Substructure

(Unterbau)

Substructure – General remarks

The bridge substructure usually comprises:

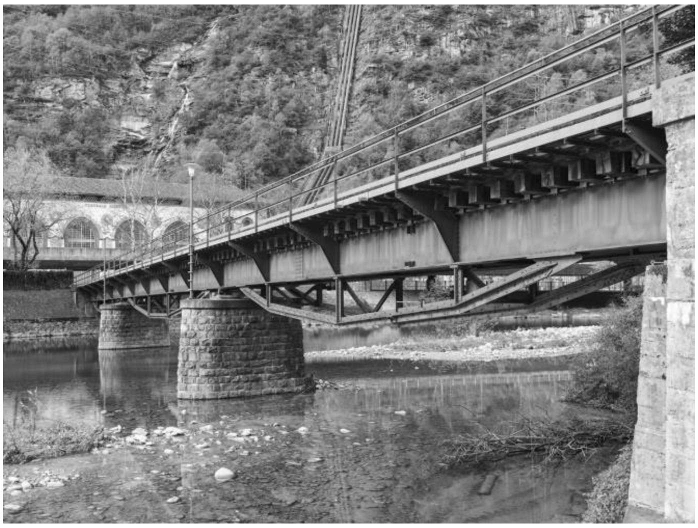
- abutments
- piers
- foundations

The piers, abutments and foundations are virtually always made from concrete (even in a “steel bridge” or “timber bridge”.

Particularly for foundations and other elements in contact with backfill or water, hardly any economical and durable alternatives to concrete exist.

Exceptions are steel piles (H-profiles) and sheet piles, which are sometimes used in abutments or foundations, and reinforced earth abutment walls → examples at end of chapter.

Stone masonry was used before concrete and would still be a viable solution in many cases, but is usually only used in the rehabilitation of existing structures for economic reasons.



Example: Steel superstructure on stone masonry piers, concrete abutments (Ticino Bridge Atel, photo © Georg Aerni)

Note that in frame bridges, the abutment walls are also part of the superstructure.

Photo: Tessinbrücke Atel, © Georg Aerni.

Abutments

1. Nomenclature and function
2. Classification / examples
3. Design
4. Dimensioning

Piers

1. General remarks and aesthetics
2. Construction
3. Design

Foundations

1. Types
2. Selected design aspects
3. Scour protection

Substructure

Abutments – Nomenclature and function

Substructure – Abutments: Nomenclature and function

SBB Rupperswil-Lenzburg, Brücke im Rohbau (1982) Photo © Ruth Hintermann

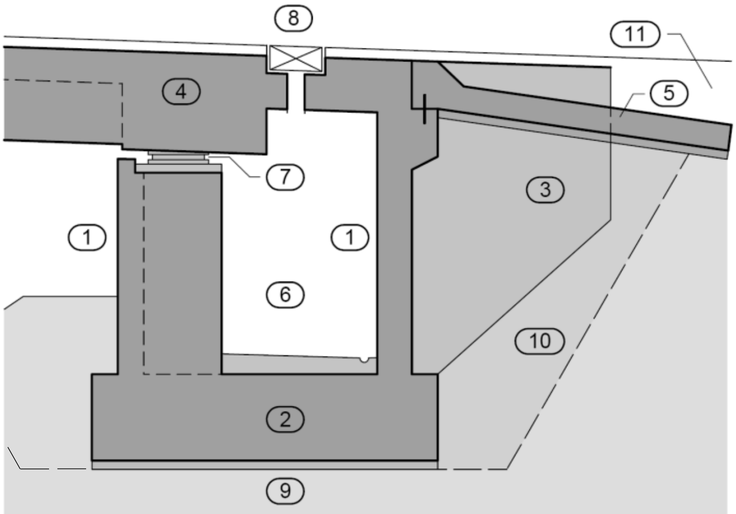
Substructure – Abutments: Nomenclature and function

Bridge ends have to ensure the structural safety and serviceability of the bridge, the adjoining roadway or railway tracks, and the transition between them.

They consist of the following components:

1. abutment walls	= Widerlagerwände
2. foundation	= Fundament
3. wing walls	= Flügelmauern
4. end diaphragm	= Endquerträger
5. transition slab	= Schleppplatte
6. access chamber	= Unterhaltsraum
7. bearings	= Lager
8. expansion joint	= Fahrbahnübergang
9. subsoil	= Baugrund
10. backfill	= Hinterfüllung
11. adjoining road	= angrenzende Strasse

The structural components of the bridge end are usually made from concrete (cast in place) and referred to as abutment = Widerlager

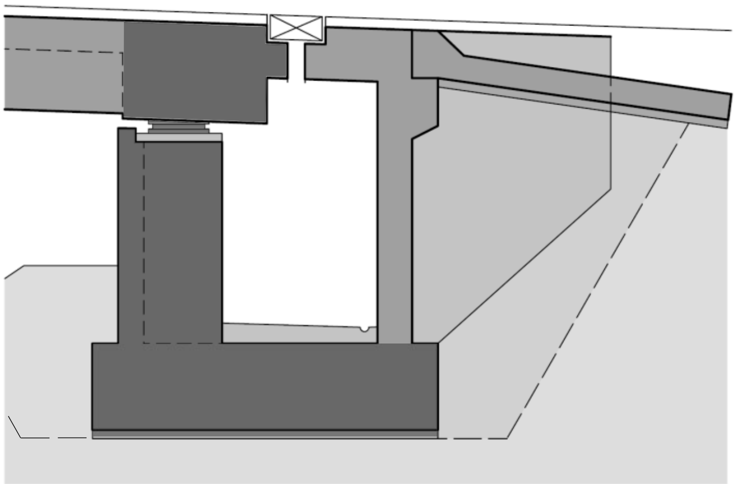


Note: The front abutment wall, carrying the bearing seat, is often referred to as “stem”. The transition slabs are also referred to as “approach slabs.”

Substructure – Abutments: Nomenclature and function

Abutments typically need to comply with all or most of the following functional requirements:

- Resist vertical and horizontal support reactions of bridge deck and transfer them to the subgrade
- Resist actions from adjoining road / rail track (earth pressure, settlements, seepage water)
- Accommodate relative movements between bridge and adjoining road / rail track (temperature, shrinkage, creep, settlements, …)
- Facilitate access for inspections and maintenance (accessibility of expansion joint, bearings, cables and pipes, drainage)
- Stiffen bridge end (limit vertical offsets of deck end and abutment, particularly for cantilevers)
- Facilitate passage of cables and pipes
- Ensure safety of bridge girder from falling in earthquakes
- Provide adequate scour protection (Kolkschutz)



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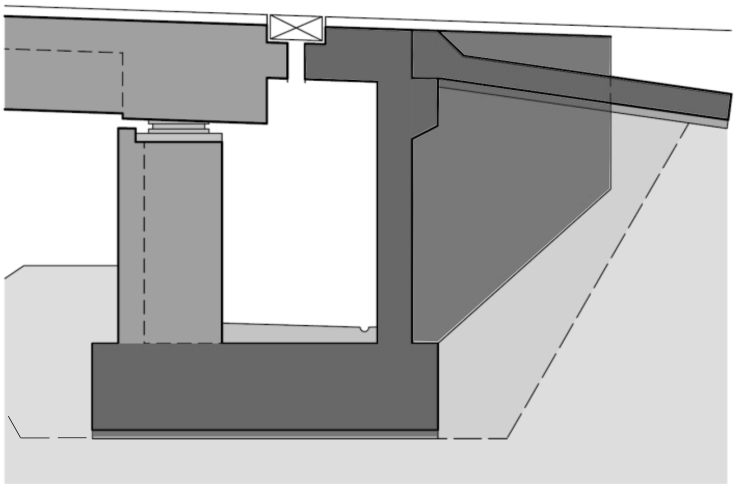

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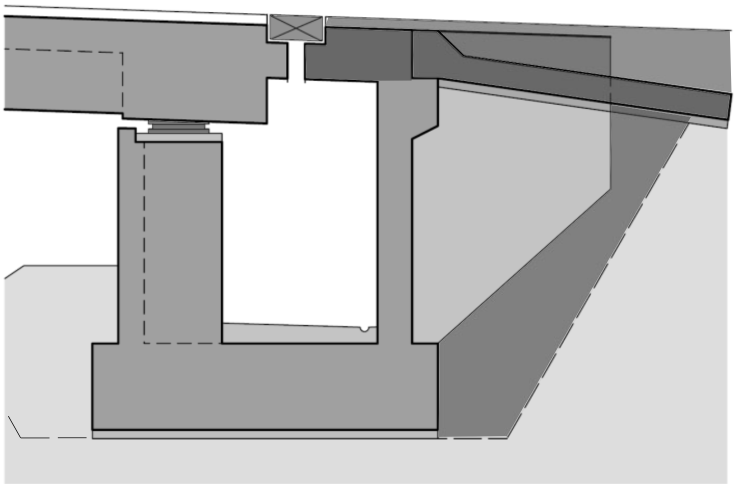

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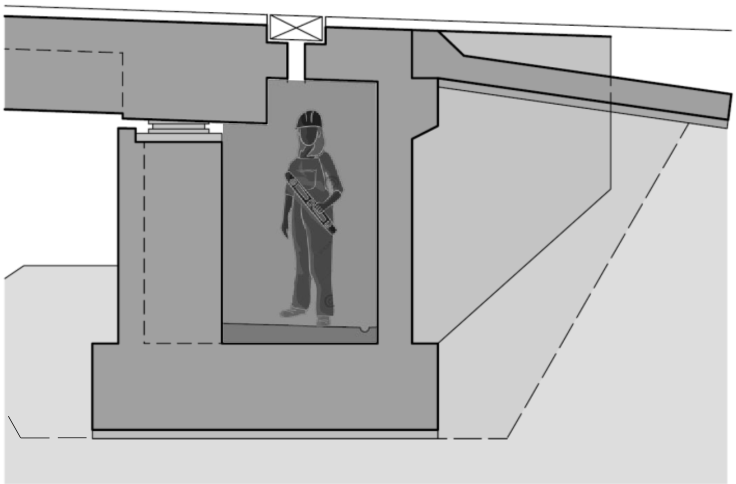

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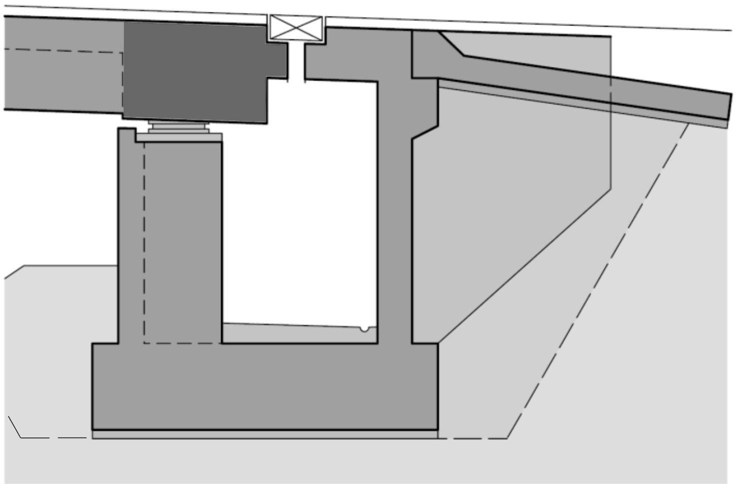

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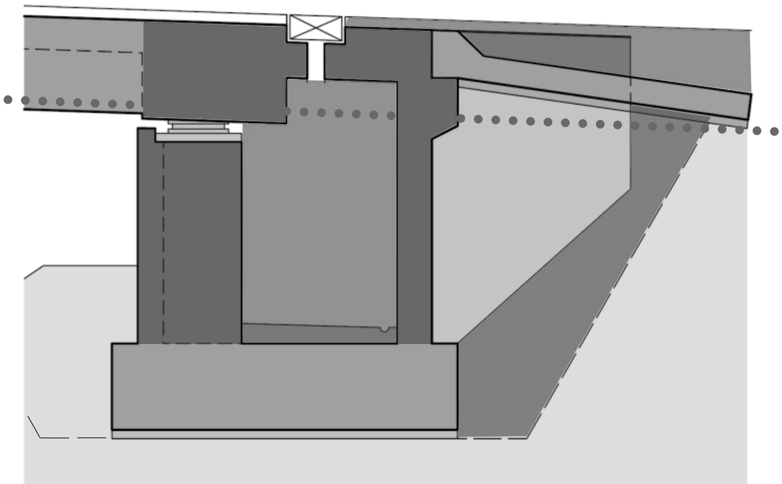

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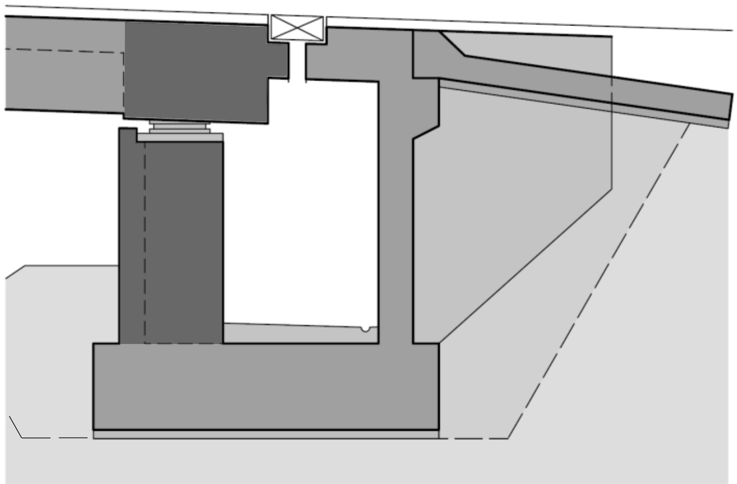

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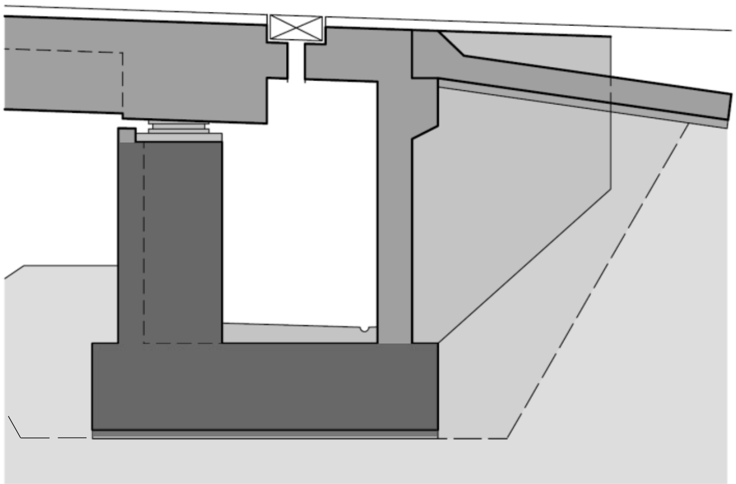


Substructure

Abutments – Classification / Examples

Substructure – Abutments: Classification / Examples

Technically, abutments can be classified depending on their functionality of the bridge end with respect to support and articulation:

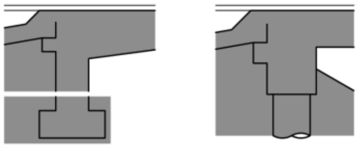
- Integral abutments (without joints nor bearings)
- Semi-integral abutments (bearings, but no joints)
- Jointed abutments (bearings and expansion joint)
  … longitudinally fixed
  … horizontally movable

Apart from this distinction, classifying abutments is difficult since their design differs strongly, depending on the local / regional preferences of clients and designers.

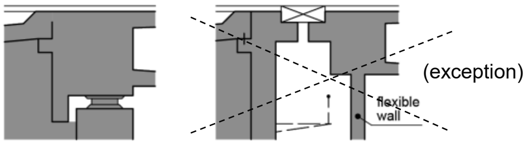
On the following slides, some basic criteria are discussed, and examples illustrating the wide range of alternatives are shown, with emphasis on the Swiss state of practice.

Note that mainly jointed abutments are shown for illustration, but integral abutments are preferred.

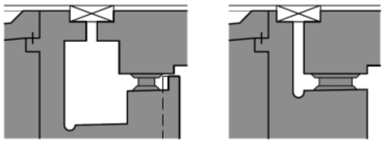
Integral bridge ends (neither expansion joint nor bearing)



Semi-integral bridge ends (bearing only)



Jointed bridge ends (with expansion joint and bearing)



Substructure – Abutments: Classification / Examples

The abutments, forming the structural part of the bridge end, connect the bridge to the adjoining road or railway. Naturally, the design of the abutments is essential for the integration of a bridge in a site.

The following parameters have to be selected in design:

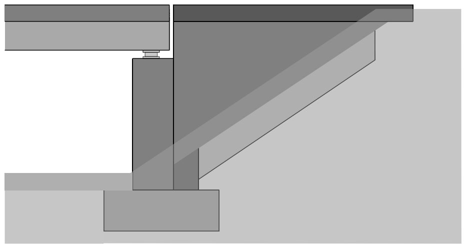
- positioning of abutments in plan
  → length of bridge and embankments, respectively
  → height of abutment (visual impact)
- orientation of the wing walls
  → embankment geometry
- design of abutment itself
  → perception by users

→ decisive for integration and aesthetic quality of a bridge

→ even more pronounced when crossing flat areas (next slides)

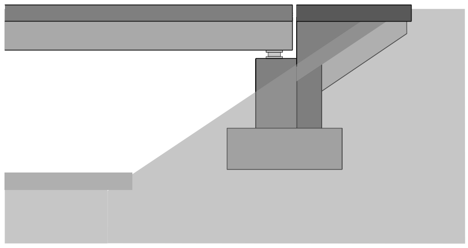
Full height /
High stem abutment:

… short bridge
… high abutment (and approach embankment)
… high visual impact



Stub abutment /
Short stem abutment:

… long bridge
… low abutment (and approach embankment)
… unobtrusive



Note that stub abutments may require a foundation at a lower level or be provided with a pile foundation. In such cases, the abutment walls do not need to be closed in the embankment, but can be divided in separated elements (or simply replaced by the piles). Such abutments are called “open” or “open ended” since there is no closed abutment wall.

