EC8 Webinars
Second Generation of Eurocode 8

Webinar 2.5
Integral abutment & cable-stayed bridges

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5th April 2024

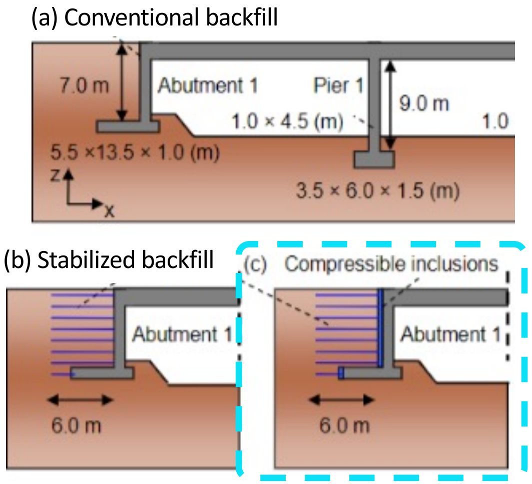
Specific rules for Integral abutment bridges

Force-based approach (linear analysis)
Displacement-based approach (nonlinear analysis)

10. INTEGRAL ABUTMENT BRIDGES

10.1 General (1/5)

- Unless intentionally reduced, SSI is an essential part of the response: vibration cannot happen without engaging embankment/soil



(1) Clause 10 should be used for the modelling, analysis and verification of integral abutment bridges.

NOTE Integral abutment bridges are continuous bridges where the connections between the deck and both the abutments are monolithic (Figure 10.1). Unless specific provisions are taken to avoid or minimize interaction, the vibration of the structure cannot happen independently of that of the surrounding medium (the approach embankments or the natural soil, depending on whether the bridge is above-ground or embedded up to the deck level).

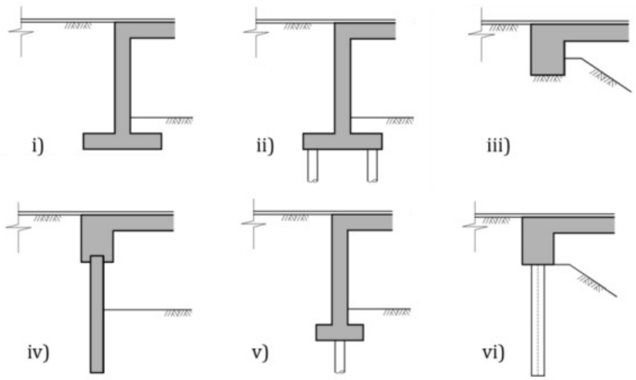


Figure 10.1 — Types of integral abutment bridges: i) full height integral abutment on pad footing; ii) full height integral abutment on piles; iii) bank pad; iv) embedded wall integral abutment; v) full height integral abutment on single row of piles; vi) bank pad on single row of piles. Other types are possible

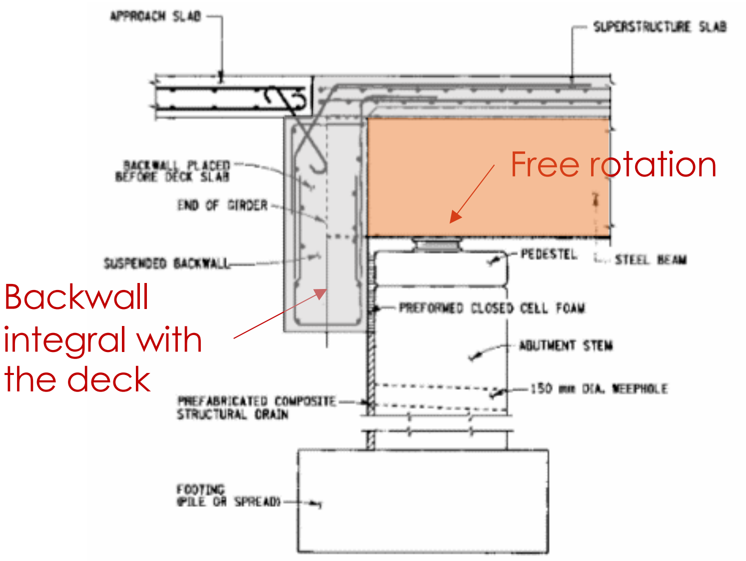
(2) Clause 10 may be applied when bridges are semi-integral, i.e. the rigid connection does not include all degrees of freedom and is realized through fixed bearings or seismic links that restrain the relative movement between the deck and one or both abutments.

Tsinidis, G., M. Papantou, and S. Mitoulis. 2019. "On the response of integral abutment bridges under a sequence of thermal loading and ground seismic shaking." Earthquakes and Structures, 16 (1): 11–28. Techno-Press.

10. INTEGRAL ABUTMENT BRIDGES

10.1 General (2/5)

- Integral vs semi-integral bridges
  (permission to use Clause 10 for the latter)



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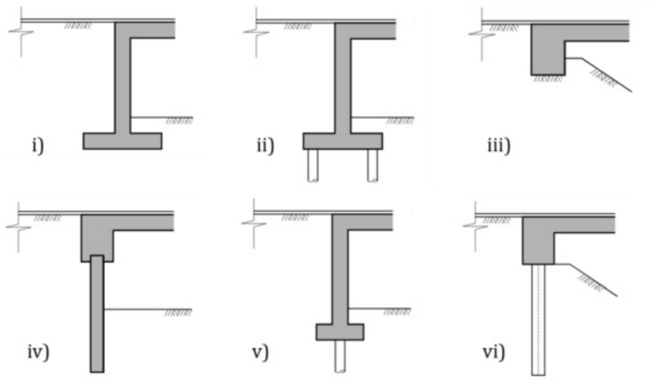

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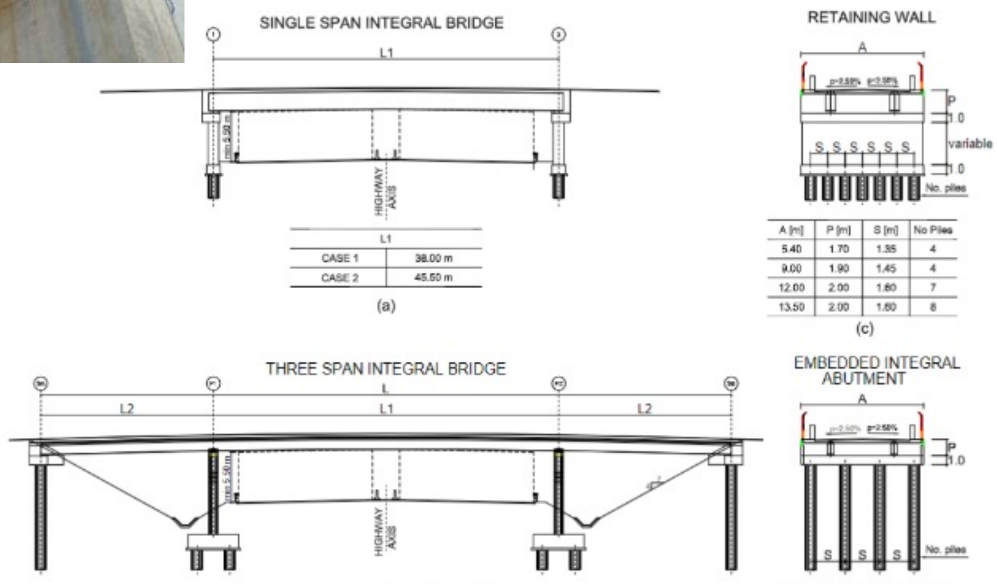
White, H. 2007. Integral Abutment Bridges: Comparison of Current Practice between European Countries and the United States of America. FHWA.

10. INTEGRAL ABUTMENT BRIDGES

10.1 General (3/5)

- Practice of IAB construction are different



	L1
CASE 1	38.00 m
CASE 2	45.50 m

(a)

A [m]	P [m]	S [m]	No Piles
5.40	1.70	1.35	4
9.00	1.90	1.45	4
12.00	2.00	1.80	7
13.50	2.00	1.80	8

(c)

	L1	L2
CASE 1	38.00 m	19.00 m
CASE 2	45.50 m	23.00 m

(b)

A [m]	P [m]	S [m]	No Piles
5.40	1.70	1.35	4
9.00	1.90	2.30	4
12.00	2.00	3.30	4
13.50	2.00	3.80	4

(d)

Ferretti Torricelli, A. Marchiondelli, R. Pefano, and R. Stucchi. 2012. “Integral bridge design solutions for Italian highway overpasses.” Proceedings of the Sixth International IABMAS Conference. Stresa, Italy.

10. INTEGRAL ABUTMENT BRIDGES

10.1 General (4/5)

- Practice of IAB construction are different

	construction	abutment $l_{sup}/h_{abutment}$	field l_{sup}/h_{field}	without haunch l_{sup}/h_{field}
Road bridges	reinforced concrete	12-18	20-25	18-21
	prestressed concrete	15-19	24-30	20-25
	steel composite	15-19	25-35	21-25
Railway bridges	reinforced concrete	10-15	20-25	16-18
	prestressed concrete	15-20	20-25	-
	steel composite	15-18	25-30	18-21

Braun A., Seidl G. and Weizenegger G. Rahmentragwerke im Brückenbau, *Beton-und-Stahlbetonbau* 101. 2006. - Heft 3. - pp.187-197.

Single-span composite deck with straight abutments

BAB A8 bridge structure 5, Germany

Single-span composite deck with inclined abutments

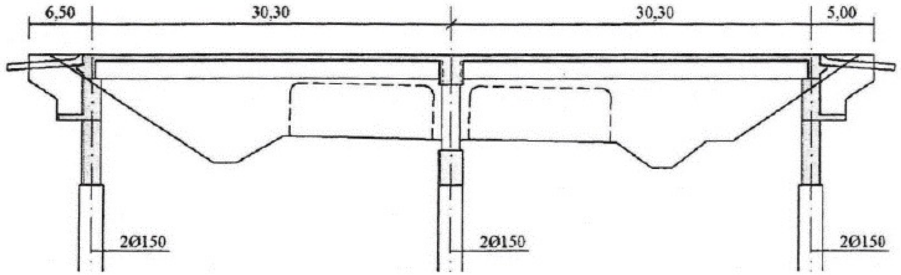
A73, Munich, Germany

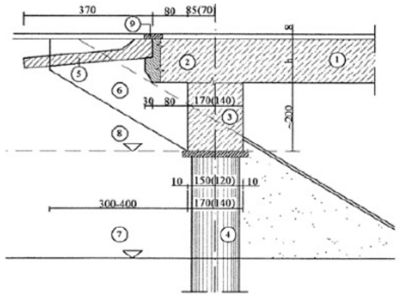
Feldmann, M., J. Naumes, D. Pak, M. Veljkovic, J. Eriksen, O. Hechler, N. Popa, G. Seidl, and A. Braun. 2010. Design guide - Economic and Durable Design of Composite Bridges with Integral Abutments. CEN/TC 250/SC 10 N 0216.

