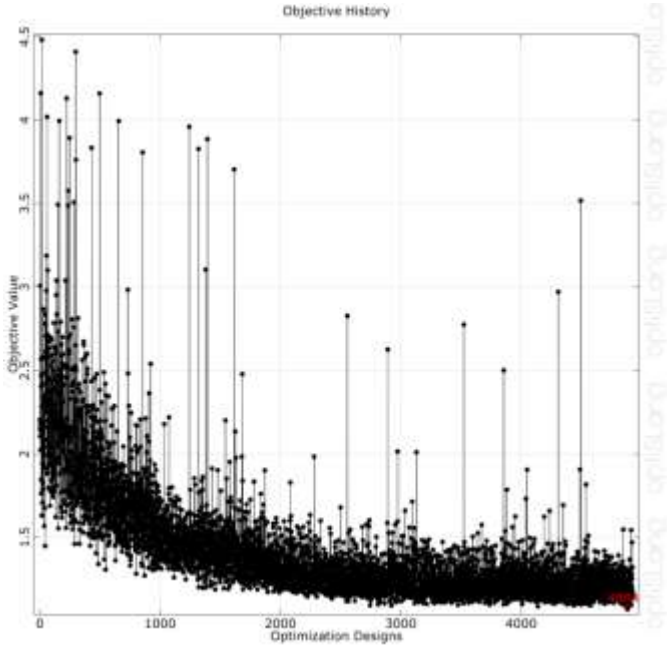
Mode 9 – f = 15.72 Hz

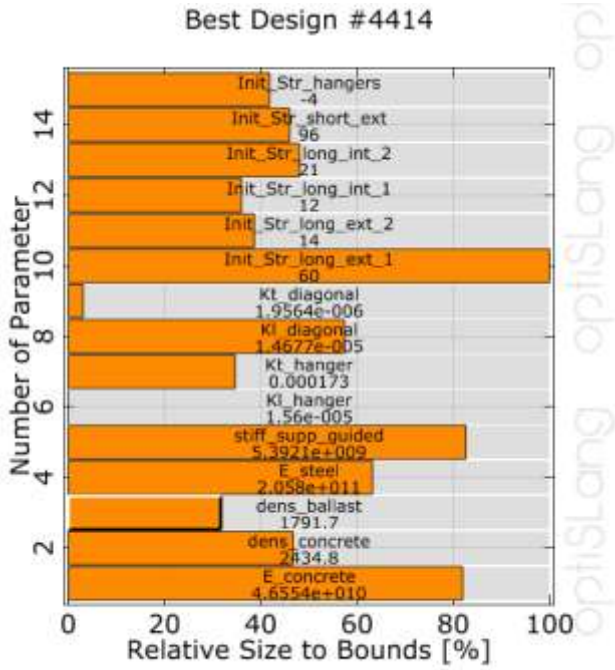
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

- Genetic algorithm (GA)

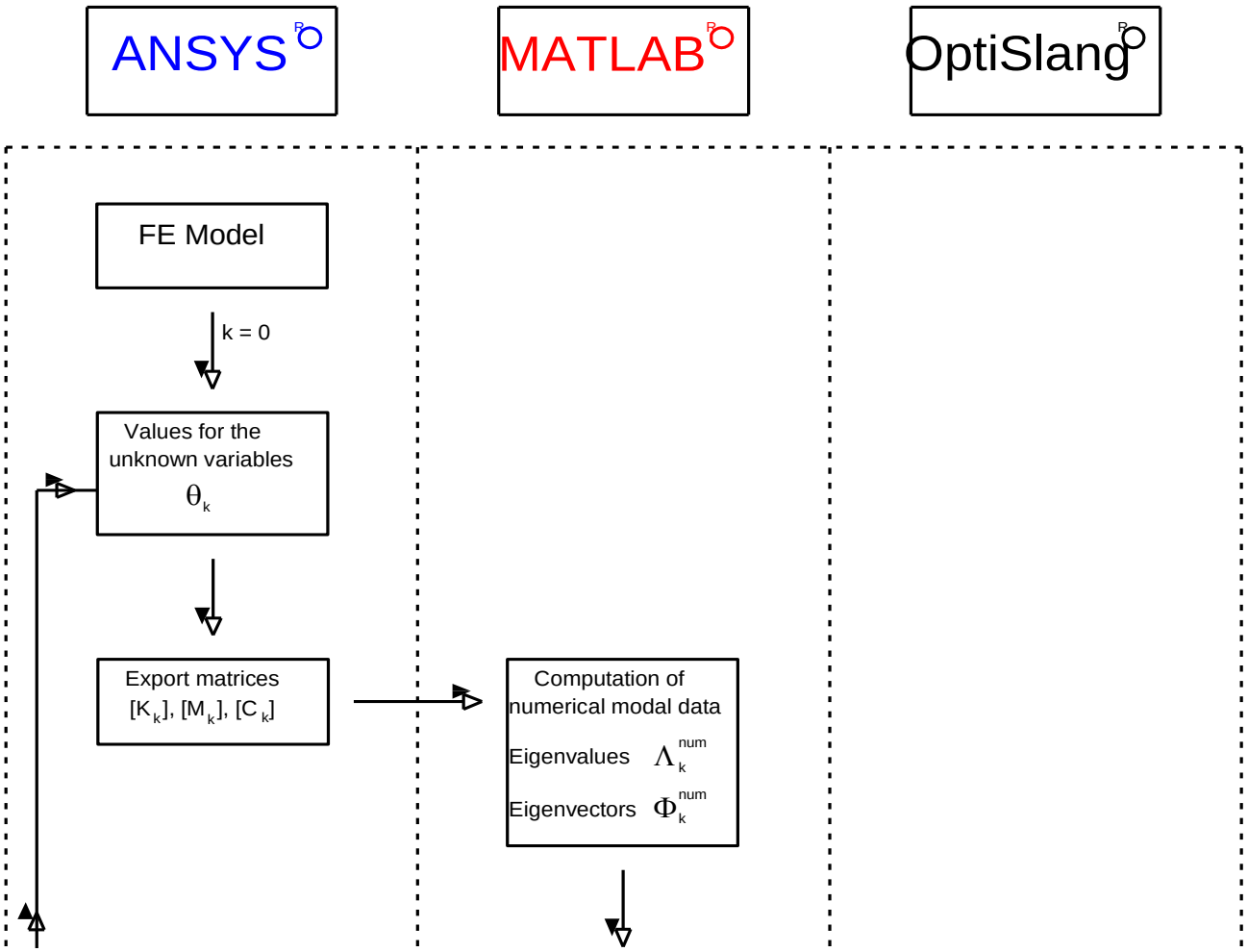
$$f = a \sum_{i=1}^{n\,\mathrm{mod}\,es} \frac{\left| f_i^{\exp} - f_i^{num} \right|}{f_i^{\exp}} + b \sum_{i=1}^{n\,\mathrm{mod}\,es} \left| MAC\left(\phi_i^{\exp}, \phi_i^{num}\right) - 1 \right|$$





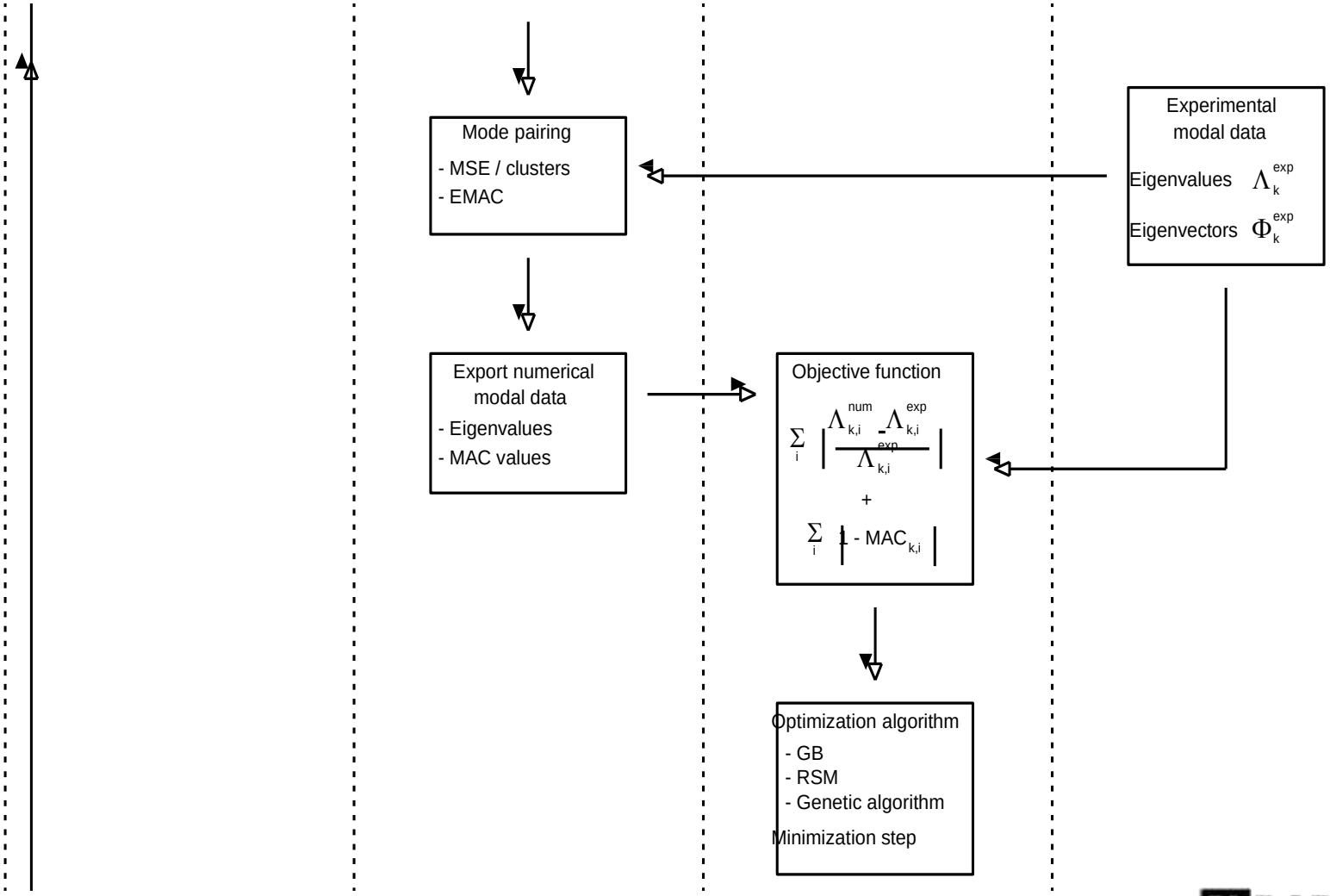
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating



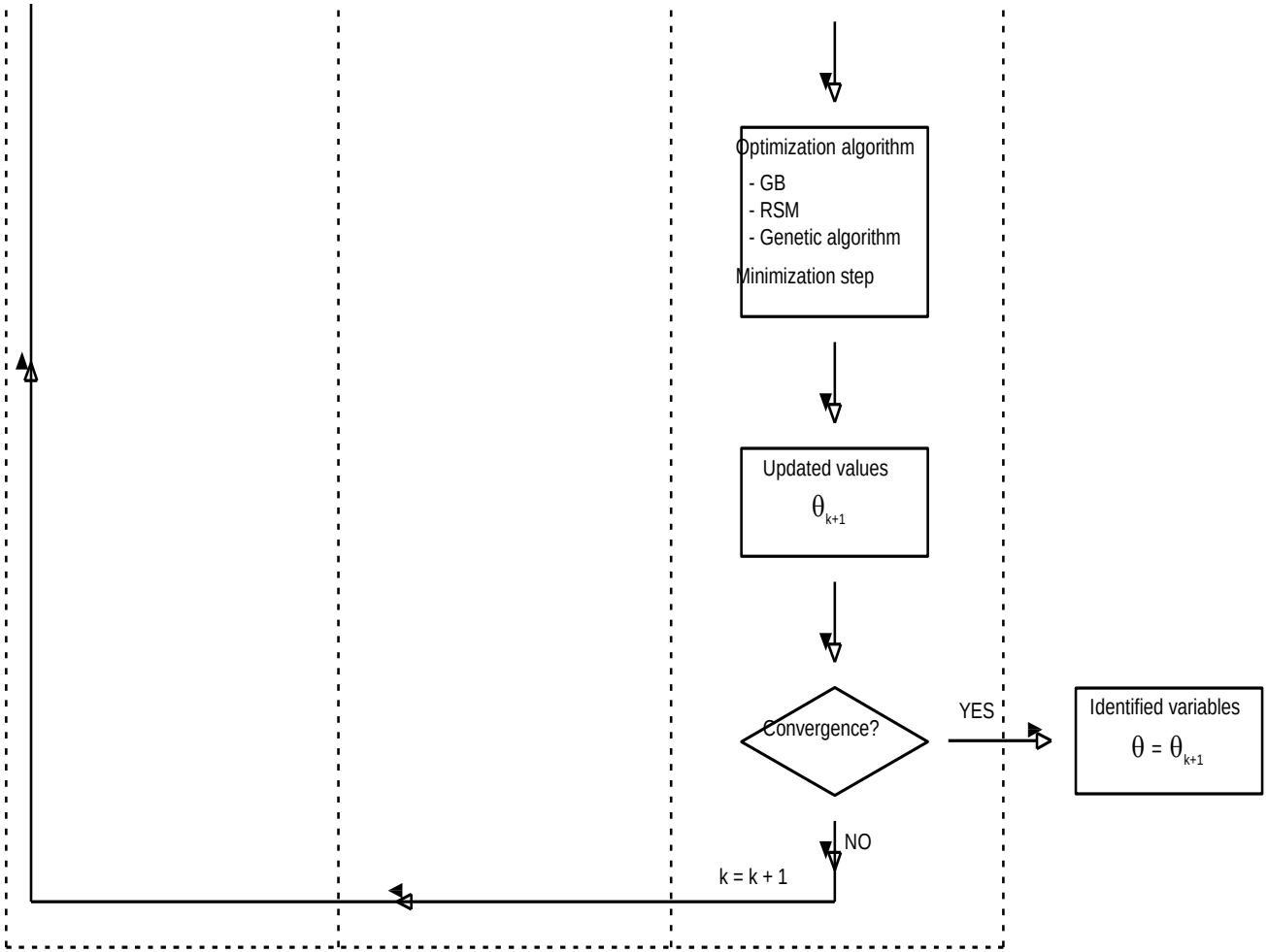
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating



4. Dynamic analysis of a bowstring arch rail bridge

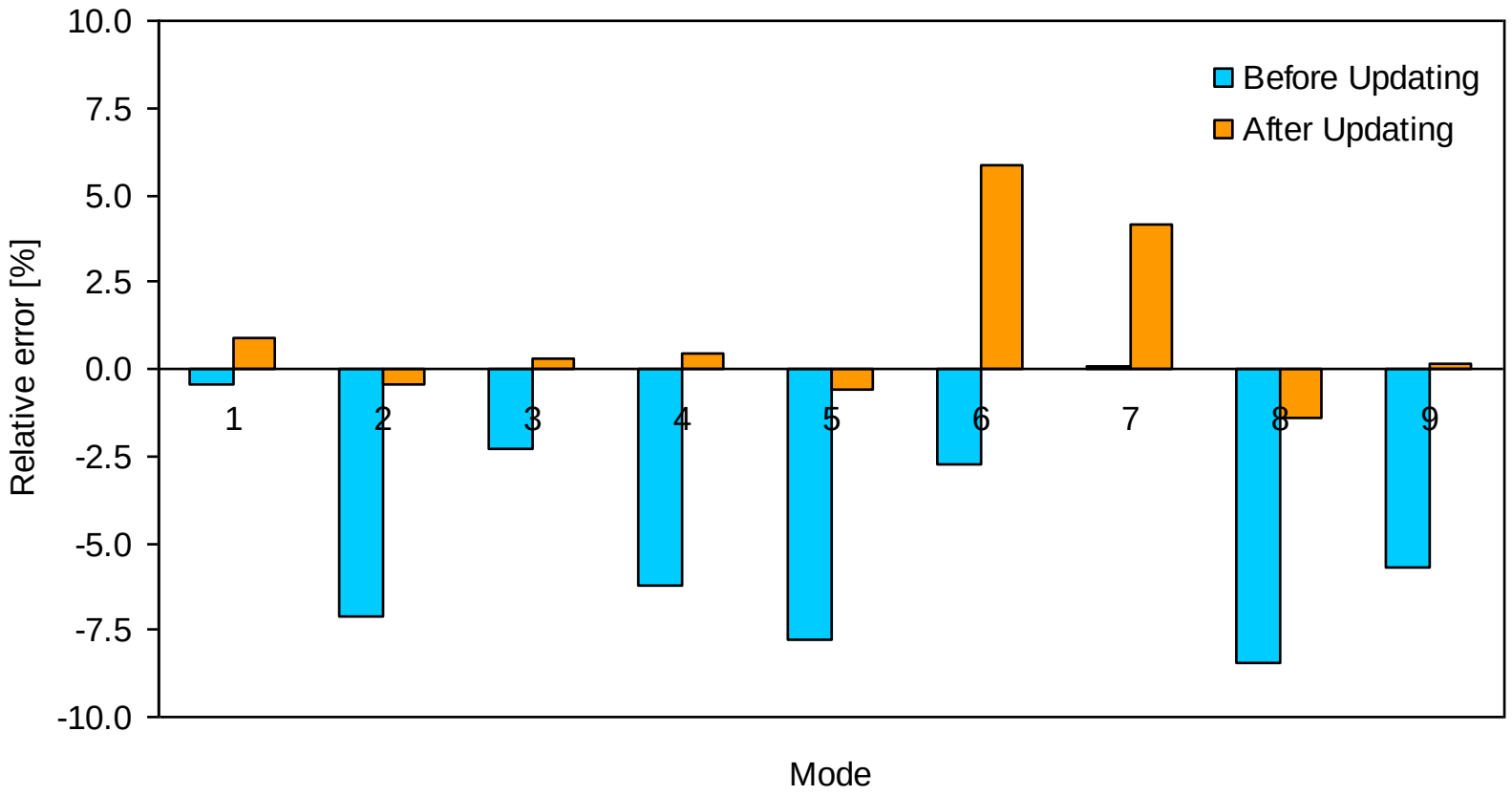
Model Updating



4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

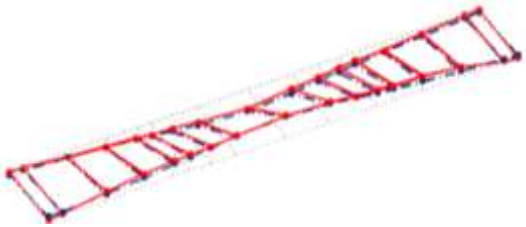
- Experimental vs numerical natural frequencies



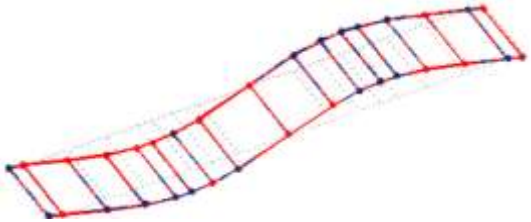
4. Dynamic analysis of a bowstring arch rail bridge

Model Updating

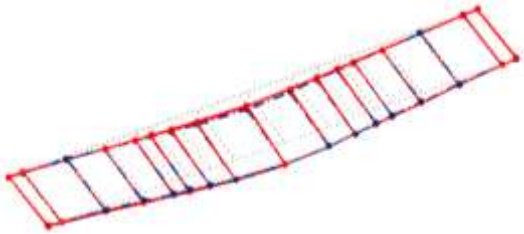
- Experimental vs numerical mode shapes



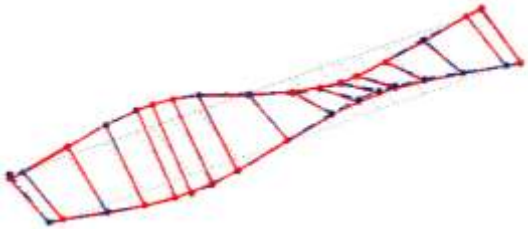
Mode 1



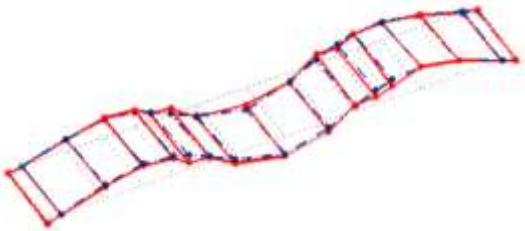
Mode 2



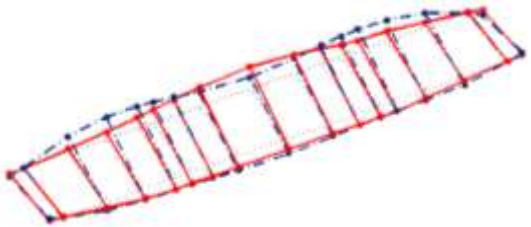
Mode 3



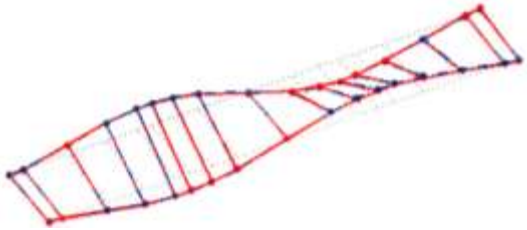
Mode 4



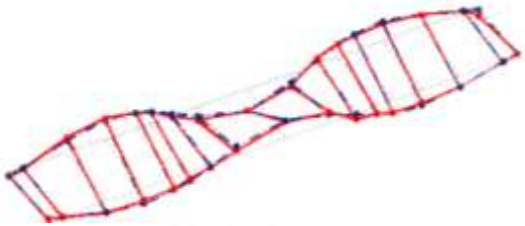
Mode 5



Mode 6



Mode 7



Mode 8

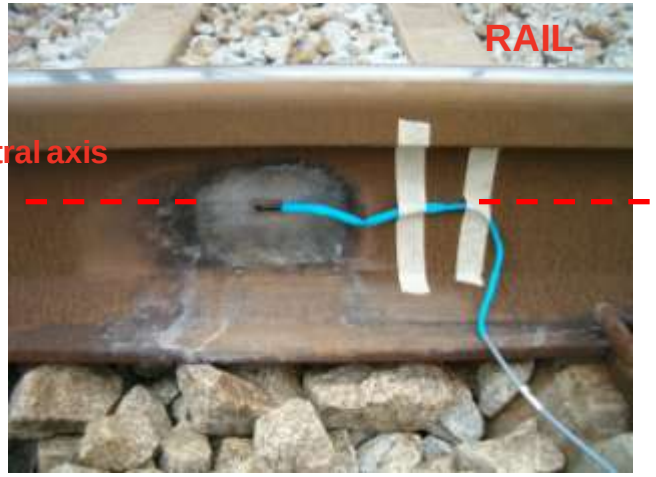
Mode 9

4. Dynamic analysis of a bowstring arch rail bridge

Dynamic tests under railway traffic

- Displacements and accelerations

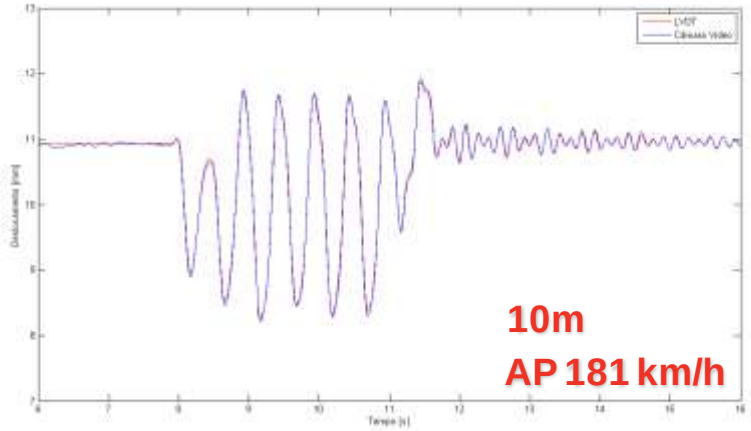
- Deformations

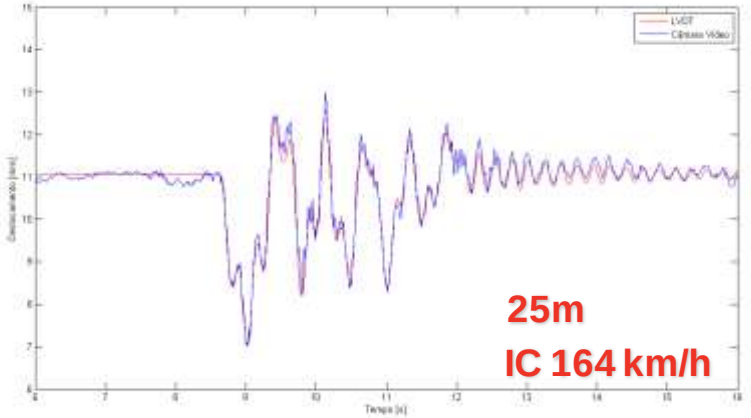


4. Dynamic analysis of a bowstring arch rail bridge

Experimental tests under railway traffic

- Measuring displacements using an advanced non-contact video system

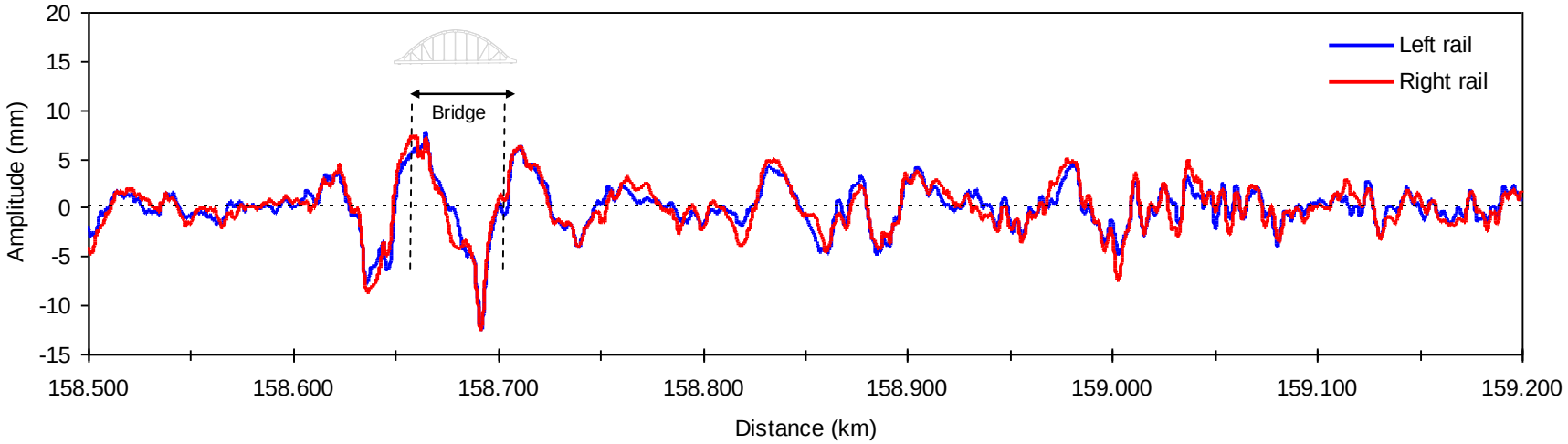




4. Dynamic analysis of a bowstring arch rail bridge

Characterization of the track irregularities

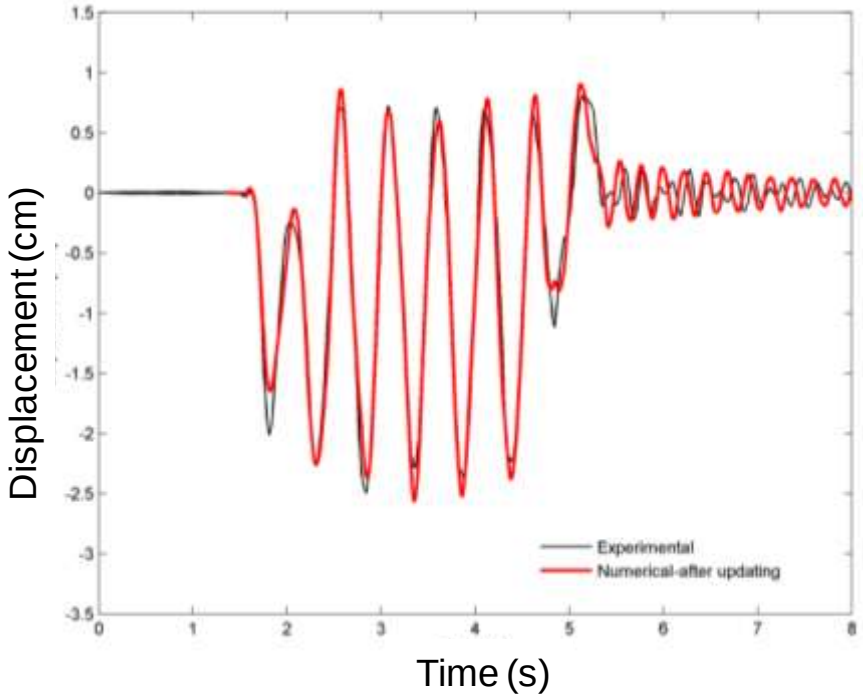
- Track inspection vehicle (EM120)