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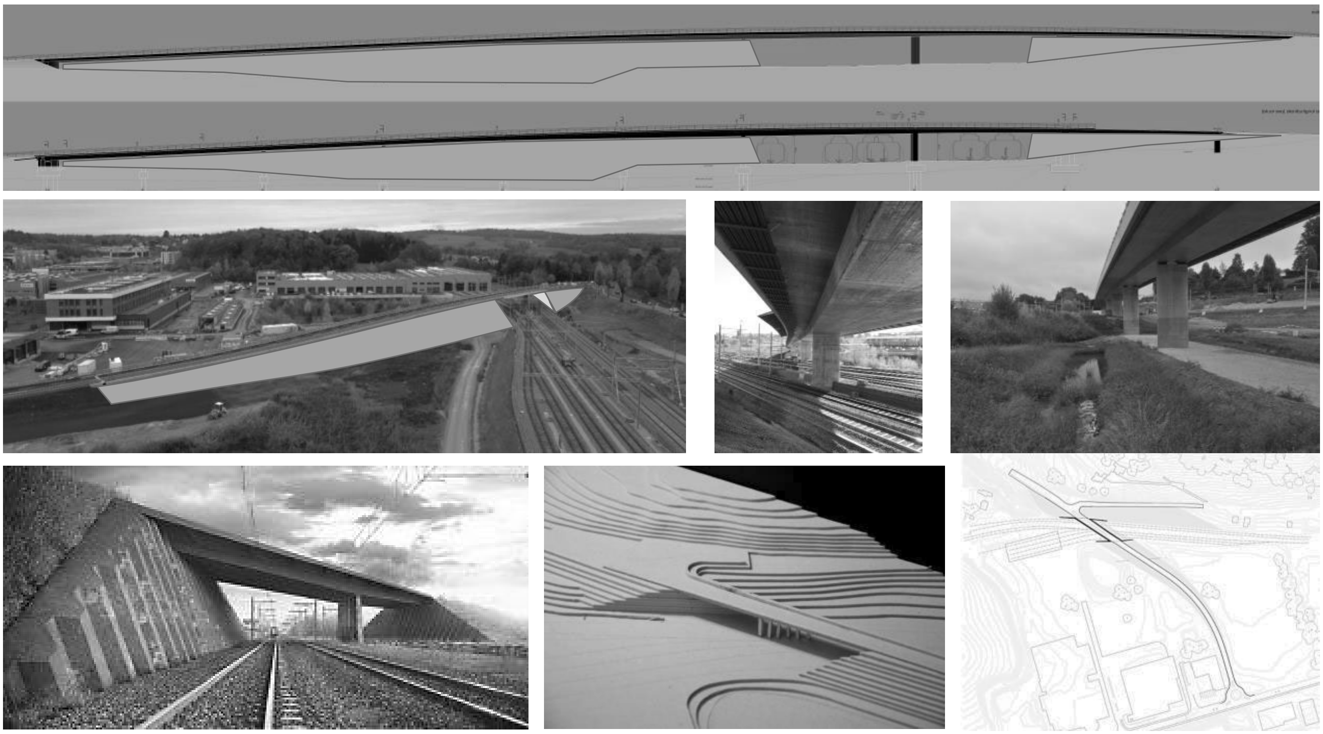
Photos: Taken from FHWA Bridge Inspector's Manual, Section 10.1 - Abutments and Wingwalls

Substructure – Abutments: Classification / Examples

Example: Stub abutment with additional span (compared to alternative with full height abutment)

Full height abutments have a much more dominant visual impact than short stem abutments. In this example, a full prefabricated girder span was added to cover the embankment length, creating an open appearance.

Photo: Crossing of US Interstate 49 (under construction) and LA 2 © https://www.alpsroads.net/roads/la/



The Pont du Tiguelet crosses a plain at low height. Apart from the selection of modest spans, enabling a very slender girder, the choice of the abutment positions was highly relevant. The project winning the design competition opted for a relatively long bridge, while many other projects proposed shorter bridges with longer approach embankments, some of them minimising the bridge length to the functionally required minimum (to cross the railway tracks). Note that in spite of the long bridge that was finally built, the «short» embankments still required soil improvements (vibrated stone columns), and probably, an even longer bridge would have been more economical.

Photos: Top/middle row built project. Long bridge, short embankments © dsp Ingenieure + Planer AG / Spataro Petoud Partner SA.

Bottom: alternative proposal in design competition: short bridge, long embankments © structurame / apaar.ch

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Photos: Top/middle row built project. Long bridge, short embankments © dsp Ingenieure + Planer AG / Spataro Petoud Partner SA.

Bottom: alternative proposal in design competition: short bridge, long embankments © structurame / apaar.ch

Setting back the abutments is not always preferable to full height abutments. The slide illustrates different solutions, including two full height abutments in locations where other solutions, with lower abutments, would at least theoretically have been possible.

Photos (clockwise from top left): Steinbachviadukt / Brücke Schönenwerd / Wildüberführung Halbmil / Pont du Tiguelet, all © dsp Ingenieure + Planer AG

Setting back the abutments is however not always possible, as illustrated in the example.

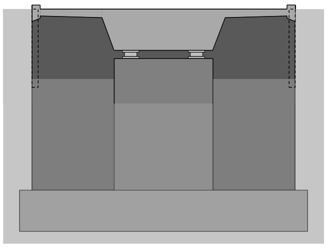
Photos: Regensbergbrücke, Durchmesserlinie Zürich, Los 4.006 (Einschnitt Oerlikon). © dsp Ingenieure + Planer AG

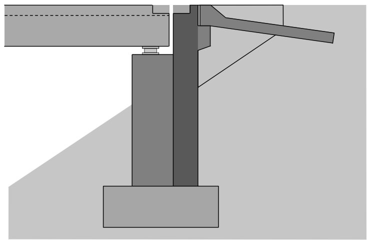
Substructure – Abutments: Classification / Examples

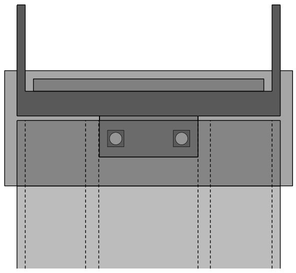
Exposed bearing seat
Without support diaphragm
Without access chamber

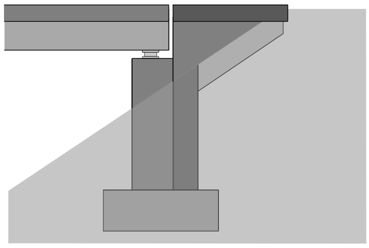
- low initial cost
- minimalist appearance
- uplift may be critical
  (limited separation of bearings)
- inconvenient maintenance
  … bearings accessible via embankment only
- limited durability
  … expansion joint inaccessible
  (leakages may remain undetected)
  … expansion joint in cantilevers subject to vertical offsets due to traffic load

Note: Integral abutments may have equally minimalist designs, without the drawbacks mentioned above.







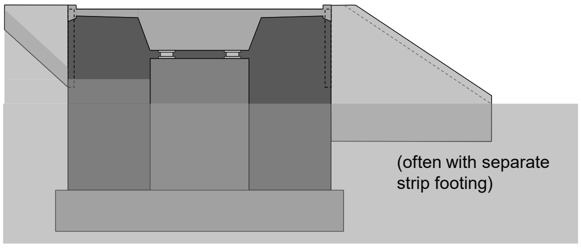


Viadukt Wangen

New access ramp: dsp Ingenieure + Planer AG, Fürst Laffranchi Bauingenieure GmbH

Substructure – Abutments: Classification / Examples

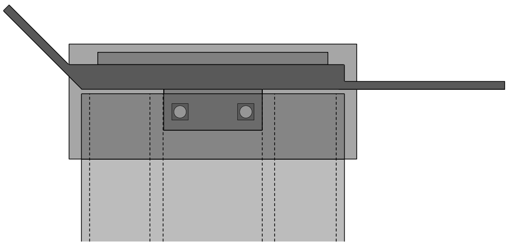
Exposed bearing seat
Without support diaphragm
Without access chamber



This slide shows alternatives with different wing wall orientation. These are

- basically possible in all solutions that follow (illustrated only here for the sake of simplicity)
- should be separated from flexible integral abutments to avoid excessive restraint

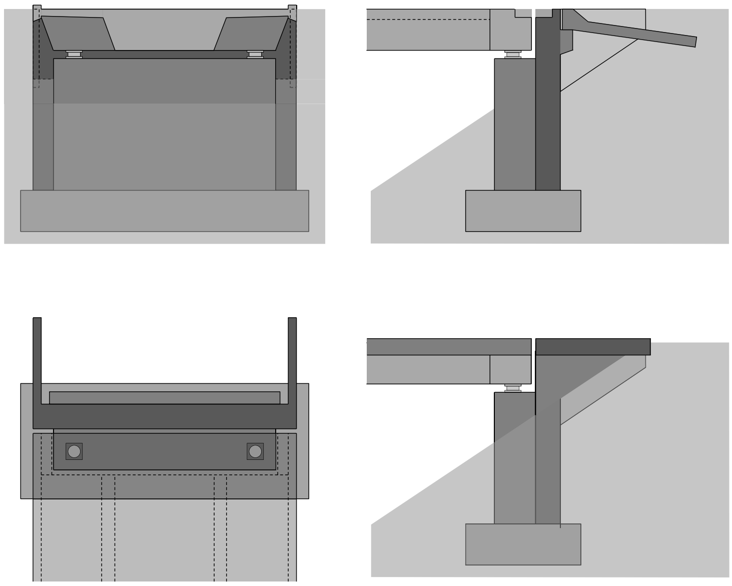
Note that the length of the wing walls depends on the embankment geometry (here, a slope of 2:3 parallel to the wing walls is assumed for simplification). Wing walls and retaining walls should always be designed with some extra length (if they are too long, they will simply be buried – but too short ones require ugly, often improvised measures).



Substructure – Abutments: Classification / Examples

Exposed bearing seat
Exposed support diaphragm
Without access chamber

- ➢ low initial cost
- ➢ uplift hardly critical
  (large separation of bearings)
- ➢ unsatisfactory appearance
  … end diaphragm fully visible
  … wide stem
- ➢ inconvenient maintenance
  … bearings accessible via embankment only
- ➢ limited durability
  … expansion joint inaccessible
  (leakages may remain undetected)



Zufahrtsrampen ETH Hönggerberg (1972)

Photos © M. Lee




Brücke Kleine Melchaa

dsp Ingenieure + Planer AG

Fuss- und Radwegbrücke Effretikon

dsp Ingenieure + Planer AG

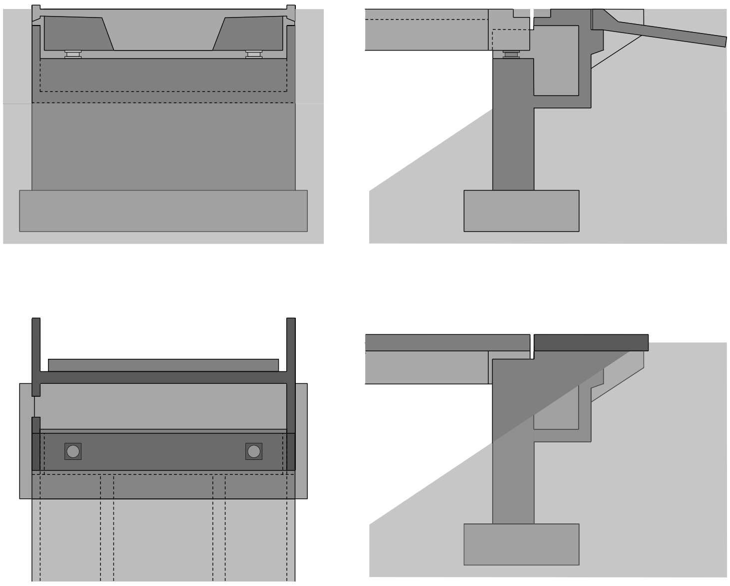
Steinbachviadukt: Abutment

Substructure – Abutments: Classification / Examples

Integrated bearing seat
Partially hidden support diaphragm (cheek walls)
With access chamber

- ➢ high durability
  … expansion joint accessible
  (leakages may be detected)
- ➢ maintenance friendly
  … bearings accessible via chamber
- ➢ uplift hardly critical
  (large separation of bearings)
- ➢ regular appearance
  … end diaphragm and bearings partly visible
  … visible horizontal offset (end diaphragm-abutment wall) due to girder contraction
- ➢ high initial cost

Note: Open abutments can also be built with an access chamber (in the drawing to the right, just the front part of wing walls needs to be removed)



Zufahrtsrampen ETH Hönggerberg (1972)

Photos © M. Lee

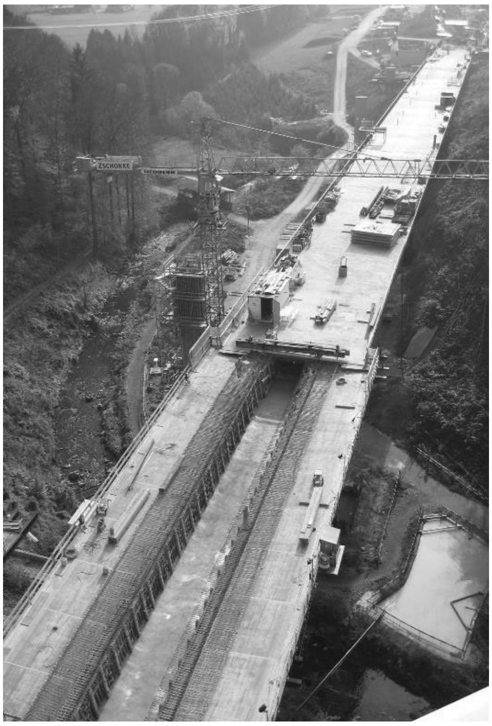
Innbrücke Vulpera



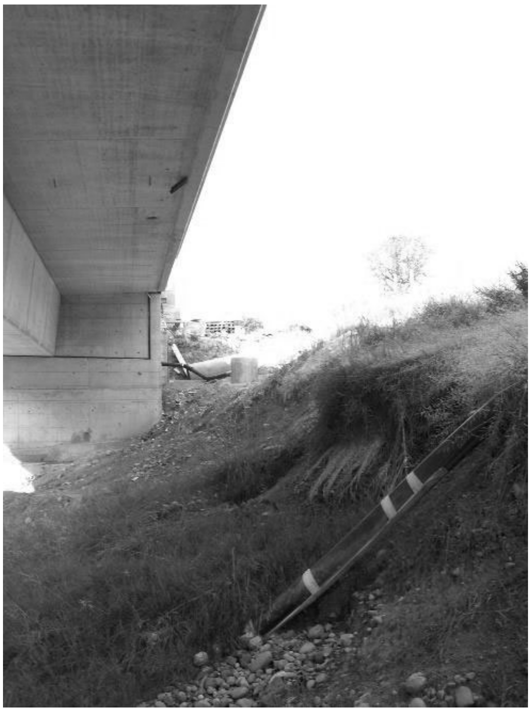


Viadukt Wangen

New access ramp: dsp Ingenieure + Planer AG, Fürst Laffranchi Bauingenieure GmbH



Jonentobelbrücke, dsp Ingenieure+Planer AG, ACS Partner AG

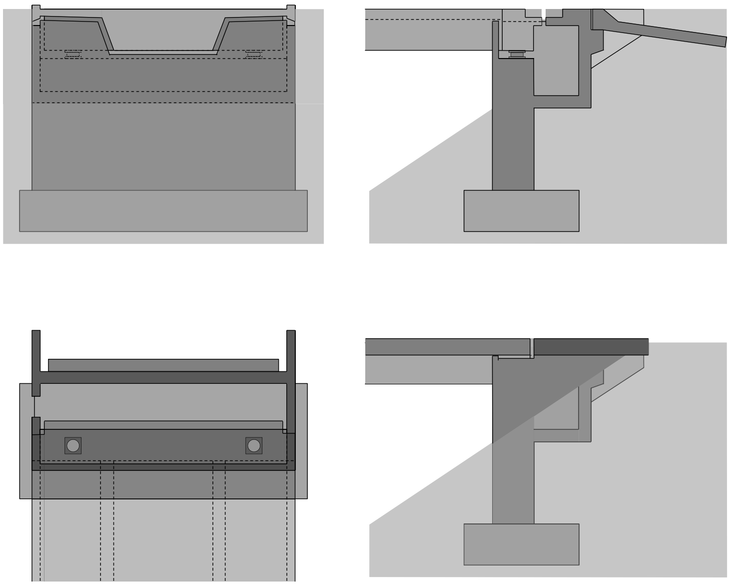


Jonentobelbrücke, dsp Ingenieure+Planer AG, ACS Partner AG

Substructure – Abutments: Classification / Examples

Integrated bearing seat
Hidden support diaphragm
With access chamber

- high durability
  … expansion joint accessible
  (leakages may be detected)
- maintenance friendly
  … bearings accessible via chamber
- uplift hardly critical
  (large separation of bearings)
- clean and tidy appearance
  … end diaphragm and bearings fully hidden
  … horizontal offset due to girder contraction hidden
- high initial cost