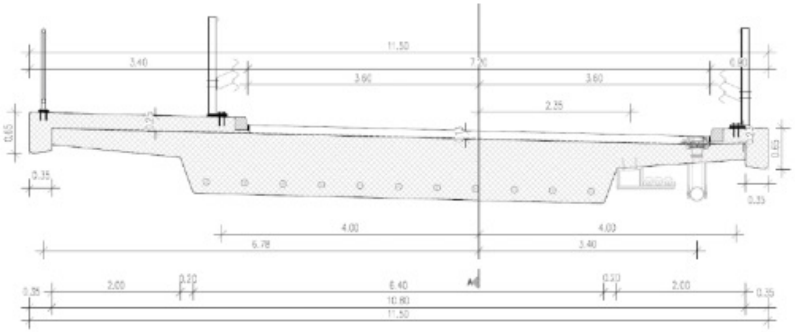
10. INTEGRAL ABUTMENT BRIDGES

10.1 General (5/5)

- Practice of IAB construction are different







Pržulj, M. (2015). Mostovi: zasnova, projektiranje, konstruiranje, zanesljivost, gradnja, gospodarjenje, obnova. Beletrina, Ljubljana
Pržulj, M. (2008). Integralni betonski mostovi. V: Zbornik 9. slovenski kongres o cestah in prometu. Portorož, 22. – 24. Oktober 2008. Ljubljana, Družba za raziskave v cestni in prometni stroki Slovenije: str. 53-72.

10. INTEGRAL ABUTMENT BRIDGES

10.2 Basis of design (1/3)

- Two sets of soil properties in general
  - Often softer soil will lead to worst condition in both piers, abutments and foundations
  - Approach enbankment material more controlled than natural soil
- Permission to consider:
  - construction sequence
  - thermal history

(1) The calculation of the effects of the seismic action should incorporate the effects of interaction between soil and abutments.

(2) Action effects should be calculated using both upper and lower bound estimates of soil properties.

NOTE The requirement in (2) intends to arrive at results which are on the safe side both for the abutments and for the piers.

(3) The calculation of the effects of the seismic action may incorporate the effects on the soil pressures against the abutments of a) and b):

a) the construction sequence;

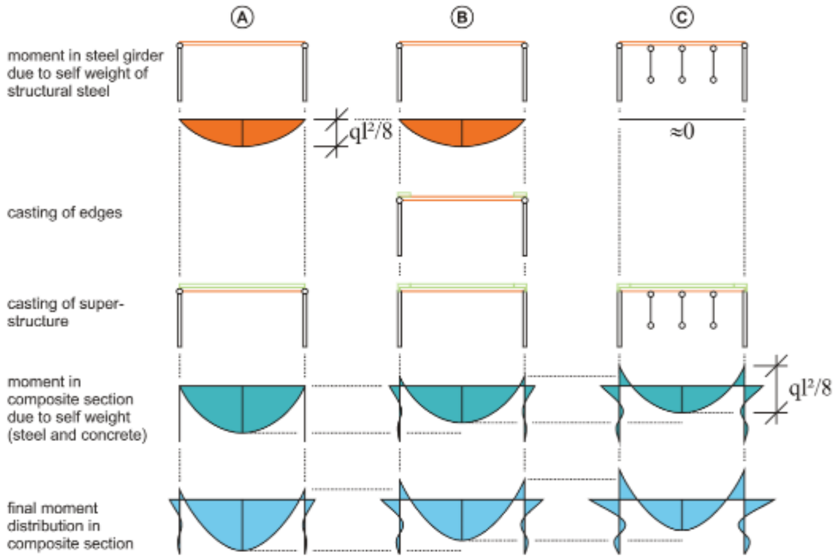


Figure 6-5: Influence of casting sequence / time of restraint

Feldmann, M., J. Naumes, D. Pak, M. Veljkovic, J. Eriksen, O. Hechler, N. Popa, G. Seidl, and A. Braun. 2010. Design guide - Economic and Durable Design of Composite Bridges with Integral Abutments. CEN/TC 250/SC 10 N 0216.

10. INTEGRAL ABUTMENT BRIDGES

10.2 Basis of design (2/3)

- Two sets of soil properties in general
  - Often softer soil will lead to worst condition in both piers, abutments and foundations
  - Approach enbankment material more controlled than natural soil
- Permission to consider:
  - construction sequence
  - thermal history

(1) The calculation of the effects of the seismic action should incorporate the effects of interaction between soil and abutments.

(2) Action effects should be calculated using both upper and lower bound estimates of soil properties.

NOTE The requirement in (2) intends to arrive at results which are on the safe side both for the abutments and for the piers.

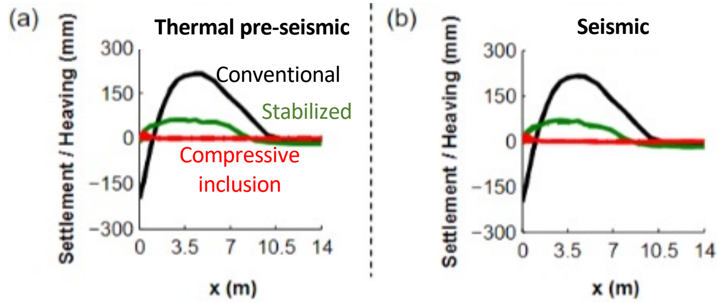
(3) The calculation of the effects of the seismic action may incorporate the effects on the soil pressures against the abutments of a) and b):

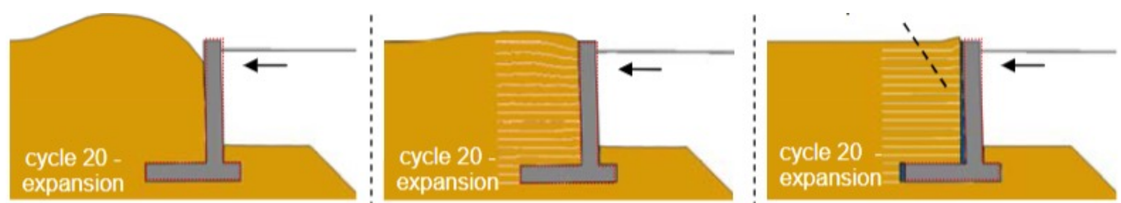
a) the construction sequence;

b) thermal cycling previous to the occurrence of an earthquake, if no special provisions are taken to prevent interaction and the material (soil or backfill) in contact with the abutments is coarse-grained.

NOTE 1 Interaction between soil and structure occurs at the foundation and through earth pressures on the vertical abutment wall. The initial pressure distribution resulting from the construction sequence is important in determining the dynamic pressure distribution during the earthquake.

NOTE 2 In coarse-grained soils and backfill, cyclic deformation induces particle realignment and progressive compaction that cause stiffening. This phenomenon, known as ratcheting, is associated, e.g. with repeated thermal cycling, and can lead to an increase in the initial at-rest pressures. Ratcheting is not present in fine-grained soils.





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10. INTEGRAL ABUTMENT BRIDGES

10.2 Basis of design (3/3)

- Displacement-compatibility = displacement-dependent soil pressures
- Response not symmetric even in symmetric IAB
- Target performance is quasi-elastic (DC1)
  - Not accessible for repair (not with reasonable difficulty, especially for tall abutments)
  - By extension, target performance for IABs is the same
  - Exception for piers framing into the deck, if design wants to exploit their ductility (e.g., Slovenian practice)→DBA most suited

(4) Seismic response should be calculated based on kinematic compatibility between the bridge structure and the free-field seismic deformation of the soil and the embankment.

(5) Verification should be carried out considering, for each component of the seismic action, the most unfavourable effects resulting from the application of the actions as defined in 10.3 in one or the opposite direction.

(6) Integral abutment bridges and culverts may be considered to be embedded structures, if the abutments are embedded in stiff natural soil formations (FprEN 1998-1-1:2024, Table B.2) over at least 80 % of their lateral area.

(7) Due to difficulties in repair, damage to abutments in integral abutment bridges should be avoided. With the exception of (8), integral abutment bridges should be designed to DC1.

NOTE The ductility class for a bridge is unique.

(8) If elastic response of abutments is ensured, integral abutment bridges may be designed to DC2 or DC3 if the energy dissipation at piers' plastic hinges rigidly connected to the deck is part of the design concept.

NOTE The displacement-based approach (10.3.3) is most suited to analyse integral abutment bridges where non-linearity occurs both in the structure and in the soil. The force-based method is not suited (see 10.3.2(1), note 1).

10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (1/9)

- Target performance for the abutments is substantially elastic under the design seismic action
  - DC1 & $q = q_S = 1{,}5$
- Note: no $S_{\mathrm{a,red}} = \frac{S_{\mathrm{a}}}{q}$ but $S_{\mathrm{a}} \rightarrow M'_{\mathrm{Ed,E}}$ and $M_{\mathrm{Ed,E}} = \frac{M'_{\mathrm{Ed,E}}}{q}$

10.3.1 General

(1) Structural members should be modelled as linear, accounting for cracking of concrete parts, according to 5.1.1(4) to (6).

NOTE Design according to DC1 implies linear response.

(2) The seismic analysis of integral abutment bridges should comply with either a) or b):

a) force-based approach according to 10.3.2;

b) displacement-based approach according to 10.3.3.

10.3.2 Force-based approach

(1) A behaviour factor q = 1,5 should be used, according to Table 5.2. The behaviour factor should be used to divide internal forces due to the seismic action, rather than the spectral acceleration acting on structural masses.

NOTE 1 Internal forces depend on pressures that, together with the foundation reaction, equilibrate the inertia forces on the structural mass. Reduction of spectral acceleration on the structural mass by q would alter the overall distribution of forces between foundation and abutment.

NOTE 2 The value of q coincides with q_S, which accounts for the difference between expected and design strength, not for reduction in spectral acceleration due to ductility.