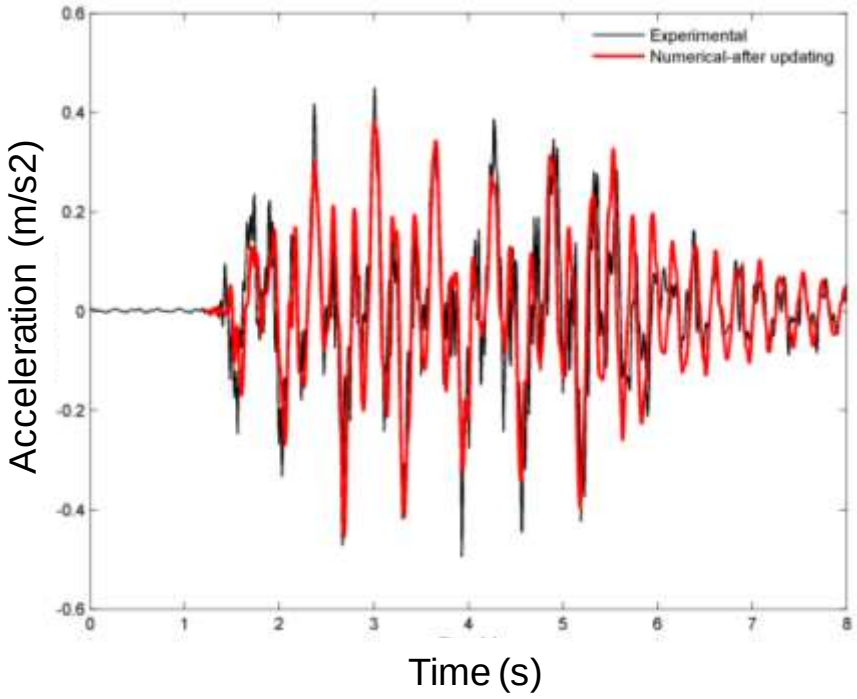


4. Dynamic analysis of a bowstring arch rail bridge

Experimental validation of the numerical model

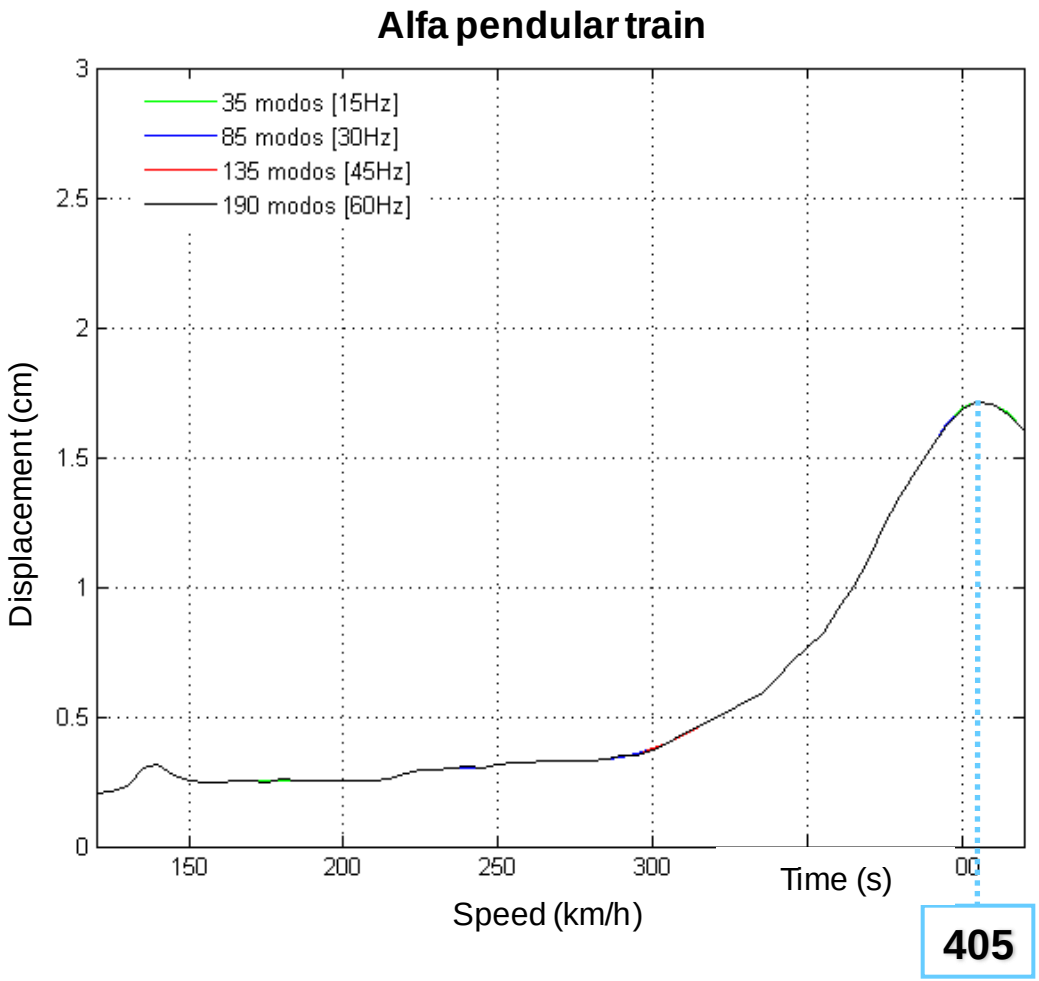
- Alfa pendular train v=185 km/h





4. Dynamic analysis of a bowstring arch rail bridge

Simulation results



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

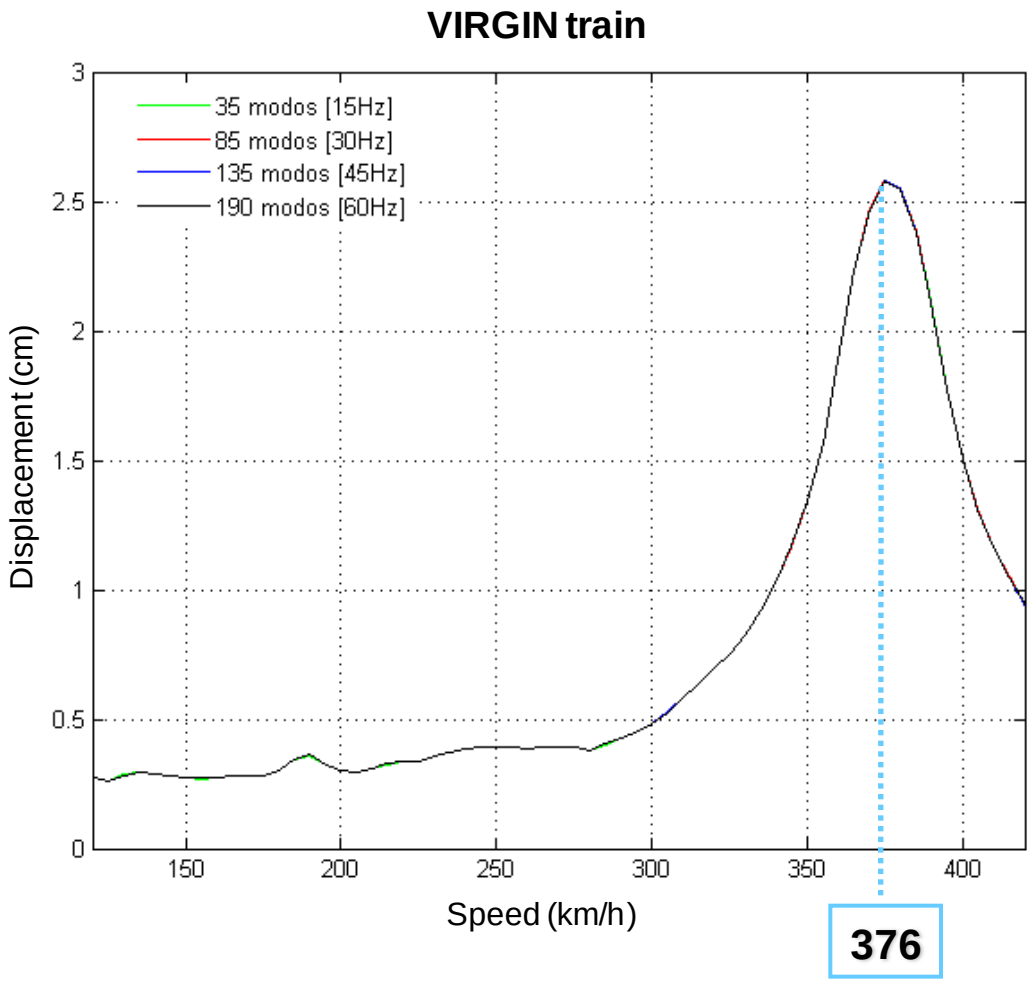
n_0=4,37 Hz

d=25,9m

$$V_{res}(1,2) = 25{,}9 \times 4{,}37 = 113\,\frac{m}{s} \approx 405\ kmh$$

4. Dynamic analysis of a bowstring arch rail bridge

Simulation results



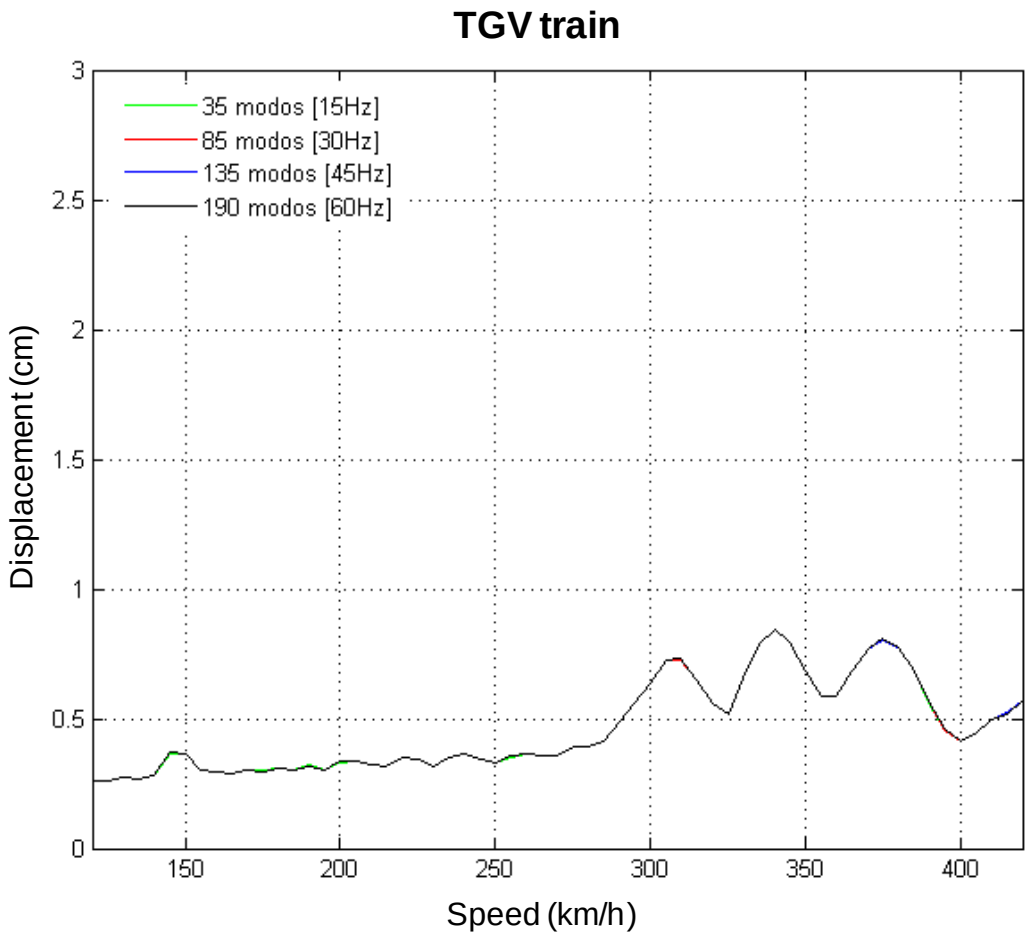
$$v_{res}(i, j) = \frac{dn_j}{i}$$

n_0=4,37 Hz

d=23,9m

4. Dynamic analysis of a bowstring arch rail bridge

Simulation results



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

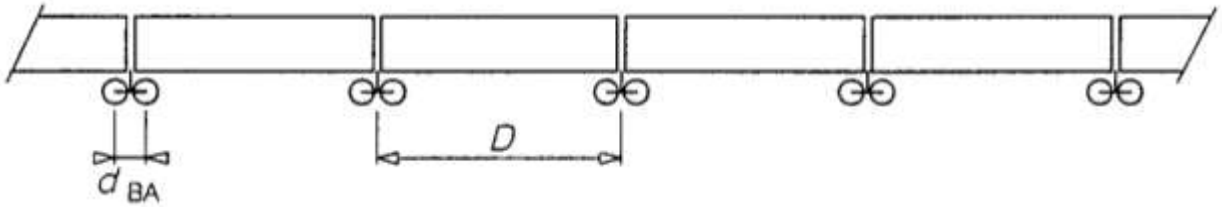
n_0=4,37 Hz

d=18,7m

4. Dynamic analysis of a bowstring arch rail bridge

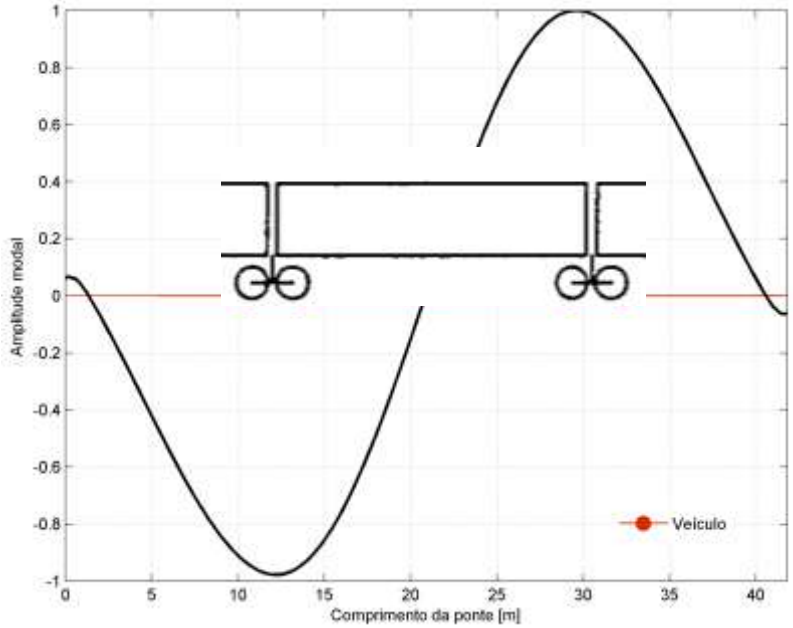
Simulation results

TGV train

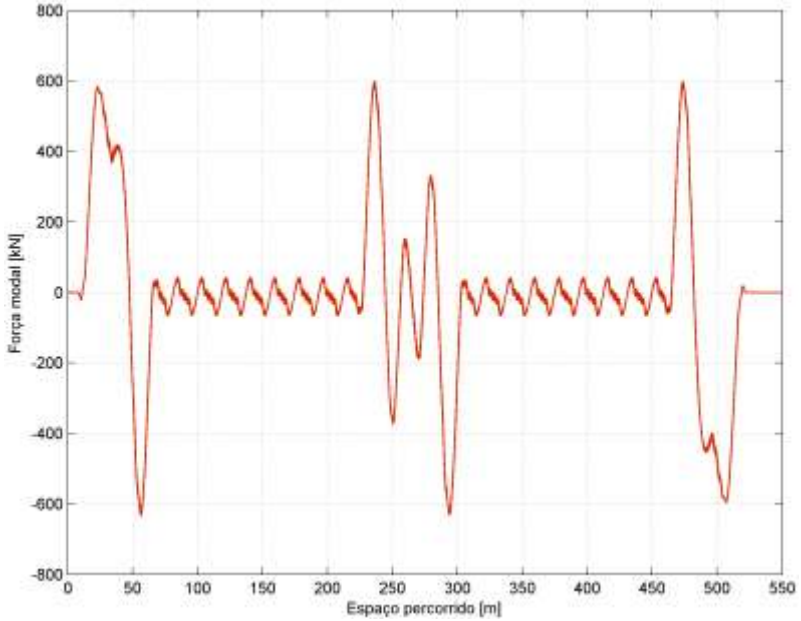


TGV (D=18.7m)