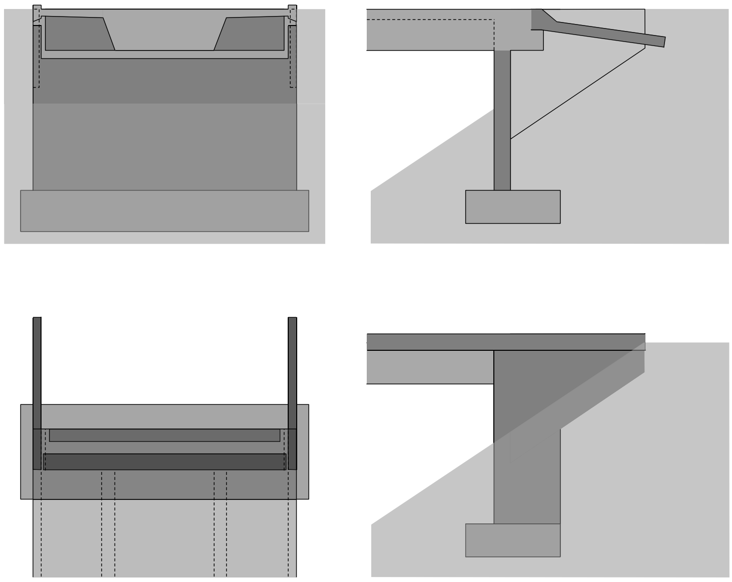

Pont du Tiguelet

dsp Ingenieure + Planer AG, Spataro Petoud Partner SA, Architekt Balz Amrein

Substructure – Abutments: Classification / Examples

Integral abutment (flexible)

- maximum durability
  … neither expansion joint nor bearings
- minimum maintenance
  … neither expansion joint nor bearings
  … pavement cracks may occur
- no uplift problems
  (abutment weight can be activated in case)
- clean and tidy appearance
  … hardly visible transition from bridge to abutment
  (joints between wing walls and front wall only)
  … horizontal offset due to girder contraction
  may become visible



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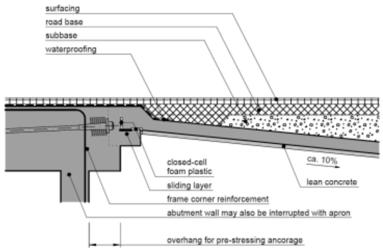
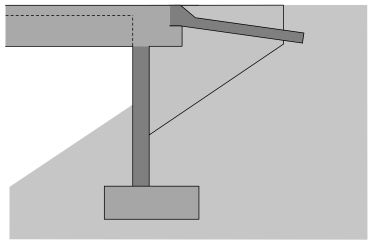
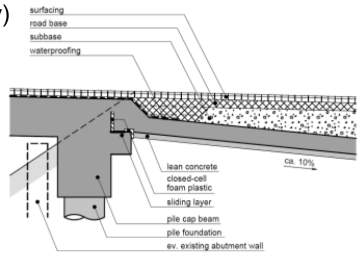
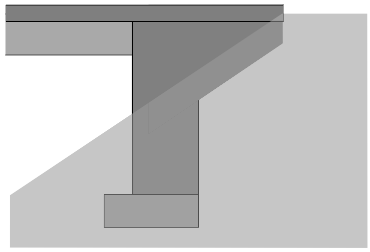








B/W Illustrations: ASTRA Richtlinie 12004

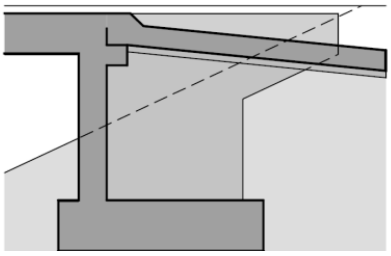
Substructure – Abutments: Classification / Examples

SBB Durchmesserlinie Zürich Oerlikon, Regensbergbrücke Photos © dsp Ingenieure + Planer AG

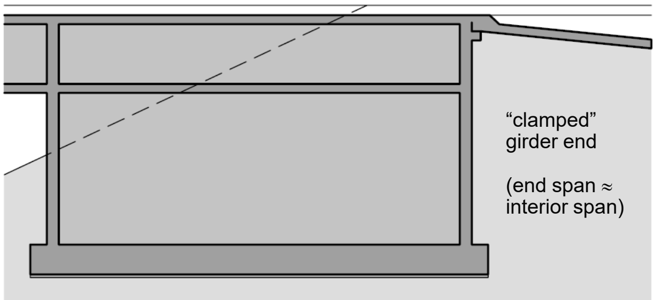
Substructure – Abutments: Classification / Examples

Integral abutment (stiff)

- maximum durability
  … neither expansion joint nor bearings
- minimum maintenance
  … neither expansion joint nor bearings
  … pavement cracks may occur
- no uplift problems
  (abutment weight can be activated in case)
- clean and tidy appearance
  … smooth transition from bridge to abutment
  (no joints between wing walls and front wall)
  … no horizontal offset due to girder contraction



frame type



“clamped” girder end

(end span ≈ interior span)

B/W Illustrations: ASTRA Richtlinie 12004

Photo: Einfahrtsrampe BW714, Dreieck Zürich West, length 120 m, spans 4x29 m (side span = interior span), dsp Ingenieure + Planer AG, 2004 © W. Kaufmann

Brücke Schönenwerd © dsp Ingenieure + Planer AG



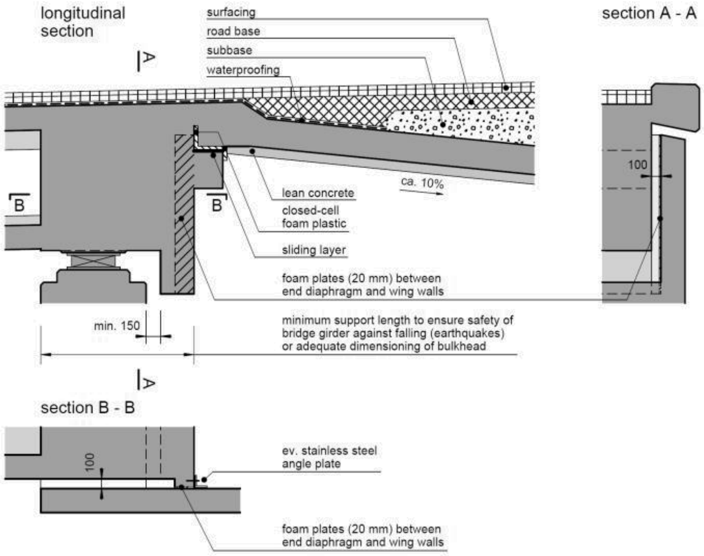

Pont du Tiguelet

dsp Ingenieure + Planer AG, Spataro Petoud Partner SA, Architekt Balz Amrein

Substructure – Abutments: Classification / Examples

Semi-integral abutment

- very high durability
  … no expansion joint, just bearings
- low maintenance
  … no expansion joint, just bearings
  … pavement cracks may occur
- uplift hardly critical
  (wide separation of bearings, load on transition slab can be activated
- regular appearance
  … end diaphragm and bearings partly visible
  … visible horizontal offset (end diaphragm-
  abutment wall) due to girder contraction



B/W Illustrations: ASTRA Richtlinie 12004



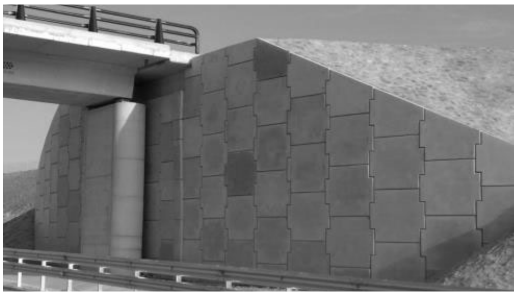
Versamertobelbrücke: Abutment

Substructure – Abutments: Classification / Examples

As already mentioned (in this chapter, as well as the support and articulation chapter), the design of abutments differs strongly, depending on the local / regional preferences of clients and designers. This slide shows a solution frequently used in some countries; more examples see support and articulation (integral abutments).

Retained earth (this slide) can be used for abutments walls, with the same advantages and drawbacks as in other retained earth walls:

- efficient system for tall approach embankments
  … use backfill as retaining wall
  … symmetric embankments: reinforcement fixed to facings at both ends
- appearance may be unsatisfactory (untidy)
- durability concerns
  … steel reinforcement: corrosion
  … geosynthetic reinforcement: degradation
- construction process
  … compaction of backfill without damaging reinforcement



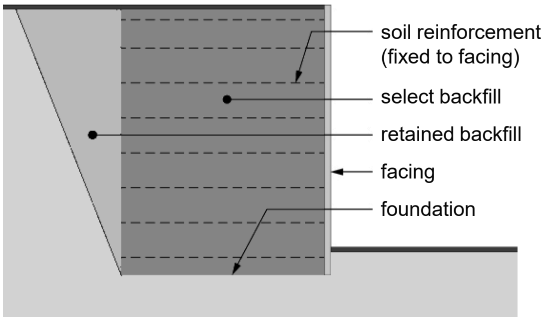


Photo: Overpass of Variante de Fuentealbilla (Albacete), reinforced earth abutment © Tierra Armada

Substructure – Abutments: Classification / Examples

Other materials than concrete may of course be used for abutments and piers, as illustrated on the slide for timber and steel, but these are exceptions.

Photos:

Right side, top Buffalo Creek Bridge (2011), steel / sheet pile abutment. Photo © ABC University Transportation Center, http://utcdb.fiu.edu/bridgeitem?id=255#

Right side, bottom Open cell sheet pile abutment, © PND Engineers

Left side Anchor River bridge, Timber abutment © Paul Seaton, https://www.peninsulaclarion.com/

Substructure

Abutments – Design

Substructure – Abutments: Dimensioning

Jointed abutments are essentially retaining walls, retaining backfill and approach embankment longitudinally.

As such, they have to be designed for ULS and SLS as structural elements, but also geotechnically, against

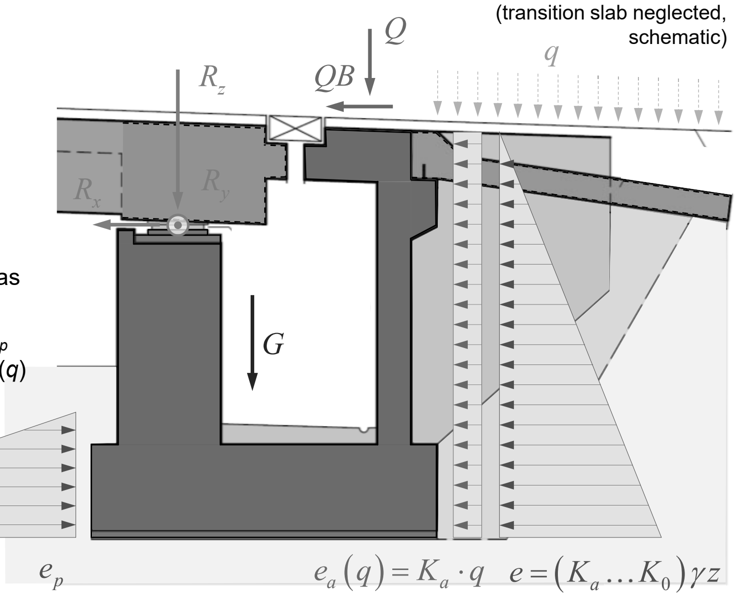
- Bearing (Grundbruch)
- Sliding (Gleiten)
- Toppling (US: Turnover) (Kippen)

Unless a pile foundation is required, the abutments act as gravity walls loaded by:

- Earth pressures from embankment / backfill $e_a \ldots e_0$, e_p
- Earth pressure due to traffic load on embankment $e_a(q)$ or traffic load + braking force (see notes)
- Vertical girder support reaction R_z
- Horizontal support reactions R_x, R_y
- Abutment self-weight G
- Further loads

Geotechnical design see lectures of IGT (and particularities, next slide)

Loads on abutment (transition slab neglected, schematic)



The figure is schematic, neglecting wall friction and other effects. Passive earth pressures acting on the restraining side of the abutment is illustrated as a favourable action (as in SIA 267). Other codes and textbooks treat it as a resistance (or mix both approaches).

The horizontal support reactions depend strongly on the bearing layout. If movable bearings are provided, they are limited to the bearing friction (or horizontal force due to deformation of elastomeric bearings).

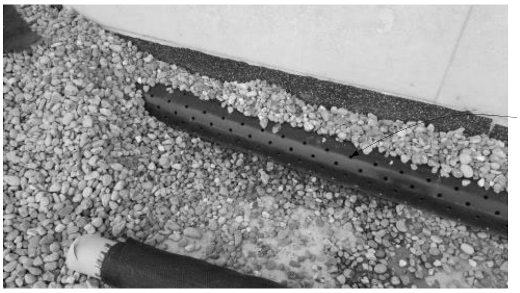
Regarding the traffic load and braking load on the top of the abutment wall, EC1991-2 (traffic loads on bridges) specifies a vertical load $\alpha_{Q1}Q_{1k}$ and a braking load of $0.6{\cdot}\alpha_{Q1}Q_{1k}$ to be applied. This corresponds to the loads of one truck axle, i.e., half the load of the tandem axle on fictitious traffic lane 1 (full load $2{\cdot}\alpha_{Q1}Q_{1k}$, braking load $1.2{\cdot}\alpha_{Q1}Q_{1k}$). EC1991-2 states that if these loads are considered, no additional traffic load on the embankment needs to be considered, e.g. either the earth pressure due to traffic loads $K_a{\cdot}q$ or the vertical and braking loads due to half a tandem axle ($\alpha_{Q1}Q_{1k}$, $0.6{\cdot}\alpha_{Q1}Q_{1k}$) are acting on the abutment, but not both at the same time. Furthermore, if half the braking load is acting on the abutment, only the remaining part of the braking load needs to be considered on the bridge (contributing to R_x).

More general cases, such as a high abutment where the access chamber cantilevers out from the abutment wall, can all be treated as in the case of retaining walls, see lectures of IGT.

Substructure – Abutments: Dimensioning

The following particularities should be observed:

- Usually, earth pressures on the active side are higher than active pressure; $(K_a + K_0)/2$ often is assumed
- Earth pressure due to traffic loads may be approximated by assuming a uniform vertical load, e.g. q_k = 25 kPa, on the entire approach embankment ($\rightarrow e_{ak}(q) \approx 10$ kPa)
- Wall friction (not shown in the figure) may be assumed where appropriate $(0.5 \ldots 0.67) \cdot \varphi$
- No water pressure is usually assumed since drainage mats and seepage pipes are provided ($\rightarrow$ maintenance, flushable!)



Seepage pipe at abutment base

Observe clean gravel and geotextile (to be unrolled and put around gravel before backfilling)

Drainage mats

Fixed to walls before backfilling)

Photos: Brücke Schönenwerd © dsp Ingenieure + Planer AG

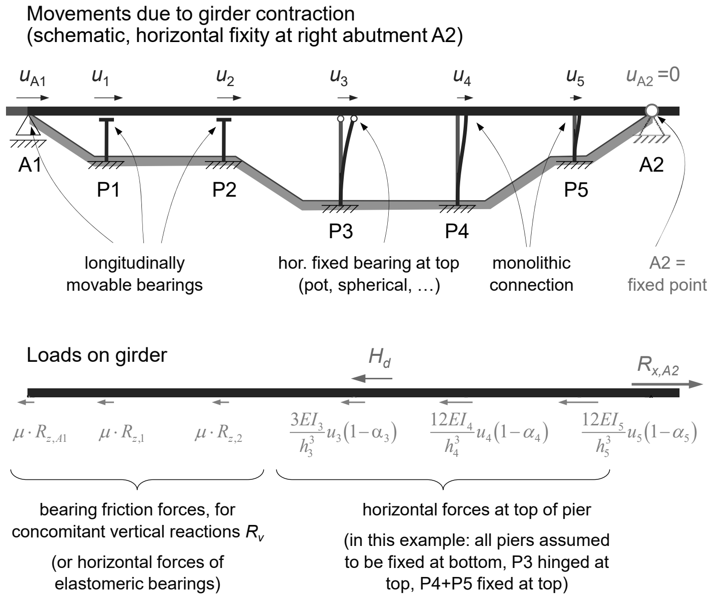
Substructure – Abutments: Dimensioning

The following particularities should be observed:

- Longitudinal support reactions are caused by (figure):
  … horizontal loads H_d (braking, wind, seismic, etc.) acting on girder, piers and abutment
  … restraint to girder contraction (or expansion), as illustrated in the figure (see section on pier design for more detailed information on values)
  → design fixed abutment providing longitudinal restraint to resist reaction $R_{x,A2}$
  → design abutment with longitudinally movable bearings to resist frictional force $\mu \cdot R_{z,A2}$
  → design piers depending on connection to girder (force or imposed pier head deformation), see section on piers

Horizontal support reactions are limited to the bearing friction in case of movable bearings.

Piers with hinges at both ends (pin-jointed members) also generate horizontal forces (see piers, system stability)



Note that in the illustration, the part of the wind loads on the piers P3/P4/P5 that is transferred to the girder is included in F_h for simplification (of the equations). These wind forces must of course be transferred to the girder at the top of the piers, which is relevant for the fixed bearing on top of pier P3 (the monolithic connections are usually not critical for horizontal force transfer).

Substructure – Abutments: Dimensioning

In jointed and integral abutments, the wing walls retain the backfill and approach embankment in the transverse direction. They may be designed

- as gravity walls (monolithically connected to the abutment walls or independent) or
- actual "wings", i.e., acting as slabs horizontally cantilevering from the abutment walls

Integral and semi-integral abutments and their behaviour were already presented in the section on support and articulation.

Cantilever wing walls may easily be dimensioned using a slab analysis software, modelling the wings and the front wall as one slab with line supports along the connecting edges

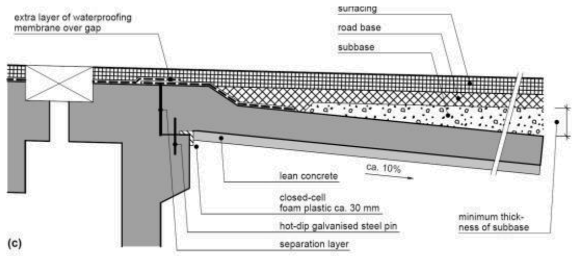
SBB Rupperswil-Lenzburg, Brücke im Rohbau (1982) Photo © Ruth Hintermann

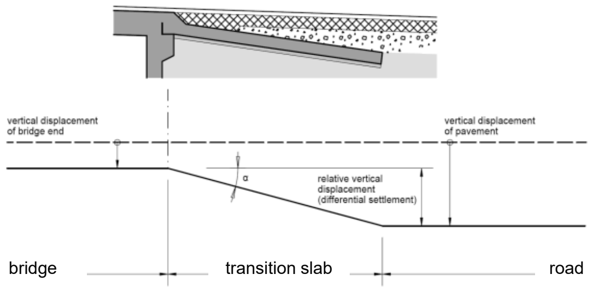
Substructure – Abutments: Dimensioning

Transition slabs (Schleppplatten) are commonly provided in road bridges to accommodate horizontal and vertical relative displacements between bridge end and embankment; in railway bridges with ballasted tracks, a backfill with stabilised material is often used instead.