Type of Ductile Members	q_R	q_D		$q = q_S\ q_R\ q_D$	
		DC2	DC3	DC2	DC3
Integral abutment bridges (see 10)	1,0	1,0	1,0	1,5	1,5

10. INTEGRAL ABUTMENT BRIDGES

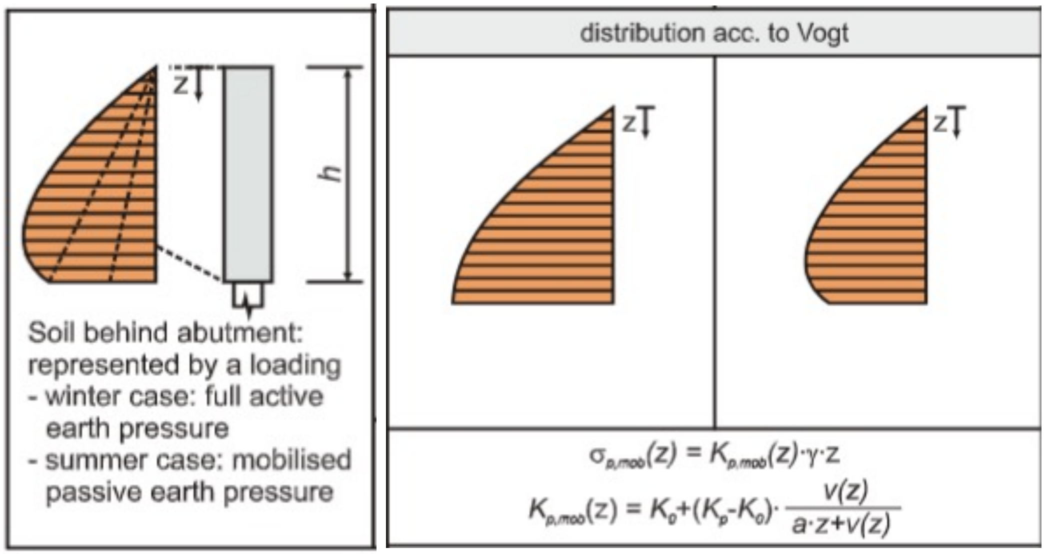
10.3 Modelling and analysis (2/9)

- Pressure distribution is not symmetric as for thermal expansion/contraction cycles
- Mobilised passive stresses: pressures on the downstream side (where the bridge leans against the soil/enbankment), similarly to thermal exapansion, are intermediate between at rest and passive
- For thermal cycles they are self-equilibrated stresses…

10.3.2 Force-based approach

(2) The actions in a) and b) should be taken into account in the longitudinal direction (Figure 10.2):

a) total (static plus seismic) earth pressures E_d acting on the abutments in the seismic design situation, calculated according to FprEN 1998-5:2024, 10.3.2, duly accounting for the effect of friction between soil and abutment wall. The pressures E_d may be assumed to correspond to the active limit on one abutment (away from which the structure's mass is accelerated, denoted as 'upstream') and intermediate between the at-rest and the passive limit on the other abutment (towards which the structure's mass is accelerated, denoted as 'downstream');



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10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (3/9)

- Mobilised passive stresses:
  Seismic case: different! external action!
  passive $\sigma_{\mathrm{p,mob}}(z) = K_{\mathrm{PE}}\gamma \cdot \min(z; \Delta H)$
  until $\Delta H = c\alpha H_{\mathrm{ab}}$
  increasing with intensity $\alpha = \dfrac{mS_{\alpha}}{\frac{1}{2}BK_{\mathrm{PE}}\gamma H_{\mathrm{ab}}^2}$
- What doesn't go to the abutment, goes to the foundation
  - Value of c permitted not recommended, can be reduced/increased depending on foundation type (e.g., steel pipes)

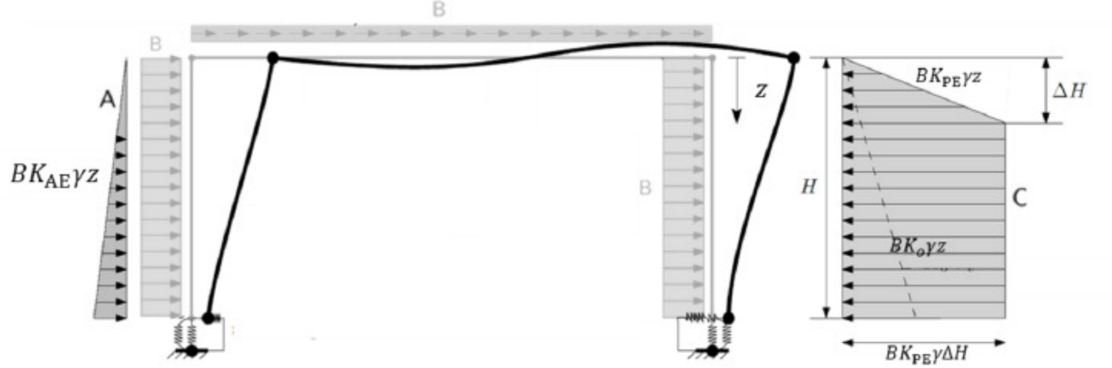
Bridge-embankment modes

10.3.2 Force-based approach

(2) The actions in a) and b) should be taken into account in the longitudinal direction (Figure 10.2):

a) total (static plus seismic) earth pressures E_{d} acting on the abutments in the seismic design situation, calculated according to FprEN 1998-5:2024, 10.3.2, duly accounting for the effect of friction between soil and abutment wall. The pressures E_{d} may be assumed to correspond to the active limit on one abutment (away from which the structure's mass is accelerated, denoted as 'upstream') and intermediate between the at-rest and the passive limit on the other abutment (towards which the structure's mass is accelerated, denoted as 'downstream');

b) inertia forces acting on the mass of the structure, evaluated as the product of structural masses and the maximum response spectral acceleration corresponding to the constant acceleration range of the elastic response spectrum S_{α}, as given in FprEN 1998-1-1:2024, 5.2.2.2.



NOTE 2 Maximum internal forces occur when the structure moves towards the soil on the 'downstream' side.

NOTE 3 The structure cannot oscillate with its natural vibration period as if it were not in contact with the surrounding medium. On the other hand, determination of the predominant period of vibration for the structural portion of the soil-embankment-structure system is not feasible within the context of the force-based approach. This period is in general shorter than T_{C}. Plateau acceleration is thus conservatively employed as an approximation.

Marchi, A., and P. Franchin. 2023. "Equivalent static methods for seismic design of straight integral abutment bridges." Earthq. Eng. Struct. Dyn. Wiley. https://doi.org/10.1002/eqe.4052.

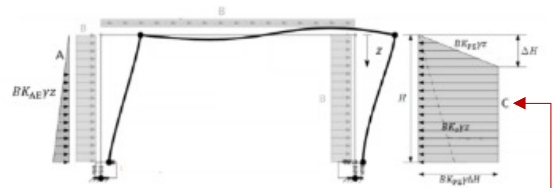
10. INTEGRAL ABUTMENT BRIDGES

10.3.2 Force-based approach

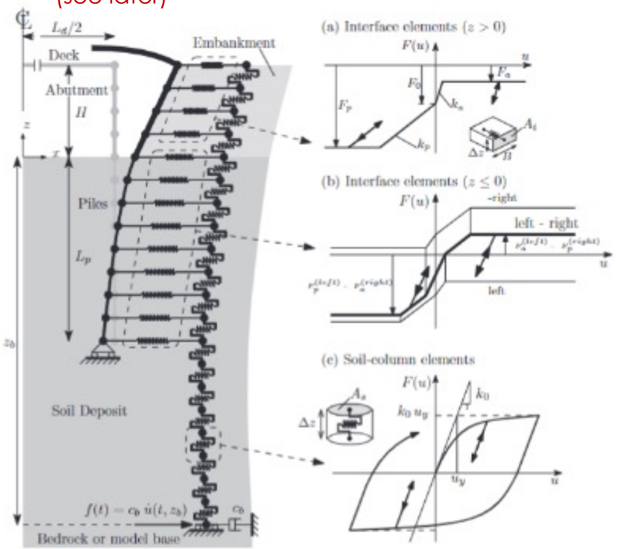
NOTE 2 Maximum internal forces occur when the structure moves towards the soil on the 'downstream' side.

10.3 Modelling and analysis (4/9)

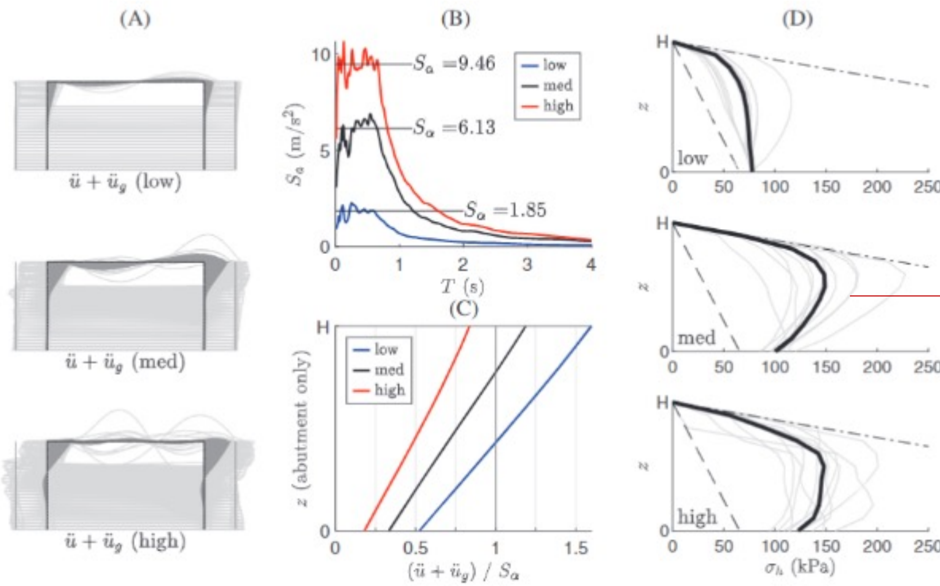
- Background to pressure distribution



Validated NL dynamic model
(see later)