Mode shape



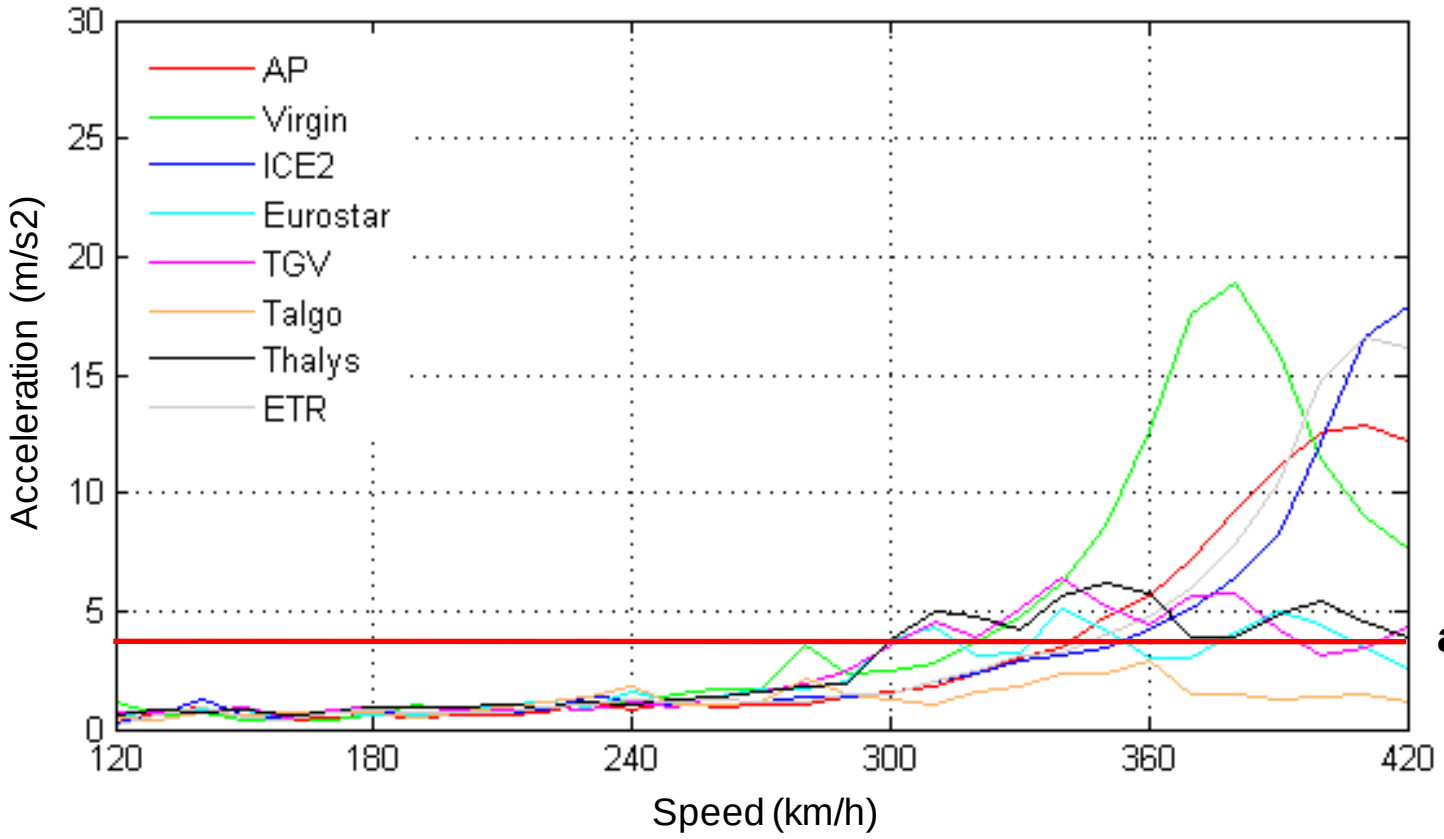
Modal force



4. Dynamic analysis of a bowstring arch rail bridge

Traffic safety

- Vertical acceleration of the deck

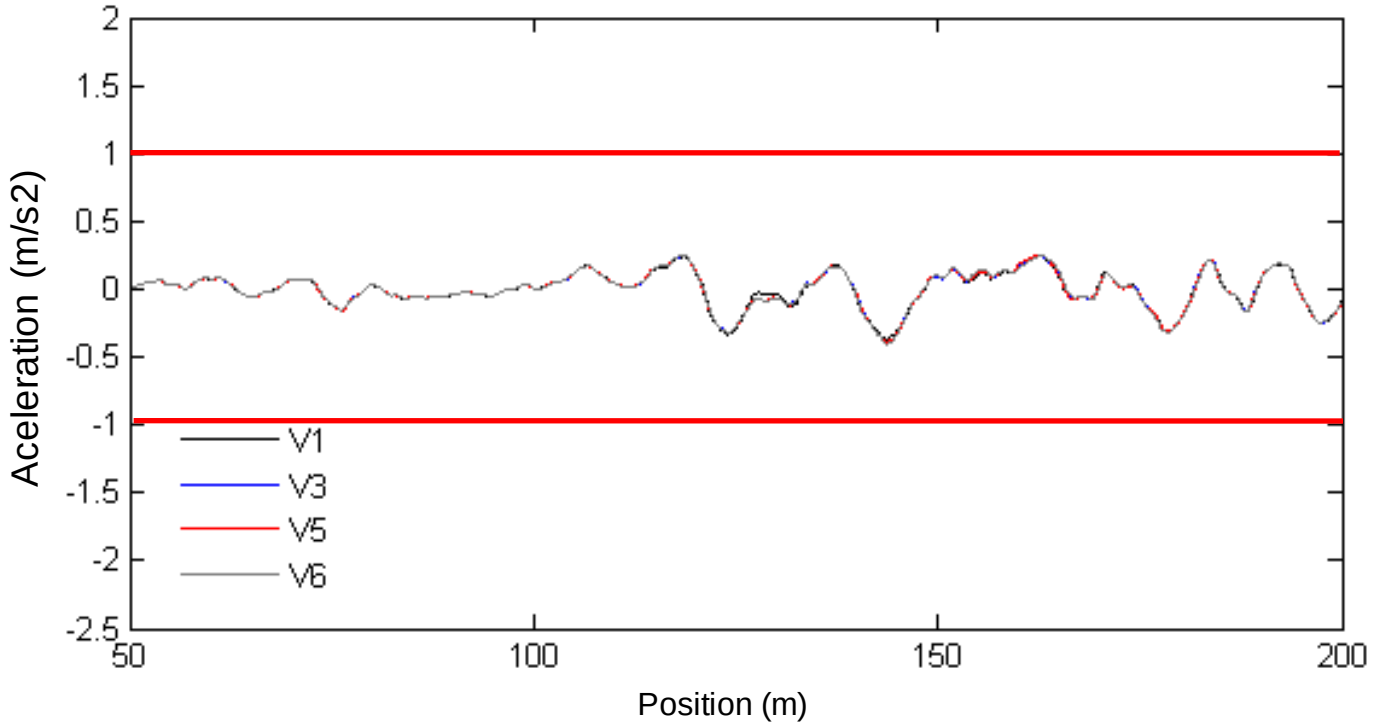


a_{max} = 3.5 m/s²

4. Dynamic analysis of a bowstring arch rail bridge

Passengers comfort

- Alfa pendular train v = 250 km/h

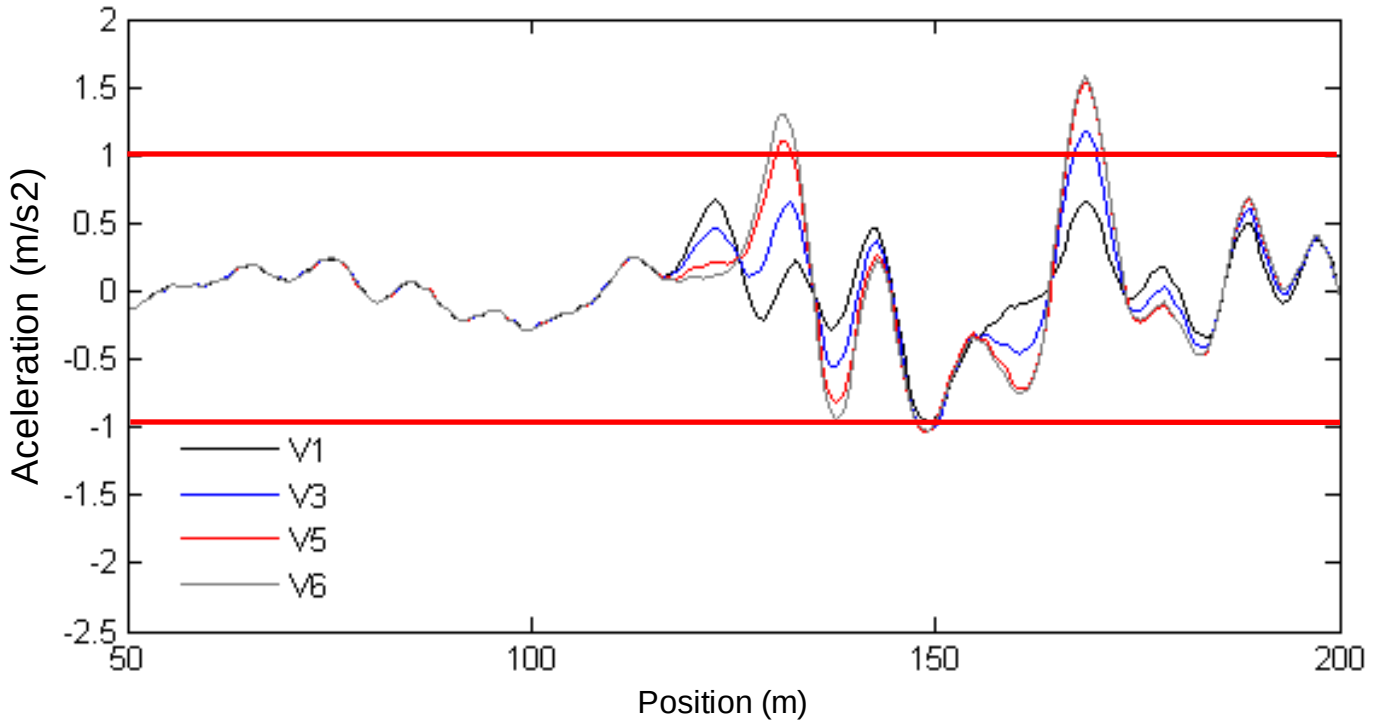


a_{max} = 1.0 m/s²

4. Dynamic analysis of a bowstring arch rail bridge

Passengers comfort

- Alfa pendular train v = 405 km/h



a_{max} = 1.0 m/s²

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

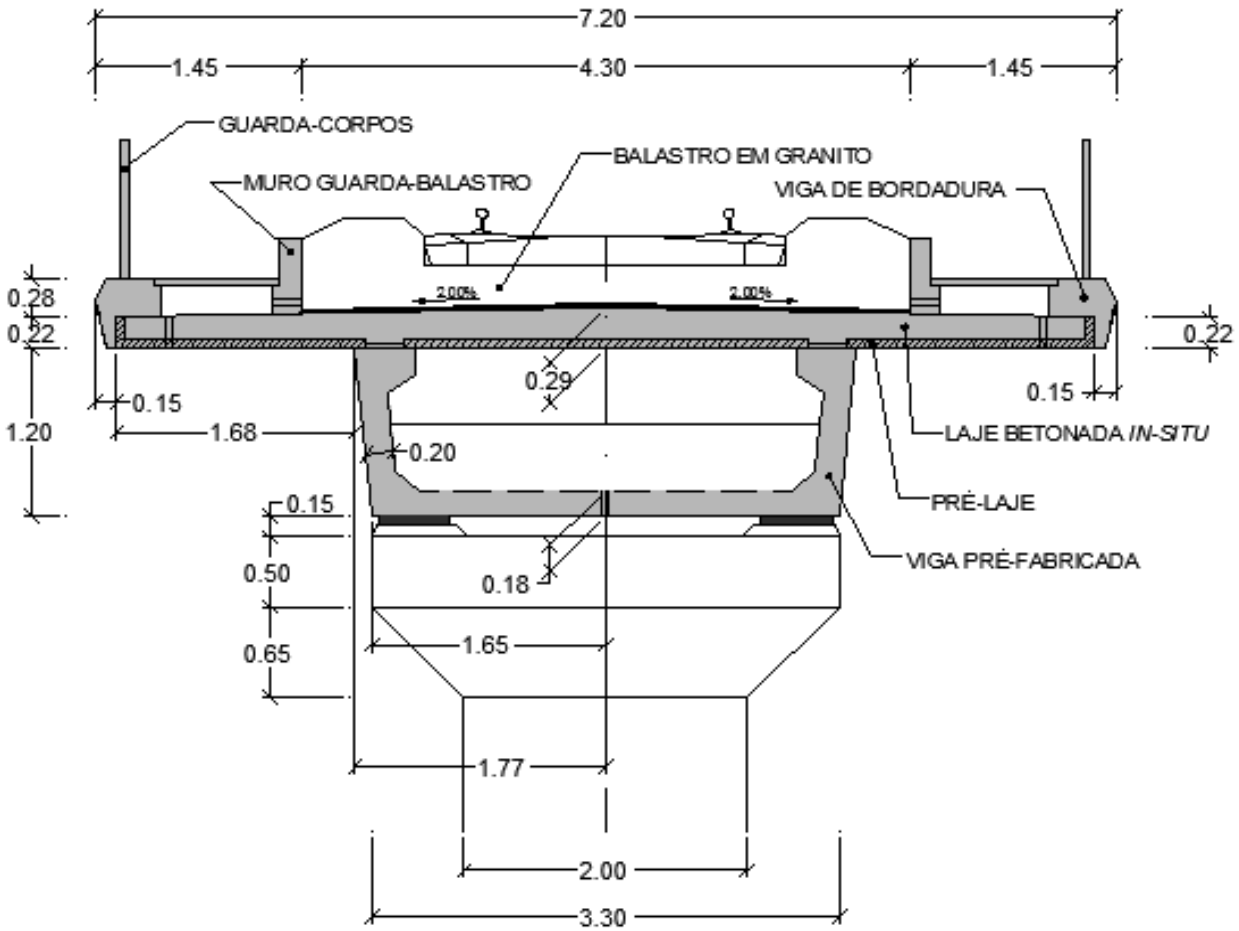
Alverca viaduct

Location and description

5. Dynamic analysis of a precast rail bridge

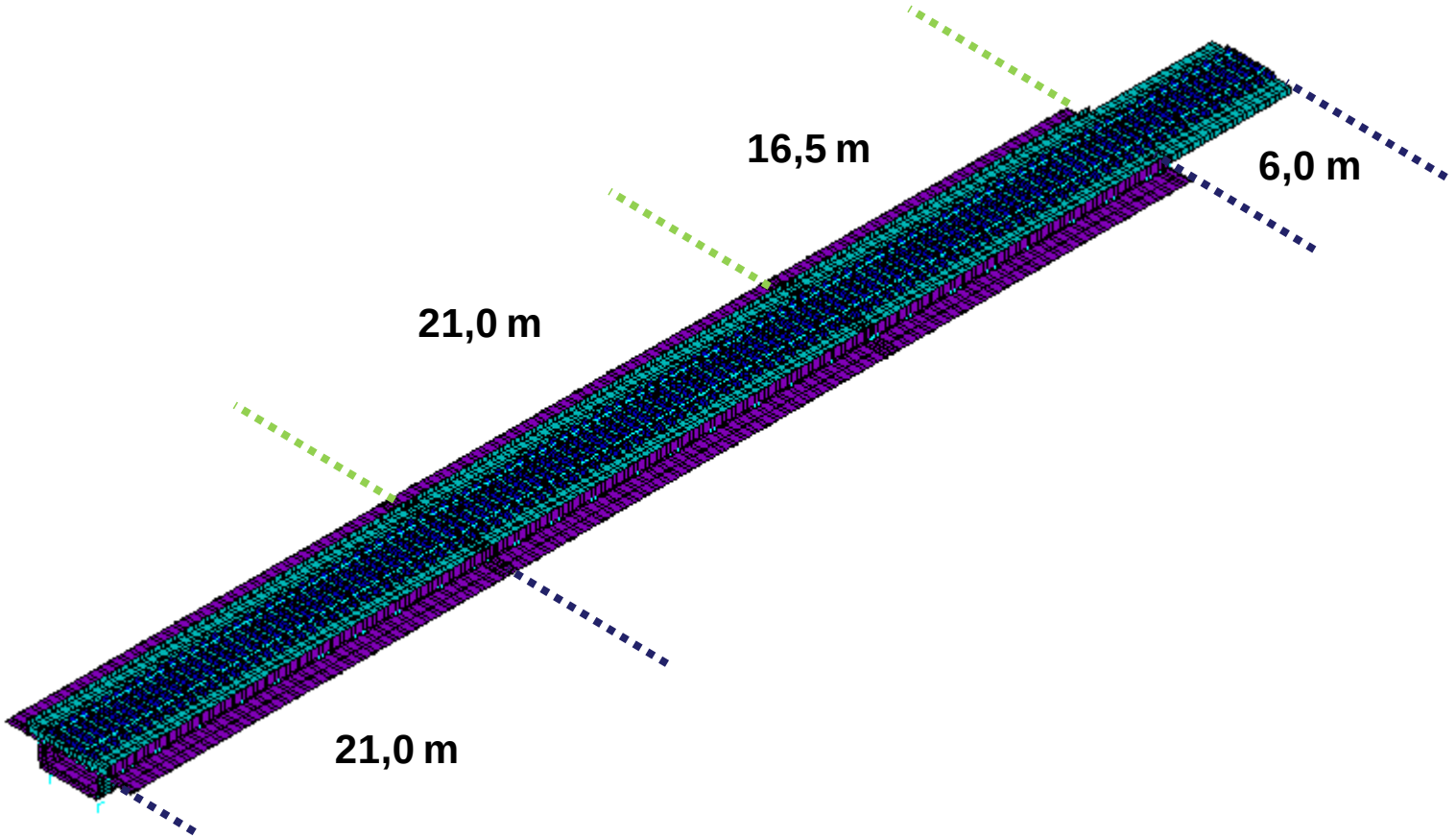
Alverca viaduct

Cross section of the deck



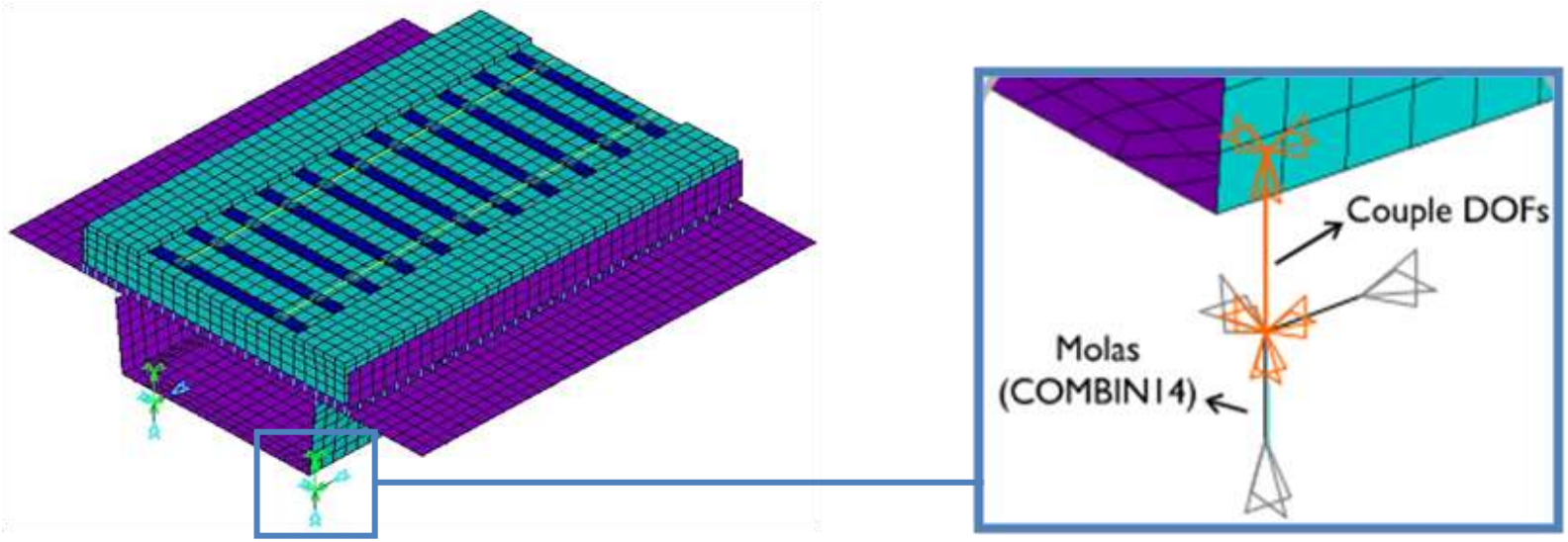
5. Dynamic analysis of a precast rail bridge

Numerical model



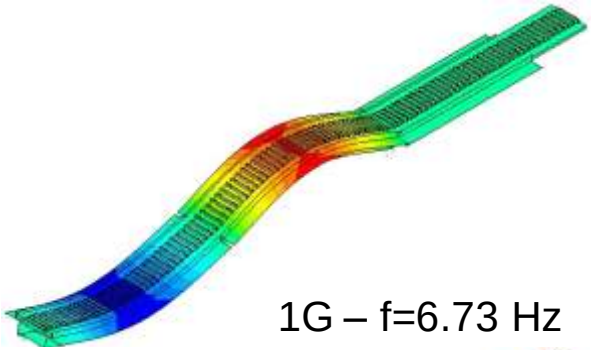
5. Dynamic analysis of a precast rail bridge

Numerical model

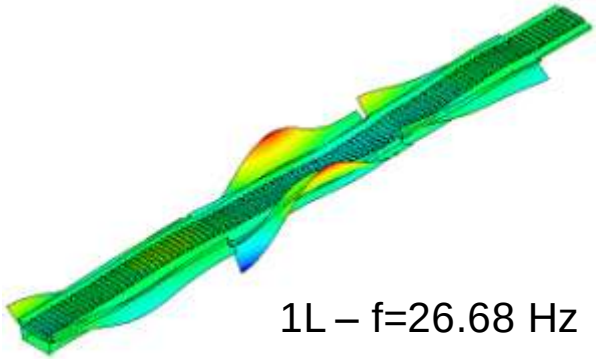


5. Dynamic analysis of a precast rail bridge

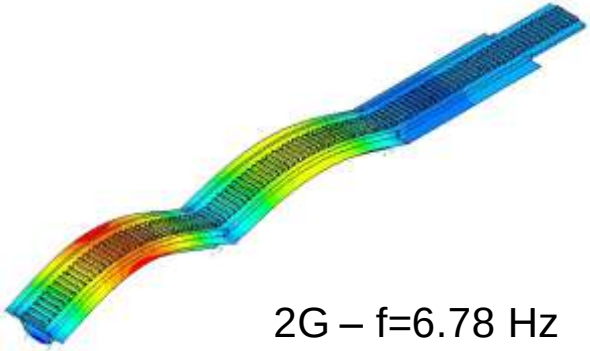
Numerical frequencies and mode shapes



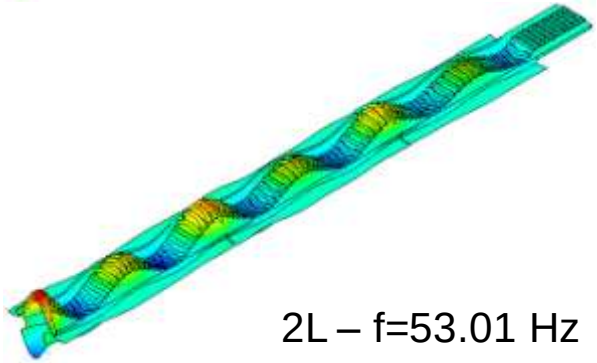
1G – f=6.73 Hz



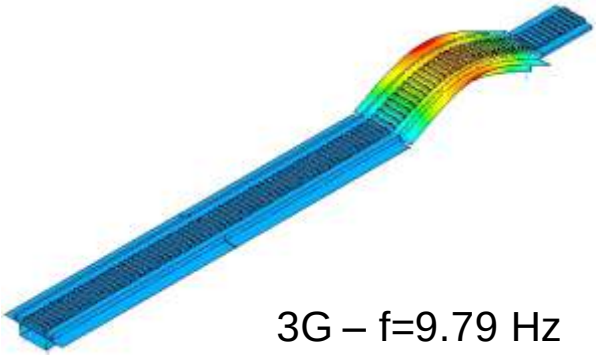
1L – f=26.68 Hz



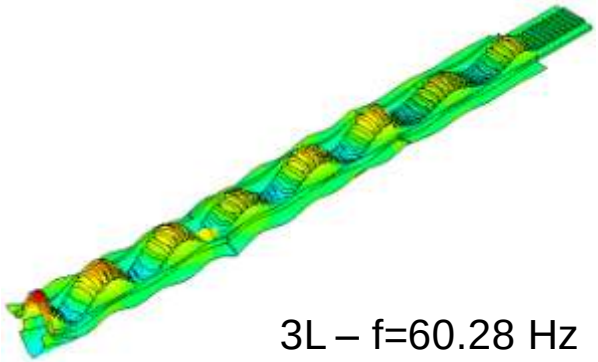
2G – f=6.78 Hz



2L – f=53.01 Hz



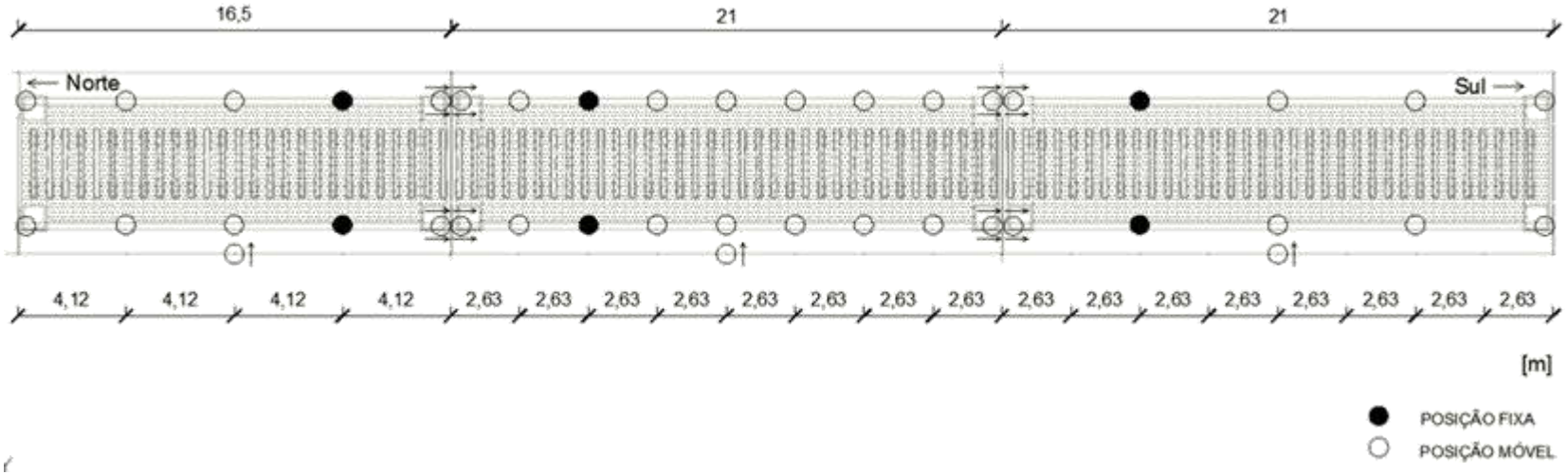
3G – f=9.79 Hz



3L – f=60.28 Hz

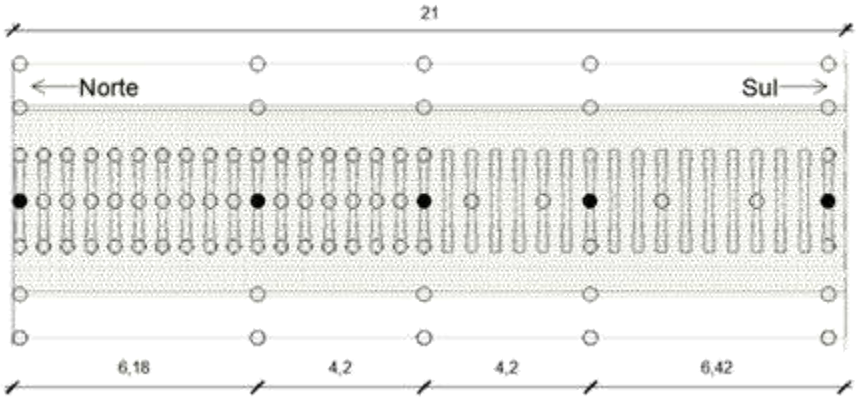
5. Dynamic analysis of a precast rail bridge

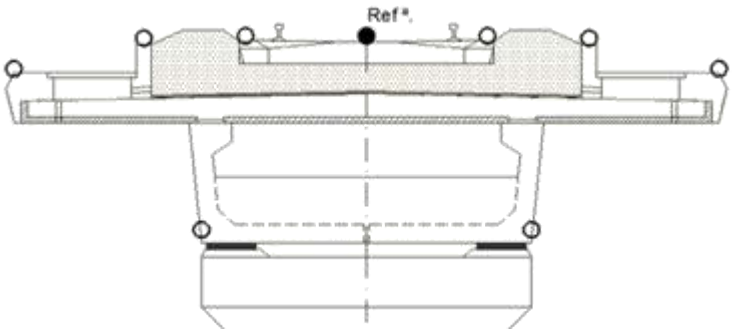
Ambient vibration test – identification of global modes



5. Dynamic analysis of a precast rail bridge

Ambient vibration test – identification of local modes

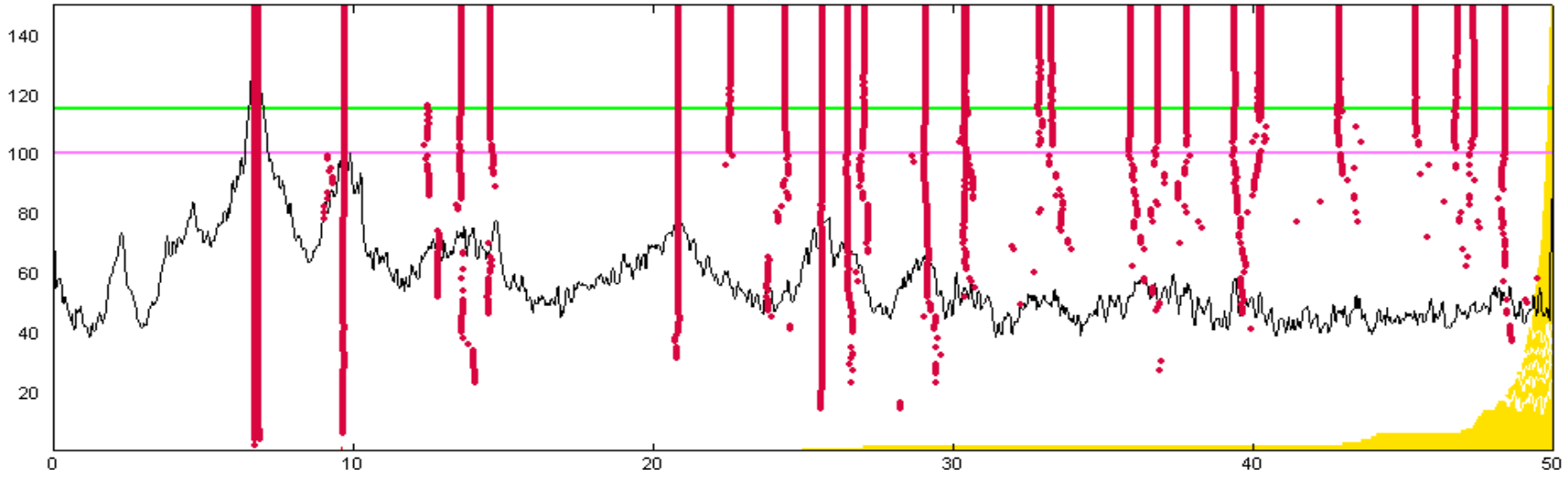




5. Dynamic analysis of a precast rail bridge

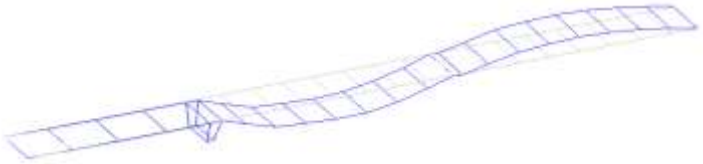
Ambient vibration test

Application of Stochastic Subspace Identification (SSI) method

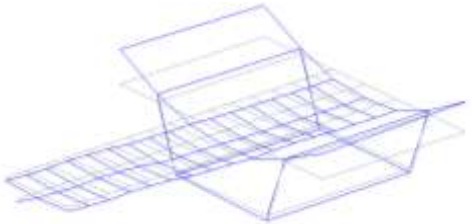


5. Dynamic analysis of a precast rail bridge

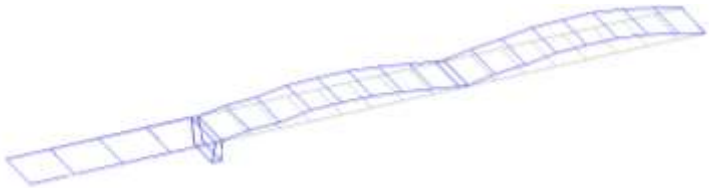
Experimental frequencies and mode shapes



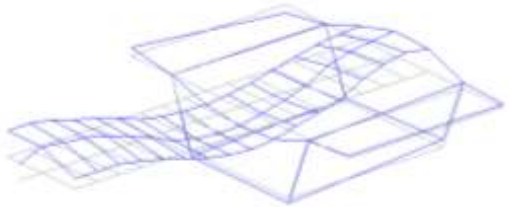
1G – f=6,76Hz ; ξ= 1,53%



1L – f=25,48Hz; ξ= 2,03%

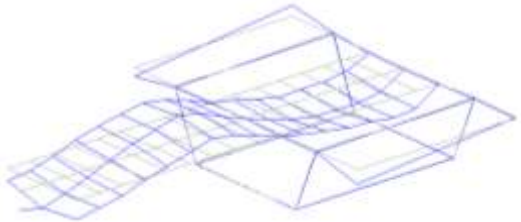


2G – f=6,95Hz; ξ= 3,72%



2L – f=53,18Hz; ξ= 2,27%

3G – f=9,65Hz; ξ= 2,21%



3L – f=60,18Hz; ξ= 3,16%

5. Dynamic analysis of a precast rail bridge

Numerical vs experimental results

Global modes

Mode	Experimental (Hz)	Numerical (Hz)	Error(%)
1G	6,76	6,73	0,44
2G	6,95	6,78	2,45
3 G	9,65	9,79	1,45

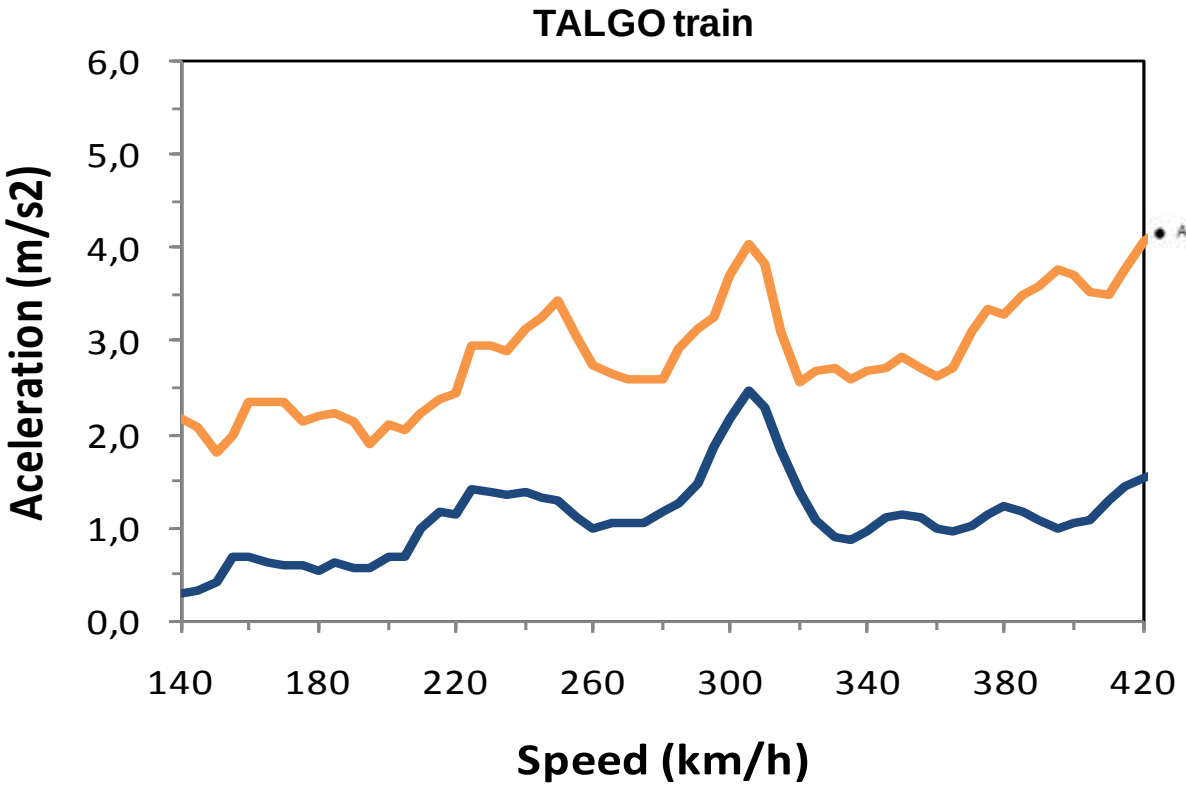
Local modes

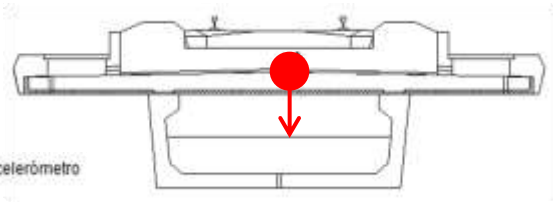
Mode	Experimental (Hz)	Numerical (Hz)	Error(%)
1L	25,48	26,68	4,71
2L	53,18	53,01	0,32
3L	60,18	60,28	0,17