Transition slabs may be positioned directly under the pavement (usual e.g. in US), or buried underneath the subbase of the road. In either case, they should ensure a smooth ride, which requires a certain length depending on the expected differential settlement (lower figure). In CH, the following angles are considered:

- $\alpha \leq 0.4$ % for motorways (v=120 km/h) (required length usually ca. 5…8 m)
- $\alpha \leq 0.8$ % for other roads ($v \leq 80$ km/h) (required length usually ca. 3…5 m)





B/W Illustrations: ASTRA Richtlinie 12004

Substructure – Abutments: Dimensioning

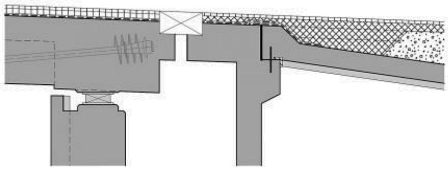
Most previous slides illustrate jointed abutments accommodating horizontal movements of the bridge end with minimal restraint.

Integral and semi-integral abutments and their behaviour were presented in the section on support and articulation. In addition to the benefits of integral abutments mentioned there, i.e.

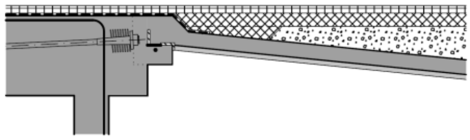
→ lower construction and maintenance costs
→ less restricted ratios of side span / interior span
→ longer or more slender end spans possible
→ noise reduction and enhanced user comfort
→ structural redundancy

they have the advantage that in many cases (particularly in frame bridges) the earth pressure on the abutments at both bridge ends can be shortcut, which is highly beneficial for the foundation design. This compensates the higher bending moments in the abutment walls due to strain ratcheting (increased earth pressure, see integral bridges).

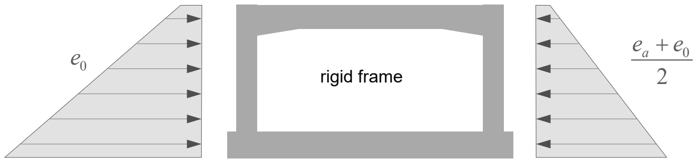
Jointed abutment



Integral abutment (much simpler)



Earth pressure transfer in closed frame bridge (underpass)



Note that even in symmetric frames, earth pressures are never fully symmetrical (particularly while backfill is installed) as in the figure. A reasonable difference between the pressures on both sides is therefore usually assumed, e.g.

- Flexible frames: Active pressure e_a on one side and enhanced earth pressure $(e_a + e_0)/2$ on other side
- Stiff frames: At-rest pressure e_0 on one side and enhanced earth pressure $(e_a + e_0)/2$ on other side

Substructure – Abutments: Dimensioning

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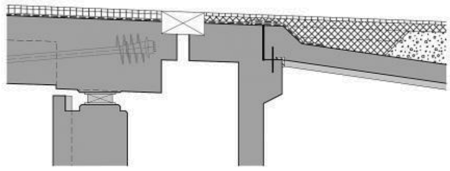
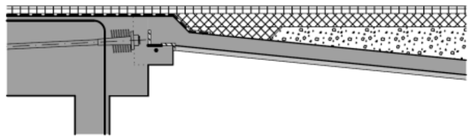
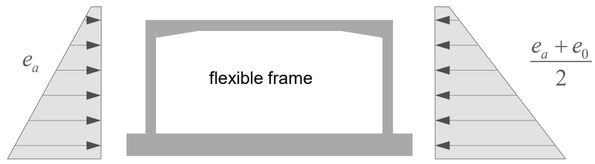


Integral abutment (much simpler)



Earth pressure transfer in closed frame bridge (underpass)



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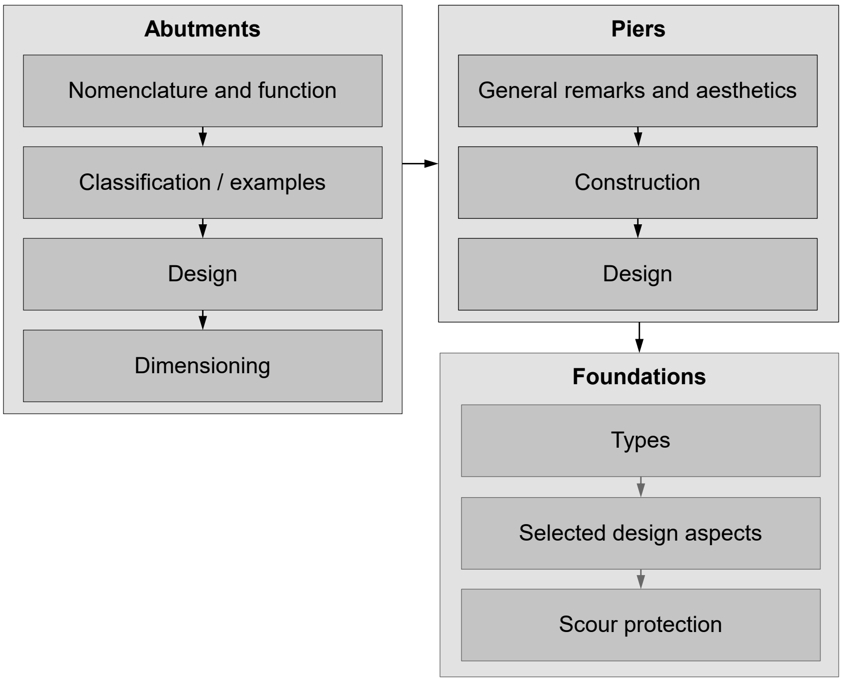

Abutments
Nomenclature and function
Classification / examples
Design
Dimensioning
Piers
General remarks and aesthetics
Construction
Design
Foundations
Types
Selected design aspects
Scour protection

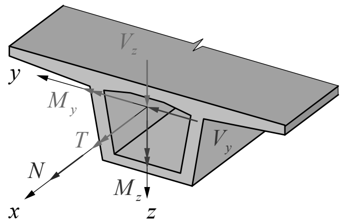
Substructure

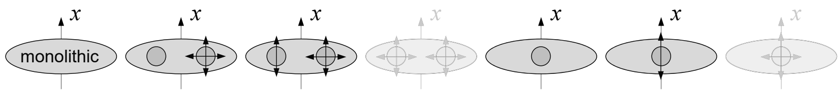
Piers – General remarks and aesthetics

Substructure – Piers: General remarks and aesthetics

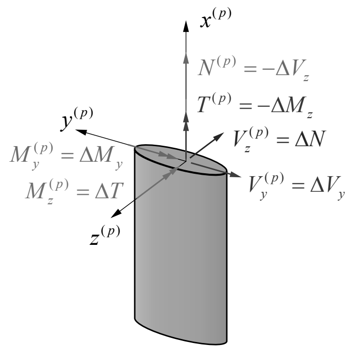
Bridge piers support the bridge girder. They provide

- vertical support (always)
- horizontal support in transverse direction (usually)
- torsional support (often)
- horizontal support in longitudinal direction (sometimes)
- longitudinal moment “support” (if monolithically connected; piers are usually much more flexible $\rightarrow \Delta M_y$ small)
- support against rotations around the vertical axis (rarely, may be required during construction (free cantilevering)





	monolithic						
ΔV_z	✓	✓	✓	✓	✓	✓	✓
ΔV_y	✓	✓	✓	–	✓	✓	–
ΔT	✓	✓	✓	✓	–	–	–
ΔN	✓	✓	–	–	✓	–	–
ΔM_y	(✓)	–	–	–	–	–	–
ΔM_z	✓	–	–	–	–	–	–



The matrix shows the support provided to the girder by monolithically connected piers and such provided with common bearing arrangements at their top.

As illustrated in the slide, bending moments around the *y*-axes are transferred between the bridge girder and monolithically connected piers. Since the girder stiffness around this axis is usually much bigger than that of the pier, the effect on the girder bending moments is usually negligible. However, the bending moments transferred to the pier at its top need to be accounted for in the pier design. The same applies to the restraint of monolithically connected piers to longitudinal girder deformations (e.g. due to thermal expansion and contraction), which is about four times bigger than that of piers connected through fixed bearings (see behind).

Substructure – Piers: General remarks and aesthetics

The piers are often decisive for the aesthetic quality and site integration of a bridge

→ selection of pier layout (single, double) highly relevant

→ design of pier geometry important

Collection of different types of bridge piers. The one on the bottom right is a «column pier» or «column bent pier» frequently used in (North American) precast bridges and consists of a number of columns joined by a pier cap (bent) on top.

Photos: Kirchtobelviadukt SOB (top middle), Anschlussbauwerke Mosi (top right), © dsp Ingenieure + Planer AG

Crossing of US Interstate 49 and LA 2 (bottom right) © https://www.alpsroads.net/roads/la/

Unknown location © https://delongsinc.com/wp-content/uploads/2019/07/555.jpg (bottom left)

Substructure – Piers: General remarks and aesthetics

Pier Layout

As already outlined (see Superstructure – Aesthetics):

- Piers are decisive for the transparency of a bridge
- Transparency of the piers depends highly on the perspective (direction of sight), particularly for wide piers
- Single, narrow piers (one slender pier per support axis) are much more transparent than wide or twin piers

Furthermore, single piers have a smaller footprint, which may be decisive for an economical span layout and the future use of the space below the bridge (urban bridges, skew crossing of roads or railway lines).

→ Avoid wide piers (including hammerhead columns and multiple-column bents)

→ Provide single, slender piers where possible

See examples on this and following slides.

Top: Aarerücke Solothurn-Zuchwil (“Rote Brücke”). Ingenieurbüro Th. Müller, 1986. Photo © kfm

Bottom: Überführung Dufourstrasse, Zürich Tiefenbrunnen / Zollikon. Photo © Google Streetview

Substructure – Piers: General remarks and aesthetics

Glattalbahn, Viadukt Glattzentrum, dsp mit Höltschi und Schurter, Beratung Gestaltung Feddersen Klostermann (2009). Photos © dsp Ingenieure + Planer

Substructure – Piers: General remarks and aesthetics

Single, slender piers are feasible (see Support and Articulation)

- in narrow bridges
- in medium width bridges if the piers provide neither horizontal nor torsional support to the girder (single longitudinal girder with high torsional stiffness and strength required)

In other cases, larger pier widths or twin piers are required.

Except for wide, low bridges (see behind), single piers are still preferred. The pier width should be limited to maximise transparency; usually, the piers should not be wider than about 25…35% of the deck width.

Twin piers, even if they are slender and well designed, risk to be perceived as perturbing or even disordered if the span is not clearly larger than the transverse spacing.

Top: Viaduc du lac de la Gruyère, Schmidt+Partner (1979), typical span 60.5 m. Photo © La Liberté

Bottom: Puente sobre la Ría de Betanzos, La Coruña, ES, Juan José Arenas (1996). Photo © Arenas y Asociados

Substructure – Piers: General remarks and aesthetics

Gänsebachtalbrücke, Erfurt-Leipzig/Halle (Schlaich Bergermann Partner, 2012).

Photo © Deutsche Bahn AG / Hannes Frank

Substructure – Piers: General remarks and aesthetics

However, in wide low bridges, to maximise the apparent slenderness (girder depth vs. clear height under the bridge), twin piers are often adequate.

Twin piers are usually also required in twin girder bridges (next slide).

Top Europabrücke Zürich (1963), bottom Hardbrücke Zürich (1972), right side both © ETH Baugeschichtliches Archiv, e-pics.ethz.ch, left side https://www.signify.com

Substructure – Piers: General remarks and aesthetics

Jonentobelbrücke, dsp Ingenieure+Planer AG, ACS Partner AG

Substructure – Piers: General remarks and aesthetics

Pier geometry – Orientation in plan and dimensions

Piers resisting horizontal transverse forces, and/or providing torsional support to the girder require a substantial width of about 25…35% of the deck width (transverse to girder axis).

Longitudinally, piers may be much more slender if they do not have to longitudinally stabilise the girder. If bearings are provided on the top of piers, these are often decisive for the minimum pier thickness.

Aesthetically, the piers should be slender to maximise transparency, but at the same convey a perception of stability.

Rectangular, prismatic single piers are the obvious option to satisfy these requirements. However, in many cases, somewhat more refined geometries are adequate:

- Increasing width towards bottom (in high piers for stability, in low piers to foster the perception of stability)
- Rounded or elliptical cross-sections (more slender appearance, particularly adequate if pier orientation varies and for hydraulic reasons in river piers)
- Circular cylindrical piers (lack orientation, which may be disconcerting but adequate for twin piers).

Photo: Puente del Milenario sobre el Ebro en Tortosa, J. Martínez Calzón, MC2 Ingeniería. 1982 (one of the first major bridges using weathering steel, main span 180 m).

Substructure – Piers: General remarks and aesthetics

Pier geometry – General observations

Refined pier geometries are feasible with little effect on cost since piers constitute <10% of total cost for moderate bridge heights.

Economy is thus no reason to design dull prismatic piers with rectangular cross-section – but there may be functional or aesthetic reasons why they are appropriate for a specific site.

In order to facilitate economical formwork fabrication, pier geometries should however (see notes for definition) be

→ prismatic (constant section)
(simple and economical, even for curved sections)

→ polyhedral (polygonal faces with straight edges)
(more complex)

→ consist of developable (complex) or even ruled surfaces
(most complex)

Complex geometries should only be used for high piers, or if the same formwork can be used for several piers.

Doubly curved formworks other than ruled surfaces are an order of magnitude more expensive and should be avoided.

Ruled surfaces can be generated by the continuous motion of a straight line (= *generator* or *ruling* of the surface) along a space curve (*directrix*). Alternatively, which is more useful for defining ruled surfaces in practice, a ruled surface can be defined by a ruling joining corresponding points on two space curves. The formwork can be built by using straight (but generally variable width) timber slats along these rulings.

Developable surfaces are special ruled surfaces, having the same tangent plane along all points of the generator (no twist along the generators). Hence, one of the principal curvatures is zero (developable surfaces = singly curved surfaces). They can be formed by bending or rolling a planar surface without stretching or tearing, which is very practical for formwork production (e.g. using thin metal panels).

Innbrücke Vulpera, dsp Ingenieure + Planer, ACS Partner with Eduard Imhof and Dr. Vollenweider (2010).
© dsp Ingenieure + Planer AG

Substructure – Piers: General remarks and aesthetics

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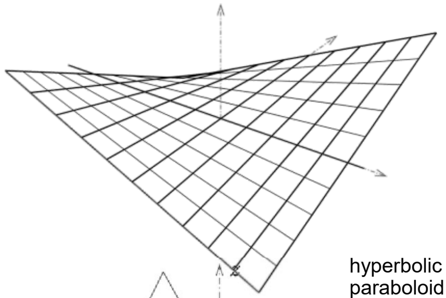
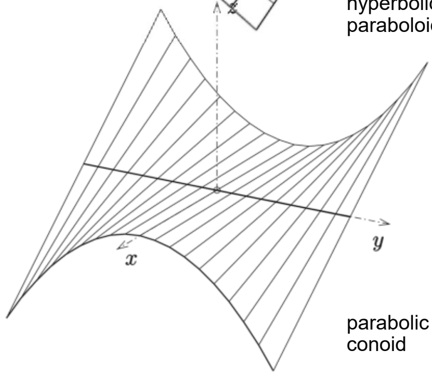
Complex geometries should only be used for high piers, or if the same formwork can be used for several piers.

Doubly curved formworks other than ruled surfaces are an order of magnitude more expensive and should be avoided.



hyperbolic paraboloid



parabolic conoid

Ruled surfaces can be generated by the continuous motion of a straight line (= *generator* or *ruling* of the surface) along a space curve (*directrix*). Alternatively, which is more useful for defining ruled surfaces in practice, a ruled surfaces can be defined by a ruling joining corresponding points on two space curves. The formwork can be built by using straight (but generally variable width) timber slats along these rulings.

Developable surfaces are special ruled surfaces, having the same tangent plane along all points of the generator (no twist along the generators). Hence, one of the principal curvatures is zero (developable surfaces = singly curved surfaces). They can be formed by bending or rolling a planar surface without stretching or tearing, which is very practical for formwork production (e.g. using thin metal panels).

Innbrücke Vulpera, dsp Ingenieure + Planer, ACS Partner with Eduard Imhof and Dr. Vollenweider (2010).

Substructure – Piers: General remarks and aesthetics

Pier geometry – Prismatic piers

Prismatic piers adapt to variable height without any particular measures and are appropriate
→ for low-moderate height
→ where no particularly expressive form is sought

Prismatic piers may be provided with complex polygonal cross-sections without excessive cost. Curved cross-sections are more expensive, but this is usually insignificant due to the low share of piers in total cost.

Pont du Tiguelet, dsp Ingenieure+Planer with Spataro Petoud Partner and Balz Amrein Photos © R. Spataro

Prismatic piers with «elliptic» cross-section

Substructure – Piers: General remarks and aesthetics

Pier geometry – Polyhedral piers

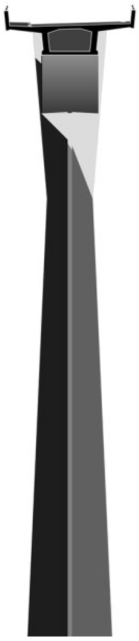
Polyhedral piers are less economical than prismatic piers, but enable structurally efficient (e.g. variable width according to bending moments in pier) and aesthetically appealing geometries. They are appropriate

→ for tall, slender piers (structural efficiency)

→ where a more expressive form is adequate

Designers and clients are often reluctant regarding polyhedral geometries for economical concerns. However, experience shows that such geometries cause little extra cost if an efficient pier formwork is part of the design.