Pressure distributions @ instant(s) of max internal forces



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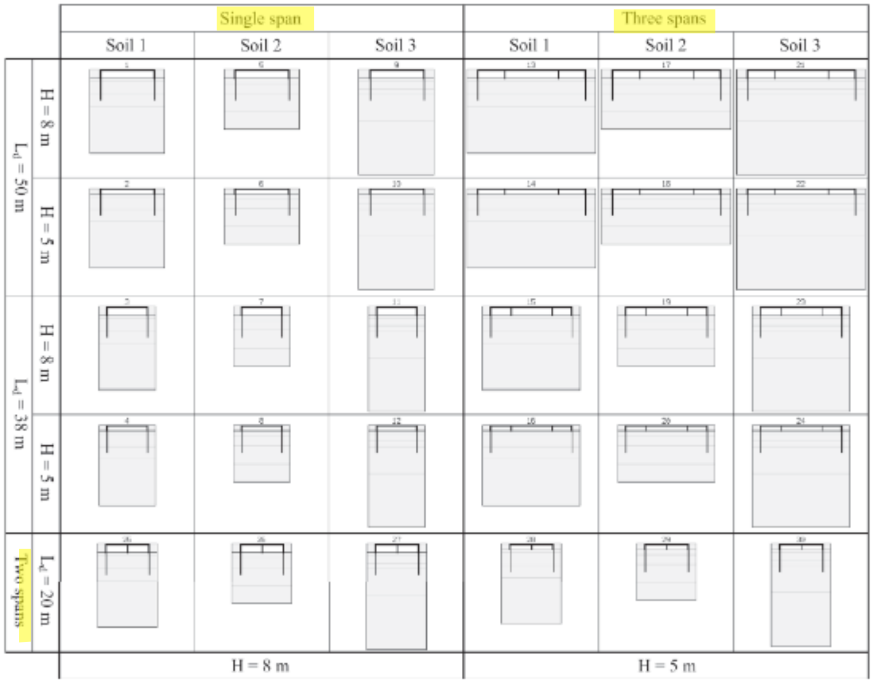
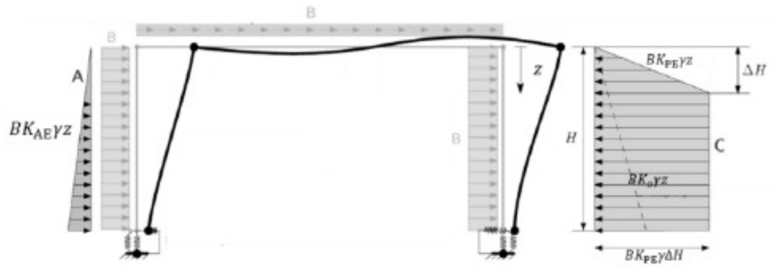
10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (5/9)

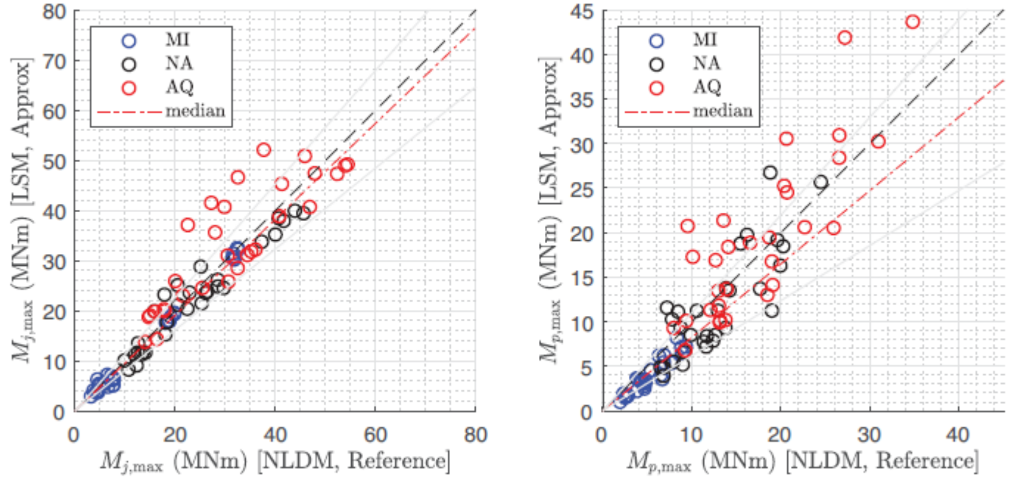
Parametric study (12+12+6 cases)



10.3.2 Force-based approach



NL dynamic vs Linear static: deck-abutment joint + piles head



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10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (6/9)

- Static impedances = springs
  - FprEN1998-5, 8.2.1:
  - (2) Ground reaction may be represented by springs for all degrees of freedom. NOTE 1 In general, the springs are non-linear and frequency-dependent.
  - (4) For certain shallow foundation shapes (circle, strip, rectangle), piles and ground profiles (for example, homogeneous half-space and soil layer on rock), values for spring stiffnesses may be obtained from available elasticity-based solutions.
  - (5) A frequency-independent stiffness value may be assigned to each spring, at the fundamental mode period, accounting for SSI in the considered direction. If this period is difficult to determine reliably, the static stiffnesses may be used instead.

10.3.2 Force-based approach

(7) The model should account for the effect of the flexibility of the abutment and piers foundation.

(8) The effect of foundation flexibility may be accounted for through static foundation impedances according to FprEN 1998-5:2024, Clause 8. Group effects may be taken equal to their static values.

NOTE FprEN 1998-5:2024, Annex D, gives guidance for calculating foundation impedances of both shallow and deep foundations.

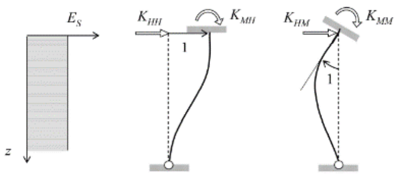
D.5 Static lateral impedance of a single pile in a homogeneous layer

(1) The static impedance functions, defined in 8.2, 8.3 of an isolated flexible pile in a homogeneous layer (see Figure D.4) may be calculated using Formulae (D.19) to (D.21).

$$K_{HH} \approx E_S d \left(\frac{E_P}{E_S}\right)^{0,20} \quad (D.19)$$

$$K_{MM} \approx 0,15 E_S d^3 \left(\frac{E_P}{E_S}\right)^{0,75} \quad (D.20)$$

$$K_{MH} = K_{HM} \approx -0,22 E_S d^2 \left(\frac{E_P}{E_S}\right)^{0,50} \quad (D.21)$$



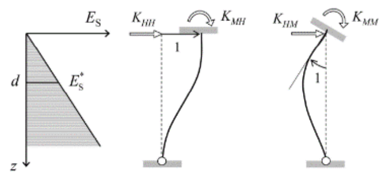
D.6 Static lateral impedance of a single pile in a linearly inhomogeneous layer

(1) The static impedance functions, defined in 8.2 and 8.3, of an isolated pile in an inhomogeneous layer (see Figure D.5) with a modulus increasing linearly with depth may be calculated using Formulas (D.22) to (D.24).

$$K_{HH} \approx 0,6 E_S^* d \left(\frac{E_P}{E_S^*}\right)^{0,35} \quad (D.22)$$

$$K_{MM} \approx 0,15 E_S^* d^3 \left(\frac{E_P}{E_S^*}\right)^{0,80} \quad (D.23)$$

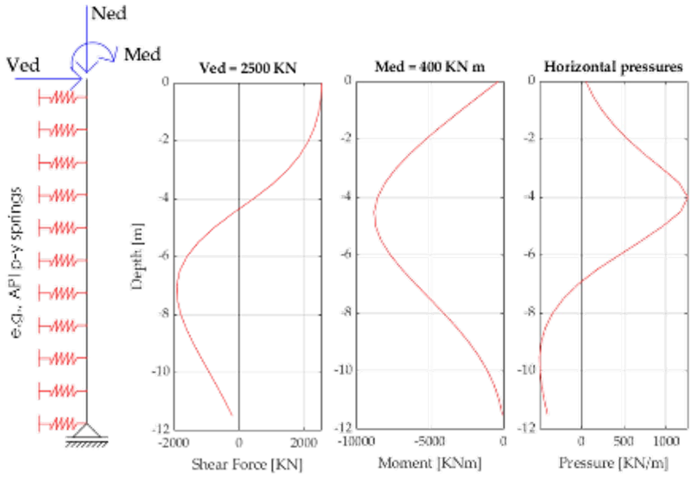
$$K_{MH} = K_{HM} = -0,17 E_S^* d^2 \left(\frac{E_P}{E_S^*}\right)^{0,60} \quad (D.24)$$



10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (7/9)

- Static impedances = springs
  - Piles must be designed with an appropriate model



  - Piles that satisfy the geotechnical verifications (bearing capacity & settlements under vertical loads) may turn out to be too heavily reinforced

FprEN1997-3:2024 (CEN/TC250/SC7 N1753)

6.5.4 Transverse resistance of single piles

(1) The transverse resistance of a single pile shall be determined by calculation or by testing.

(2) The transverse resistance of a single pile may be determined assuming rotation or translation of a rigid body or bending failure and local yielding depending on the ground properties and the flexural stiffness of the pile.

EN1997-1:2004

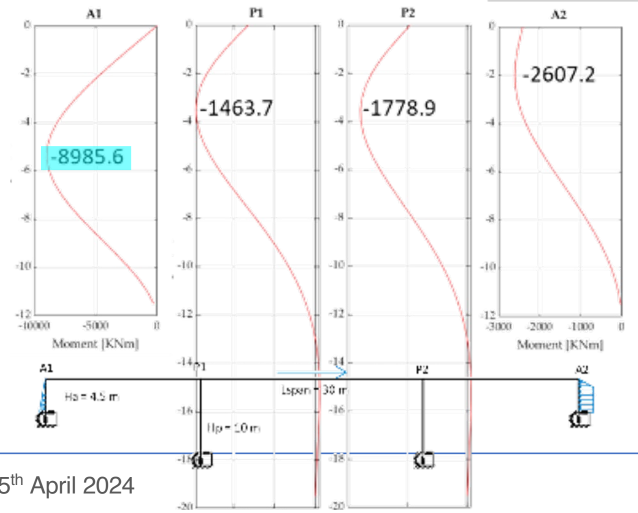
7.7 Transversely loaded piles

7.7.1 General

(3) One of the following failure mechanisms should be considered:
— for short piles, rotation or translation as a rigid body;
— for long slender piles, bending failure of the pile, accompanied by local yielding and displacement of the soil near the top of the pile.

7.7.3 Transverse load resistance from ground test results and pile strength parameters

(3) The calculation of the transverse resistance of a long slender pile may be carried out using the theory of a beam loaded at the top and supported by a deformable medium characterised by a horizontal modulus of subgrade reaction.



Credits: Tomic, M. 2024

10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (8/9)

- Static impedances = springs
- Transverse directon: in general separate model and even RSA (note that RSA not considered for longitudinal direction)
    - Warning: some abutment types can be very flexible, assuming them rigid will decrease transverse deflections/internal actions
- Skew bridge = spatial model… easier said than done, more easily implemented with nonlinear analysis methods (DBA)

10.3.2 Force-based approach

(7) The model should account for the effect of the flexibility of the abutment and piers foundation.