5. Dynamic analysis of a precast rail bridge

Results

Influence of frequency range

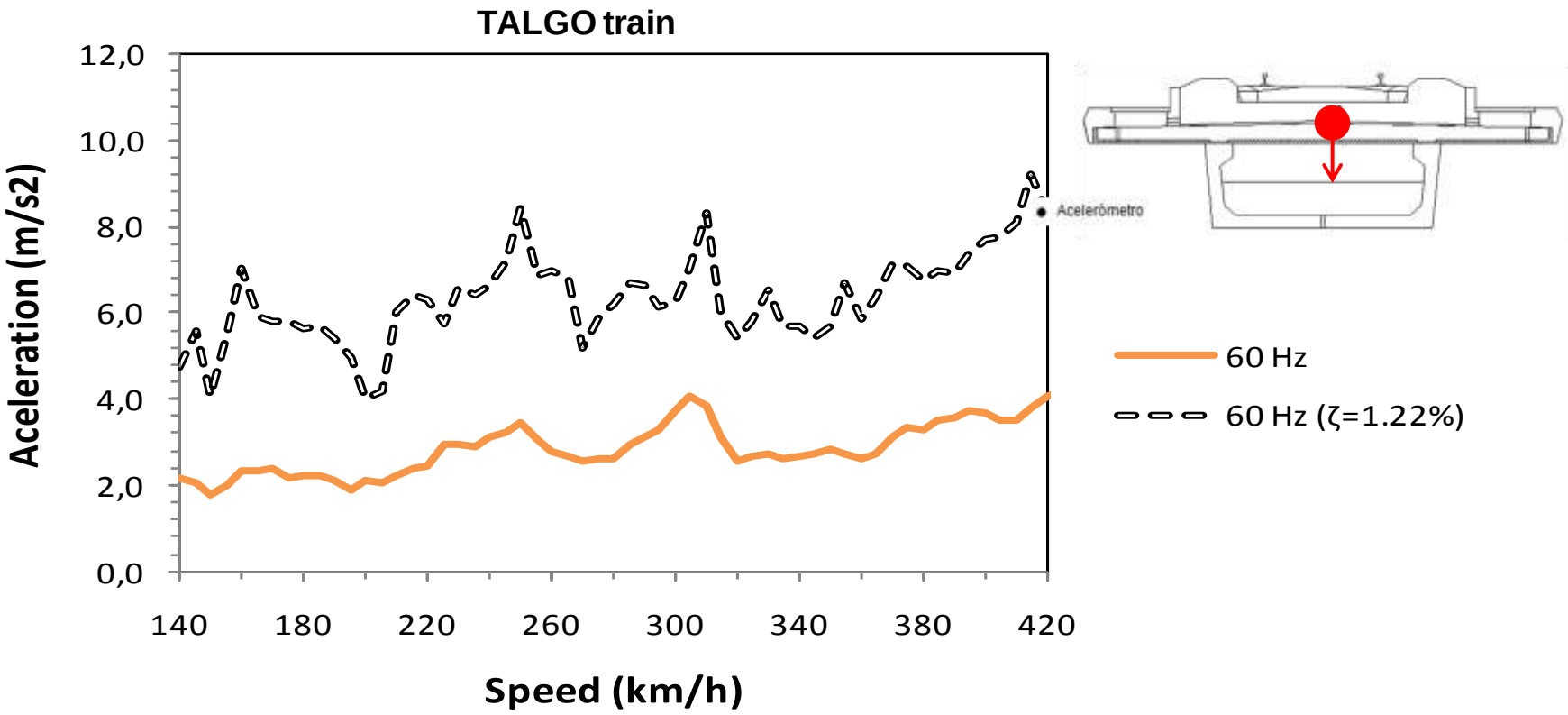




5. Dynamic analysis of a precast rail bridge

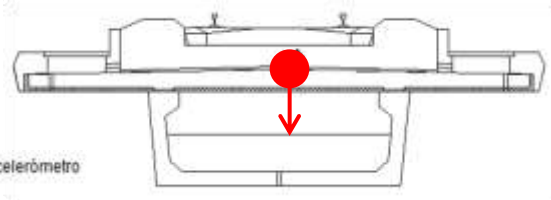
Results

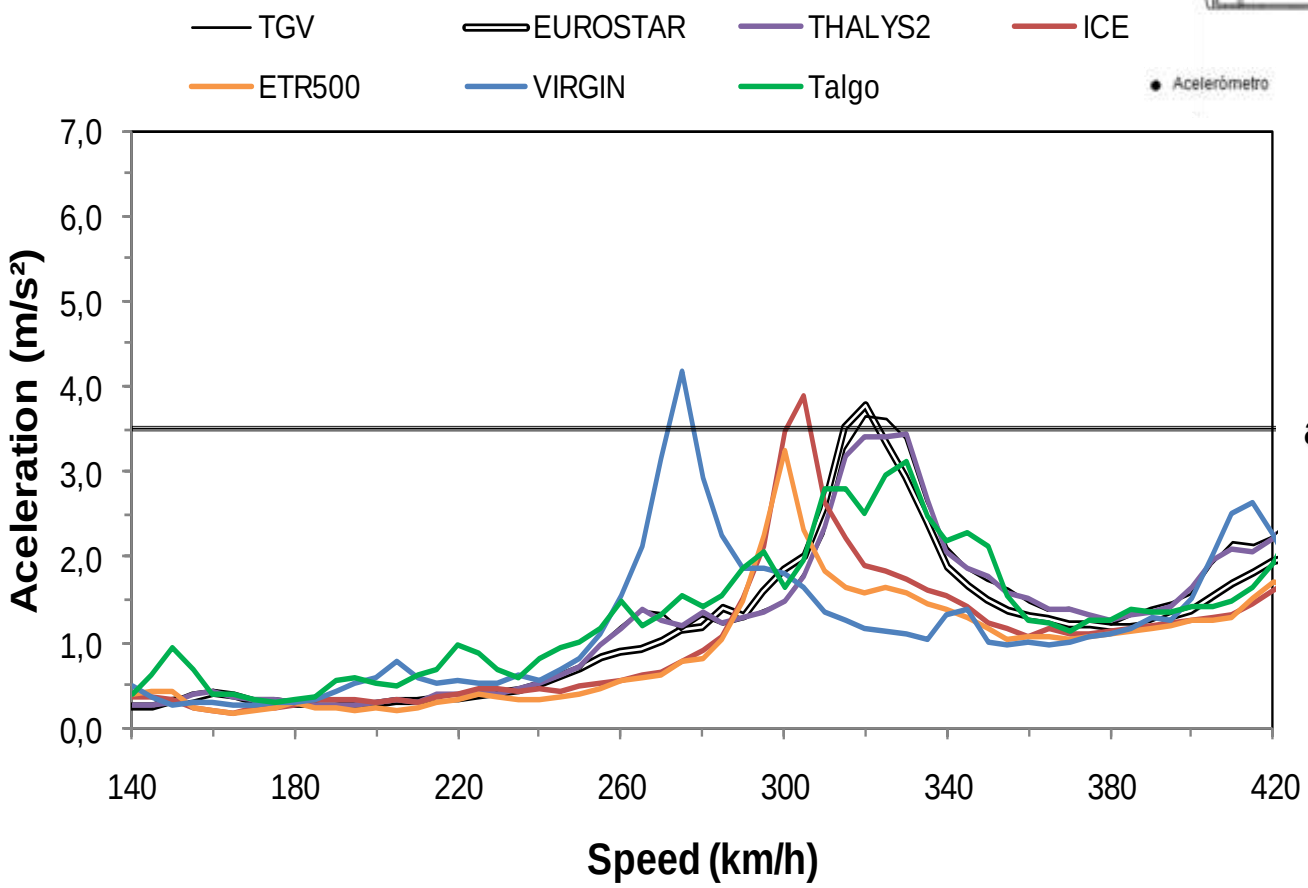
Influence of structural damping



5. Dynamic analysis of a precast rail bridge

Traffic safety





a_{max} = 3.5 m/s²

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6. Dynamic analysis of a short span filler beam bridge

6. Dynamic analysis of a short span filler beam bridge

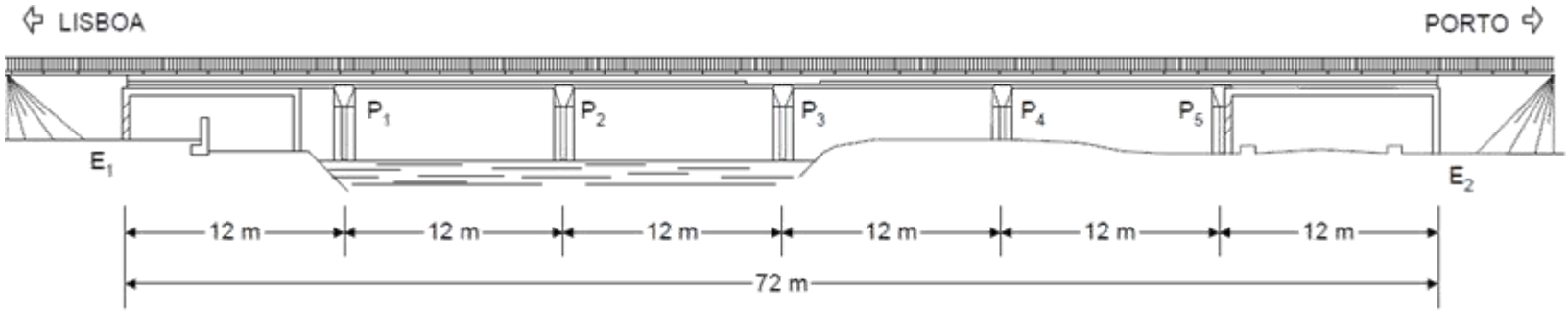
Canelas bridge

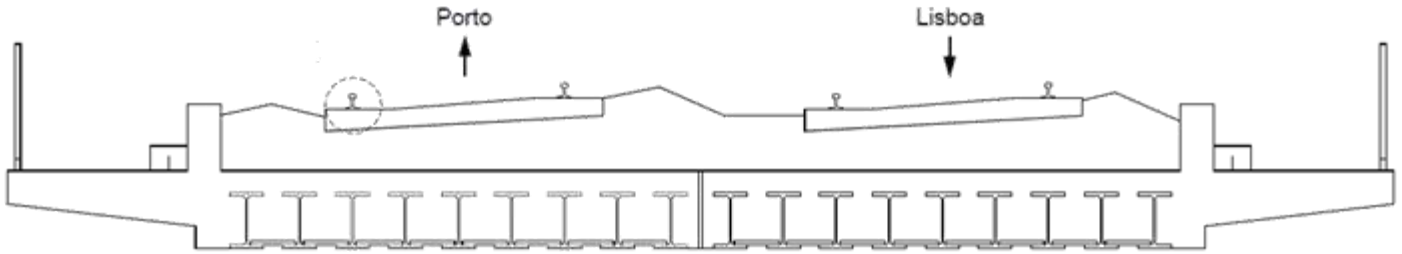
Location

6. Dynamic analysis of a short span filler beam bridge

Canelas bridge

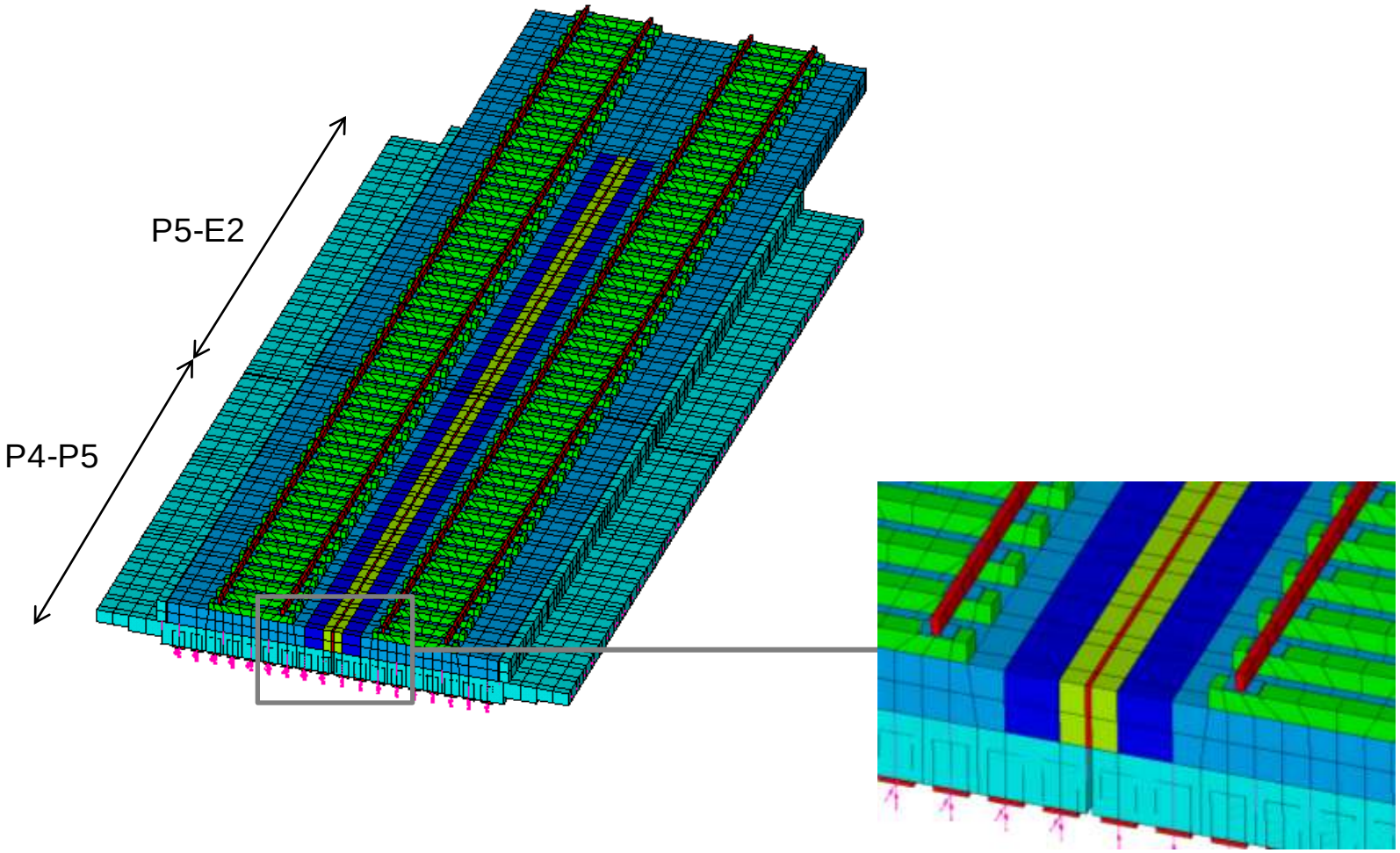
Description





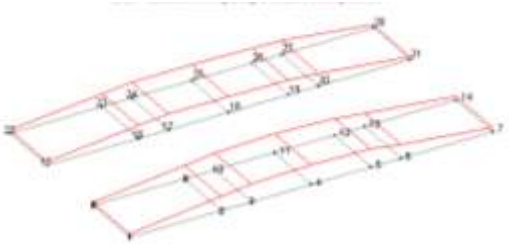
6. Dynamic analysis of a short span filler beam bridge

Numerical model

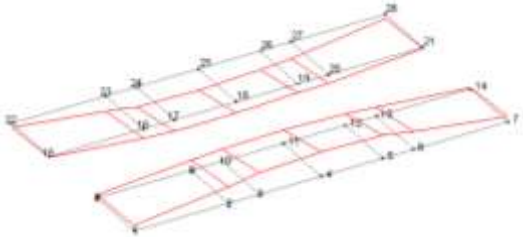


6. Dynamic analysis of a short span filler beam bridge

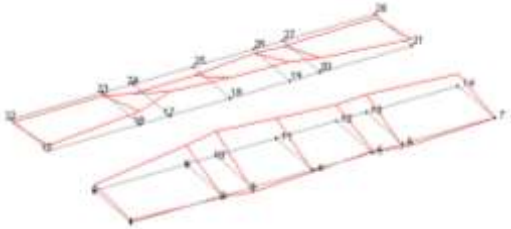
Experimental frequencies and mode shapes



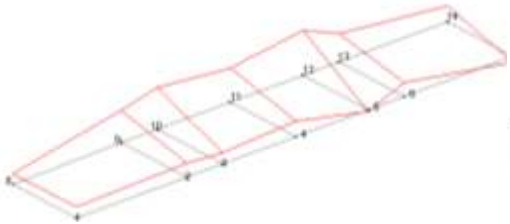
Mode 1 – f = 8,7 Hz



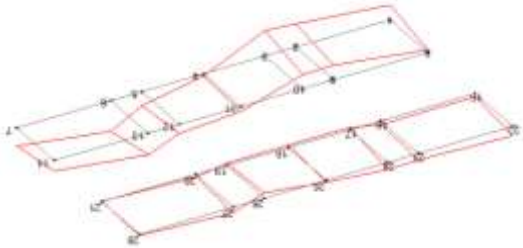
Mode 2 – f = 9,8 Hz



Mode 3 – f = 14,9 Hz



Mode 4 – f = 16,6 Hz



Mode 5 – f = 28,3 Hz

6. Dynamic analysis of a short span filler beam bridge

Model updating

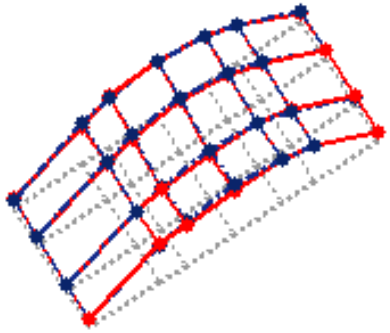
- Experimental vs numerical frequencies

Mode	Experimental (Hz)	Numerical Initial model (Hz)	Error (%)	Numerical Updated model (Hz)	Error (%)
1	8,7	9,43	8,38	9,07	4,23
2	9,8	11,01	12,39	9,78	- 0,21
3	14,9	16,62	11,57	14,89	- 0,04
4	16,6	30,37	82,97	16,60	0,00
5	28,3	27,38	- 3,26	27,25	- 3,73

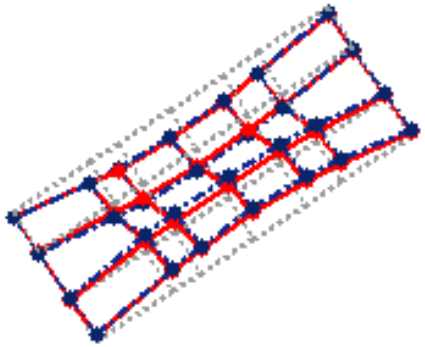
6. Dynamic analysis of a short span filler beam bridge

Model updating

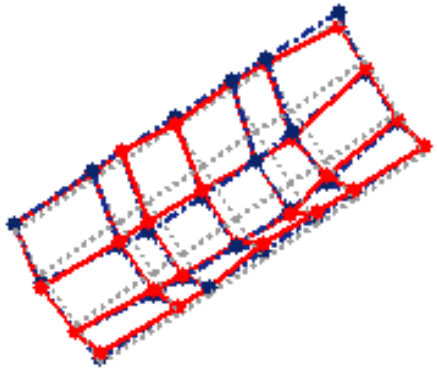
- Experimental vs numerical mode shapes



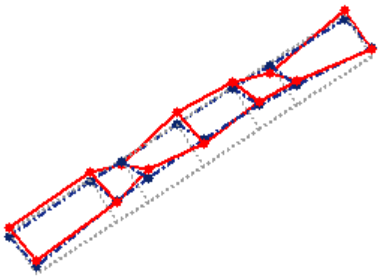
Mode 1



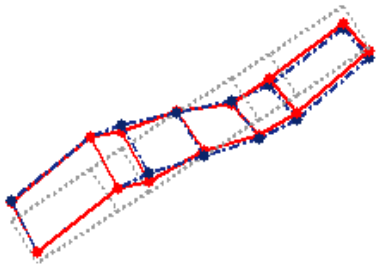
Mode 2



Mode 3



Mode 4



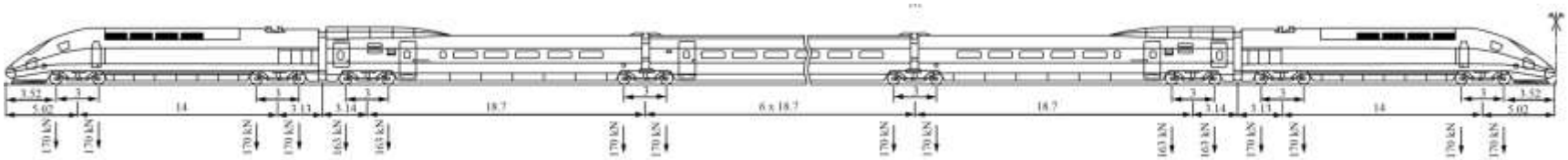
Mode 5

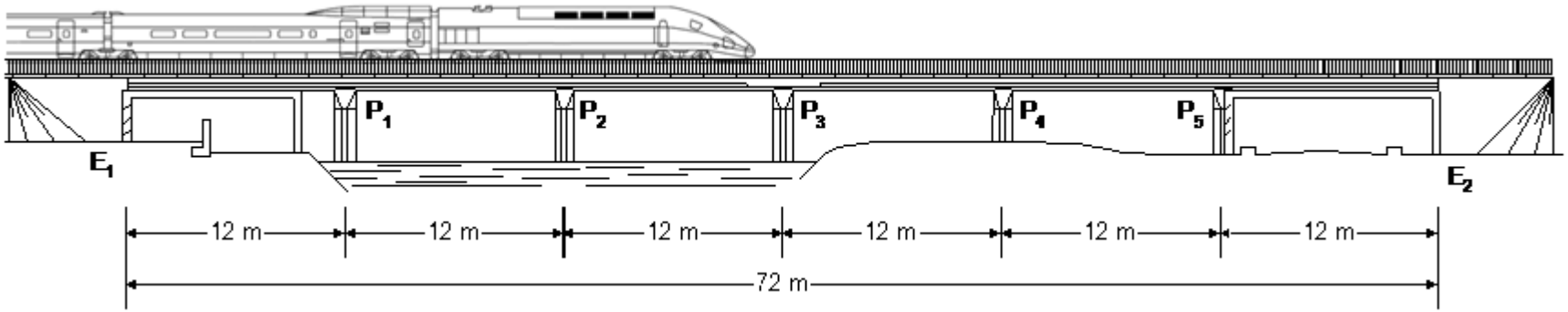
Numerical
Experimental

6. Dynamic analysis of a short span filler beam bridge

Simulation scenario

- TGV train

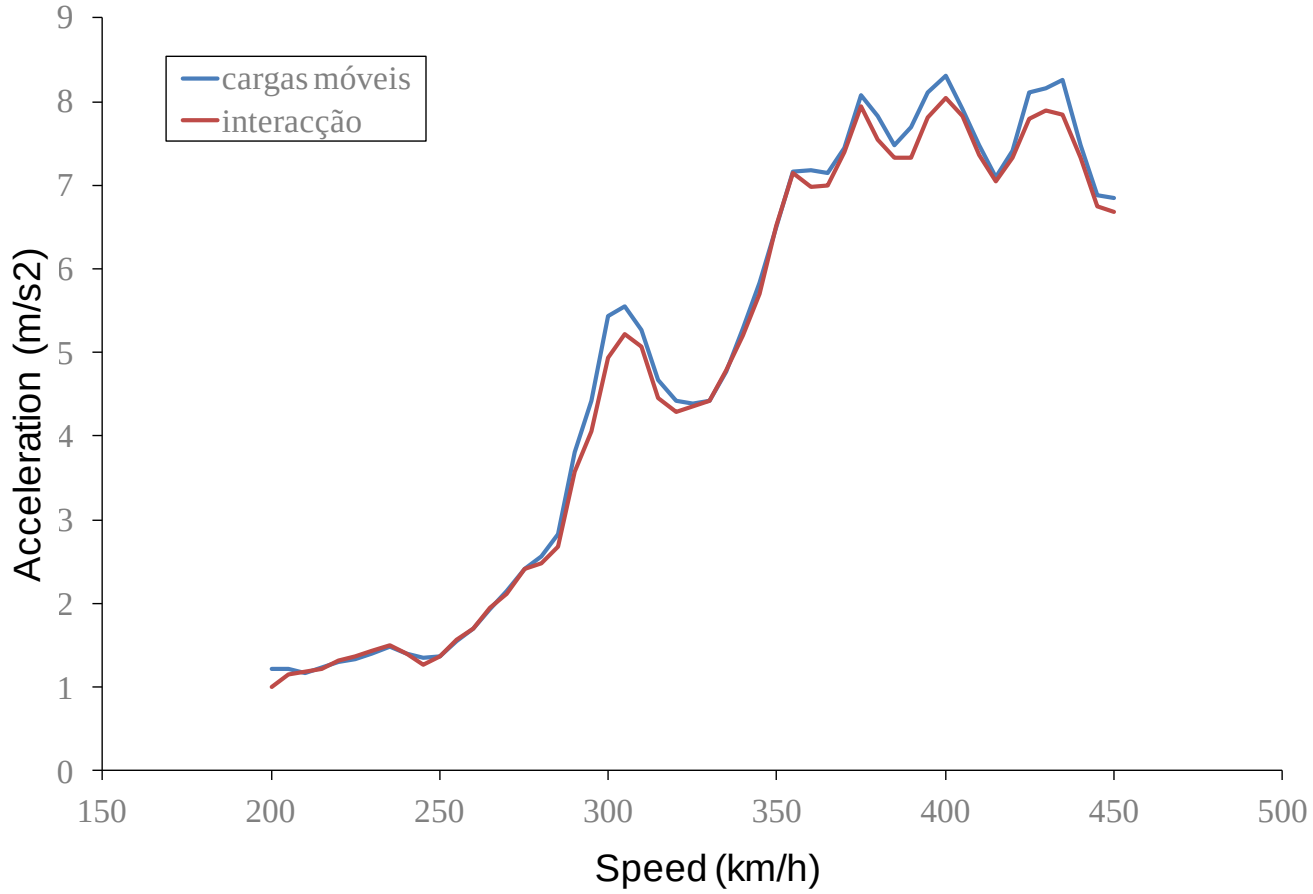




6. Dynamic analysis of a short span filler beam bridge

Deterministic response

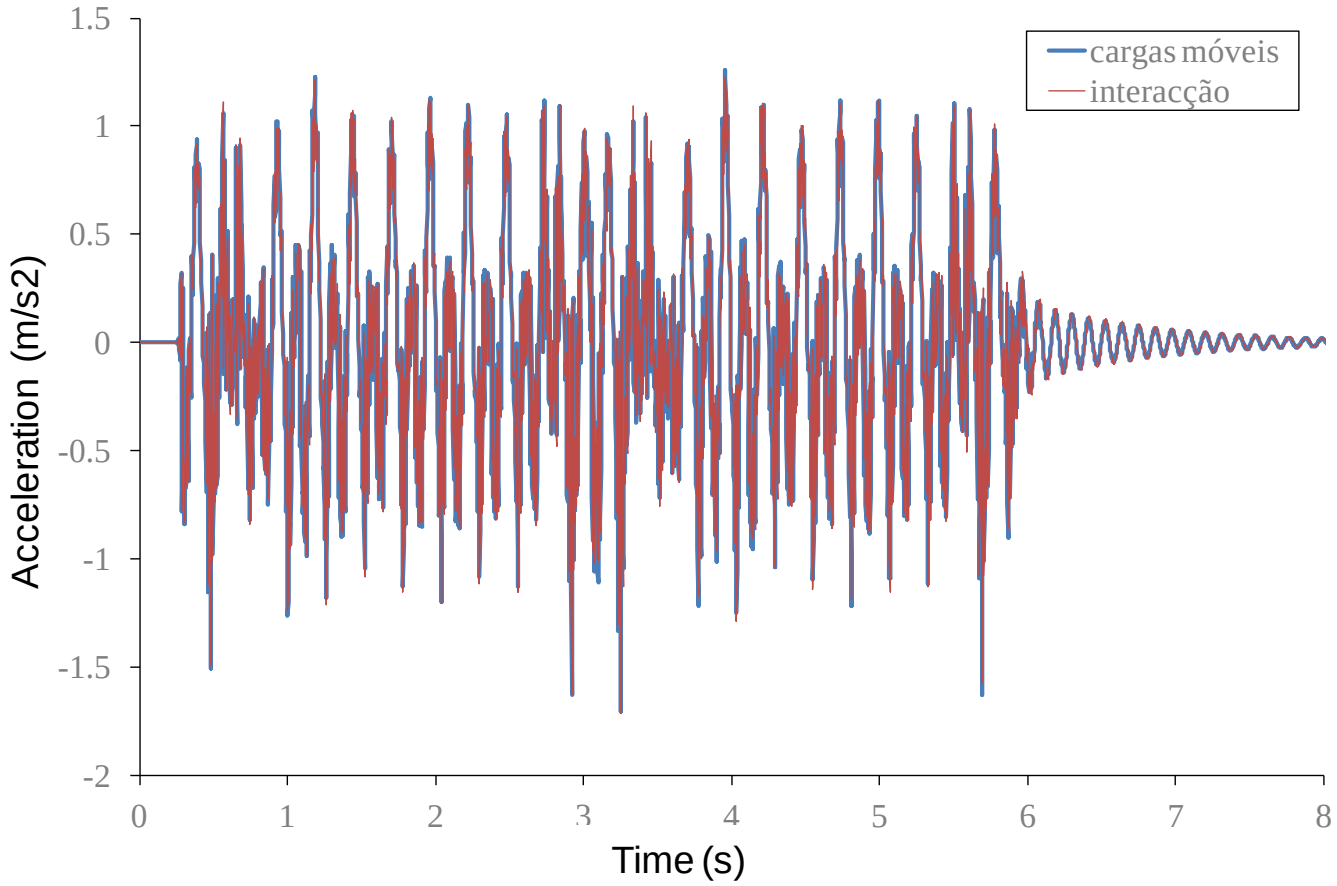
- TGV train



6. Dynamic analysis of a short span filler beam bridge

Deterministic response

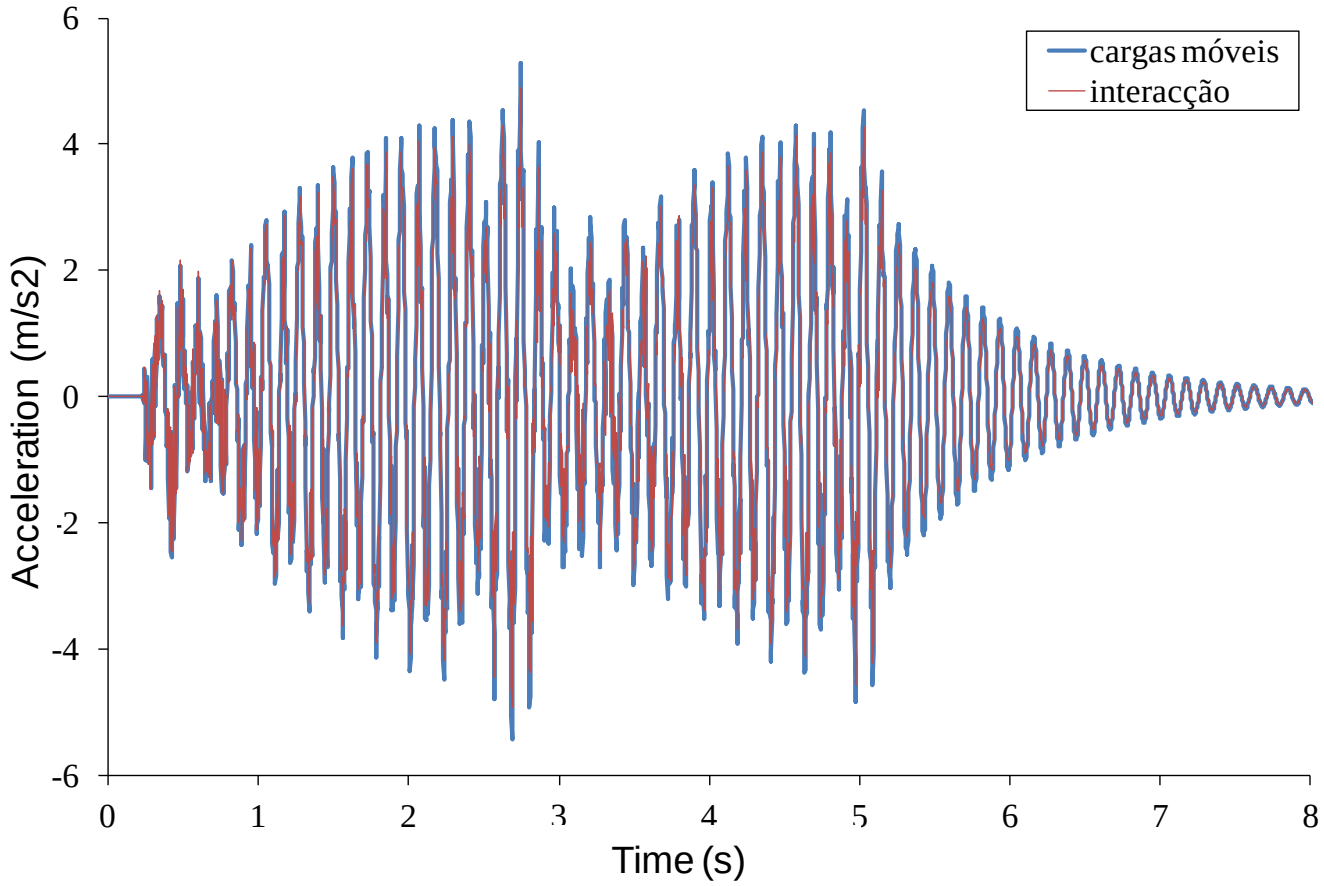
- TGV train v = 250 km/h



6. Dynamic analysis of a short span filler beam bridge

Deterministic response

- TGV train v = 300 km/h



6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

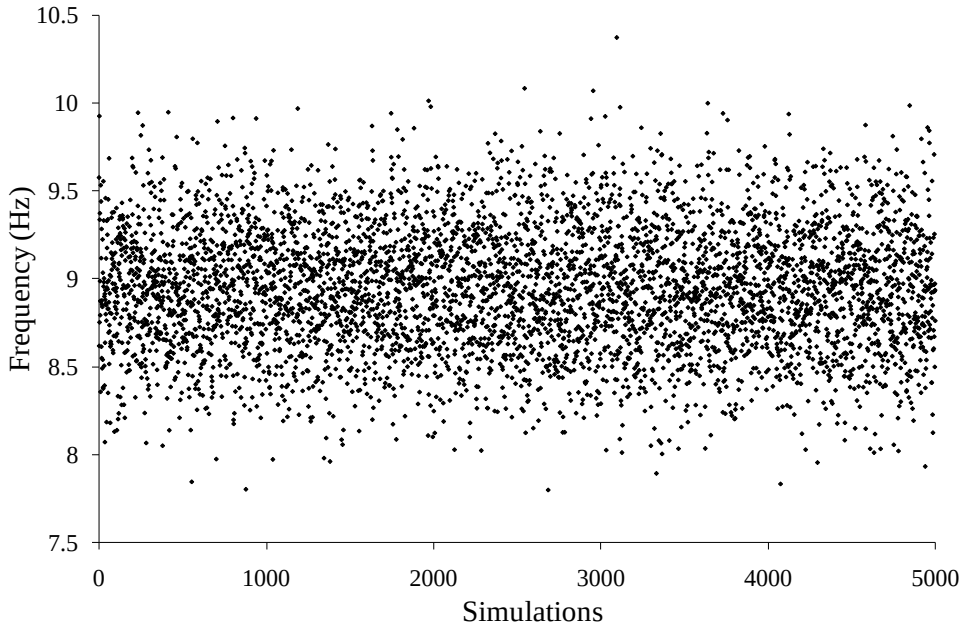
- Random variables

Variable [simulation]	Distribution	Mean (gaussian) or Min. (uniform)	Std. Deviation (gaussian) or Max. (uniform)
Concrete density	Gaussian	2,5 t/m^3	0,1 (CV = 4%)
Ballast density	Uniform	17 kN/m^3	21 kN/m^3
Ballast area	Uniform	1,48659 m^2	2,76081 m^2
HEB 500 area	Gaussian	Nominal area	0,04 x nominal area
Elasticity modulus concrete	Gaussian	36,1 GPa	2,888 (CV = 8%)
Concrete height	Gaussian	Nominal value	10 mm
Concrete width	Gaussian	Nominal value	5 mm
Distortion modulus neoprene	Uniform	0,75 MPa	1,18 MPa