In the example shown on the right and the next slide, the apparently complex geometry essentially consists of four slightly inclined planes (same formwork panels used for full height of pier), cut off laterally by variable end pieces. The pier was cast using a formwork corresponding largely to the concept proposed in the design competition already (climbform, 14 segments @ 4.50 m, vertical precamber 30 mm).

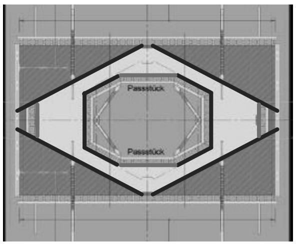


Innbrücke Vulpera, dsp Ingenieure+Planer, ACS Partner mit Eduard Imhof und Dr. Vollenweider (2010). Photos © dsp Ingenieure+Planer

Polyhedral pier

Substructure – Piers: General remarks and aesthetics

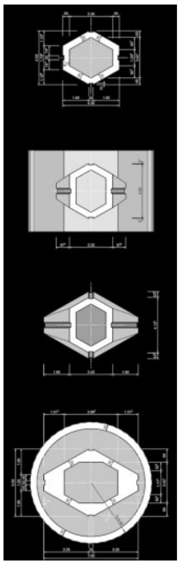
cardboard model used in conceptual design



formwork panels used for full pier height

fitting panels (adjusted per segment)

3D-model used in final design



Innbrücke Vulpera, dsp Ingenieure+Planer, ACS Partner mit Eduard Imhof und Dr. Vollenweider (2010). Photos © dsp Ingenieure+Planer / Isometric drawing © ACS Partner AG / cardboard model © Eduard Imhof

Polyhedral pier

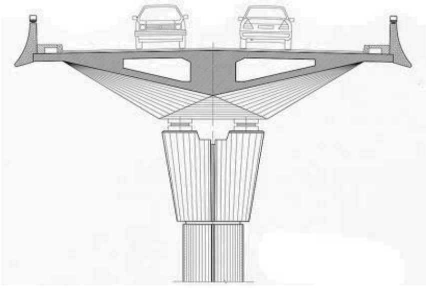
Substructure – Piers: General remarks and aesthetics

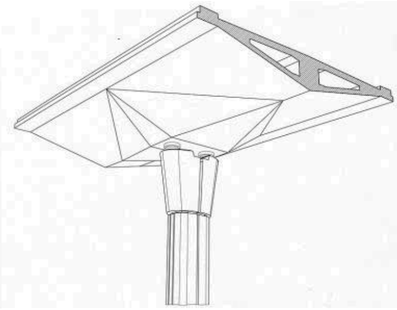
Pier geometry – Developable surfaces

Developable surfaces may be used to achieve geometries including curved parts. These may be unpretentious (conical, cylindrical, etc., this slide) or expressive (next slide).

Steel formwork is often used for such geometries, as thin “plates” can readily be curved uniaxially → inlays unless very smooth surface is desired.

(Note the polyhedral soffit of the girder → observations on pier geometry apply to girders, but economy is more relevant for girder formwork)





Access viaduct of the Puente Puerta de Europa, Barcelona, Arenas&Asociados (2000). Photo © W. Kaufmann

Unpretentious elliptic cone pier heads to accommodate bearings.

Substructure – Piers: General remarks and aesthetics

Pier geometry – Developable surfaces

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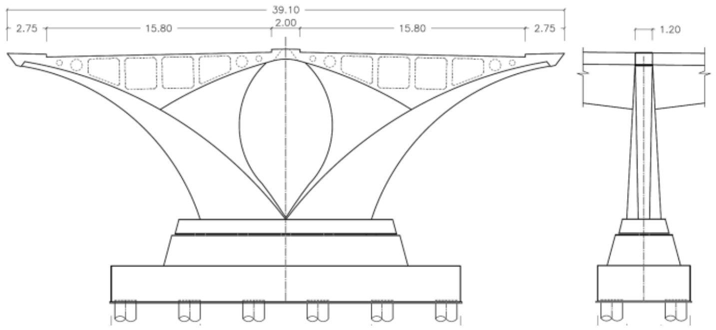
Steel formwork is often used for such geometries, as thin “plates” can readily be curved uniaxially → inlays unless very smooth surface is desired.



Puente sobre el Rio Besos, Carlos Fernandez Casado S.L. Photo © CFCSL

Expressive pier geometry.

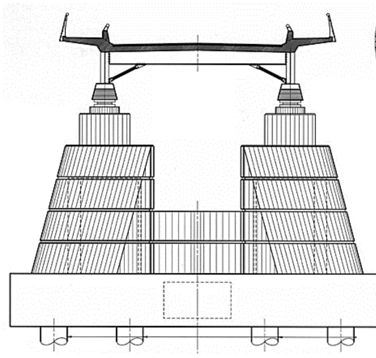
Substructure – Piers: General remarks and aesthetics

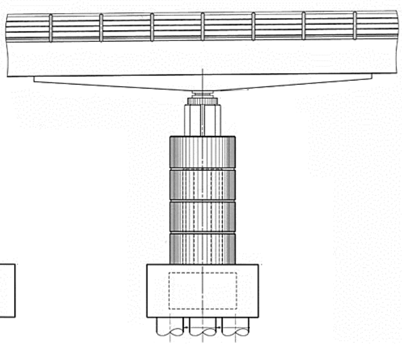
Pier geometry – Ruled surfaces

Ruled surfaces may be used to achieve expressive geometries including curved parts.

They can be produced using timber slats, making them more expensive than prismatic or polyhedral piers, yet still much less expensive than free-form double curved surfaces.

The following slide shows a girder with ruled surface geometry.





Puente sobre el Embalse del Ebro, Reinosa-Corconte, Arenas & Asociados (2001). Illustrations © Arenas & Asociados

Pier geometry consisting of ruled surfaces.

Substructure – Piers: General remarks and aesthetics

Ponte del Risorgimento, Verona. Pier Luigi Nervi, 1968. Ruled surfaces. Photo © Concorso Fotografico Nazionale Comuni-Italiani.it

Substructure – Piers: General remarks and aesthetics

Pier geometry – Variable height

Hardly any bridge with several piers has a constant height above ground, not even in the case of road bridges across a river or lake: A longitudinal gradient is usually provided for drainage. On the other hand, the piers usually have a constant width at their top (bearings or connection to girder).

Hence, unless prismatic piers are used, finding a pier geometry that fits for the tallest as well as the shortest piers of a bridge may be challenging.

Prismatic parts at the bottom of low-medium height piers, are often useful to achieve a consistent appearance of all piers in a bridge with strongly varying height above ground, see next slides.

Alternatively / additionally, the upper part of the piers may adopt the superstructure geometry, see photo and other slides (Vulpera, Steinbach, Tortosa …)

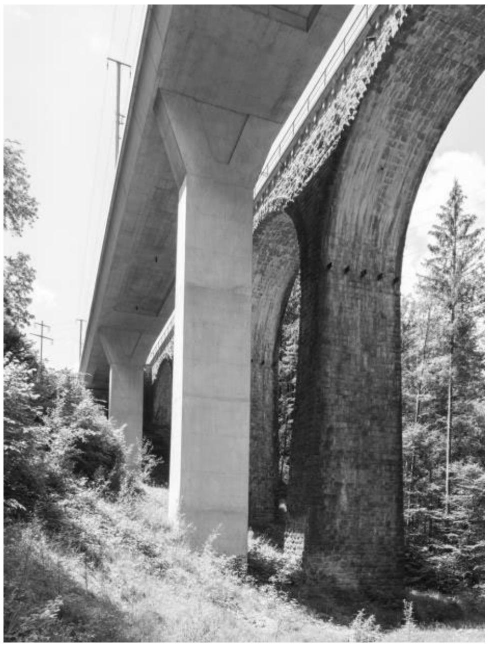
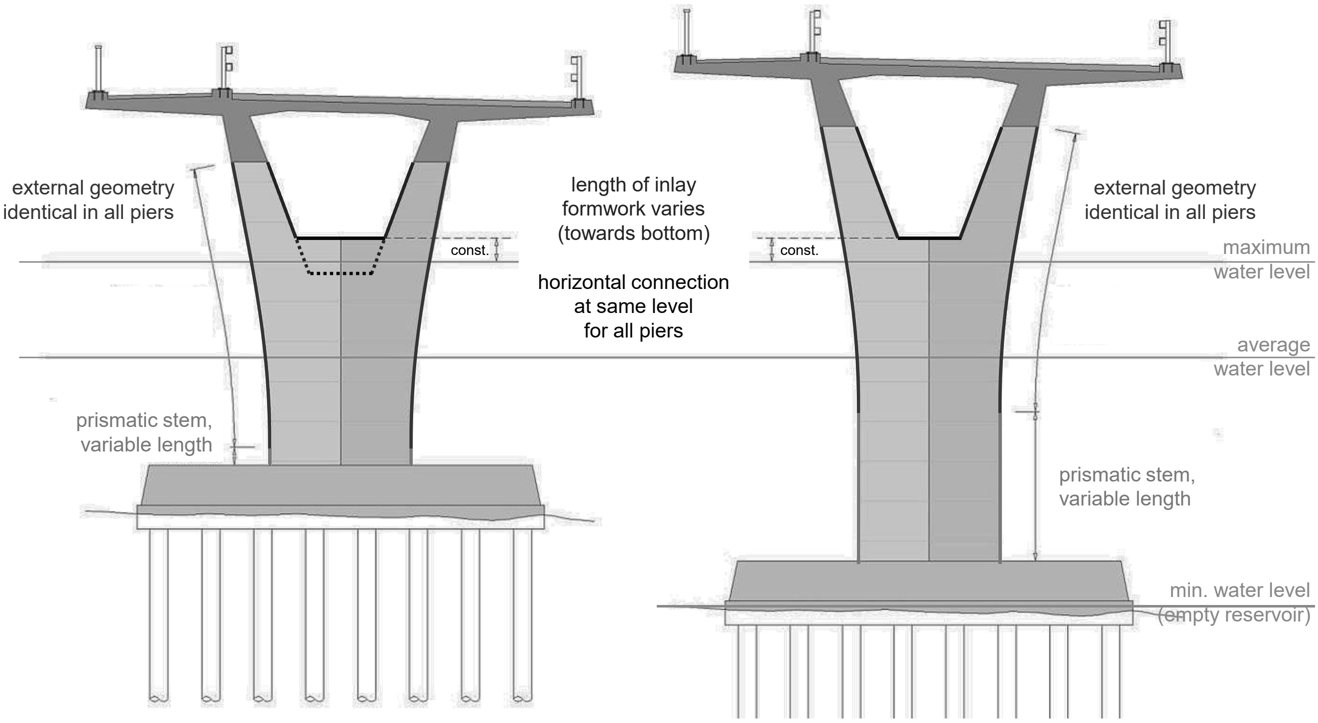
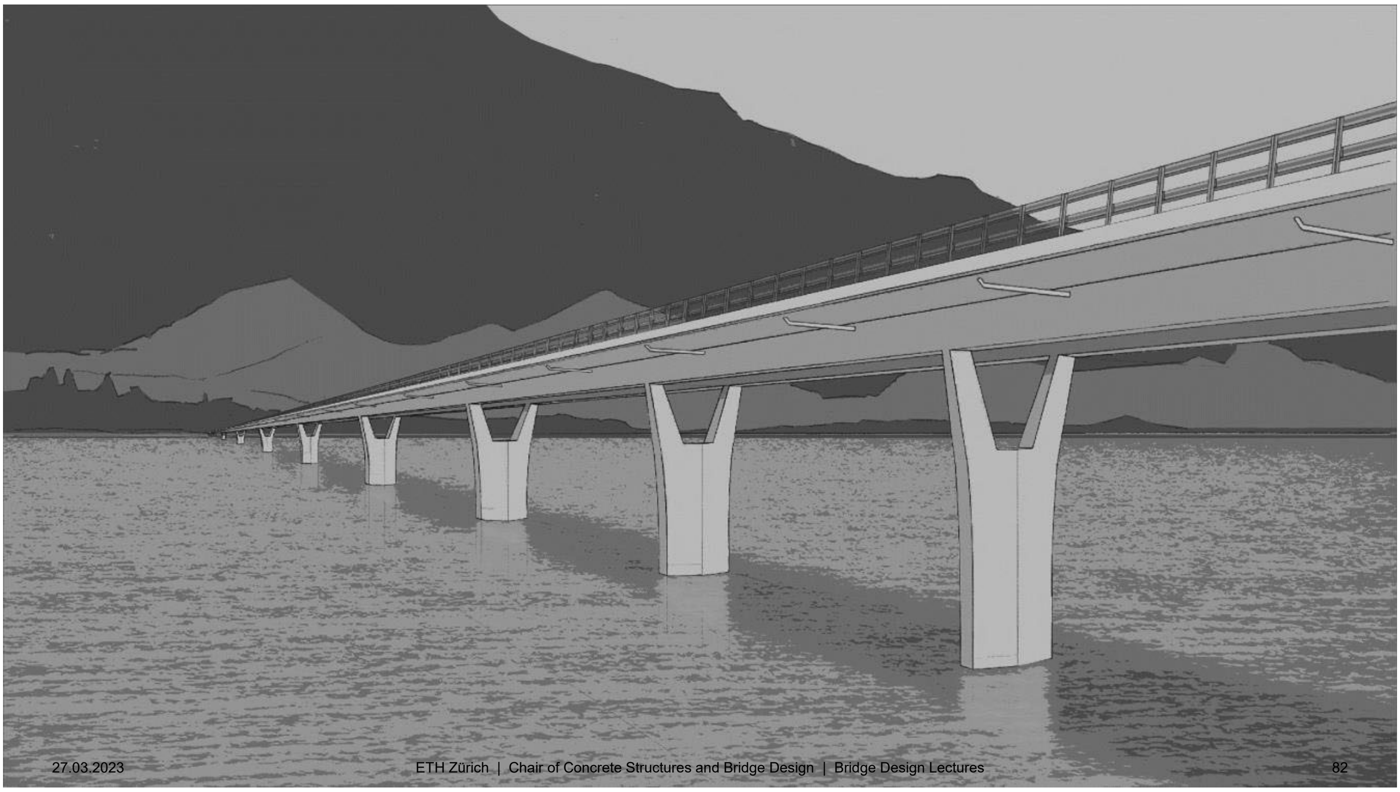


Photo: Kirchtobelviadukt SOB, © dsp Ingenieure + Planer AG



Steinbachviadukt, dsp Ingenieure+Planer mit F. Preisig AG, Feddersen Klostermann und Fellmann Geotechnik, 2012. © dsp Ingenieure+Planer AG



In a bridge over lake or ocean, the water level constitutes a strong reference, that must be considered when designing the piers. The piers should refer to the water level, yet without obstructing the construction sequence of the girder (which needs to be built either using movable scaffold systems or by incremental launching since no conventional scaffold can be used). Furthermore, the pier geometry needs to account for the variation of the girder height above the lake (longitudinal gradient required for drainage or to achieve navigational clearance), but the girder width is usually constant. Unless a trivial (vertical prismatic) geometry is used, this is challenging.

In a reservoir with strongly varying water level (example: Steinbach Viaduct over Sihlsee Reservoir, variation > 8 m every year), this is even more difficult, since the piers should look decent at all water levels.

Substructure

Piers – Construction

Substructure – Piers: Construction

Pier geometry – Mechanised equipment

If mechanised construction equipment is used (launching girders, movable scaffold systems MSS etc.), the piers must be designed to enable their efficient use. In some cases, pier diaphragms need to be cast beforehand.

Requirements depend strongly on the specific system. If possible, the use of underslung or lateral gantries should be enabled (overhead gantries are more expensive).

In case of incremental launching, the pier heads must accommodate larger bearings and the piers be designed to resist the frictional forces during launching (and, where appropriate, accidental forces due to manipulation errors etc.).

Jonentobelbrücke, dsp Ingenieure+Planer AG, ACS Partner AG, Photo © dsp Ingenieure + Planer

Substructure – Piers: Construction

Pier geometry – Mechanised equipment

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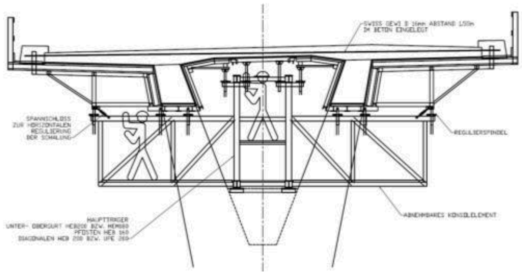


Steinbachviadukt, dsp Ingenieure+Planer mit F. Preisig AG, Feddersen Klostermann und Fellmann Geotechnik, 2012. © dsp Ingenieure+Planer AG

Substructure – Piers: Construction

Pier construction methods

Prismatic piers can be built very efficiently using climbforms (Kletterschalung):

→ segments of usually about 4…6 m height
→ formwork / scaffold is fixed to the previously cast pier segment
→ lifting of formwork with crane or hydraulic device (self-climbing)

Polyhedral and developable surfaces can also be built using climbforms, provided that the geometry is defined appropriately (see previous slides, Innbrücke Vulpera).

One lift per week can usually be achieved; cycles of 3 days (for 4 m lifts) are possible in perfect conditions. If short cycles are used, curing of the concrete requires additional measures (protect one segment below the climbform from evaporation)

Jonentobelbrücke, dsp Ingenieure+Planer AG, ACS Partner AG, Photo © dsp Ingenieure + Planer

Substructure – Piers: Construction

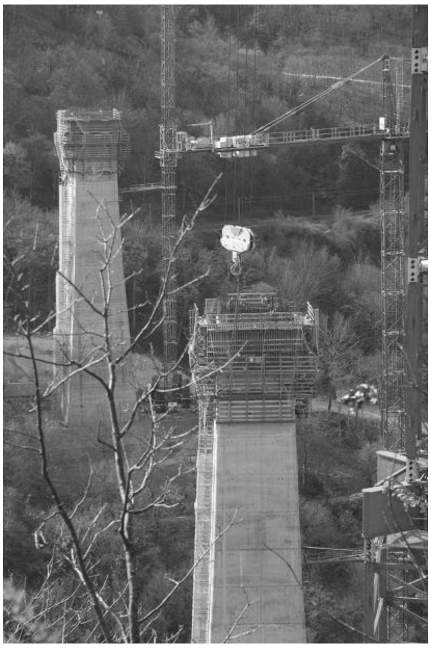


Photo: Viaducto de Montabliz, Cantabria, Spain, Apia XXI (2008). Spans 110+155+175+155+126 m, Maximum pier height 145 m.