(8) The effect of foundation flexibility may be accounted for through static foundation impedances according to FprEN 1998-5:2024, Clause 8. Group effects may be taken equal to their static values.

NOTE FprEN 1998-5:2024, Annex D, gives guidance for calculating foundation impedances of both shallow and deep foundations.

(10) In the transverse direction, analysis may be carried out with any of the methods in 5.2, with due consideration of the deck restraint at the abutments.

(11) For determining the soil-abutment stiffness at the deck-abutment connection, the abutment wall may be considered rigid to the foundation level with flexibility contributed only by the foundation.

(9) If the bridge is skew ($\varphi > 20°$), response in the transverse direction should be obtained from the same spatial model used for the longitudinal response. For smaller skew angles and straight bridges, separate models may be used.

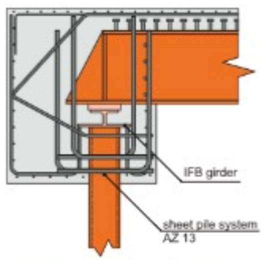


Figure 9-7: Sheet pile connection, type "INTAB"

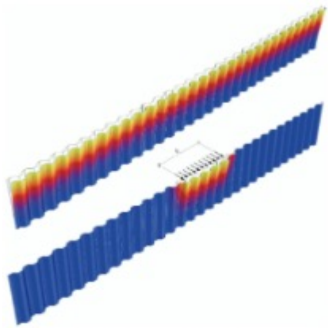


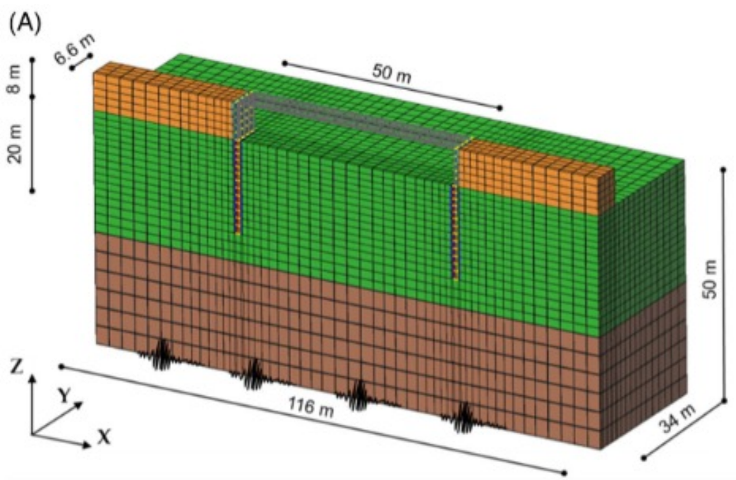
Figure 9-8: Sheet pile deformation

Feldmann, M., J. Naumes, D. Pak, M. Veljkovic, J. Eriksen, O. Hechler, N. Popa, G. Seidl, and A. Braun. 2010. Design guide - Economic and Durable Design of Composite Bridges with Integral Abutments. CEN/TC 250/SC 10 N 0216.

10. INTEGRAL ABUTMENT BRIDGES

10.3 Modelling and analysis (9/9)

- DBA = nonlinear analysis
- NL-RHA: model should include everything
  - Complete SSI, refer to EN1998-5



10.3.3 Displacement-based approach

(1) The displacement-based approach should be implemented by either a) or b):

a) non-linear static analysis;

b) response-history analysis.

(2) For the purpose of the displacement-based approach, the soil should be modelled as a discretised inelastic continuum.

(4) For response-history analysis, the model should include the entire soil-foundation-structure system. The analysis model should allow for the transmission of seismic waves across the lateral and bottom boundaries of the system, according to FprEN 1998-5:2024, 8.5(2).

NOTE 1 In (4), soil means the natural soil deposit beneath the structure, as well as the backfill material and soil, natural or embankment, beside the abutments.

NOTE 2 Informative Annex D also provides guidance on this aspect.

Marchi, A., D. Gallese, D. N. Gorini, P. Franchin, and L. Callisto. 2023. “On the seismic performance of straight integral abutment bridges: From advanced numerical modelling to a practice-oriented analysis method.” Earthq. Eng. Struct. Dyn., 52 (1): 164–182. Wiley.

D. DBA for IABs

D.2 Scope & field of application

- No discretised inelastic continuum? Winkler approach then
- Relevant material in Informative Annex D

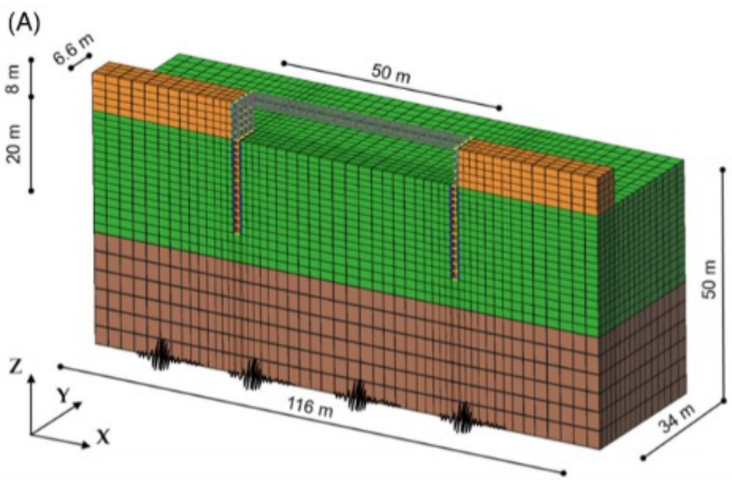
10.3.3 Displacement-based approach

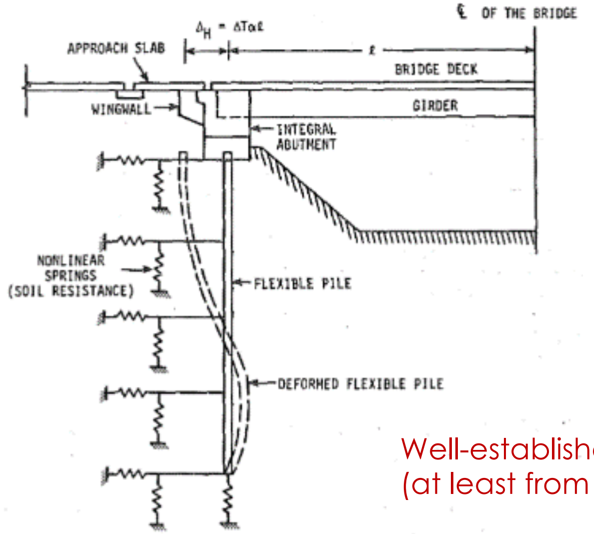
(3) If (2) is not applied, mutually independent inelastic springs may be used to model the soil in contact with the abutment walls.

NOTE Informative Annex D provides guidance on this aspect.

D.2 Scope and field of application

(1) This Informative Annex provides indications on modelling of soil in contact with the abutment walls through mutually independent inelastic springs and other aspects related to non-linear static and response-history analysis for integral abutment bridges.





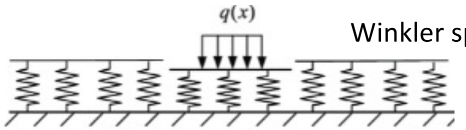
Well-established technique
(at least from the '80s)

Greimann, L. F., P.-S. Yang, and A. M. Wolde-Tinsae. 1986. "Nonlinear analysis of integral abutment bridges." Journal of Structural Engineering, 112: 2263–2280.

D. DBA for IABs

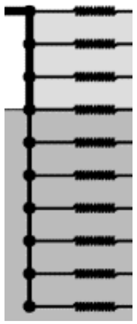
D.3 Modelling for NL analysis

- Common case, foundations:
  horizontal distribution of costant springs



Winkler spring k_z

- Earth-retaining structures (and abutments): vertical distribution of depth-dependent springs



Winkler spring $k_x(z)$

(1) If springs are used according to 10.3.3(3), they should describe a depth-dependent non-linear pressure-deflection ($\sigma - \delta$) relation between the active σ_a and passive σ_p resistance limits (FprEN 1997-1:2024, 9.5.4), in the seismic design situation, according to FprEN 1998-5:2024, Clause 10.

NOTE 1 Mutually independent springs can be used to represent vertical or horizontal reaction of the soil. The former case is of interest when they model soil reaction along the horizontal contact surface of a shallow foundation. In the context of integral abutment bridges, and more in general of retaining structures, springs represent soil reaction along vertical contact surfaces. In the latter case, if the soil is granular, its stiffness and strength vary with depth, along with vertical stress.

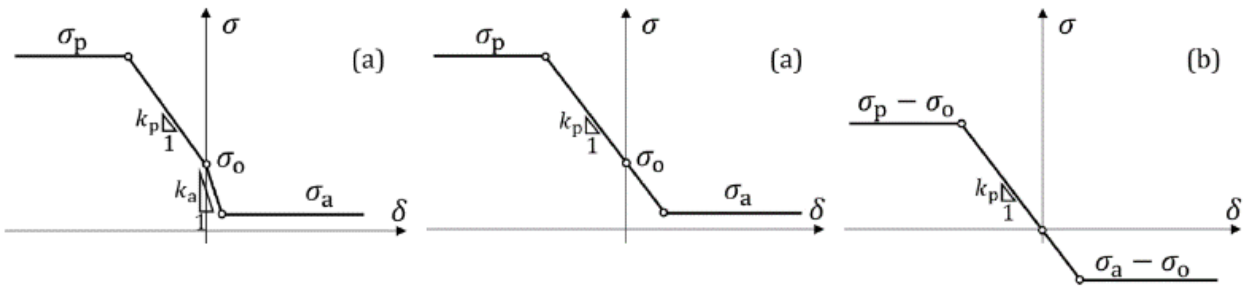
NOTE 2 FprEN 1998-5:2024, Annex F, F.3, gives guidance for calculating active and passive earth pressures in the seismic design situation.

(2) The constitutive law of springs should be composed of at least four linear branches: one elastic, from the initial pressure σ_0 to passive resistance σ_p, one elastic from the initial pressure to the active resistance σ_a, and two horizontal branches at the active and passive resistance levels (Figure D.1a).

(7) As an approximation, trilinear springs with a single elastic branch of stiffness k_p may be used (Figure D.2a).

(8) As an approximation, non-symmetric tension-compression springs may be used if the 'at-rest' pressures are applied as a force distribution on the abutment back-walls (Figure D.2b).