6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

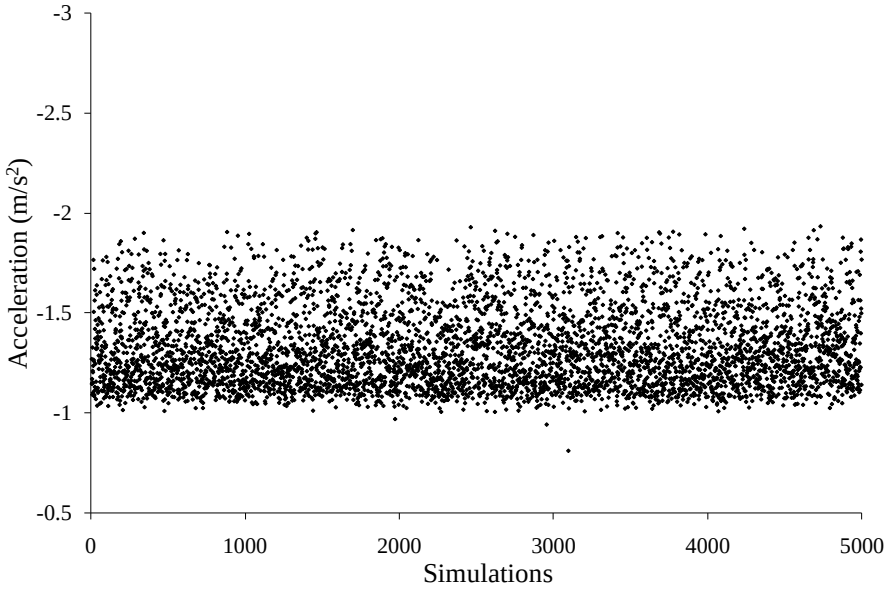
- First natural frequency for the simulated bridges



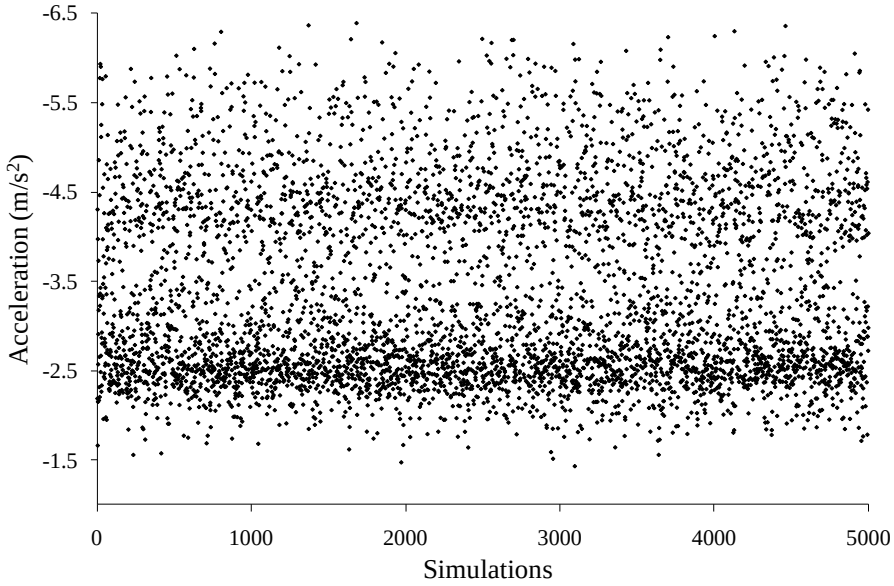
6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- Maximum deck acceleration for the simulated bridges



v = 250 km/h

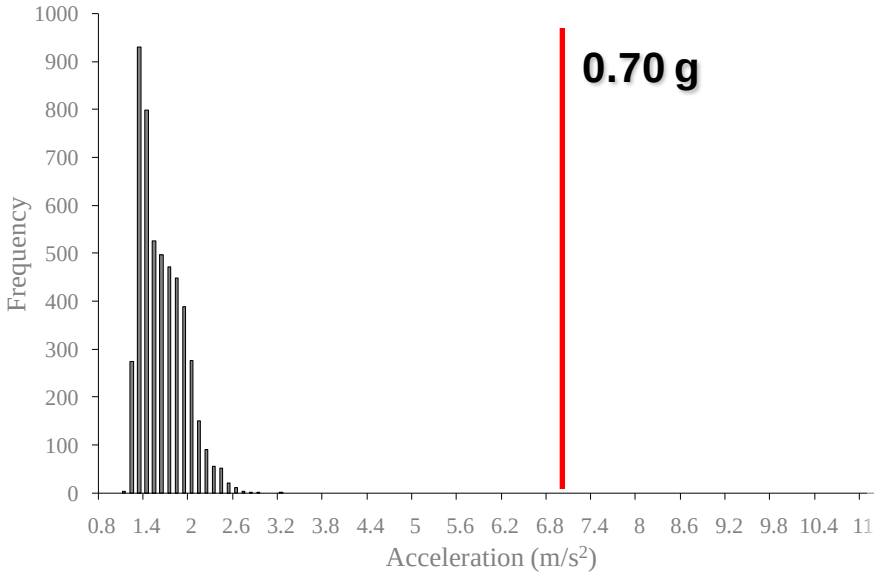


v = 285 km/h

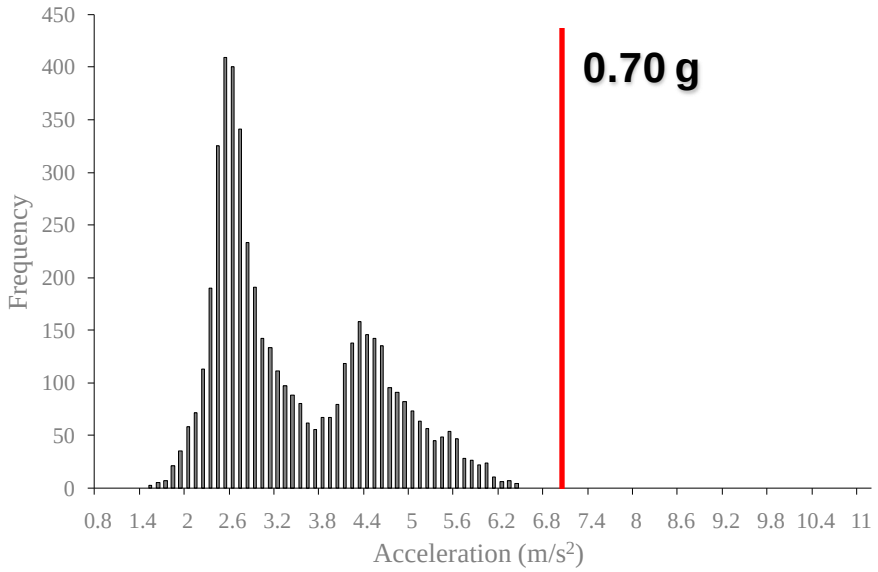
6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- Maximum deck acceleration for the simulated bridges



v = 250 km/h



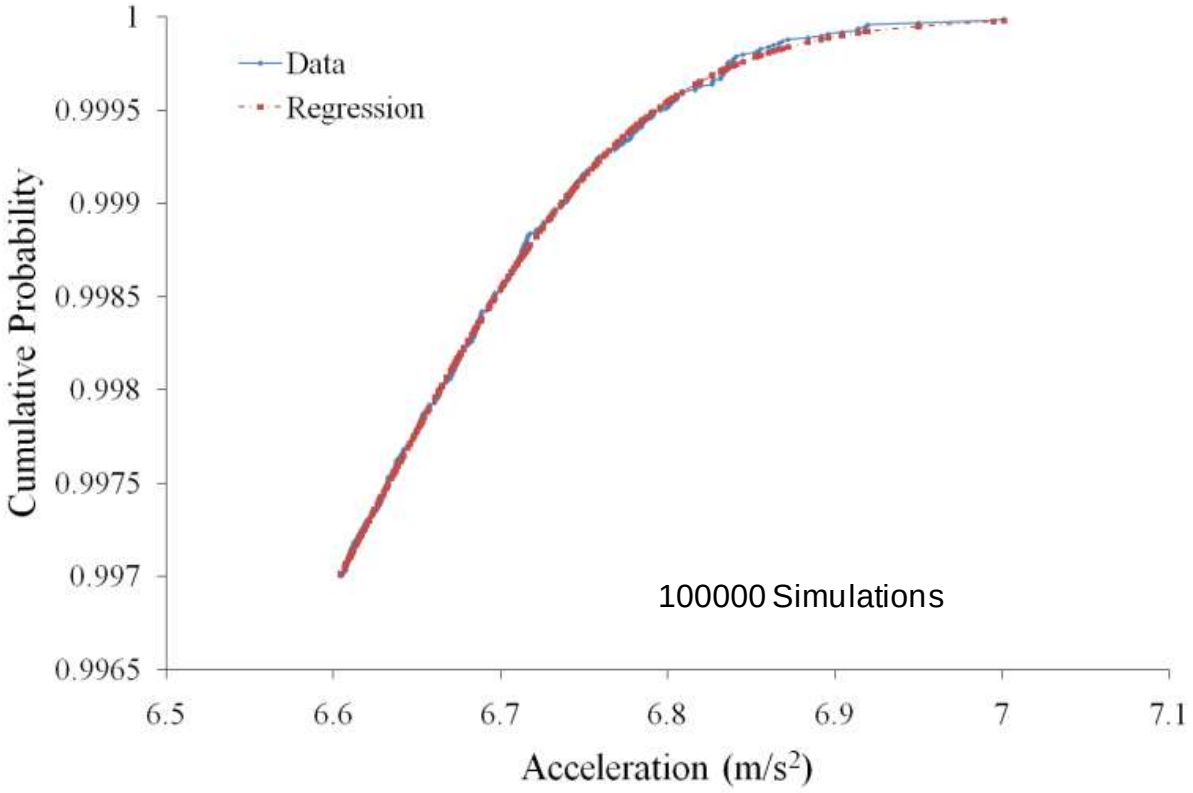
v = 285 km/h

6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- Cumulative probability

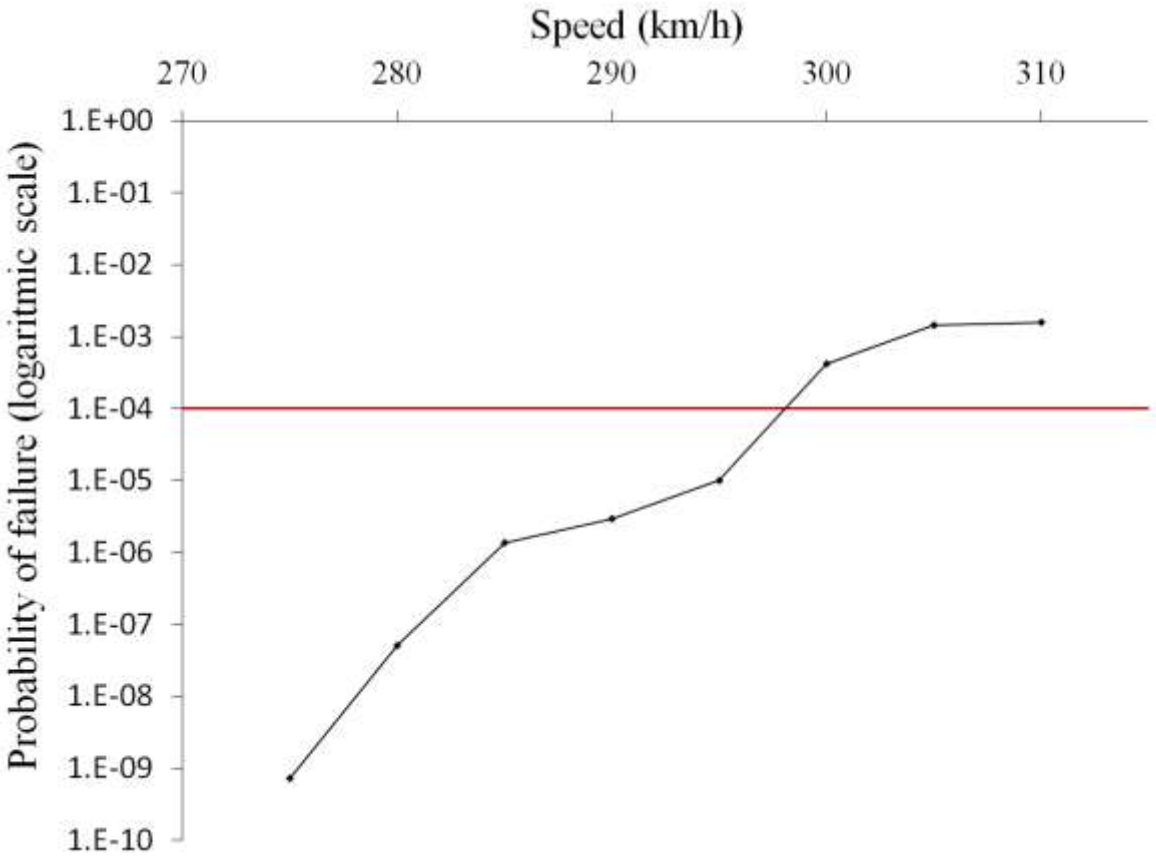
v = 285 km/h



6. Dynamic analysis of a short span filler beam bridge

Stochastic simulation of the dynamic response

- Probability of failure

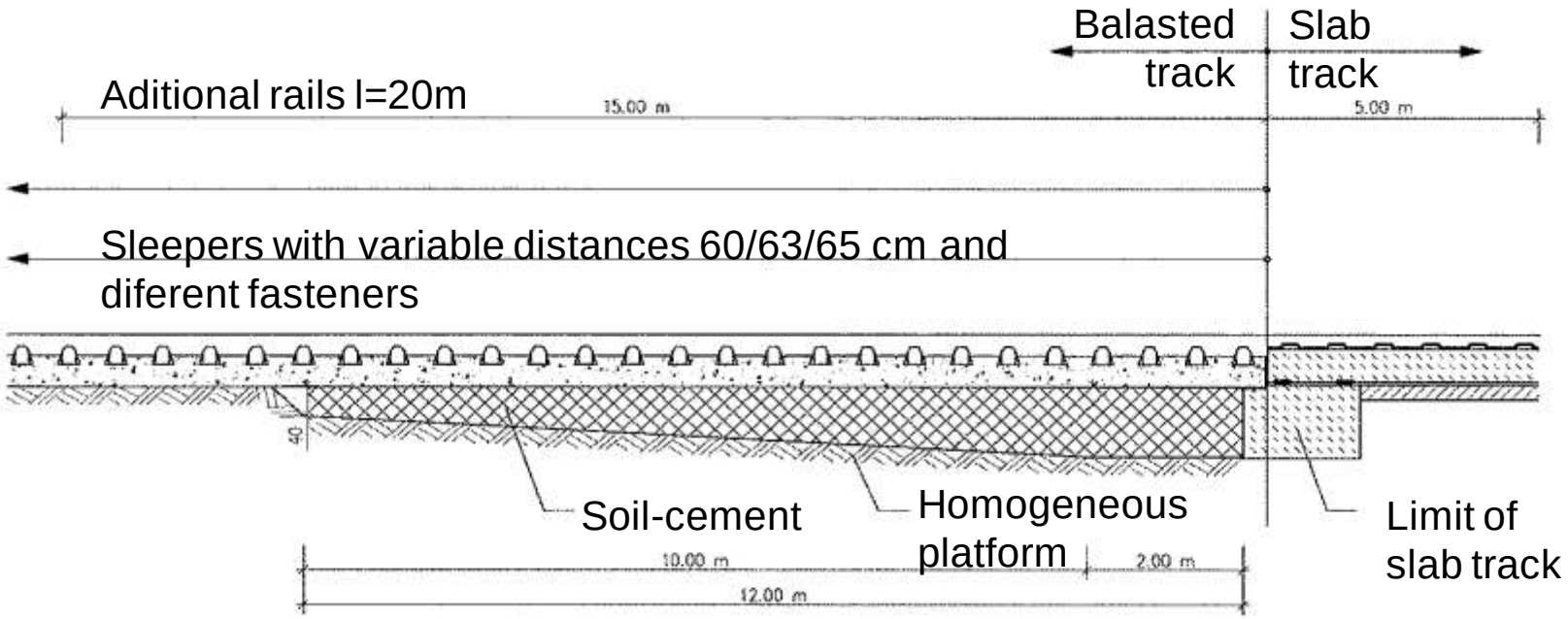


7. Dynamic effects of the train passage on transition zones

7. Dynamic effects on transition zones

Introduction

- Balasted track to slab track transition



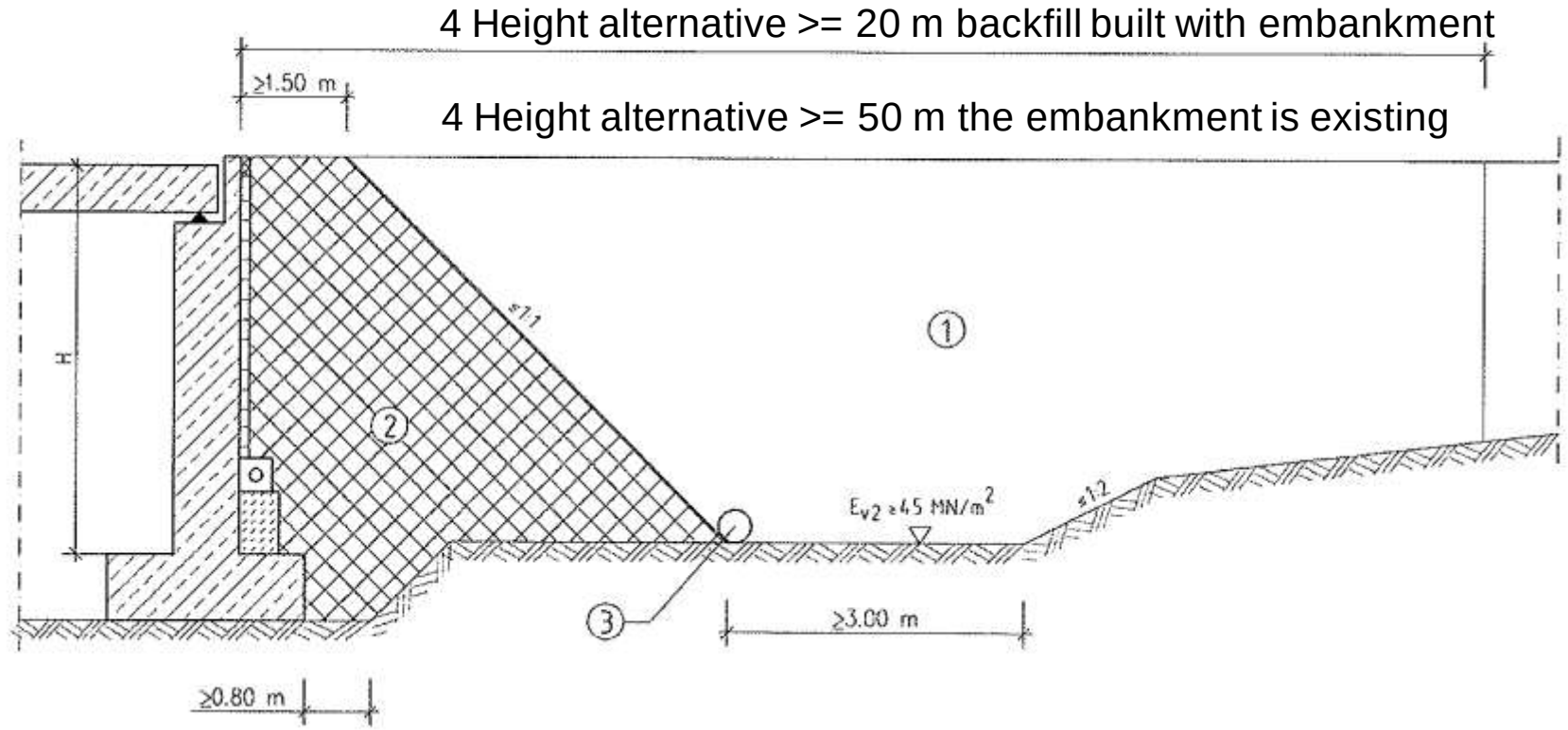
7. Dynamic effects on transition zones

Introduction

- Embankment to bridge transition

Bridge

Embankment



7. Dynamic effects on transition zones

Introduction

- Maintenance interventions (Madrid - Seville high speed line)

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

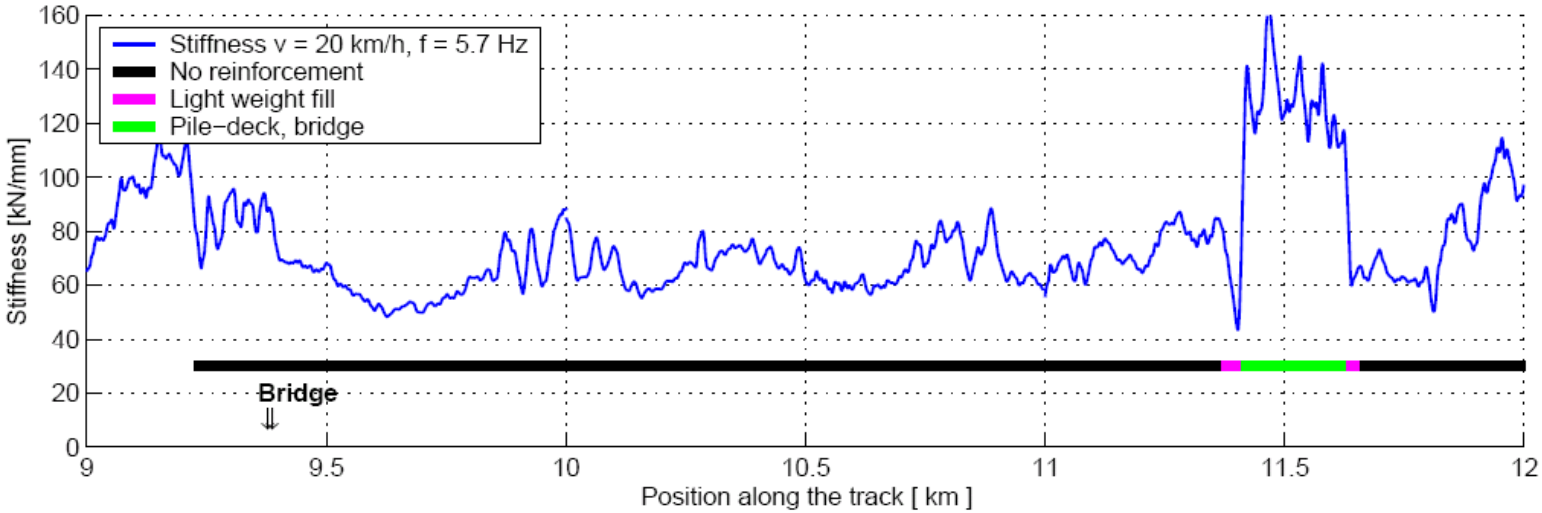
in Pita, A. Lopez - *Deterioration of track geometric quality on high speed lines: the experience of the Madrid - Sevilha line*, Workshop Track for high-speed railways, Portugal, 2006

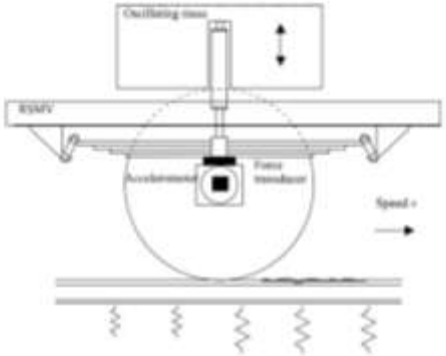
(*) número de vezes por quilómetro de via que a aceleração vertical do eixo do comboio AVE ultrapassou 30m/s^2.