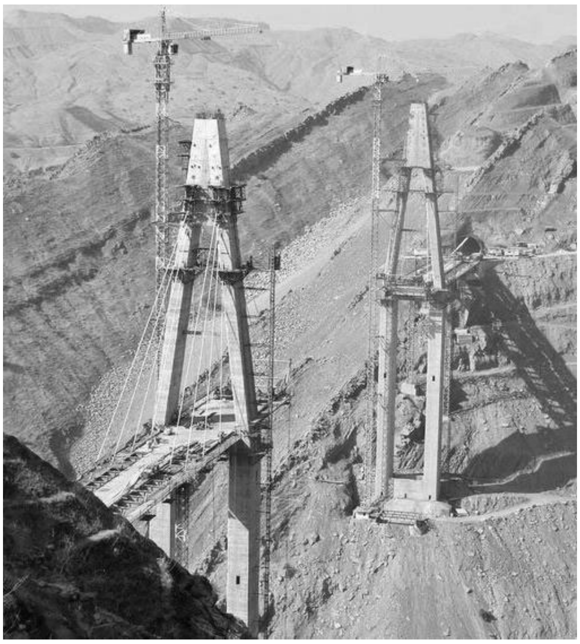
R. Villegas Gómez, M. Pantaleón Prieto, R. Revilla Angulo, P. Olazábal Herrero: «Viaducto de Montabliz». Hormigón y Acero, Vol. 59, nº 248, pp. 9-40, 04/2008

Substructure – Piers: Construction

Pier construction methods

Large and tall prismatic piers may alternatively be built by slipforming (Gleitschalung):
→ short formwork, ca. 1.2 m high, advancing continuously 24hx7d
→ supported by cast-in vertical bars, extended as slipform moves
→ casting speed 5-7 m per day (20…30 cm/h)

However, slipforming is only economical for very tall piers with large cross-section (specialist (sub-)contractor required. Furthermore, 24/7 work is difficult / prohibited (noise emissions, concrete delivery, …) → hardly ever used today for bridge piers



Photos: Lali Bridge, Boland Payeh / Hexa (design consultant dsp Ingenieure + Planer), 2011. Photo © Highestbridges.com

Pylon shafts (h=90 m) / Pylon upper parts (h=60 m) built by slipforming.

Substructure – Piers: Construction

Pier geometry – Solid or hollow cross-section

Piers with a solid cross-section are much simpler to build than hollow piers, requiring an inner formwork.

Saving weight is less relevant in vertical piers than in girders, since no bending moments are caused by the pier dead load and higher vertical loads may even be favourable for spread footing foundations.

However, hollow cross-sections have a higher decompression moment under a given vertical load (superstructure is usually dominant), see "Strategies for efficient bridge girders"), which is favourable for the stiffness

→ use hollow cross-section for tall, slender piers

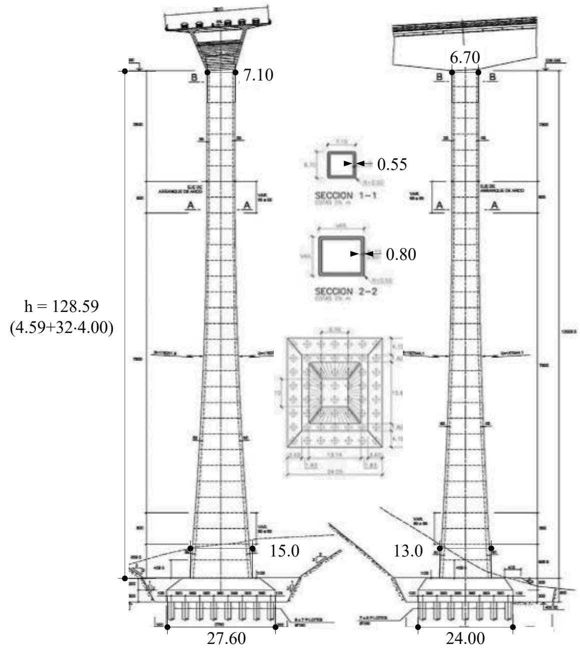
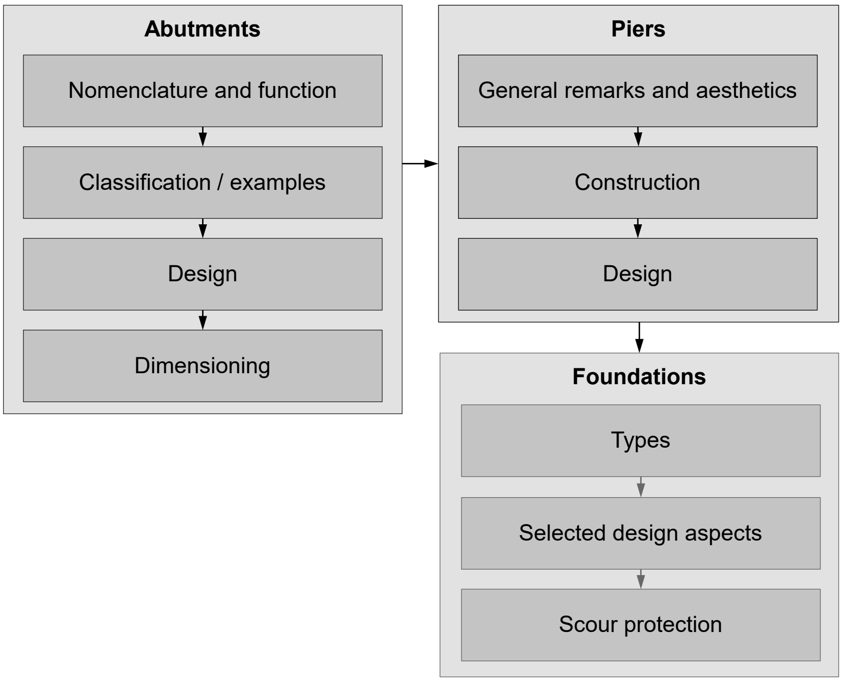


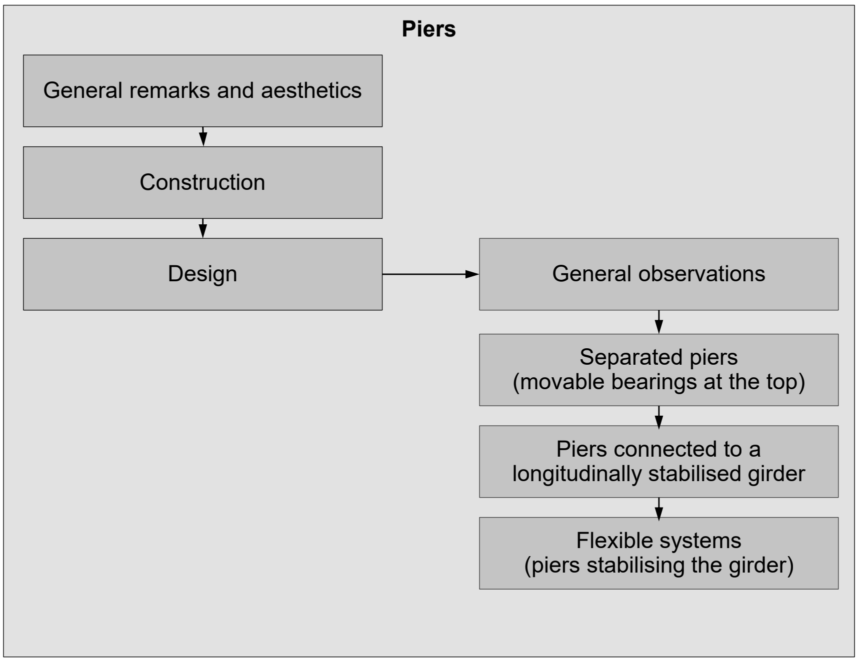
Illustration: Viaducto de Montabliz, Cantabria, Spain, Apia XXI (2008). Spans 110+155+175+155+126 m, Maximum pier height 145 m. Photo © Roberto Revilla http://www.robertorevillaestudio.es/

R. Villegas Gómez, M. Pantaleón Prieto, R. Revilla Angulo, P. Olazábal Herrero: «Viaducto de Montabliz». Hormigón y Acero, Vol. 59, nº 248, pp. 9-40, 04/2008

Substructure

Piers – Design


Abutments
Nomenclature and function
Classification / examples
Design
Dimensioning
Piers
General remarks and aesthetics
Construction
Design
Foundations
Types
Selected design aspects
Scour protection


Piers
General remarks and aesthetics
Construction
Design
General observations
Separated piers
(movable bearings at the top)
Piers connected to a
longitudinally stabilised girder
Flexible systems
(piers stabilising the girder)

Substructure

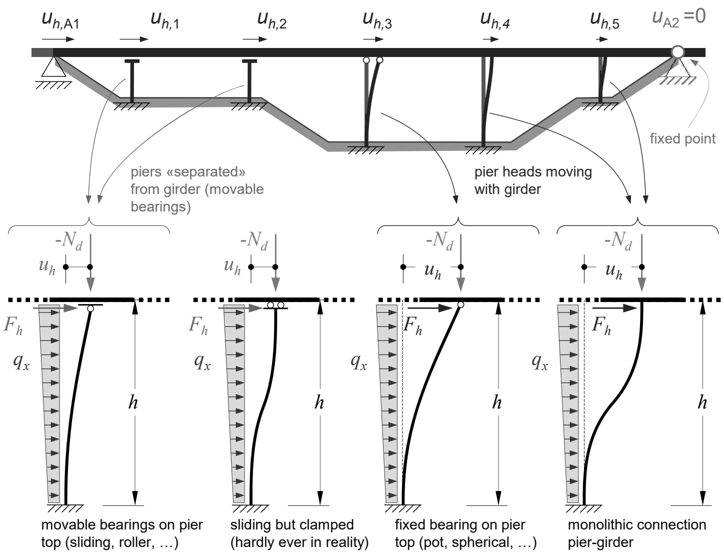
Piers – Design
General observations

Substructure – Piers: Design / General observations

Internal actions on bridge piers

- Bridge piers provide vertical support to the girder → high compressive normal forces
- Due to the movements of the bridge girder, horizontal displacements and corresponding loads are imposed to the piers at their top unless longitudinally movable bearings are provided
- Bending moments in bridge piers are caused by
  → horizontal loads applied at the top
  (bearing friction if provided with sliding bearings, horizontal forces transferred from deck otherwise)
  → horizontal loads applied to the pier shaft (wind, impact, seismic) (variable over height generally)
  → second order effects
- Bridge piers are often slender (longitudinally)
  → account for geometric second order effects when determining the relevant internal actions
- The response of concrete piers is nonlinear (cracking, concrete stress-strain relationship, creep)
  → account for material nonlinearities

Movements of superstructure (due to girder contraction) imposed to piers (schematic, horizontal fixity at right abutment A2)



Substructure – Piers: Design / General observations

Internal actions on bridge piers and static systems

Bridge piers are generally subjected to

- high compressive normal forces N<0, combined with biaxial bending moments M_y, M_z
  - → design for combination of {N, M_y, M_z} = maximum / minimum values of each action, combined with concomitant values of other actions
  - → check 3D interaction diagrams for verification (see Stahlbeton I / figure)
  - → in preliminary design, check interaction of {$M_{y,Ed}$, $M_{z,Ed}$} vs {$M_{y,Rd}$, $M_{z,Rd}$} at $N_{Rd,min}$ and $N_{Rd,max}$

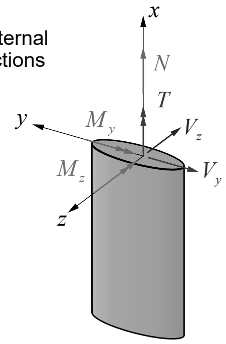
Bridge piers are typically wide, and thus much stiffer and stronger in the transverse direction than longitudinally

→ design often mainly governed by {N, M_y)
(neglect M_z in preliminary design where appropriate)

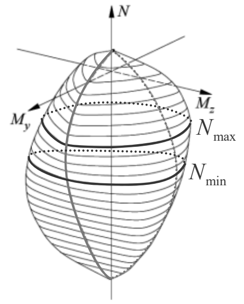
Bridge piers are often relatively stiff compared to the foundation; on the other hand, the girder is commonly much stiffer than the piers

→ no full fixity at pier base, but clamped at top

Internal actions



Cross-section resistance under {N, M_y, M_z}
3D interaction surface



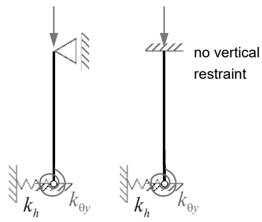
Required verifications of cross-section resistance in general case

$$\begin{Bmatrix} N_{\min} \\ N_{\max} \end{Bmatrix} \text{ with concomitant } \{M_y, M_z\}$$

$$\begin{Bmatrix} M_{y,\min} \\ M_{y,\max} \end{Bmatrix} \text{ with concomitant } \{N, M_z\}$$

$$\begin{Bmatrix} M_{z,\min} \\ M_{z,\max} \end{Bmatrix} \text{ with concomitant } \{N, M_y\}$$

Static systems of stiff piers on soft soil



Substructure – Piers: Design / General observations

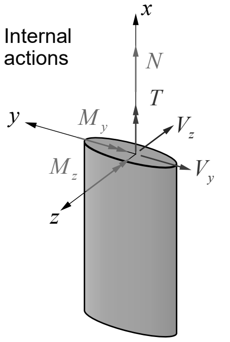
Second order analysis of reinforced concrete piers

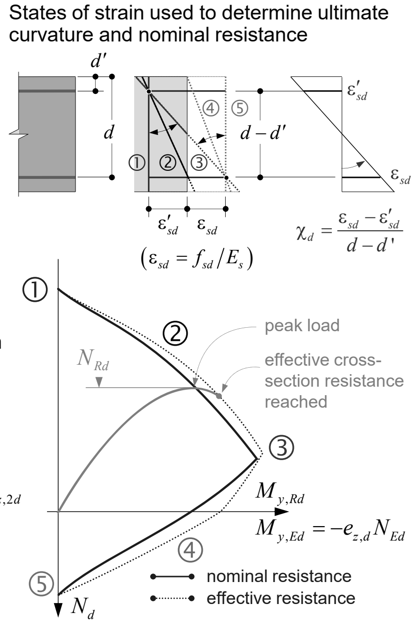
- The pier stiffness in the 2nd-order analysis must correspond to the states of strain used to determine the resistance (interaction diagram)
- At the effective cross-section resistance M_{Rd}, large strains occur (i.e. large curvature χ = low stiffness EI)
  - → large deflections of (slender) piers and second-order bending moments at effective resistance
  - → load when effective cross-section resistance is reached is lower than actual ultimate load
  - → carry out nonlinear analysis or define nominal cross-section resistance using reduced state of strain
- Usual assumption for design: M_{Rd} limited by the onset of yielding of reinforcement ($\varepsilon_s = \pm\varepsilon_{sd} = \pm f_{sd}/E_s$):

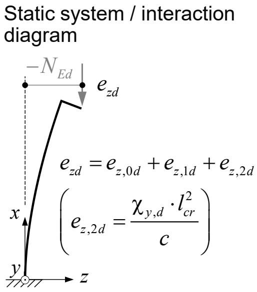
$$\chi_d \leq \frac{\varepsilon_{sd} - \varepsilon'_{sd}}{d-d'} \rightarrow EI_d = \frac{M_{Rd}}{\chi_d} \approx M_{Rd} \cdot 230(d-d') \quad \text{(for B500B)}$$

- Creep may be accounted approximately for by adding a term $\chi_{irr,d}$:

$$\chi_d \leq \frac{\varepsilon_{sd} - \varepsilon'_{sd}}{d-d'} + \chi_{irr,d} \qquad \chi_{irr,d} \approx \frac{|\varepsilon_{c\infty}|}{d}$$







Note that using the nominal cross-section resistance defined by the onset of yielding of the reinforcement, the real peak load (which can be determined with a nonlinear analysis) is not exactly met. However, for usual slenderness ratios, it is reasonably approximated (and underestimated = safe side).

In design, the design curvature may be adjusted to the governing strain state at reaching the nominal cross-section resistance, which yields higher ultimate loads (since second-order effects are reduced when using a lower curvature = higher stiffness)

Substructure – Piers: Design / General observations

Geometrical imperfections

- Design codes provide values of geometrical imperfections that need to be considered in standard cases.
- According to SIA 262, a base rotation α has to be considered for vertical members

$$\frac{1}{200} \geq \alpha_i = \frac{0.01}{\sqrt{h}} \geq \frac{1}{300} \left(h = \text{ height of pier [m]}\right)$$

  and in the design of compression members the following eccentricity must be accounted for:

$$e_{0d} = \max\begin{pmatrix} \dfrac{d}{30} \\ \alpha_i \cdot \dfrac{l_{cr}}{2} \end{pmatrix} \quad \begin{pmatrix} d = \text{static depth of cross-section} \\ l_{cr} = \text{buckling length} \end{pmatrix}$$

- In exceptionally tall or slender piers, special considerations may be appropriate. For example, in the 145 m tall pier shown to the right, a base rotation of 1/300 would correspond to an eccentricity of 483 mm at the pier head (during construction, where $l_{cr} = 2h$). This eccentricity could be reduced by adopting strict geometrical control measures.