Minimum four… …but then again three… …even tension-compression

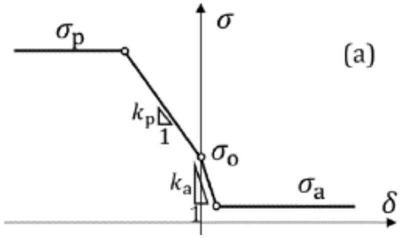


Becci, B., and R. Nova. 1987. "Un metodo di calcolo automatico per il progetto di paratie." Rivista Italiana di Geotecnica.

D. DBA for IABs

D.3 Modelling for NL analysis

- **At-rest pressure σ_o** possibly increased
  ←10.2(3): 'Calculation of the effects of the seismic action may incorporate the effects of the construction sequence and thermal cycling'
- **Stiffnesses** given in Annex D
- **Strength** from EN1998-5, Annex F
  ←D.3(1) NOTE 2: 'FprEN1998-5, Annex F,F.3 gives guidance for calculating active and passive earth pressures in the seismic design situation'
  - $\sigma_a = K_{AE\gamma}(\sigma_v - u) + K_{AEq}q - K_{AEc}c + u$
  - $\sigma_p = K_{PE\gamma}(\sigma_v - u) + K_{PEq}q + K_{PEc}c + u$
  - in practice $\sigma_a = K_{AE\gamma}\sigma_v$ and $\sigma_p = K_{PE\gamma}\sigma_v$



(3) Initial pressures may be assigned values different from 'at-rest' pressures, due to preloading, according to 10.2(3).

(4) The secant stiffness for the active-side pressure may be calculated using Formula (D.1).

$$k_a(z) = \frac{E_s(z)A_{co}(z)}{L_a} \quad \text{(D.1)}$$

A_{co} = contact area

$$L_a = \frac{2}{3}min(H_{ab} + D; 2H_{ab})\, tan\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \quad \text{(D.2)}$$

H_{ab} = abutment height

(5) The secant stiffness for the passive-side pressure may be evaluated by Formula (D.3).

$$k_p(z) = \frac{E_s(z)A_{co}(z)}{L_p} \quad \text{(D.3)}$$

where L_p is the characteristic length, measuring the volume of soil involved in the deformation behind the abutment, in passive conditions, which may be calculated using Formula (D.4).

$$L_p = \frac{2}{3}min(D; H_{ab})\, tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \quad \text{(D.4)}$$

(6) The secant stiffness should be evaluated with soil properties compatible with its expected level of deformation. In the absence of more accurate determinations, FprEN 1998-5:2024, Table 6.1, may be used for the ratio of secant to initial soil stiffness.

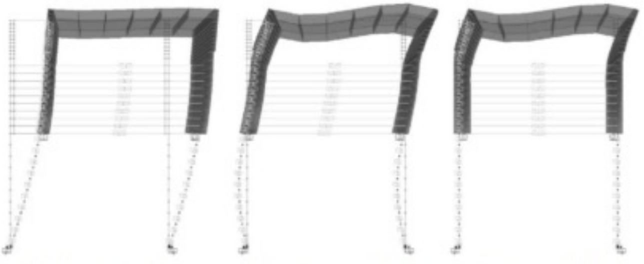
NOTE The ratios of G/G_0 in FprEN 1998-5:2024, Table 6.1, apply also to E/E_0.

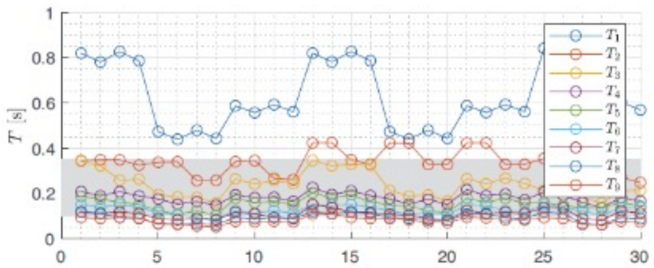
Becci, B., and R. Nova. 1987. "Un metodo di calcolo automatico per il progetto di paratie." Rivista Italiana di Geotecnica.

D. DBA for IABs

D.4 Nonlinear static analysis

- Lateral forces
- Concurrent displacement profile

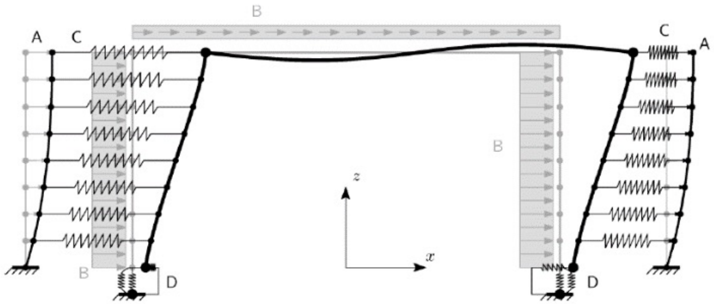




(1) The non-linear static analysis should be carried out by imposing a) and b) (see Figure D.3):

a) the free-field displacement profile at the soil-end of the springs on both abutments of the bridge;

b) equivalent lateral forces on the structure according to 10.3.2(2)b).



(2) If the integral abutment bridge is above-ground and in contact with approach embankments, the free-field displacements δ_{ff} should be taken equal to as given in Formula (D.5).

$$\delta_{\mathrm{ff}}(z) = S_{\mathrm{De}}(T_{\mathrm{emb}})\boldsymbol{\phi}(z) \tag{D.5}$$

(3) If a more refined evaluation is not carried out, the embankment fundamental period in the bridge longitudinal direction may be evaluated by Formula (D.6), and a quarter-sine wave may be used as first-mode shape.

$$T_{\mathrm{emb}} = 4H_{\mathrm{emb}}\sqrt{\frac{\rho_{\mathrm{emb}}}{G_{\mathrm{emb}}}} \tag{D.6}$$

(5) If the integral abutment bridge is embedded, the free-field displacements should be taken as a linear profile with maximum value given by Formula (D.7).

$$\delta_{\mathrm{ff}}(z=0) = \frac{PGV_{\mathrm{e}}}{v_{\mathrm{s,H}}}H_{\mathrm{ab}} \tag{D.7}$$

where PGV_{e} is the design peak value of horizontal ground velocity, as given in FprEN 1998-1-1:2024, 5.2.2.4, for the limit state under consideration.

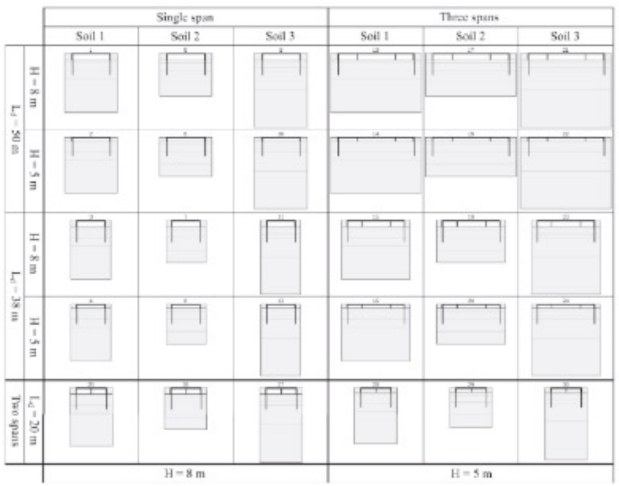
Franchin, P., and P. E. Pinto. 2013. “Performance-based seismic design of integral abutment bridges.” Bull. Earthquake Eng., 12 (2): 939–960. Springer.

D. DBA for IABs

D.4 Nonlinear static analysis

- Validation

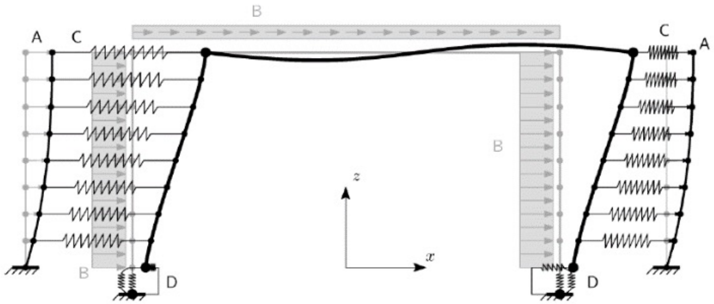
  Same NL dynamic model & parametric study (12+12+6 cases) used for FBA
  Moments @ deck-abutment joint + piles head

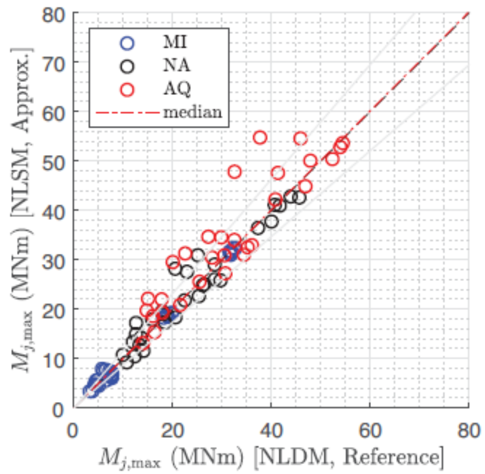


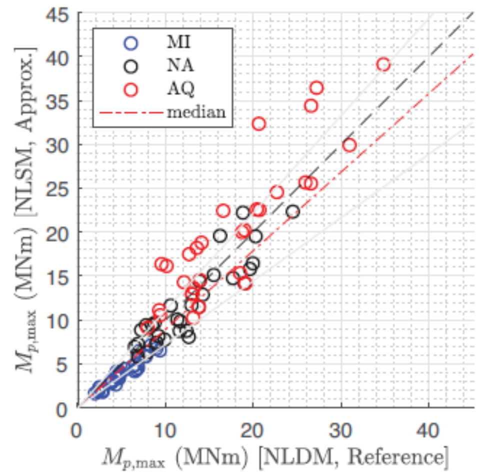
(1) The non-linear static analysis should be carried out by imposing a) and b) (see Figure D.3):

a) the free-field displacement profile at the soil-end of the springs on both abutments of the bridge;

b) equivalent lateral forces on the structure according to 10.3.2(2)b).