7. Dynamic effects on transition zones

Introduction

- Vertical track stiffness variation





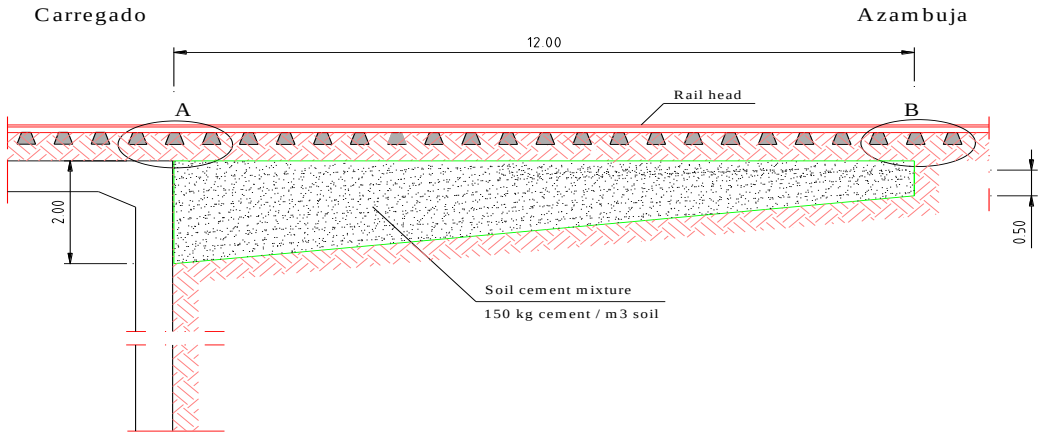
7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Location and description

Linha do Norte

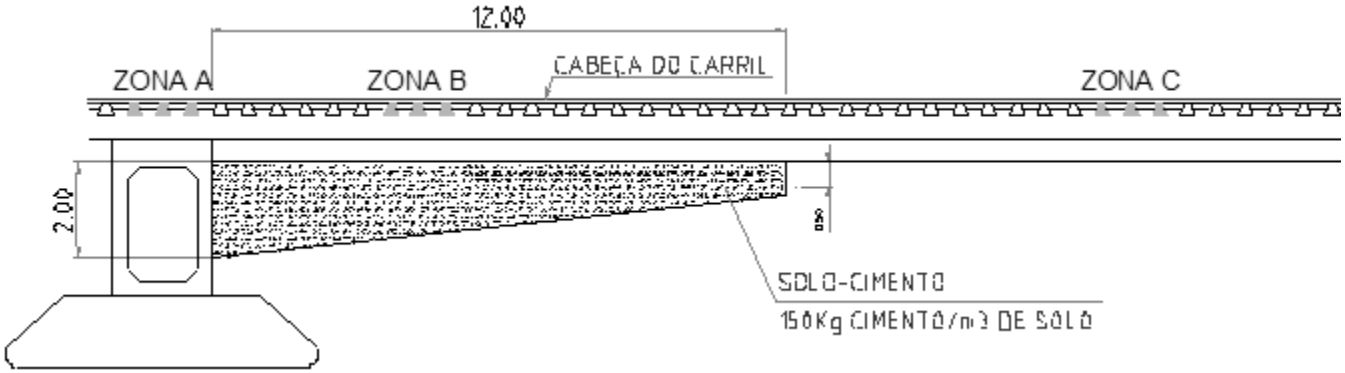
km 40+250

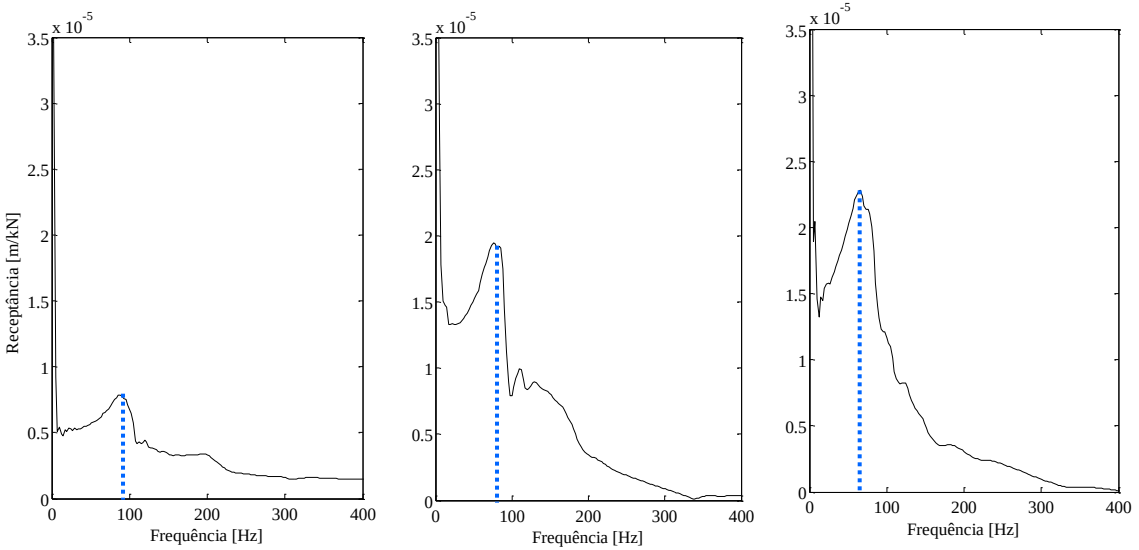


7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Track receptance tests

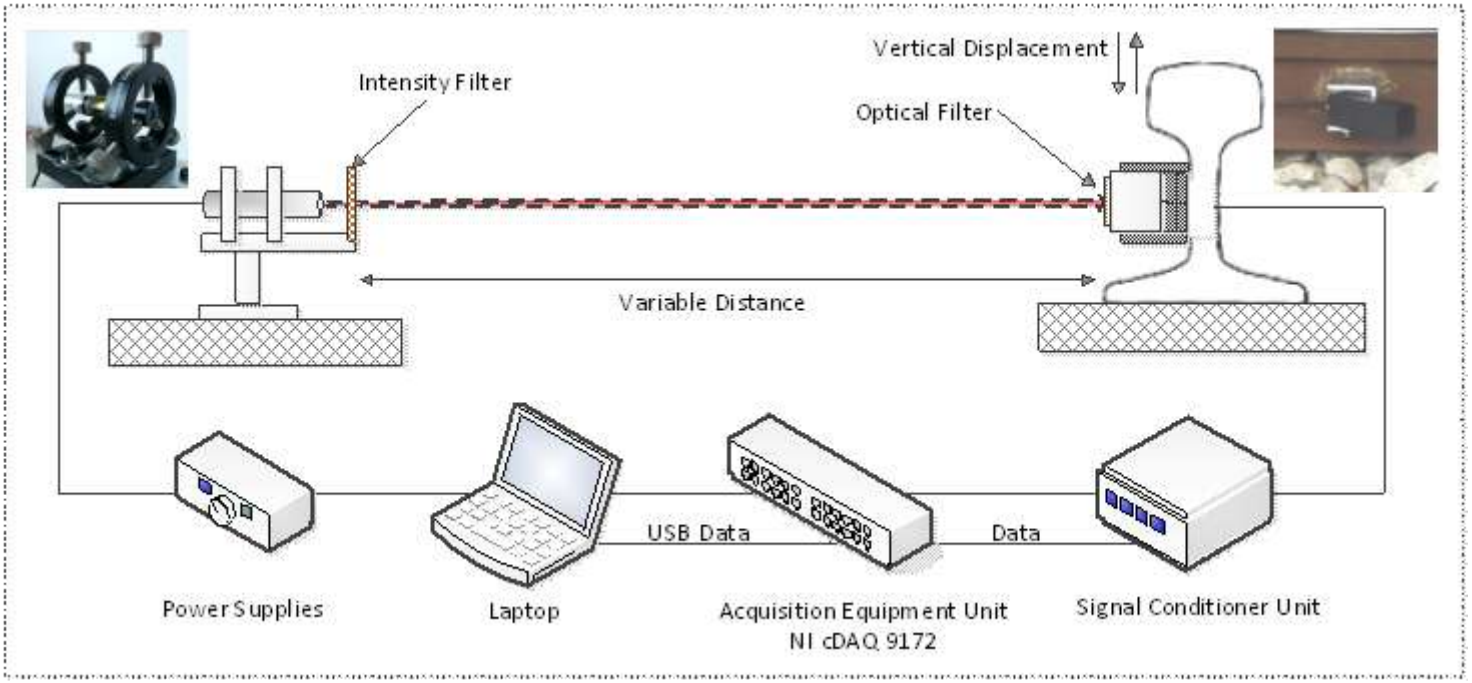


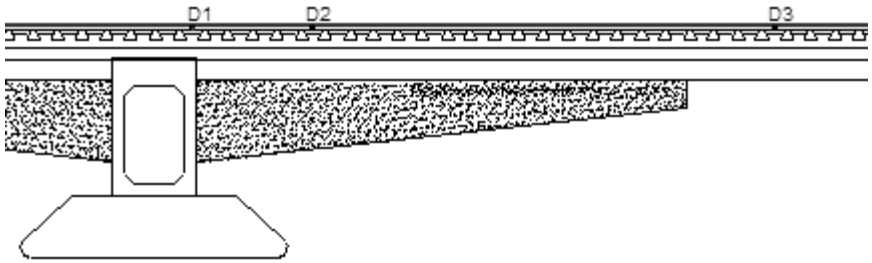


7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Dynamic monitoring of track displacement

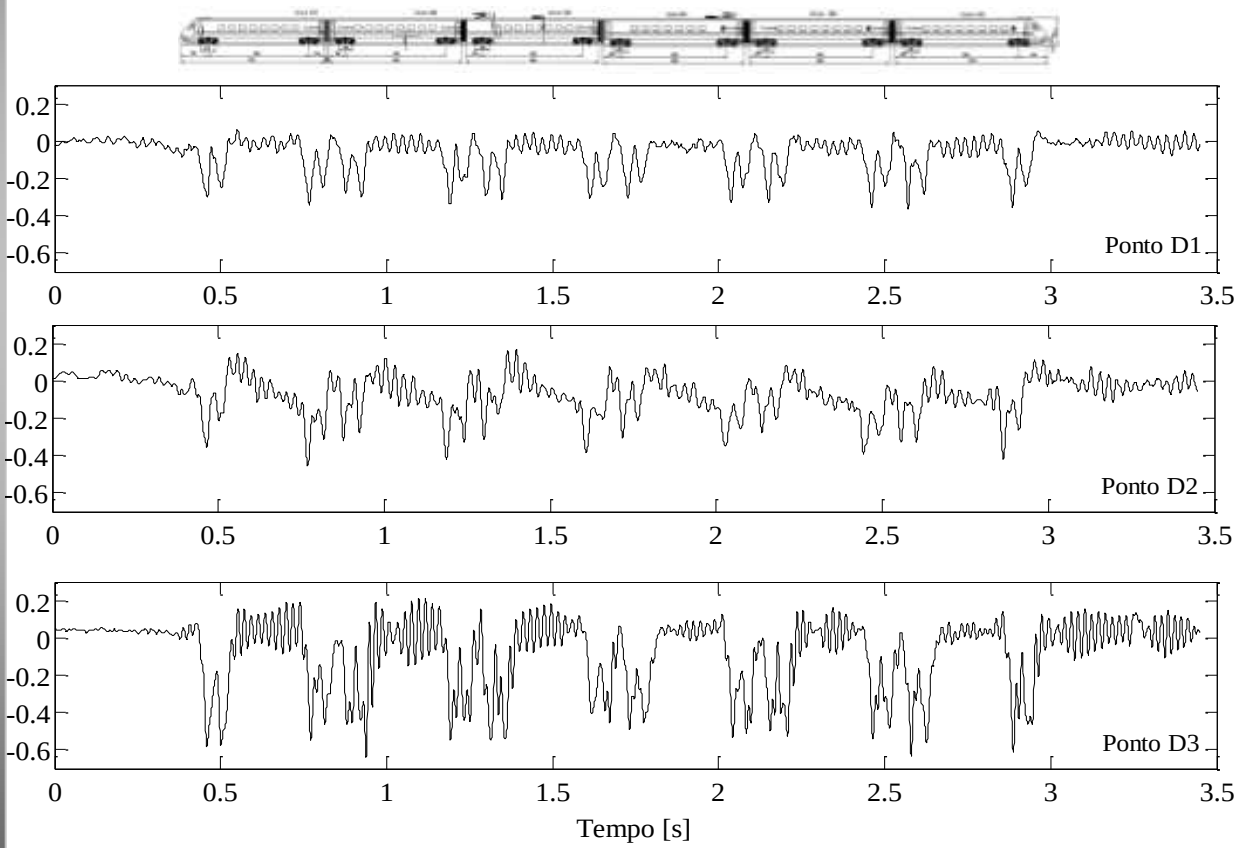




7. Dynamic effects on transition zones

Hydraulic underpass PH126A

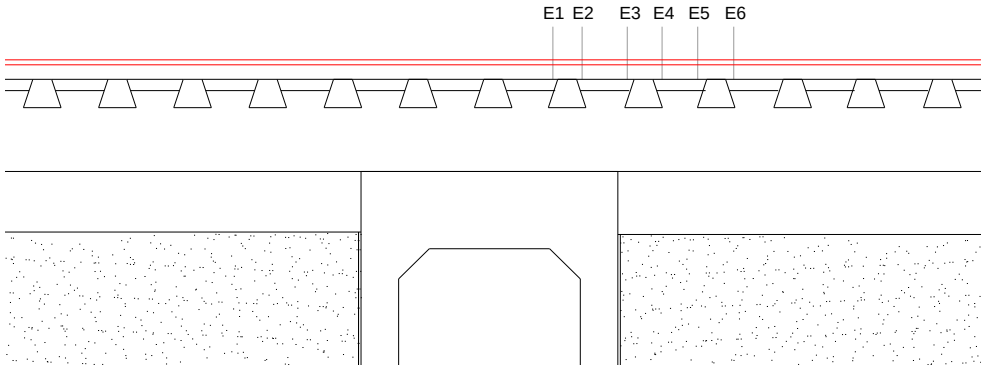
- Dynamic monitoring of track displacement

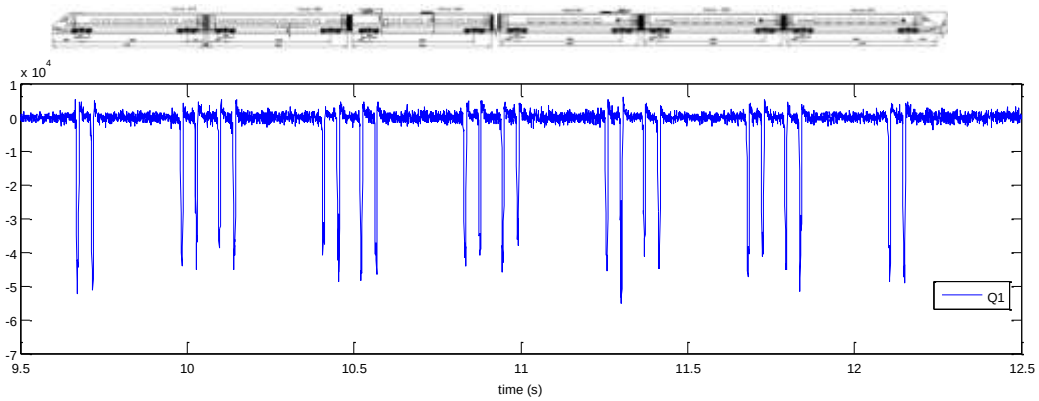


7. Dynamic effects on transition zones

Hydraulic underpass PH126A

- Dynamic monitoring of axle loads

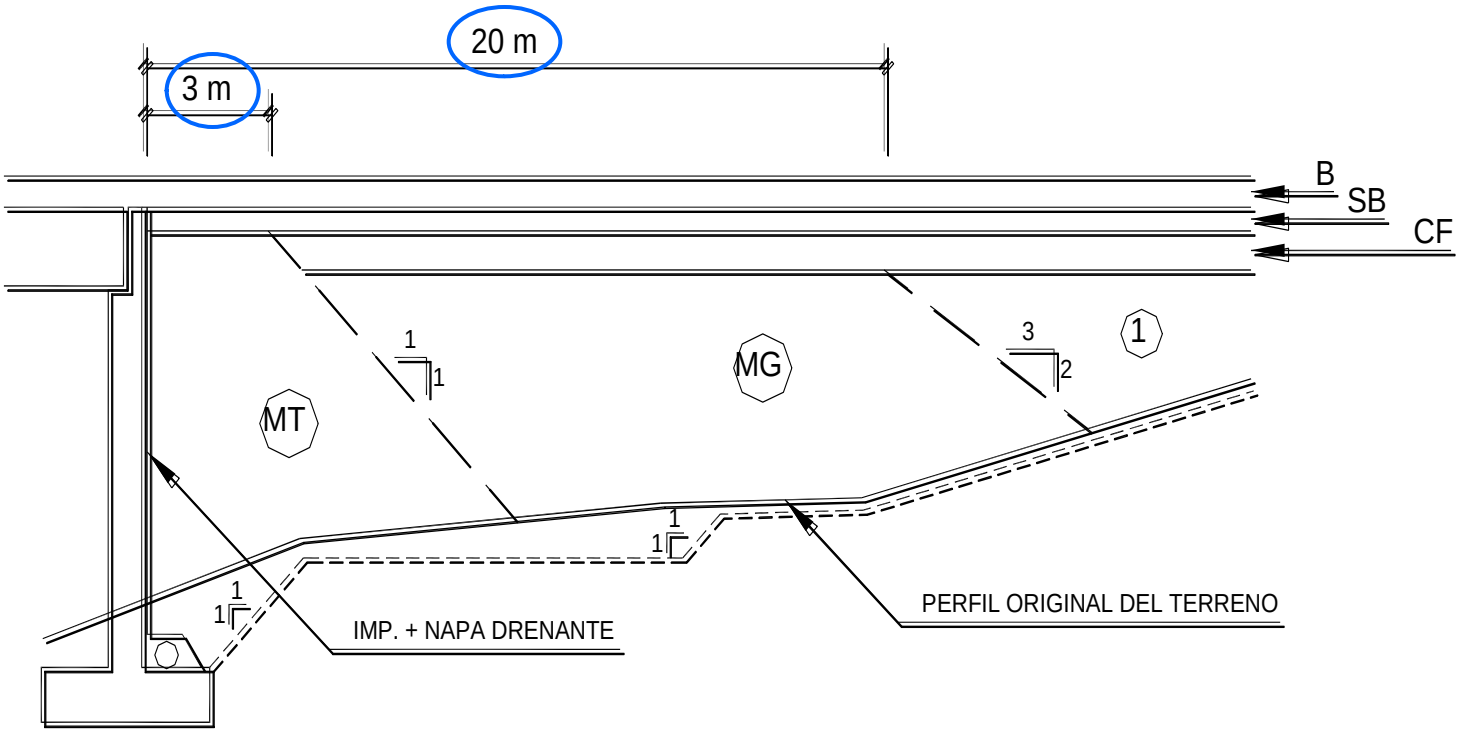




7. Dynamic effects on transition zones

Case studies

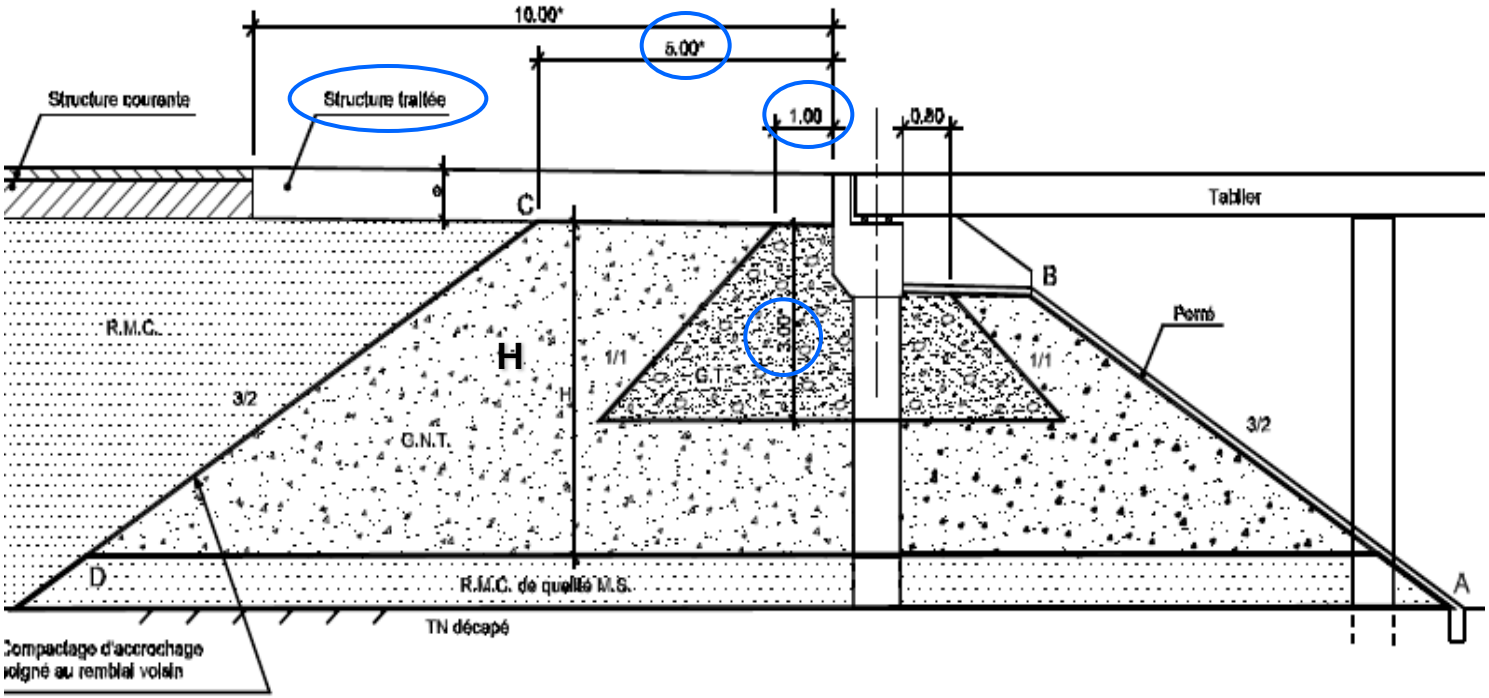
- ADIF solution (Spain)



7. Dynamic effects on transition zones

Case studies

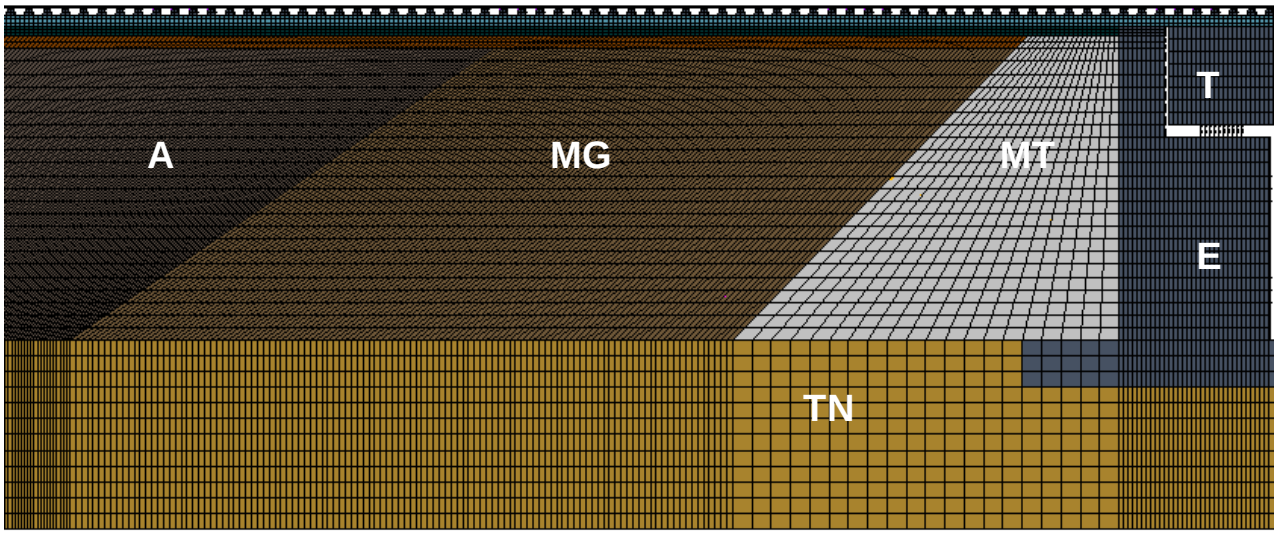
- SNCF solution (France)



7. Dynamic effects on transition zones

Numerical modelling

- ADIF solution (Spain)

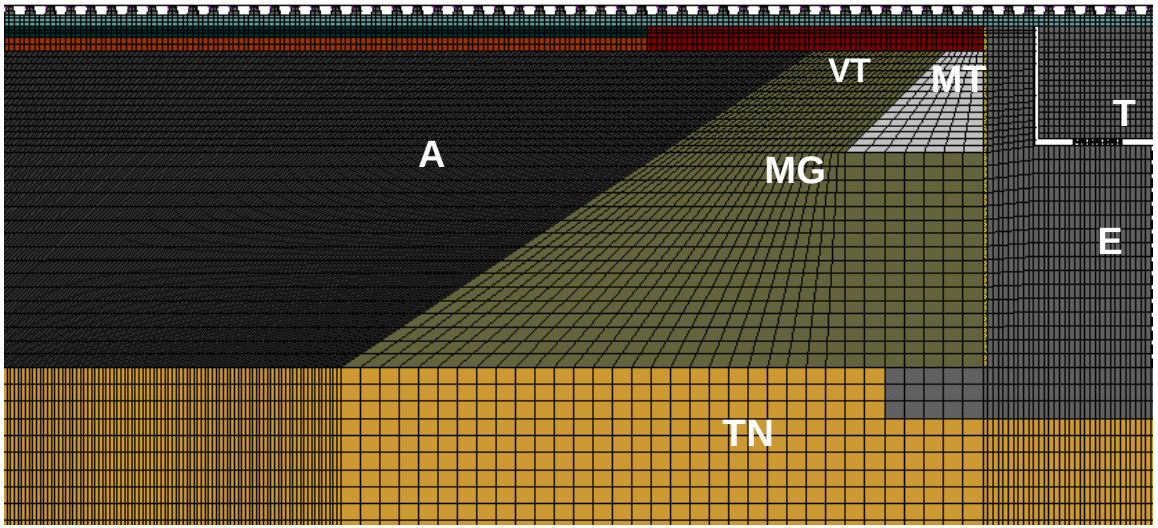


A = Enbankment soil
MG = Gravel
MT = Gravel-cement
TN = Natural ground
T = Deck
E = Abutment

7. Dynamic effects on transition zones

Numerical modelling

- SNCF solution (France)

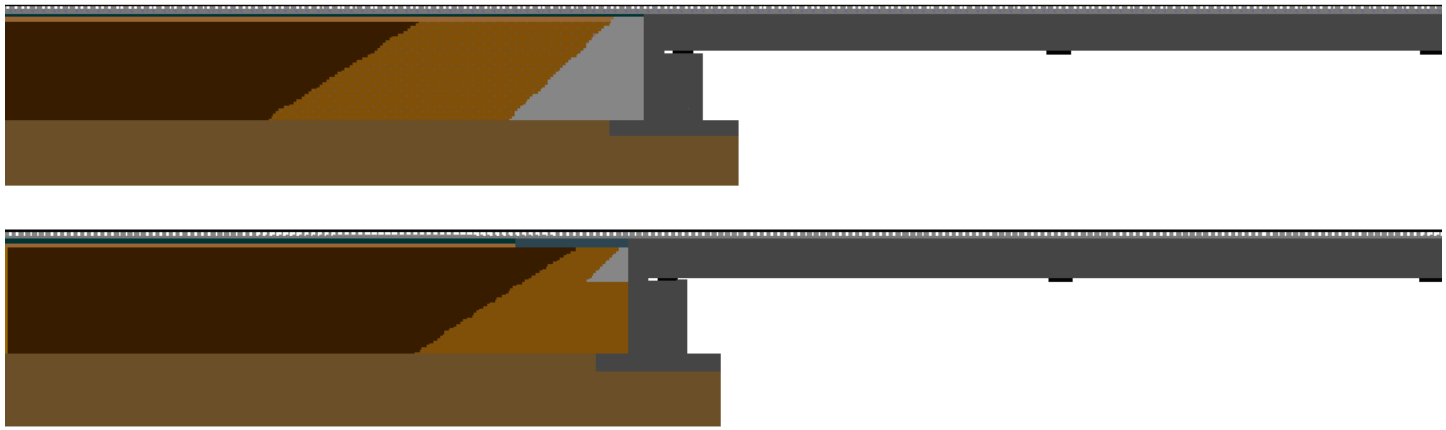


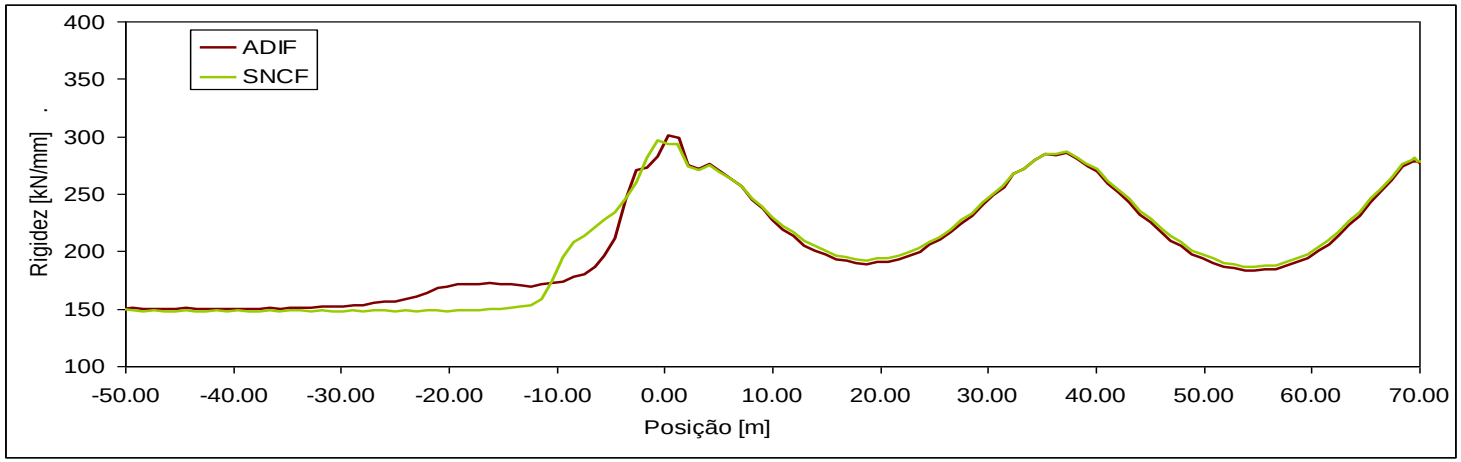
A = Enbankment soil
MG = Gravel
MT = Gravel-cement
TN = Natural ground
T = Deck
E = Abutment
VT = Soil-cement under the track