Photo: Viaducto de Montabliz, Cantabria, Spain, Apia XXI (2008). Spans 110+155+175+155+126 m, Maximum pier height 145 m.

R. Villegas Gómez, M. Pantaleón Prieto, R. Revilla Angulo, P. Olazábal Herrero: «Viaducto de Montabliz». Hormigón y Acero, Vol. 59, nº 248, pp. 9-40, 04/2008

Substructure

Piers – Design

Horizontally separated piers
(piers with movable bearings at the top)

Substructure – Piers: Design / Horizontally separated piers

Horizontally separated piers (movable bearings at top)

Piers provided with longitudinally movable bearings at their top can be analysed separately from the superstructure

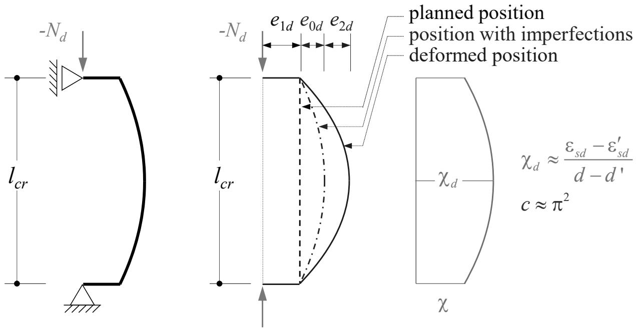
They may be safely designed e.g. using the curvature based method of SIA 262, see Stahlbeton I.

Compared to columns in buildings, there are some differences:

- static systems
  … beam columns (= pin-jointed piers) unusual in bridges
  … often statically indeterminate support
- much higher loads, pier dimensions and cross-sections
- usually horizontal loads (at pier top, not just accidental loads due to impact as in buildings)

The same design approach may also be used to estimate second order effects when pre-dimensioning piers with a horizontally fixed connection to the superstructure. In their final design, imposed pier head displacements need however to be accounted for → see behind

Design procedure according to SIA 262 (curvature based design)



$$\chi_d \approx \frac{\varepsilon_{sd} - \varepsilon'_{sd}}{d - d'}$$

$$c \approx \pi^2$$

$$e_d = e_{0d} + e_{1d} + e_{2d} \quad \begin{cases} e_{0d} = \max\begin{pmatrix} \dfrac{d}{30} \\ \alpha_i \cdot \dfrac{l_{cr}}{2} \end{pmatrix} & \text{eccentricity due to geometric imperfection} \quad \dfrac{1}{200} \geq \alpha_i = \dfrac{0.01}{\sqrt{h[m]}} \geq \dfrac{1}{300} \\ e_{1d} = \dfrac{M_{1d}}{-N_d} & \text{first order eccentricity of action} \\ e_{2d} = \displaystyle\int_0^{l_{cr}} \chi \cdot \overline{M} dx = \chi_d \frac{l_{cr}^{\,2}}{c} & \text{eccentricity due to member deformation} \end{cases}$$

The geometric imperfections can generally be determined according to formulae (17) and (74) of SIA 262. For the special case $l = l_{cr}$, the following simplified relationships result: $e_{0d} = l_{cr}/400$ for $l_{cr} \leq 4$ m and $e_{0d} = l_{cr}/600$ for $l_{cr} \geq 9$ m, and linearly interpolated for 4 m < l_{cr}< 9 m.

Note that when using elastomeric bearings to accommodate longitudinal displacements (without sliding plate), an additional eccentricity of the vertical load, corresponding to the deformation of the elastomer, needs to be accounted for. When using sliding or roller bearings, the fixed part should be located on the pier head (i.e. sliding plate fixed to the girder), such that no such eccentricity results.

Substructure – Piers: Design / Horizontally separated piers

Horizontally separated piers (movable bearings at top)

Using the curvature based design approach of SIA 262, different loads (particularly horizontal loads) may be accounted for as illustrated (derivation see Stahlbeton I).

This slide recaps the factors c_i for the basic cases treated in Stahlbeton I (beam column and cantilever column). Other, statically indeterminate systems, common in bridge piers, and corresponding factors c_i are shown on the next slide.

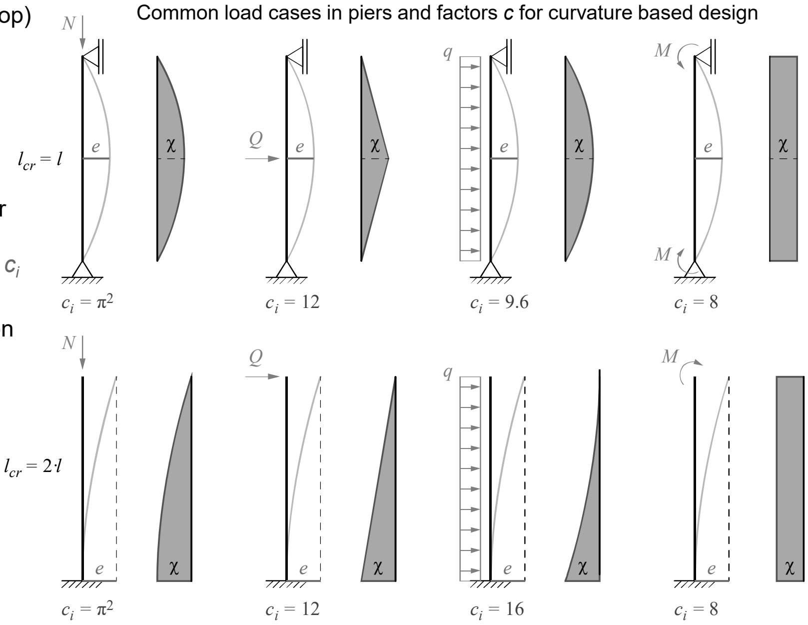
If piers are stiff and/or the foundation is not sitting on rock, piers are not fully clamped at the base → use appropriate buckling length l_{cr} and c-factors.

Superposition of basic cases to a common factor c (derivation see Marti, Theory of Structures):

$$N_{cr,d} = -\frac{\pi^2 EI_d}{l_{cr}^2} \qquad \alpha = \frac{N_d}{N_{cr,d}}$$

$$\rightarrow c = \alpha \cdot \pi^2 + (1-\alpha)\frac{\Sigma M_{di}}{\Sigma \frac{M_{di}}{c_i}} \qquad \left(c_i = \frac{\chi_{mi} l^2}{e_{1i}} = \frac{M_{1i}\chi_{mi} l^2 / M_m}{\int_0^l \chi_i \overline{M} dx} \right)$$

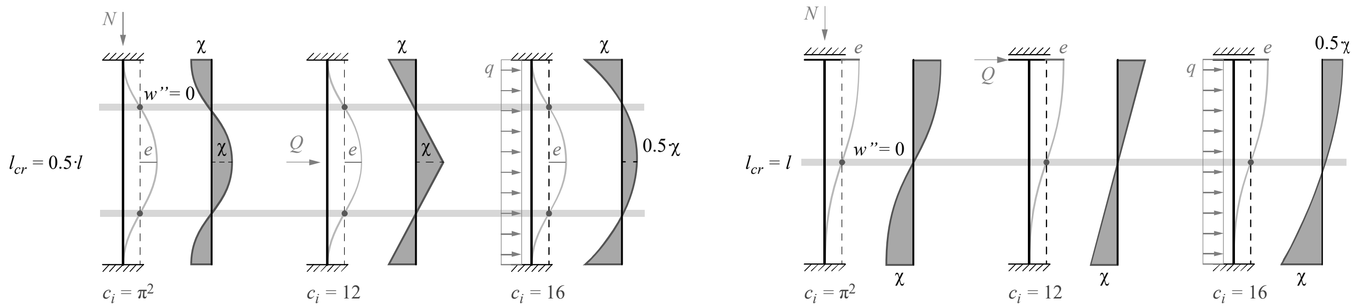
Common load cases in piers and factors c for curvature based design



The individual c_i factors are calculated using the work theorem, see e.g. Marti, Baustatik, Chapter 22.3.2.3. When combining different load cases, the sums in the formula shown on the slide must be applied over all n components of the first-order moments, including the component $N \cdot e_0$ due to imperfections.

This procedure leads to exact 2nd order values only if the shape of the 1st order deformation line caused by q, Q and M is affine to the first eigenform (deformation line for normal force on top). If that is not the case (as generally occurs), this procedure results in (good) approximate solutions.

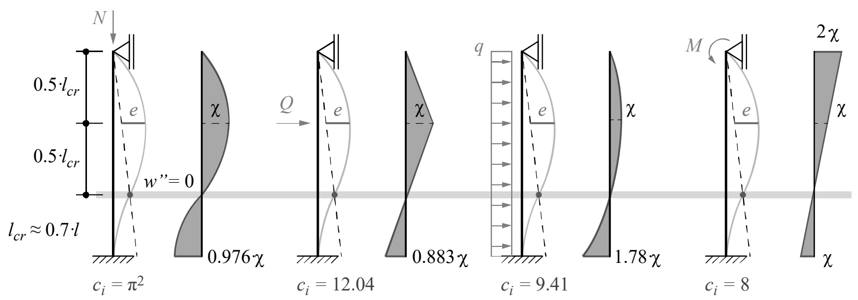
Substructure – Piers: Design / Horizontally separated piers



Statically indeterminate piers

For statically indeterminate piers, the eccentricity e that effectively contributes to the increase of the moment is the distance between the deflection curve and the secant through the points of inflection (where $\chi = 0$).

To allow the superposition of various load cases to a single c-factor despite different curvature distributions, the assumption is made that the points of inflections are the same as in the base case with a normal force on top (see notes).



The eccentricity e that effectively contributes to the 2nd order moment ($N \cdot e$) is the distance between the column deflection curve (blue) and the secant (dashed line) through the points of inflection (grey dots).

For statically determinate systems (previous slide), the maximum eccentricity corresponds to the maximum total deflection, which can be discretely calculated with the principle of virtual work (Arbeitsgleichung). (Note that the eccentricity in the case of the cantilever does not follow the deflection curve but refers to the maximum deformation).

In statically indeterminate systems (this slide), the points of inflection are generally not at the same locations for different load cases. The simplifying assumption is made that the deflection curves are affine to the first eigenform for all load cases. This allows the superposition of different load cases to a single, weighted c-factor. The errors made using this simplification are rather small as the exact locations for the points of inflection differ only slightly for the ones of the first eigenform.

Point of inflection ($\chi = 0$), location x from bottom support				
	N	Q	q	M
fixed-fixed	$0.25 \cdot l$	$0.25 \cdot l$	$0.21 \cdot l$	-
clamped-fixed (no horizontal restraint)	$0.50 \cdot l$	$0.50 \cdot l$	$0.42 \cdot l$	-
pinned-fixed	$0.30 \cdot l$	$0.31 \cdot l$	$0.25 \cdot l$	$0.33 \cdot l$

Note that in case of the pinned-fixed system, the location of the governing section (i.e. section with maximum 2nd order moment) can vary depending on the load combination (in particular on the ratio q/N *and* M/N) and magnitude of the geometric imperfections. Therefore, one should always check the 2nd order moment distribution along the entire pier in order to identify the governing section.

Substructure

Piers – Design

Piers connected to a longitudinally stabilised girder (typically fixed at an abutment)

Substructure – Piers: Design / Piers connected to long. stabilised girder

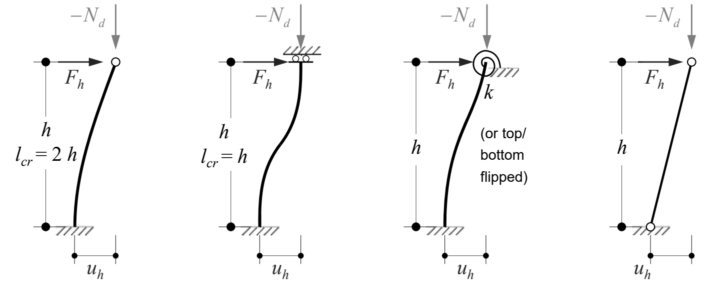
Imposed pier head displacements – General

Piers longitudinally fixed to the girder can still be analysed individually, but the pier head displacements due to superstructure movements need to be accounted for.

While the design of cross-section resistances is analogous to piers with longitudinally movable bearings (see previous slides), determining the relevant internal actions and dimensioning the pier reinforcement is more complex:

- Imposed pier head displacements cause first and second order bending moments in the piers
- Different buckling lengths apply:
  … horizontal forces due to imposed pier head displacements (figure on right): pier head movable
  … buckling of individual piers: pier head hor. fixed
- Additional verifications are required to ensure appropriate behaviour in serviceability SLS (crack widths due to imposed deformation):
  … ULS: lower-bound value of pier stiffnesses EI_d
  … SLS: characteristic value of pier stiffnesses EI_k

Imposed pier head displacements and corresponding horizontal forces (second-order, EI=const; $N_{cr,d}$ and N_d are both < 0 = compression)



$$F_h = u_h \frac{3EI_d}{h^3}(1-\alpha) \qquad \alpha = \frac{N_d}{N_{cr,d}} \qquad N_{cr,d} = -\frac{\pi^2 EI_d}{4 \cdot h^2}$$

$$F_h = u_h \frac{12EI_d}{h^3}(1-\alpha) \qquad \alpha = \frac{N_d}{N_{cr,d}} \qquad N_{cr,d} = -\frac{\pi^2 EI_d}{h^2}$$

$$\left(N_{cr,d} \quad \text{for longitudinally unrestrained pier head relevant here} \right)$$

$$F_h = -u_h \frac{12N_{cr,d}}{\pi^2 h}(1-\alpha) \qquad N_{cr,d} = \frac{-\pi^2 EI_d}{h^2 \cdot \left(4 - \dfrac{3}{1+\dfrac{EI_d}{k \cdot h}}\right)}$$

$k = 0:$ hinged at top $\rightarrow$ as Fig. (a)
$k = \infty:$ clamped at top $\rightarrow$ as Fig. (b)

$$F_h = N_d \frac{u_h}{h}$$

(see notes)

compression negative $\rightarrow N_d \leq 0, N_{cr,d} < 0$

Note that the pin-jointed case is only relevant here for the design of the abutment (which must resist the sum of horizontal forces F_h at all pier tops (in addition to applied horizontal loads), see abutment design). It is more important for flexible systems, see there.

Substructure – Piers: Design / Piers connected to long. stabilised girder

Imposed pier head displacements – Behaviour (1)

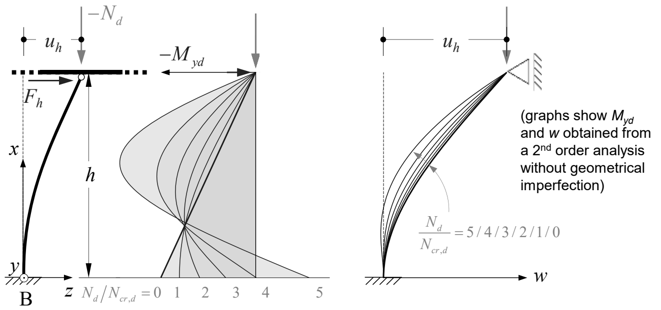
For a given imposed pier head displacement u_h, the following behaviour is observed under increasing vertical compressive normal force:

- The horizontal forces F_h caused by u_h (= the forces F_h required to displace the pier head by u_h) decrease with $|N_d|$ due to second order moments
- The magnitude of the «1st order bending moment», i.e. ($-F_h \cdot h$ at the pier base) decreases
- On the other hand, the magnitude of the 2nd order bending moment ($N_d \cdot u_h$ at pier base) increases
- Overall, the magnitude of the total bending moment at the pier base

  $$M_{y,B} = -F_h \cdot h + N_d \cdot u_h$$

  decreases, and the bending moment eventually changes sign (the moment diagram approaches that of the buckled individual pier)

Imposed pier head displacements and corresponding horizontal forces (second-order, EI=const; $N_{cr,d}$ and N_d are both < 0 = compression)



$$F_h = u_h \cdot \frac{3EI_d}{h^3}\left(1 - \frac{N_d}{N_{cr,d}}\right) \rightarrow u_h = \frac{F_h \cdot h^3}{3EI_d\left(1 - \frac{N_d}{N_{cr,d}}\right)} \qquad N_{cr,d} = -\frac{\pi^2 EI_d}{4 \cdot h^2}$$

$$M_{y,B} = -F_h \cdot h + N_d \cdot u_h = -u_h \cdot \frac{3EI_d}{h^2}\left(1 + N_d \cdot \frac{4 \cdot h^2}{\pi^2 EI_d}\right) + N_d \cdot u_h$$

$$= -u_h \cdot \left[\frac{3EI_d}{h^2} + N_d \cdot \left(\frac{12}{\pi^2} - 1\right)\right] \qquad \left(\text{compression, i.e. } N_d < 0, N_{cr,d} < 0\right)$$

The «1st order bending moment» is in quotation marks because it accounts for second order effects in the value of F_h

$$N_{cr,d} = -\frac{\pi^2 EI_d}{4 \cdot h^2}$$

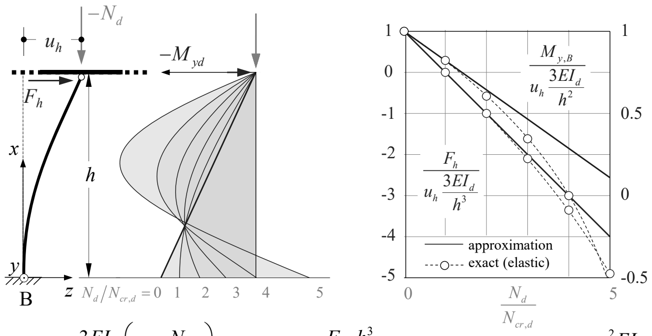
Substructure – Piers: Design / Piers connected to long. stabilised girder

Imposed pier head displacements – Behaviour (2)

(… continued …)