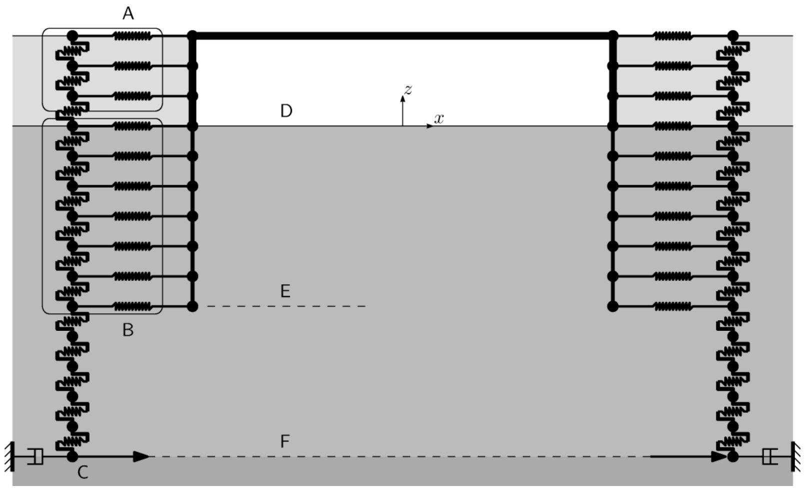


Marchi, A., and P. Franchin. 2023. "Equivalent static methods for seismic design of straight integral abutment bridges." Earthq. Eng. Struct. Dyn. Wiley. https://doi.org/10.1002/eqe.4052.

D. DBA for IABs

D.5 NL response-history analysis

- Text



(1) FprEN 1998-1-1:2024, 5.2.3.1, 6.6 and D.3 should be applied. Spectral compatibility should be checked as for site-specific seismic soil amplification and geotechnical analyses (FprEN 1998-1-1:2024, D.3(2)).

(2) 10.3.3(4) should be applied.

(3) If mutually independent non-linear springs are used, a) to c) should be applied:

a) for the soil-abutment interface, D.3 should be applied;

b) soil springs on the foundation members should comply with FprEN 1998-5:2024, 8.3(2);

c) the seismic action should be applied by exciting a one-dimensional soil column connected to the soil-side of the above springs (Figure D.4), according to FprEN 1998-5:2024, 8.3(5). As an alternative, if the soil is not included in the model, seismic action may be applied as displacement time-series at the soil-side of Winkler springs, calculated by one-dimensional soil response analysis according to FprEN 1998-5:2024, 8.3(4).

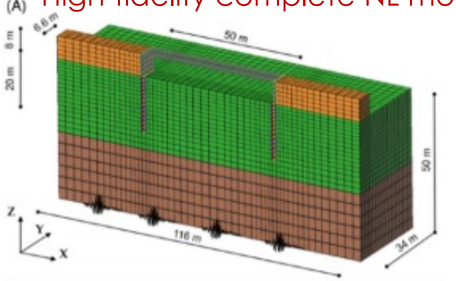
NOTE A one-dimensional soil column is a discrete shear-type (multi-degree of freedom mass-spring) model of a soil deposit commonly used for one-dimensional site response analysis.

Marchi, A., D. Gallese, D. N. Gorini, P. Franchin, and L. Callisto. 2023. "On the seismic performance of straight integral abutment bridges: From advanced numerical modelling to a practice-oriented analysis method." Earthq. Eng. Struct. Dyn., 52 (1): 164–182. Wiley.

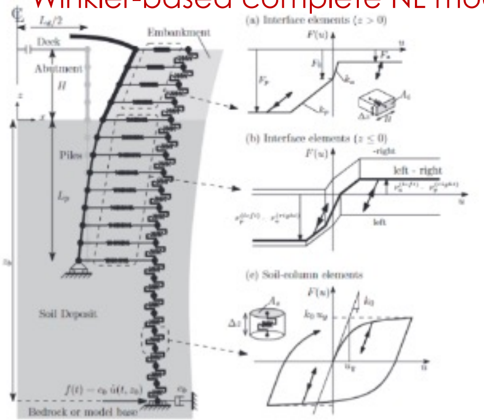
D. DBA for IABs

D.5 NL response-history analysis

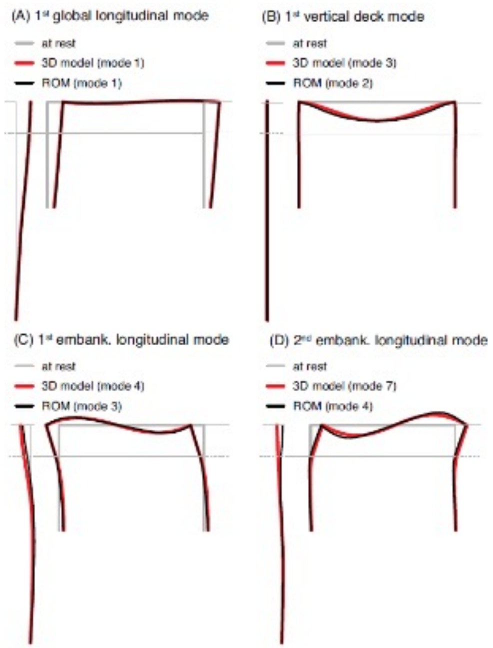
High-fidelity complete NL model



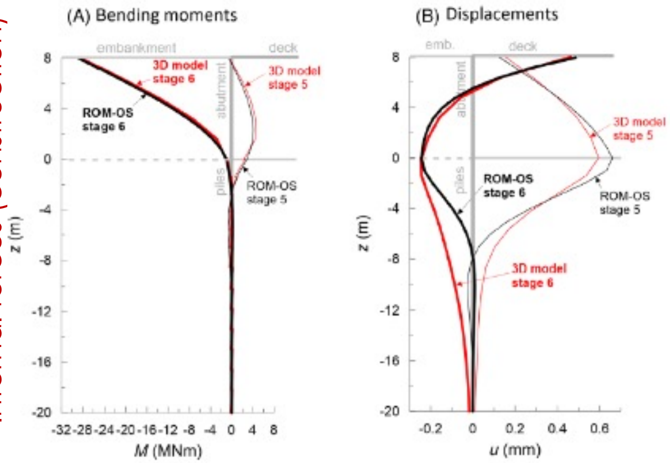
Winkler-based complete NL model

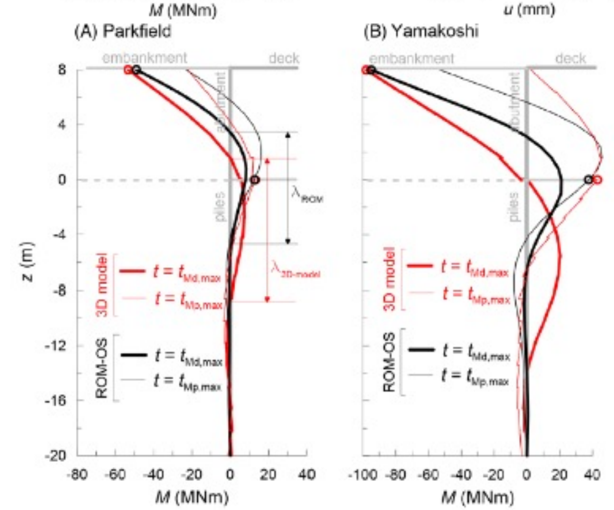


Modal properties



Internal forces (construction)





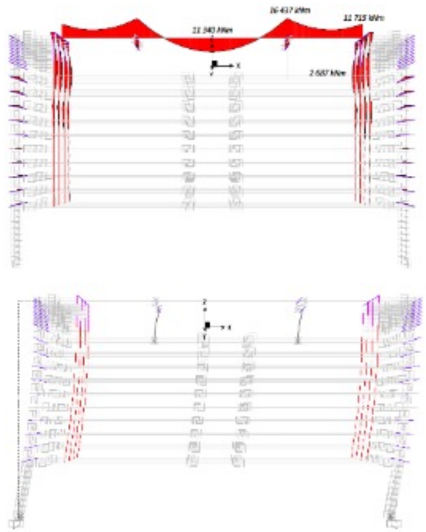
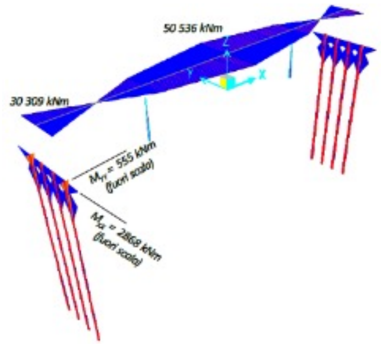
Internal forces (seismic)

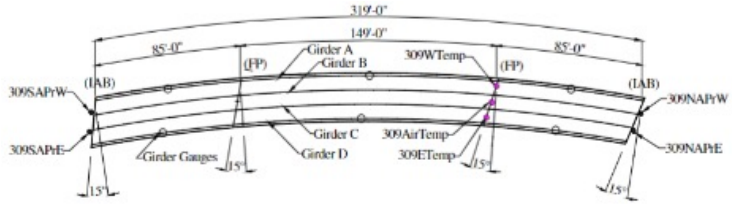
Marchi, A., D. Gallese, D. N. Gorini, P. Franchin, and L. Callisto. 2023. “On the seismic performance of straight integral abutment bridges: From advanced numerical modelling to a practice-oriented analysis method.” Earthq. Eng. Struct. Dyn., 52 (1): 164–182. Wiley.

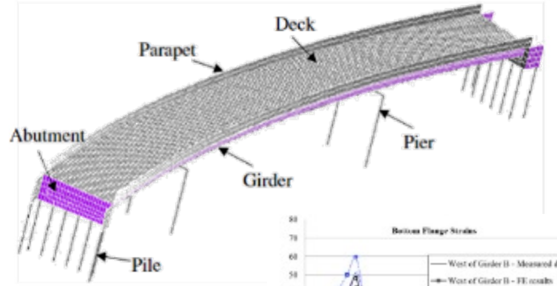
D. DBA for IABs

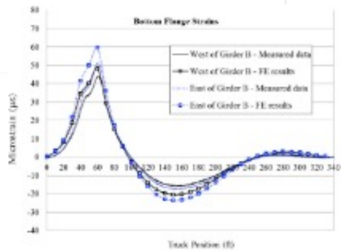
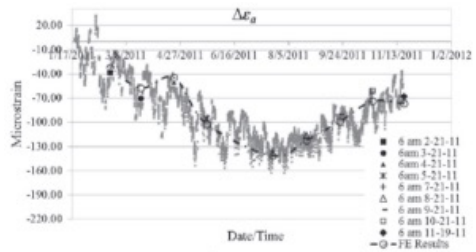
D.5 NL response-history analysis

- Single three-dimensional model for longitudinal and transverse seismic action analysis (mandatory for curved and/or skew bridges)
- Well-established, shown to be reliable for temperature and live (traffic) loads





Deng, Y., B. M. Phares, L. Greimann, G. L. Shryack, and J. J. Hoffman. 2015. “Behavior of curved and skewed bridges with integral abutments.” J. Constr. Steel Res., 109: 115–136.