7. Dynamic effects on transition zones

Numerical modelling

- Comparison of vertical track stiffness

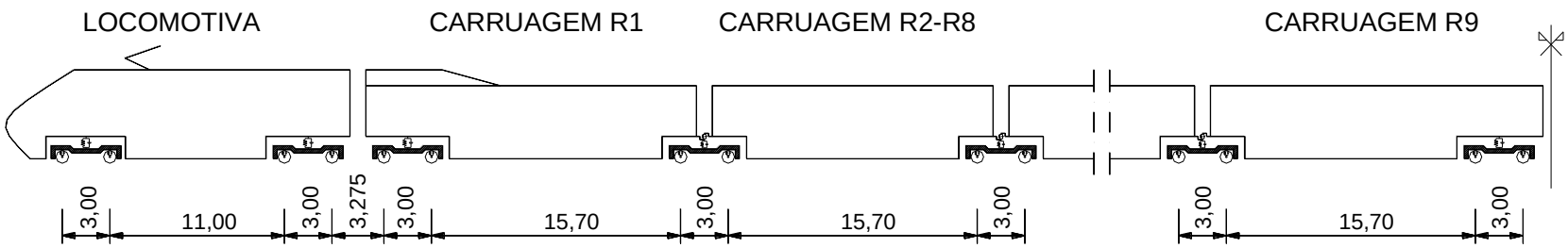


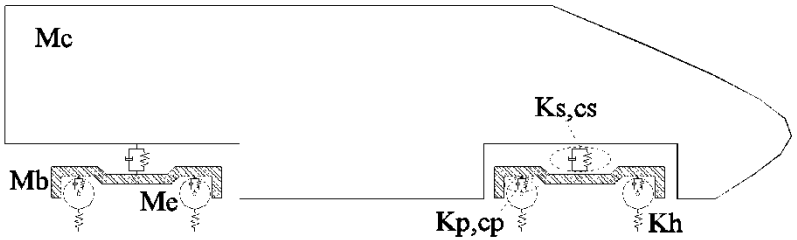


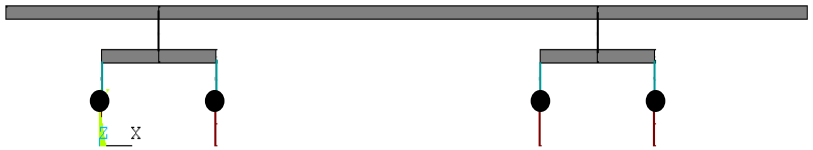
7. Dynamic effects on transition zones

Numerical modelling

- Eurostar train



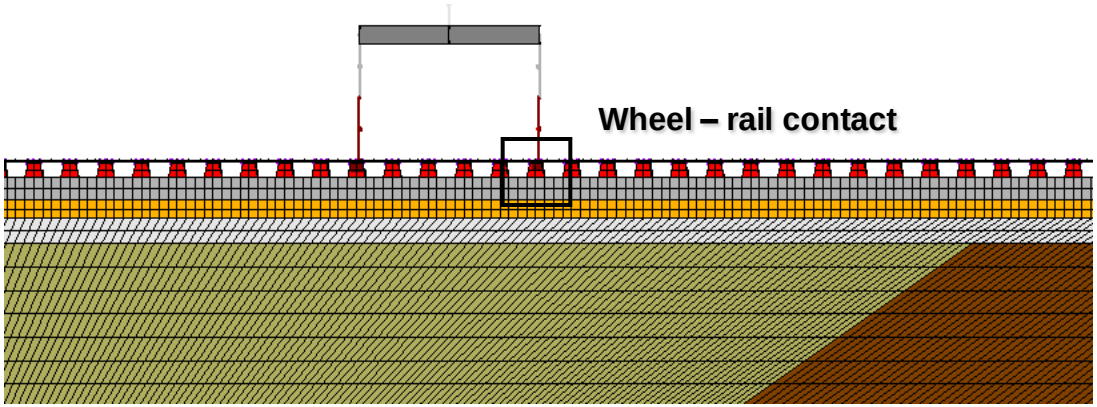


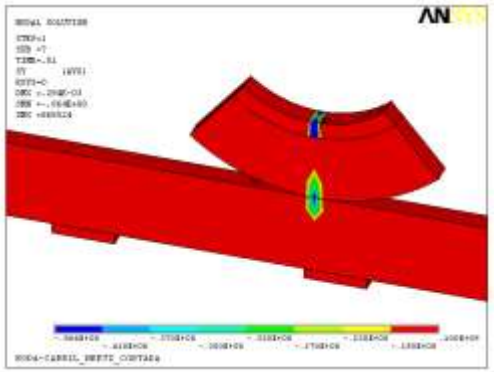


7. Dynamic effects on transition zones

Numerical modelling

- Train-track interaction



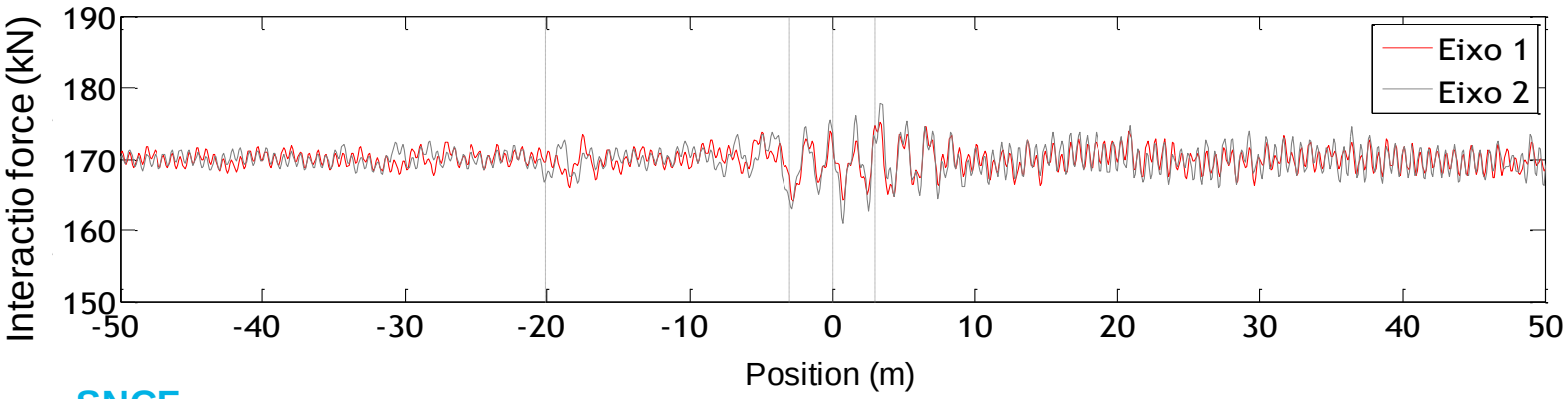


7. Dynamic effects on transition zones

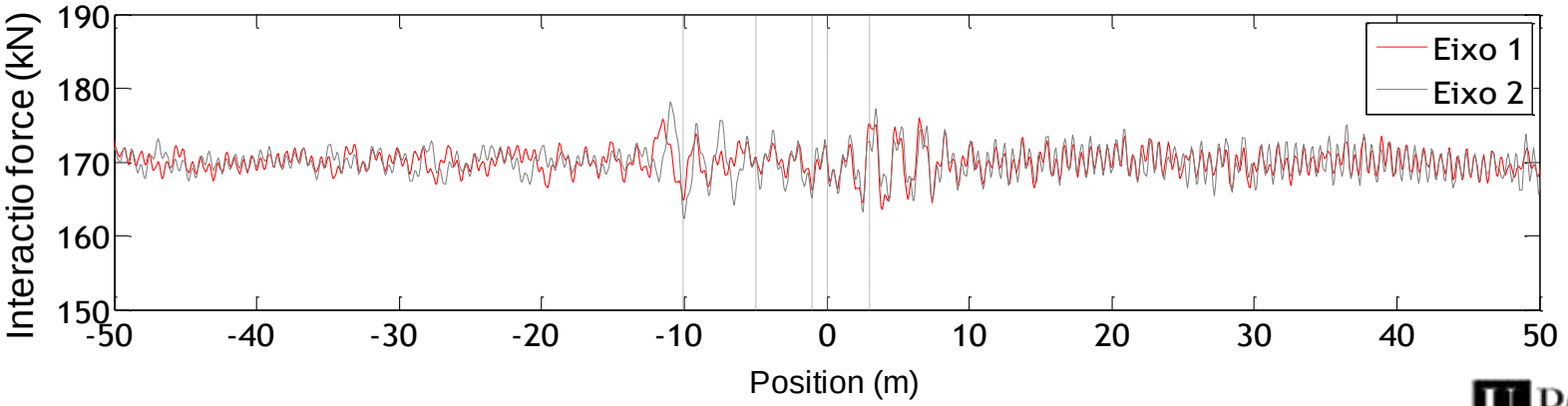
Short term behaviour

- Wheel rail interaction force

ADIF



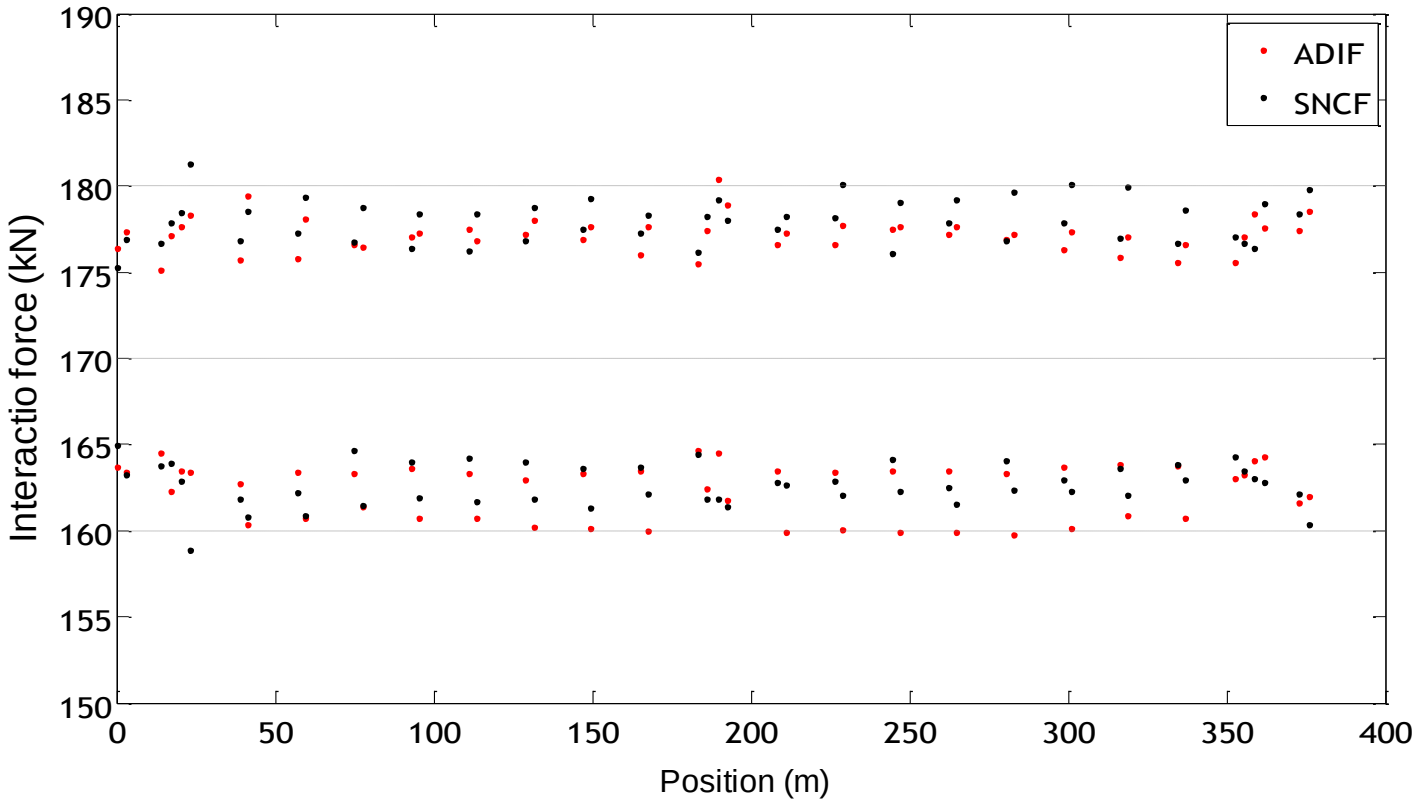
SNCF



7. Dynamic effects on transition zones

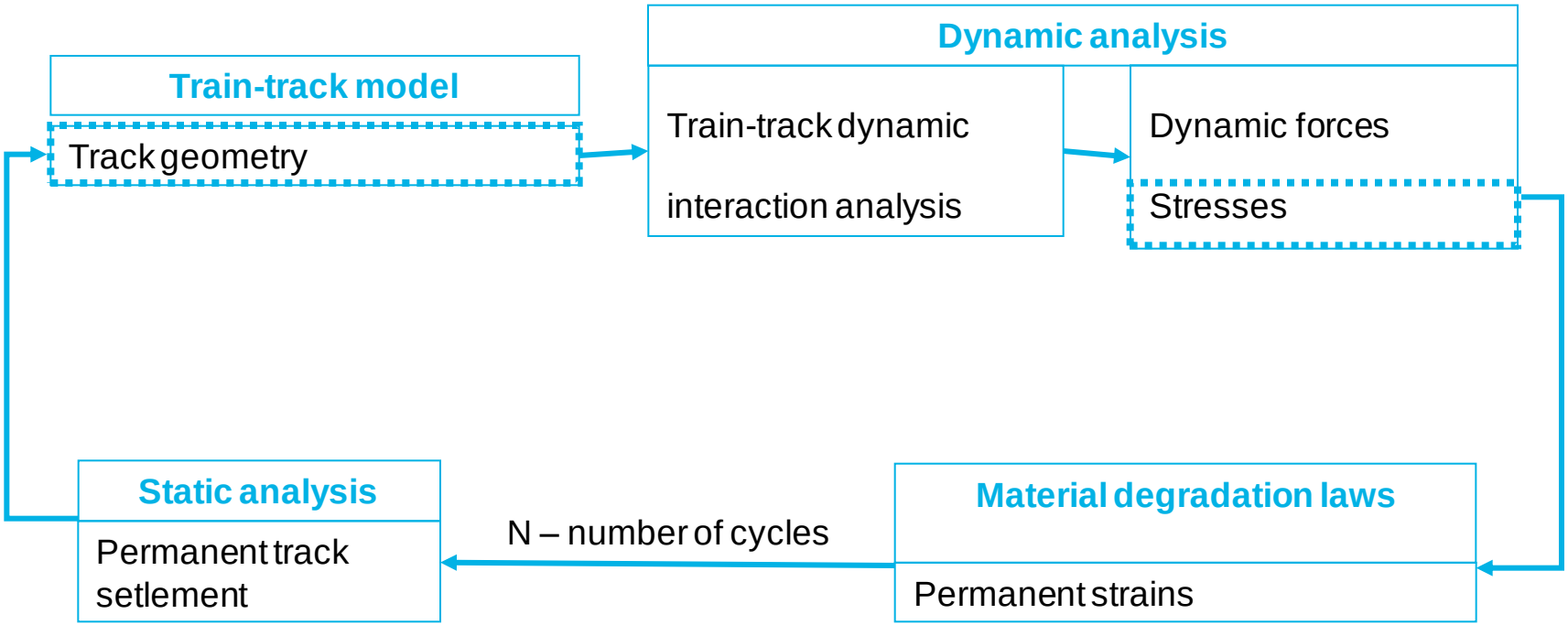
Short term behaviour

- Wheel rail interaction force



7. Dynamic effects on transition zones

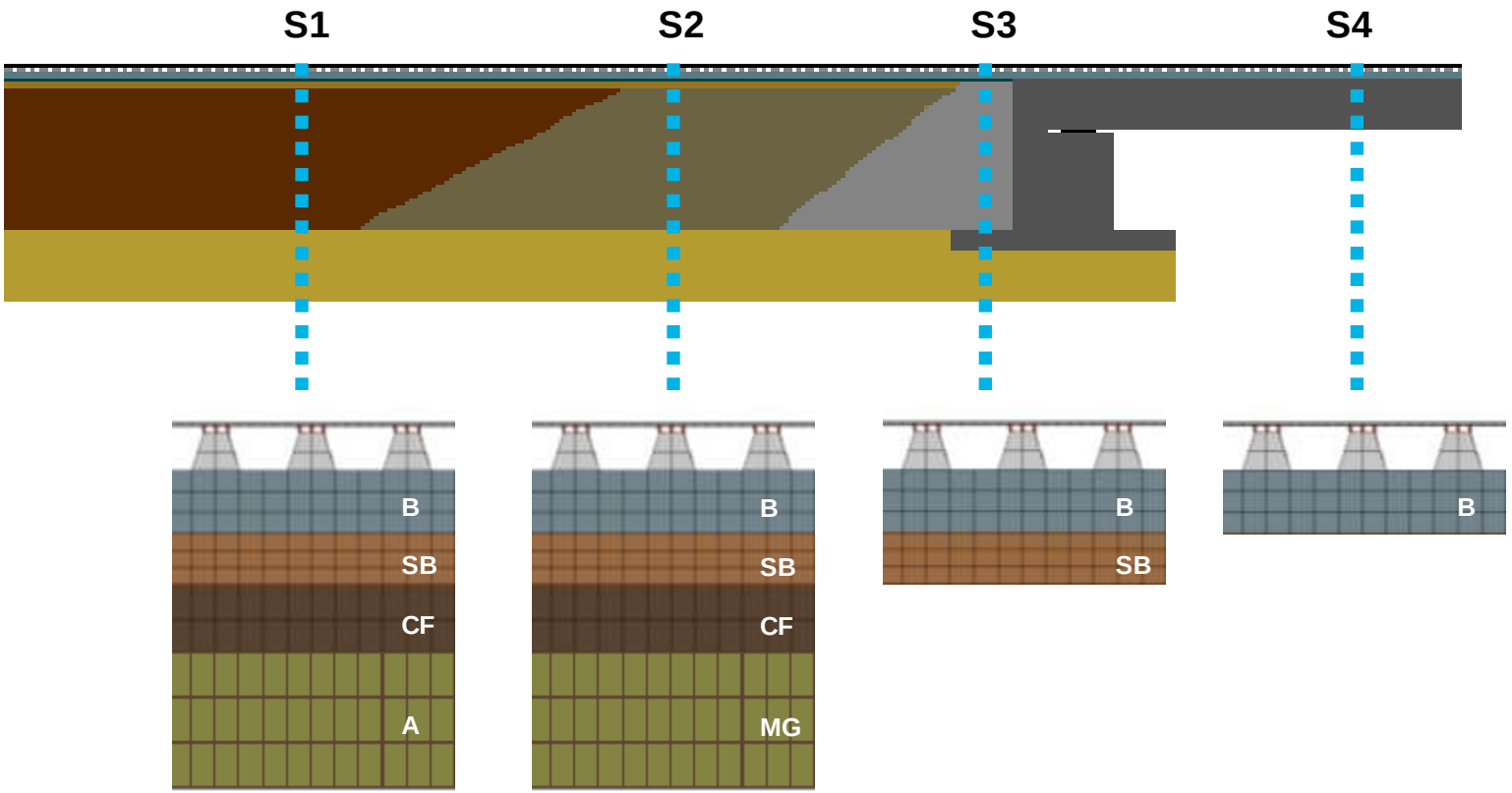
Long term behaviour



7. Dynamic effects on transition zones

Long term behaviour

- Initial stresses

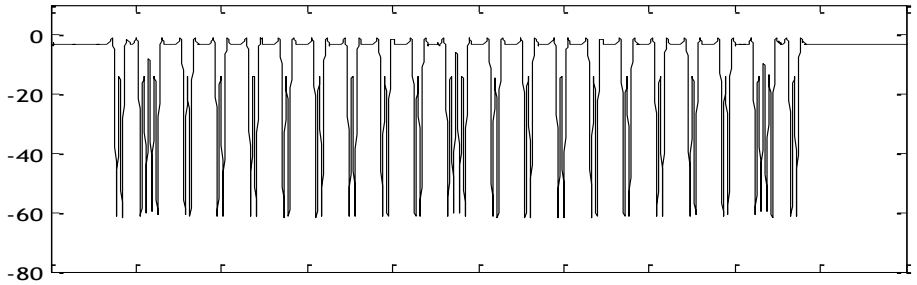


7. Dynamic effects on transition zones

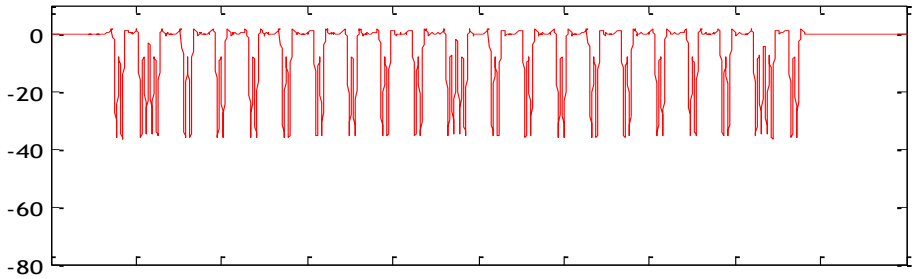
Long term behaviour

- Initial stresses – section S1

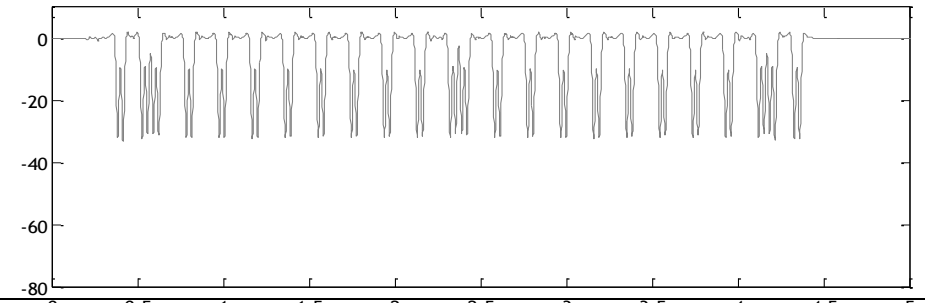
Ballast - top

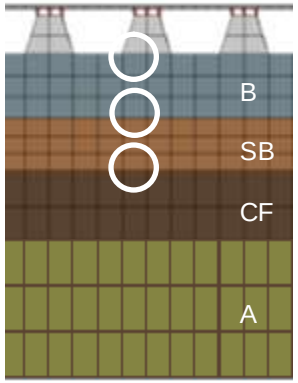


Subballast - top



CF - Top



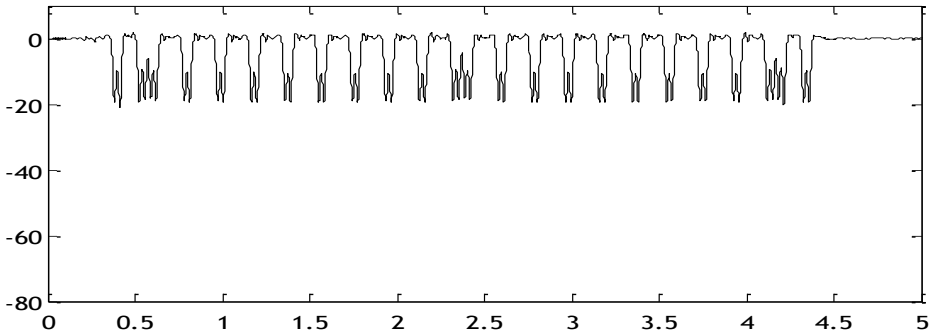


7. Dynamic effects on transition zones

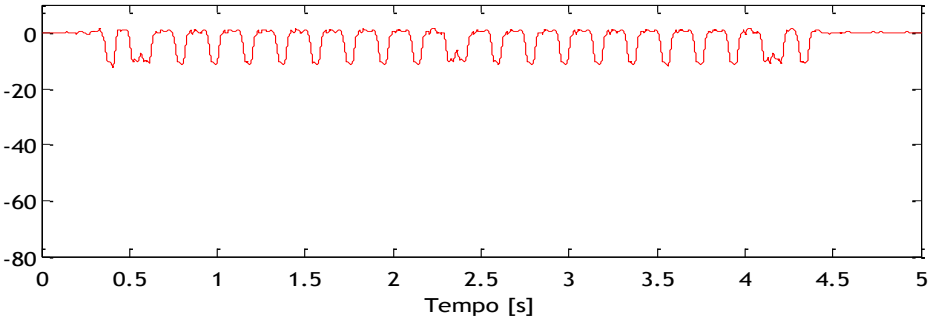
Long term behaviour

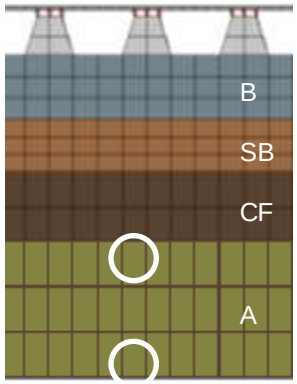
- Initial stresses – section S1

A - top



A - 4 m depth





7. Dynamic effects on transition zones

Long term behaviour

- Material degradation laws

$$\varepsilon_P = f(N)\, g(p_{max}, q_{max})$$

Number of cycles — Material stresses

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

N > 100 cycles

A e B: Parameters

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

$$q = \sigma_1 - \sigma_3$$

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

$$l_{max} = \sqrt{{p_{max}}^2 + {q_{max}}^2}$$

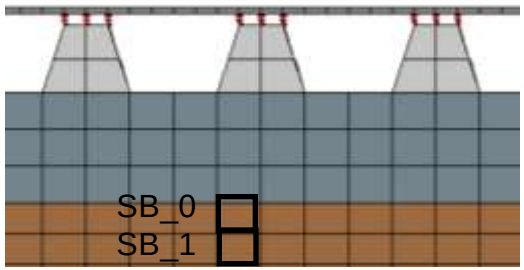
$$p_a = 100\,kPa$$

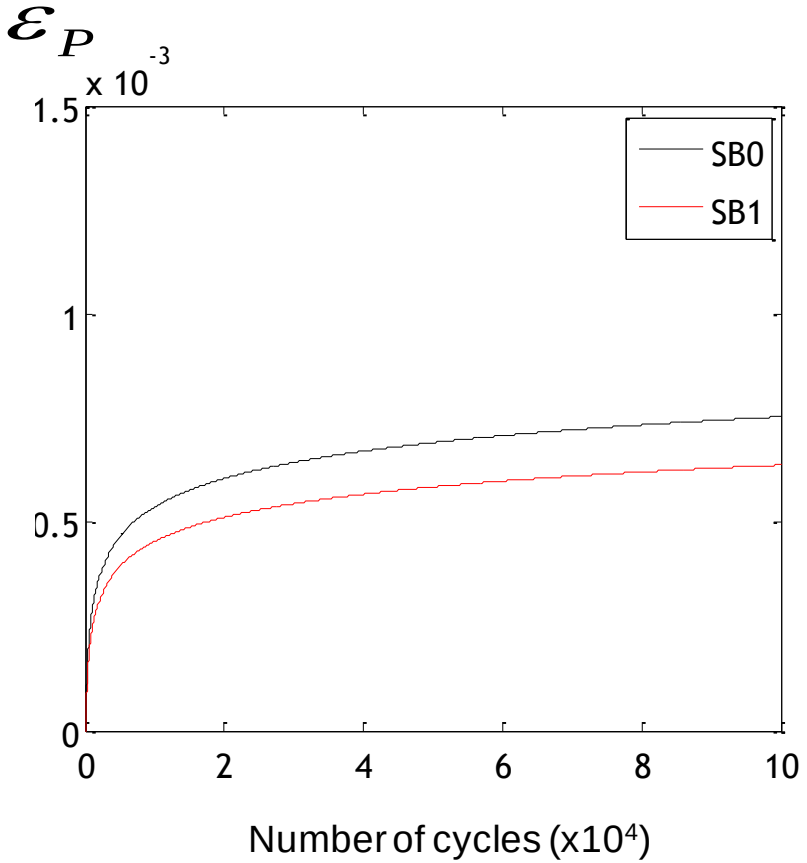
Material dependent parameters: ε_1, m, n, s