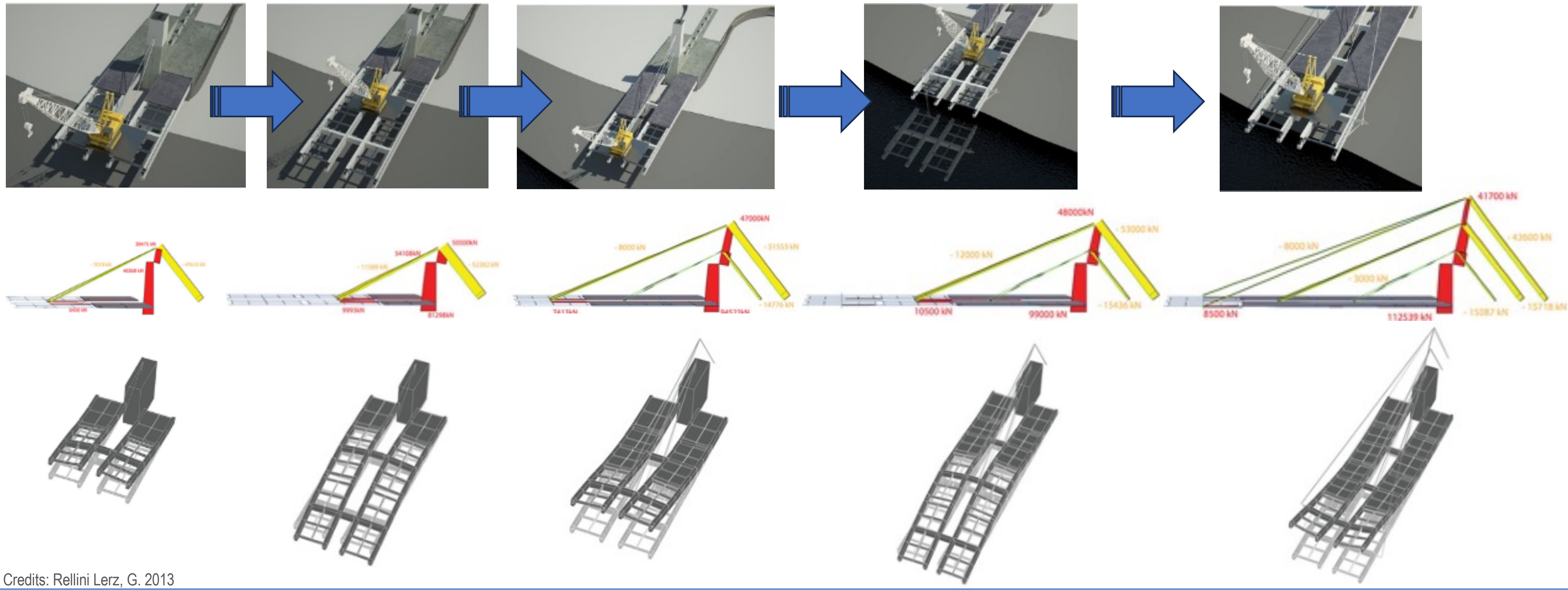
Cable-stayed and extradosed bridges

Force-based approach (linear analysis)
Displacement-based approach (nonlinear analysis)

9. CABLE-STAYED & EXTRADOSED BRIDGES

9.2 Basis of design

(1) The calculation of the action effects in the seismic design situation should consider the influence of the construction sequence on the effects of permanent actions.



Credits: Rellini Lerz, G. 2013

9. CABLE-STAYED & EXTRADOSED BRIDGES

9.3 Modelling & analysis

- Strong statement: RHA recommended (only exception, permission for multimode equivalent RSA in low s.a.c.)
- Global 3D model recommended

(1) Response-history analysis should be the preferred method of analysis for cable-stayed bridges. The dynamic analysis should start from the deformed configuration of the bridge under the permanent actions.

Effect of construction sequence (see 9.2)

NOTE The seismic response of cable-stayed bridges can present significant material and/or geometric non-linearities due to non-linear response of the cables, second-order effects in the deck and the pylons, and large displacements.

(2) In low seismic action class, multi-mode equivalent linear response spectrum analysis may be used for cable-stayed bridges without antiseismic devices.

(3) The modelling of the bridge should reflect with sufficient accuracy the coupling between the transverse bending of the deck and its torsional response.

NOTE This coupling is governed by the distribution of mass and stiffness in the deck as well as the cable arrangement.

(4) Second-order effects should be taken into account in the calculation when they are relevant due to slenderness of the deck and/or the pylons, according to 5.1.3.

(5) A global three-dimensional model should be used to capture the flexural-torsional coupling as well as the geometric non-linearity of the cable elements, pylons and deck.

9. CABLE-STAYED & EXTRADOSED BRIDGES

9.3 Modelling & analysis

- Further reason for NL-RHA: Damping is non proportional!
  - Damping of cables easily < 5%
  - Antiseismic devices often employed (with damping >5%)
  - Large forces trasnferred to foundations → radiation damping (at least impedances)

(6) The stay cable internal damping coefficient should be consistent with the calculated cable displacement.

NOTE The total damping depends on the relative contribution of each member (pylons, cable-system and deck), and their interaction, and can be significantly lower than 5 % of the critical damping.

(7) Energy dissipation of antiseismic devices located at the deck-pylon interface or at the cables should be considered explicitly in the analysis by their non-linear response.

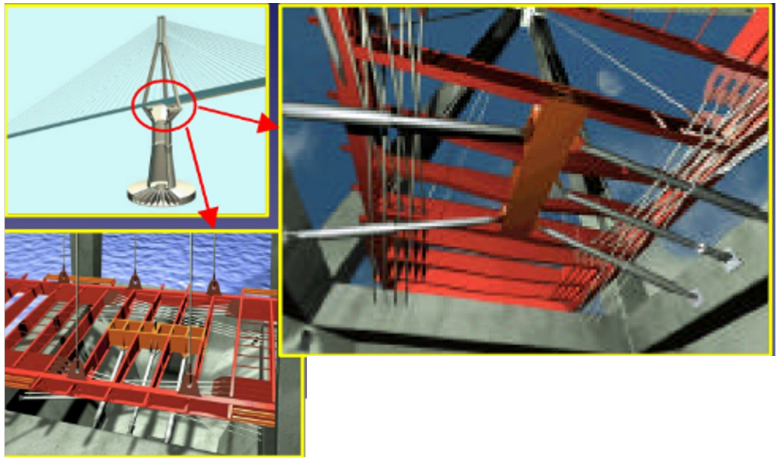
(8) If abutments' and piers' foundations are not included in the model, the model should account for the effect of their flexibility through foundation impedances, according to FprEN 1998-5:2024, Clause 8.

NOTE FprEN 1998-5:2024, informative Annex D, gives guidance for calculating foundation impedances.

9. CABLE-STAYED & EXTRADOSED BRIDGES

9.4 Verifications

- Performance requirement: except for antiseismic devices, elastic!
- Avoid deck-pylon impact!
- ΔN



Rion-Antirion Main bridge deck damping @ Pylon with transverse viscous dampers, Greece

9.4.1 General

(1) In cable-stayed bridges, all the components except non-linear antiseismic devices should remain within the elastic range in the seismic design situation.

(2) In any horizontal direction, the displacement of the deck should be limited to avoid impact between deck and pylon.

(3) Verification of displacement compatibility should take into account all potential aggravating effects such as second-order effects, contribution of higher modes or spatial variability of seismic demand (including active fault crossing).

(4) In multi-leg pylons, the additional axial load due to seismic response should be considered at each individual leg.

9.4.2 Avoidance of brittle failure of specific non-ductile components

(1) Non-ductile structural components, such as fixed bearings, sockets and anchorages for cables and stays, and other non-ductile connections, should be designed to resist capacity design effects. These capacity design effects should be taken equal to the minimum of a) and b):

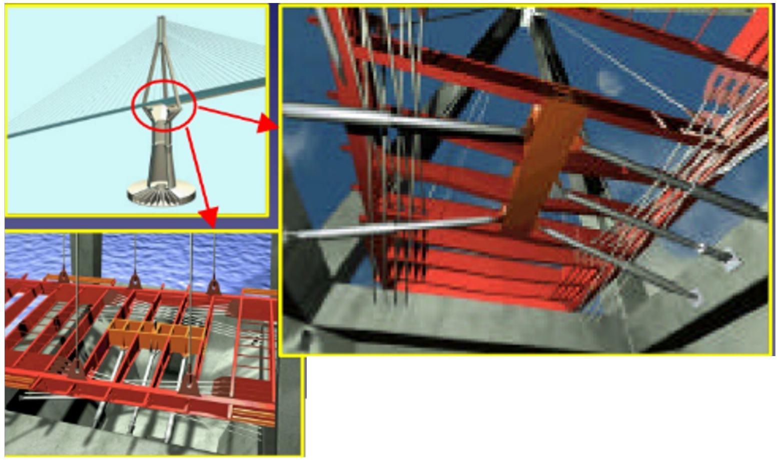
a) those obtained in the seismic design situation with $q = 1$;

b) those obtained in the assumption that the relevant ductile members (e.g. the cables) have developed their strength, multiplied by an overstrength factor $\gamma_{Rd} \geq 1{,}3$.

9. CABLE-STAYED & EXTRADOSED BRIDGES

9.5 Detailing

- Continuous deck!
- Devices @ deck-pylon interface, for horizontal deck restraint (cables ineffective)



Rion-Antirion Main bridge deck damping @ Pylon with transverse viscous dampers, Greece

(1) In cable-stayed bridges, the deck should be continuous.

(2) Antiseismic devices may be used at the deck-pylon interface or at the deck-abutment interface in order to provide restraints and/or energy dissipation.

NOTE Other layouts are possible. For instance, special seismic cable damping devices can be used.

(3) Horizontal deck restraint in the transverse direction should be provided at the deck-pylon interface and/or at the abutments.

NOTE 1 In the transverse direction, the resistance is provided mainly by the deck-tower interface since the cables provide little restraint to deck movements. In the longitudinal direction, the resistance is provided by both the cable-pylon system and the deck-pylon interface, if any.

NOTE 2 The cables can be either connected to the pylon top (fan arrangement) or distributed over the height in a harp or semi-fan type of arrangement. Distributed type of arrangements provide a stiffer solution than a fan arrangement.

(4) Vertical restraint of the deck at the deck-to-pylon interface may be used.

Questions & answers

Webinar 2.5 Integral abutment & cable-stayed bridges