7. Dynamic effects on transition zones

Long term behaviour

- Permanent strain evolution

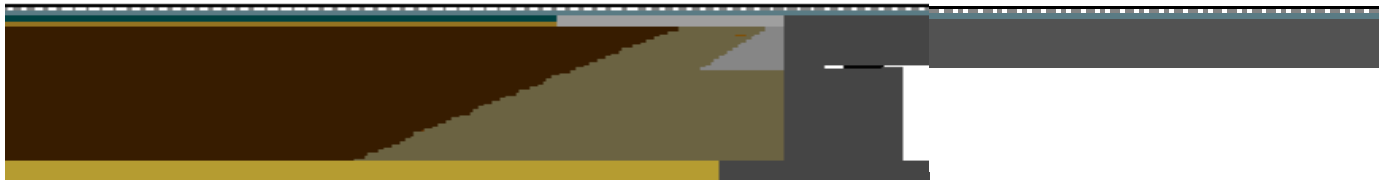


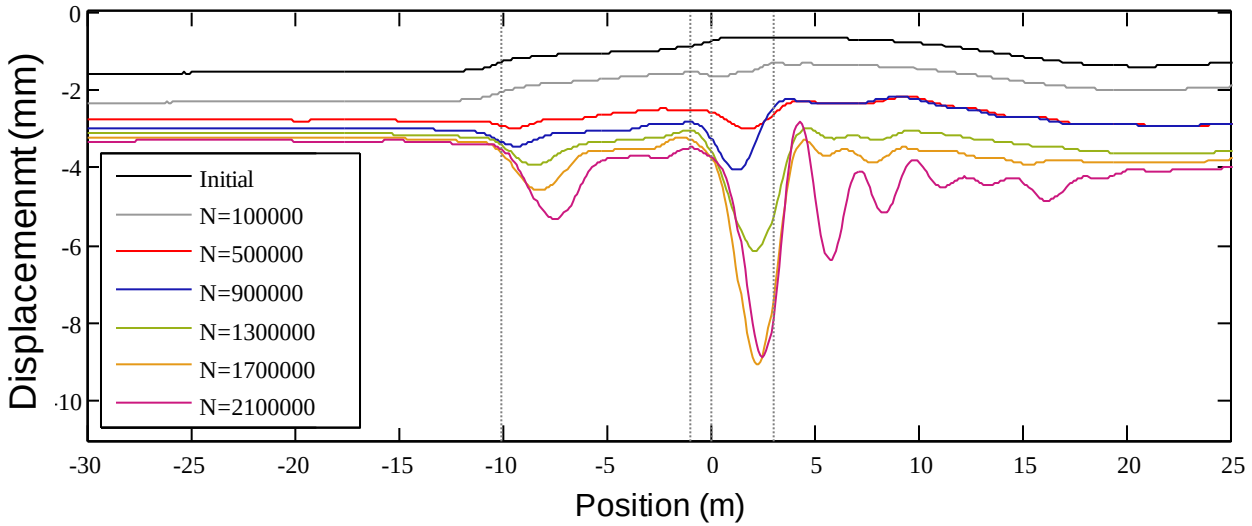


7. Dynamic effects on transition zones

Long term behaviour

- Wheel displacement

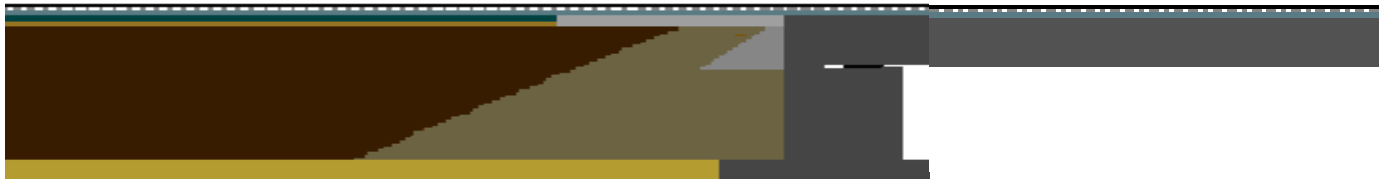


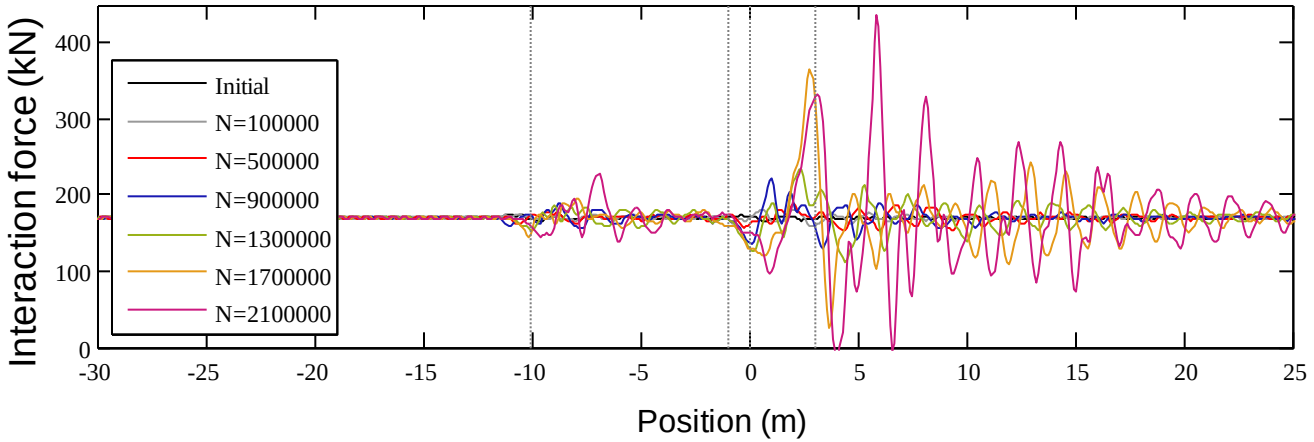


7. Dynamic effects on transition zones

Long term behaviour

- Wheel-rail interaction force

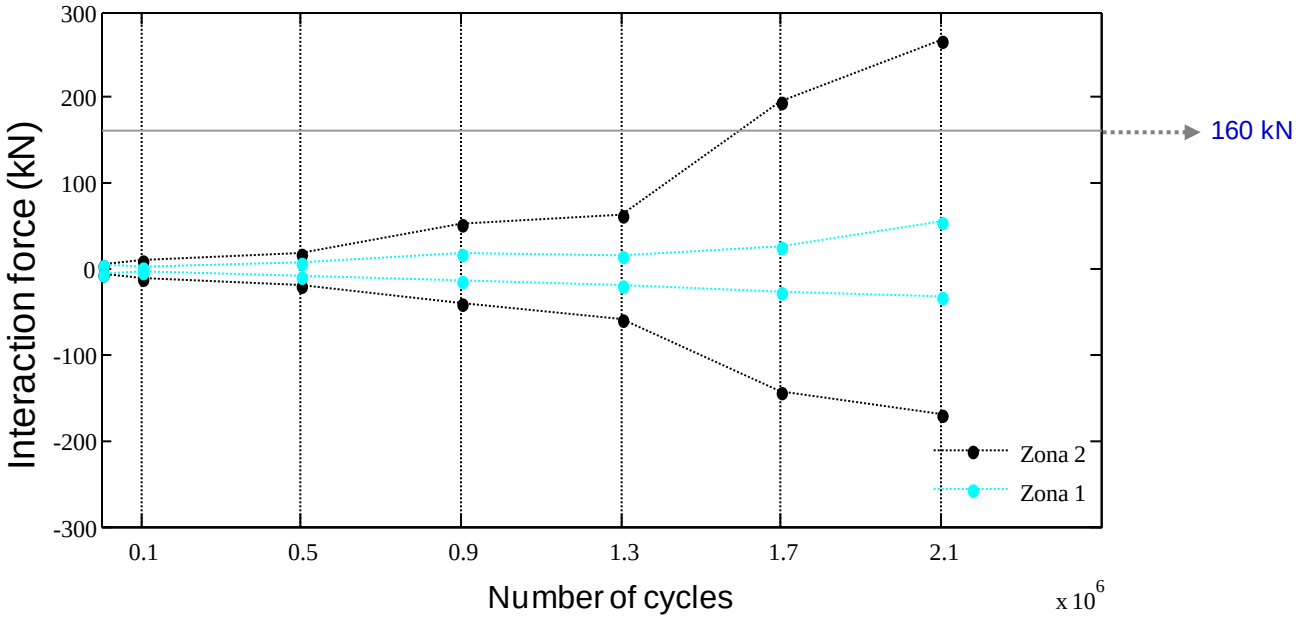




7. Dynamic effects on transition zones

Long term behaviour

- Wheel-rail interaction force

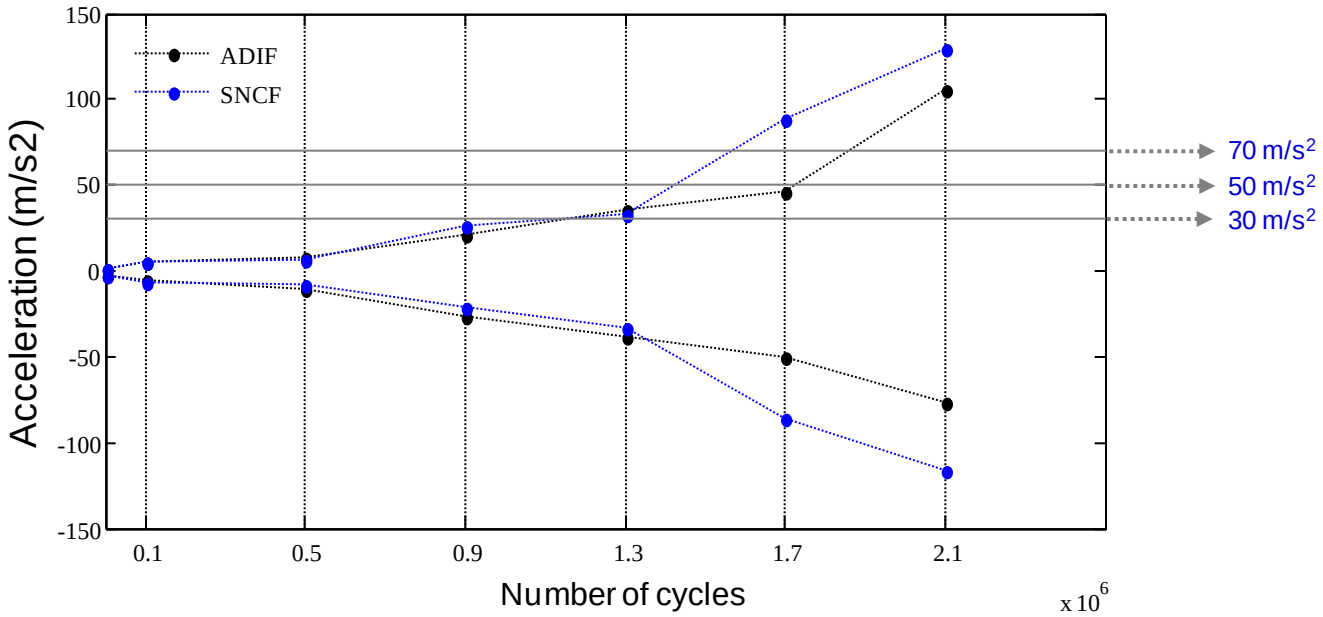


Speed (km/h)	Maximum dynamic load (kN)
V = 250	180
250 < V ≤ 300	170
V > 300	160

7. Dynamic effects on transition zones

Long term behaviour

- Axle acceleration

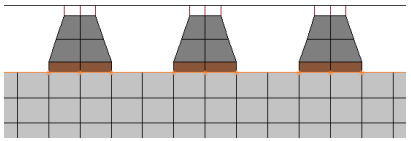


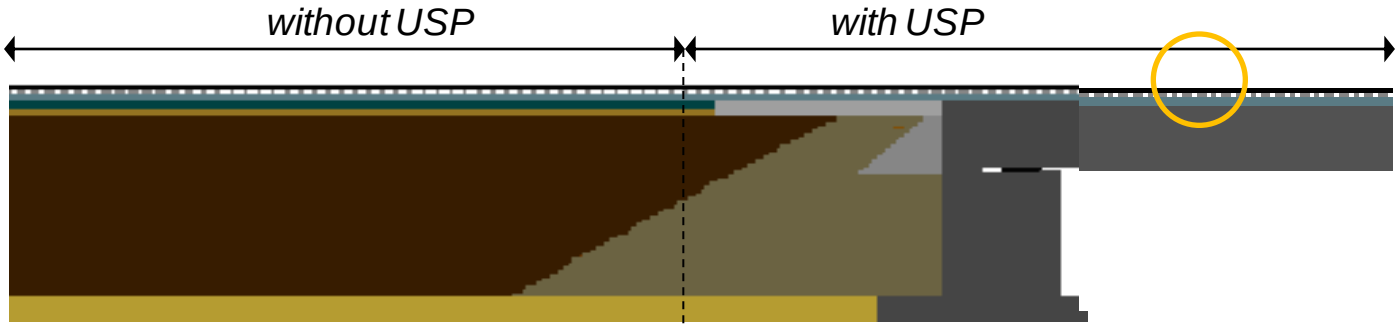
Axle acceleration (m/s²)
a < 30
30 < a < 50
50 < a < 70
a > 70

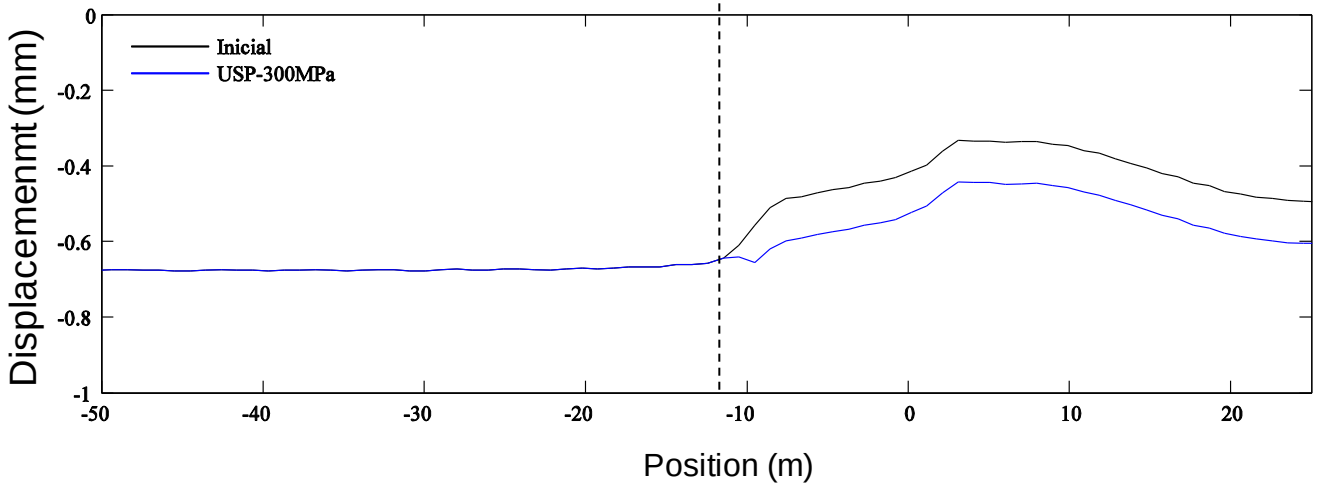
7. Dynamic effects on transition zones

Long term behaviour

- Introduction of Under Sleeper Pads (USP's)



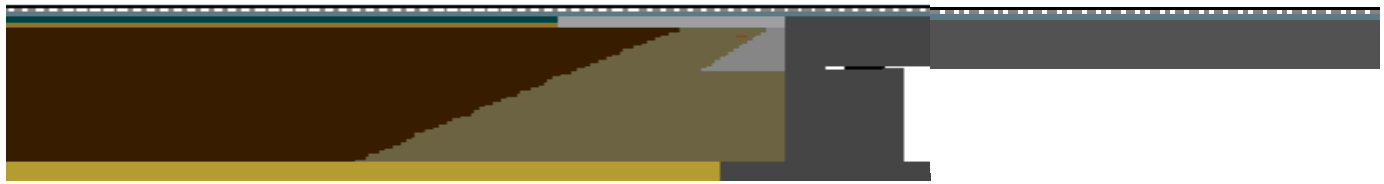




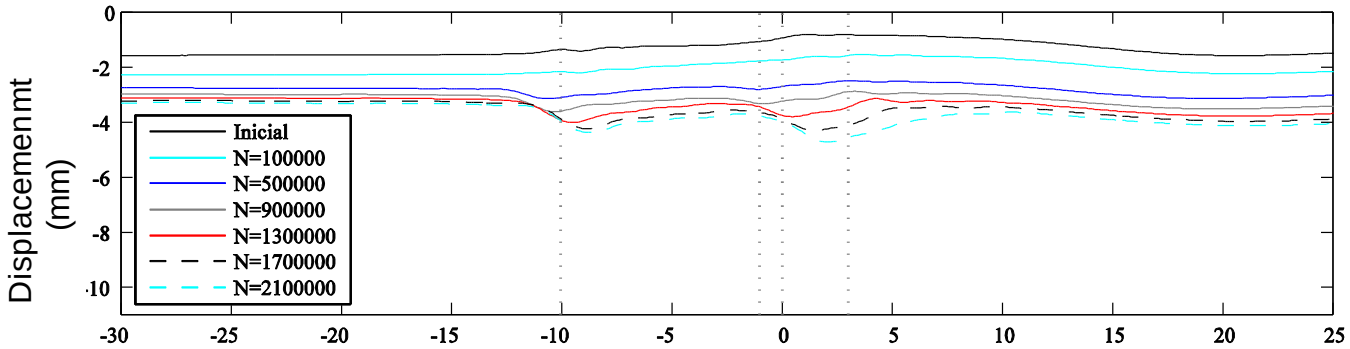
7. Dynamic effects on transition zones

Long term behaviour

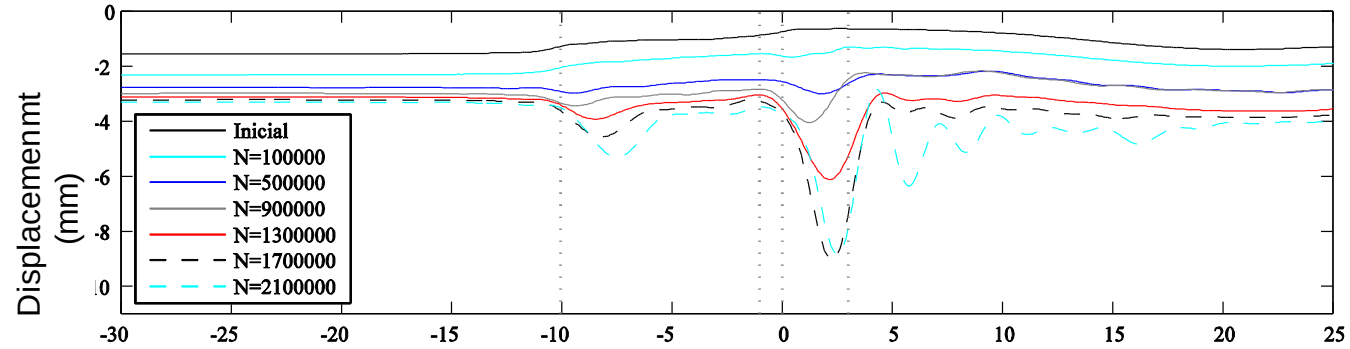
- Introduction of Under Sleeper Pads (USP's)



With USP



Without USP



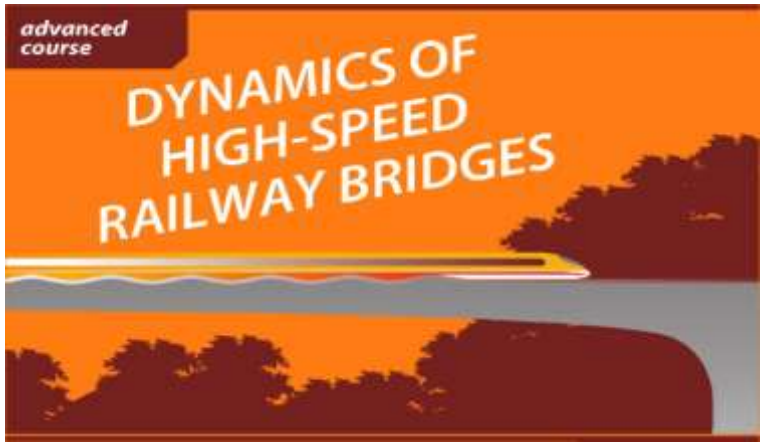
8. Dissemination and training activities

8. Dissemination and training activities

Workshops / advanced courses

3-4 June 2004, FEUP, Porto

12-13 October 2006, FEUP, Porto

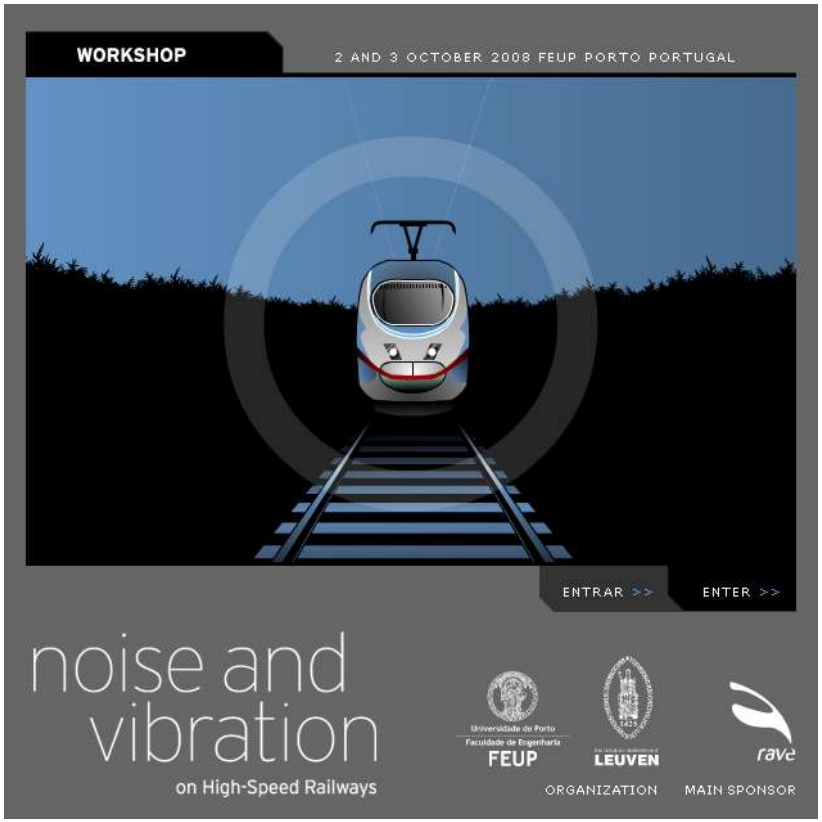


20-23 September 2005, FEUP, Porto

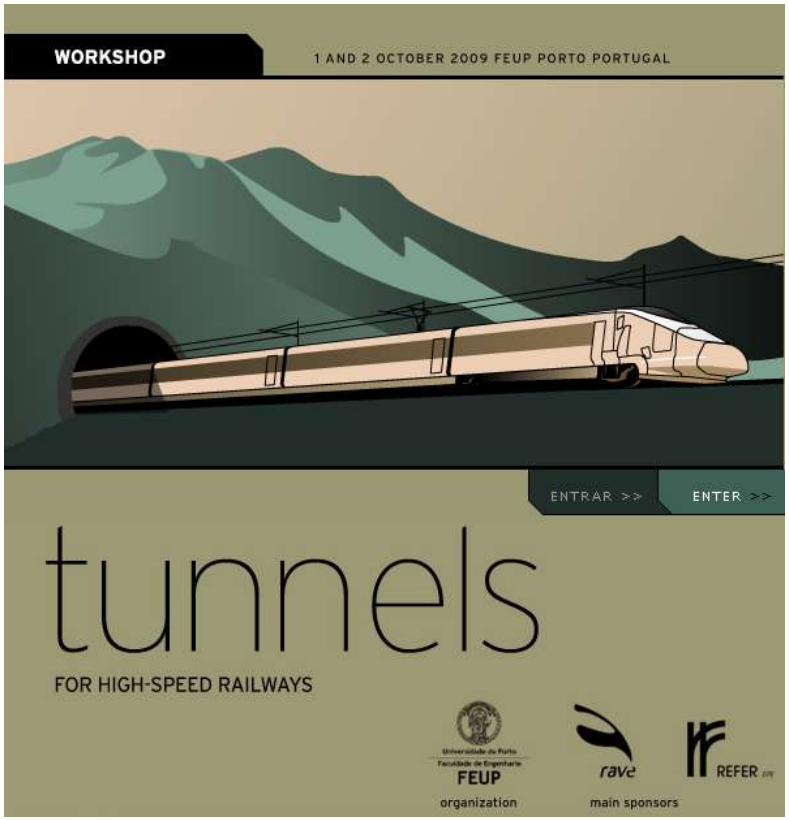
15-16 October 2007, FEUP, Porto

8. Dissemination and training activities

Workshops / advanced courses



2-3 October 2008, FEUP, Porto



1-2 October 2009, FEUP, Porto

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8. Dissemination and training activities

Workshops / advanced courses

9. Research team

9. Research team

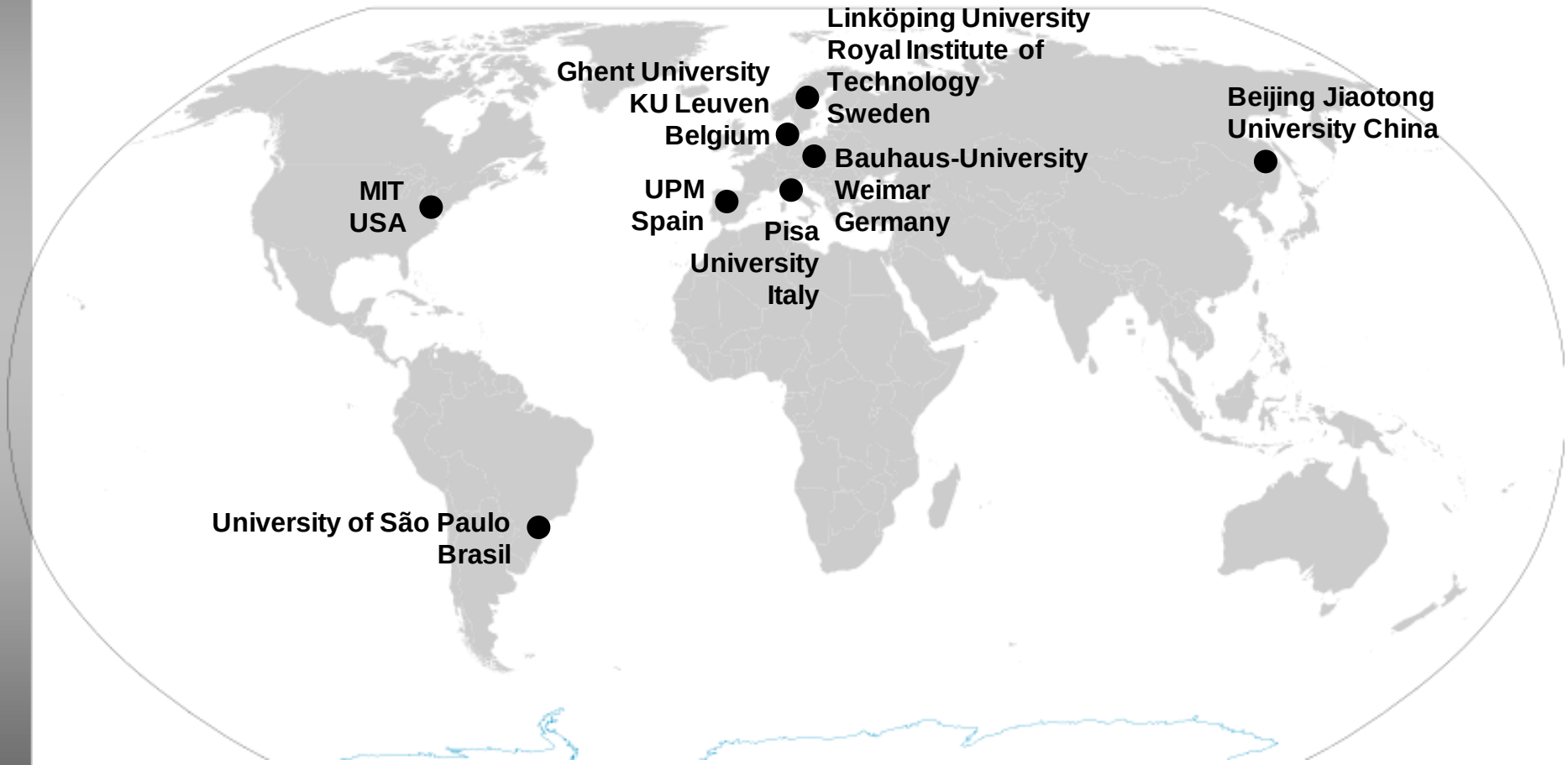
PhD Researchers: 4

PhD Students: 14

Research Assistants: 6

10. International links

10. International links



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Acknowledgments

FCT Fundação para a Ciência e a Tecnologia
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