- The normal force N_d can exceed the buckling load of the pier horizontally unrestrained at the top, i.e., $|N_d|>|N_{cr,d}|$ is possible since the pier head is fixed after imposing the displacement u_h (the buckling load of the restrained pier is $2^2/0.7^2 \approx 8$ times larger than $N_{cr,d}$ in accordance with the buckling lengths)
- For normal forces N_d exceeding the buckling load of the pier horizontally unrestrained at the top, i.e., $|N_d|>|N_{cr,d}|$, negative values of F_h result → the pier head needs to be held back to avoid instability
- The equation relating horizontal forces and pier head displacement (factor $1-N_d/N_{cr,d}$) presumes affinity of deflections, which is less accurate at higher loads (buckled shape of pier differs strongly from deflection due to pier head displacement)
- The diagram to the right compares the results of the approximation with an elastic 2nd order analysis

Imposed pier head displacements and corresponding horizontal forces (second-order, EI=const; $N_{cr,d}$ and N_d are both < 0 = compression)



$$F_h = u_h \cdot \frac{3EI_d}{h^3}\left(1-\frac{N_d}{N_{cr,d}}\right) \rightarrow u_h = \frac{F_h \cdot h^3}{3EI_d\left(1-\frac{N_d}{N_{cr,d}}\right)} \qquad N_{cr,d} = -\frac{\pi^2 EI_d}{4\cdot h^2}$$

$$M_{y,B} = -F_h \cdot h + N_d \cdot u_h = -u_h \cdot \frac{3EI_d}{h^2}\left(1+N_d \cdot \frac{4\cdot h^2}{\pi^2 EI_d}\right) + N_d \cdot u_h$$

$$= -u_h \cdot \left[\frac{3EI_d}{h^2} + N_d \cdot \left(\frac{12}{\pi^2}-1\right)\right] \qquad \left(\text{compression, i.e. } N_d < 0, N_{cr,d} < 0\right)$$

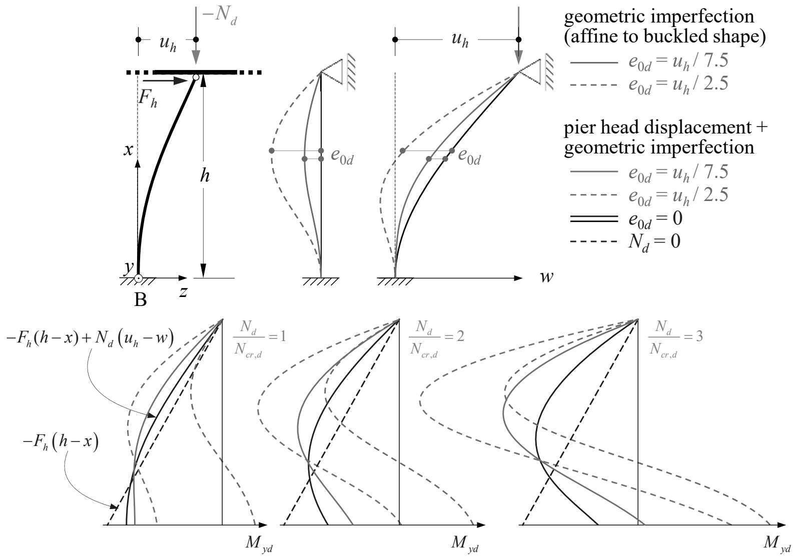
Substructure – Piers: Design / Piers connected to long. stabilised girder

Imposed pier head displacements – Imperfections

- So far, no geometric imperfections e_{0d} were considered. These may have a beneficial or detrimental effect on the bending moments in the pier, depending on
  - $\rightarrow$ the slenderness and level of compressive force
  - $\rightarrow$ the ratio between imposed deformation and geometric imperfection
  - $\rightarrow$ the position along the pier

Typically, including e_{0d} is favourable at the pier base but unfavourable higher up, and less relevant for low normal force and/or slenderness

Imposed pier head displacements and corresponding horizontal forces (second-order, EI=const; $N_{cr,d}$ and N_d are both < 0 = compression)



Substructure – Piers: Design / Piers connected to long. stabilised girder

Imposed pier head displacements – Design (1)

- The application of approximate methods (e.g. curvature-based design) is not straightforward, since the geometric imperfection e_{0d} and the imposed deflections w have completely different shapes (much more pronounced than different positions of points of inflection when determining factors *c* in statically indeterminate piers, see previous slides)

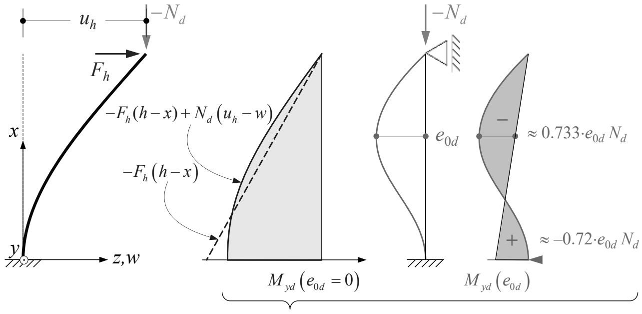
→ except for low normal force / slenderness (→ neglect beneficial e_{0d}), a 2nd order analysis is recommended

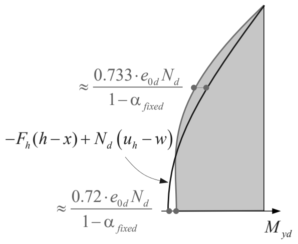
→ constant, conservative value of EI_d sufficient except for slender piers, where refined calculations accounting for material nonlinearity are adequate

- For low slenderness and preliminary design, the bending moments may be estimated as indicated in the figure, assuming parabolic $w(x)$ and checking pier base and position where $M_{yd}(e_{0d}) = 0.733 e_{0d} N_d (1\text{-}\alpha)$

Note: The principles outlined for a pier hinged at its top also apply to piers monolithically connected to the girder.

Approximate determination of bending moments due to imposed pier head displacement accounting for 2nd order effects





$$M_y(x) = -F_h(h-x) + N_d\left(u_h - w(x)\right) + \frac{M_{yd}(e_{0d})}{1-\alpha_{fixed}} =$$

$$= -u_h\cdot(h-x)\cdot\left[\frac{3EI_d}{h^3} + N_d\cdot\left(\frac{12}{\pi^2} - \frac{1-\frac{w(x)}{u_h}}{h-x}\right)\right] + \frac{M_{yd}(e_{0d})}{1-\alpha_{fixed}}$$

$$\left(\alpha_{fixed} = \frac{N_d}{N_{cr\_fixed,d}};\, N_{cr\_fixed,d} = \frac{\pi^2 EI_d}{l_{cr,fixed}^2} \approx \frac{\pi^2 EI_d}{(0.7h)^2}\right)$$

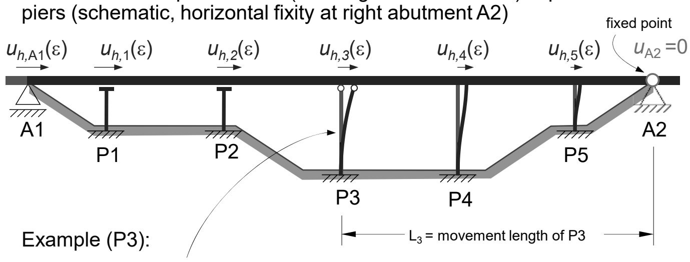
Substructure – Piers: Design / Piers connected to long. stabilised girder

Imposed pier head displacements – Design (2)

As outlined in *Support and Articulation:*

- The displacements imposed to the pier heads in fixed systems are caused by movements of the girder due to temperature variations $\varepsilon_{\Delta T}$, shrinkage ε_{cs} , prestressing ε_{cp}, and creep $\varepsilon_{cc} = \varphi \cdot \varepsilon_{cp}$
- The axial stiffness of the girder is orders of magnitude higher than the flexural stiffness of piers
  - $\rightarrow$ the design pier head displacements can be determined using the free (unrestrained) girder expansion and contraction, considering that
  - $\rightarrow$ the movement lengths vary in staged construction
- While bending moments are reduced by long-term effects (creep and relaxation), pier head displacements – causing 2nd order moments and relevant for bearing (and expansion joint) movement capacity – are not!
- The fixed abutment needs to be designed to resist the sum of horizontal forces F_h at all pier tops (in addition to applied horizontal loads), see abutment design

Movements of superstructure (due to girder contraction) imposed to piers (schematic, horizontal fixity at right abutment A2)



Example (P3):

Total imposed pier head displacement (assuming one-casting system):

$$u_{h,3}(\varepsilon) = \left(\varepsilon_{\Delta T} + \varepsilon_{cs} + \varepsilon_{cp} \cdot (1+\varphi)\right) \cdot L_3$$

Pier head displacement $\leftrightarrow$ design of slender piers (previous slides):

$u_{h,3}(\varepsilon) = \varepsilon_{\Delta T} \cdot L_3 + \ldots$ use short-term pier stiffness

$\ldots + \left(\varepsilon_{cs} + \varepsilon_{cp} \cdot (1+\varphi)\right) \cdot L_3$ use long-term pier stiffness , e.g. $E_c / (1+\varphi)$

Approximation of restraint force applied to pier head of non-slender piers:

$F_h = F_h\left(\varepsilon_{\Delta T} \cdot L_3\right) + \ldots$ use short-term pier stiffness

$\ldots + F_h\left(\varepsilon_{cs} \cdot L_3\right) + \ldots$ use age-adjusted pier stiffness (see notes), $E_c / (1+\mu\varphi)$

$\ldots + F_h\left(\varepsilon_{cp} \cdot L_3\right)$ use short-term pier stiffness (see notes)

[or $F_h\left(\varepsilon_{cp} \cdot (1+\varphi) \cdot L_3\right)$ with long-term pier stiffness]

Notes on the approximation for non-slender piers / SLS

- Prestressing, creep and shrinkage as well as temperature reductions cause a contraction of the deck, which generates restraint forces in the piers.
- *Restraint forces* in the piers due to initial prestressing (elastic contraction of the deck) would reduce over time according to «time-independent restraint», i.e., to 20…30% of their initial value. However, since the contraction of the deck increases due to creep under prestressing (which would build up restraints according to «time-dependent restraint» if considered on its own), restraint forces due to prestressing remain approximately constant over time (if piers and deck have the same creep properties and the longitudinal stiffness of the deck is much higher than the longitudinal restraint caused by the piers).
- Over time, *restraint forces* due to shrinkage of the deck imposed to the piers will build up according to «time-dependent restraint», i.e., to about 40% of the value obtained without considering relaxation.
- *Restraint forces* caused by daily as well as seasonal temperature changes (length change of deck) needs to be accounted for with almost their full elastic value, since they will also occur after many years when the concrete's ability to creep is much reduced.

Substructure

Piers – Design

Flexible systems
(piers longitudinally stabilising the girder)

Substructure – Piers: Design / Flexible systems

General behaviour

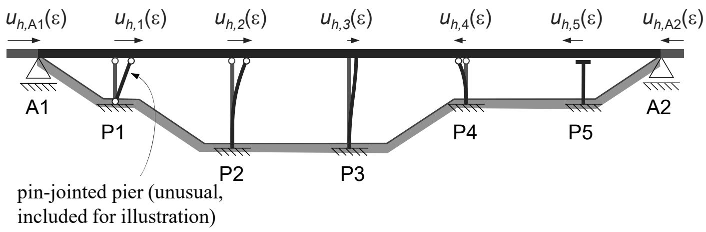
In bridges longitudinally stabilised by (slender) piers, the following verifications of the piers are required:

- System stability, i.e., safety against instability of the entire system → rigid body movement of girder
- ULS and SLS of individual piers, accounting for imposed pier head displacements (previous slides) including the rigid body movement determined above

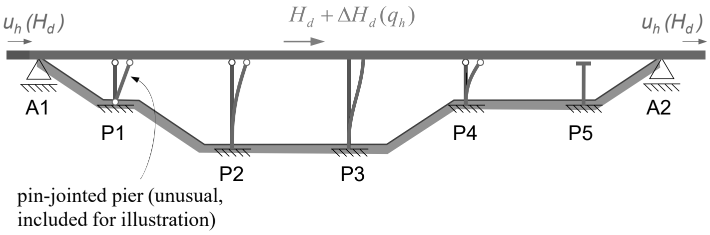
Considering that the girder is axially very stiff, the pier head displacements consist of two main contributions:

- displacements caused by (≈ unrestrained) girder expansion and contraction due to temperature variations $\varepsilon_{\Delta T}$, shrinkage ε_{cs}, prestressing ε_{cp}, and creep $\varepsilon_{cc} = \varphi \cdot \varepsilon_{cp}$ (upper figure)
  → different $u_{h,i}(\varepsilon)$ for each pier
  → horizontal forces $F_{h,i}$ of all piers cancel out
- displacements caused by rigid body movements of the girder due to applied loads (lower figure)
  → equal for all piers $u_{h,i}(H_d) = u_h(H_d)$
  → sum of horizontal forces $F_{h,i}$ = applied load H_d

Bridge longitudinally stabilized by piers: Movements due to girder contraction (schematic)



Bridge longitudinally stabilized by piers: Rigid body girder movement due to load (schematic)



Substructure – Piers: Design / Flexible systems

System stability – Basics

A rigorous verification of the system stability is complex and subjected to many uncertainties.

→ usually, a simplified approach is therefore used, see below and next slides

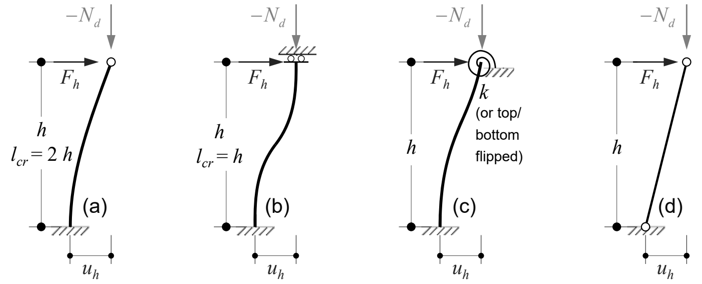
As long as piers are not extremely slender, system stability can be verified using the *linear relationship* between pier head displacement and horizontal loads used before (Figures a-c):

- Piers (a)-(c) with $|N_d| < |N_{cr,d}|$ stabilise the system
- Piers (a)-(c) with $|N_d| > |N_{cr,d}|$ destabilise the system

In addition, pin-jointed members (d) need to be considered. These are always destabilising (for compression N_d <0), and more so if they are short.

To account for the normal forces due to pier weight, a third of the pier weight should be added to N_d in the analysis.

Imposed pier head displacements and corresponding horizontal forces (second-order, EI=const; $N_{cr,d}$ and N_d are both < 0 = compression)



(a): $F_h = u_h \dfrac{3EI_d}{h^3}(1-\alpha)$, $\alpha = \dfrac{N_d}{N_{cr,d}}$, $N_{cr,d} = -\dfrac{\pi^2 EI_d}{4 \cdot h^2}$

(b): $F_h = u_h \dfrac{12EI_d}{h^3}(1-\alpha)$, $\alpha = \dfrac{N_d}{N_{cr,d}}$, $N_{cr,d} = -\dfrac{\pi^2 EI_d}{h^2}$

(c): $F_h = -u_h \dfrac{12N_{cr,d}}{\pi^2 h}(1-\alpha)$

$$N_{cr,d} = \frac{-\pi^2 EI_d}{h^2 \cdot \left(4 - \dfrac{3}{1 + \dfrac{EI_d}{k \cdot h}}\right)}$$

$k = 0$: hinged at top → as Fig. (a)

$k = \infty$: clamped at top → as Fig. (b)

(d): $F_h = N_d \dfrac{u_h}{h}$

note: compression negative → $N_d \leq 0, N_{cr,d} < 0$

Substructure – Piers: Design / Flexible systems

System stability – Determination of girder displacement

Approximation neglecting the contribution of geometrical imperfections and imposed deformations to u_h:

Horizontal loads q_h applied to the piers (earthquake, wind, …) need to be resisted by the system as well
→ add reactions $\Delta H_d\,(q_h)$ at pier tops to H_d.

Since the relationship between u_h and F_h is linear for all piers (and the displacements caused by girder expansion and contraction do not contribute to H_d), the rigid body displacement u_h of the girder can be determined as follows (stable if u_h >0):

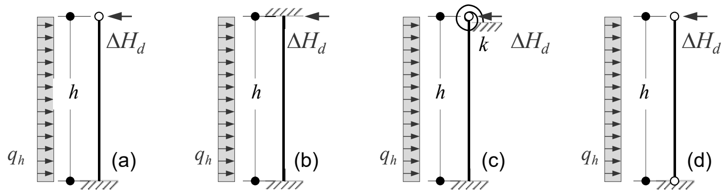
$$\sum_i F_{h,i} + \sum_j F_{h,j} = H_d + \Delta H_d(q_h)$$

$$u_h \cdot \left(\frac{12}{\pi^2} \sum_i \frac{N_{d,i} - N_{cr,d,i}}{h_i} + \sum_j \frac{N_{d,j}}{h_j} \right) = H_d + \Delta H_d(q_h)$$

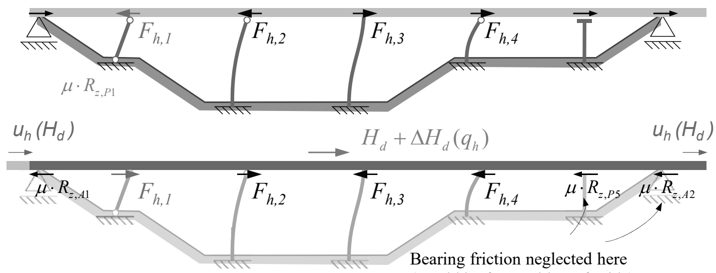
$$\rightarrow u_h = \frac{H_d + \Delta H_d(q_h)}{\frac{12}{\pi^2} \sum_i \underbrace{\frac{N_{d,i} - N_{cr,d,i}}{h_i}}_{\substack{\text{"i": all non pin-jointed piers} \\ \geq 0 \text{ for } |N_{cr,d,i}| \geq |N_{d,i}| \\ <0 \text{ else (slender piers)}}} + \sum_j \underbrace{\frac{N_{d,j}}{h_j}}_{\substack{\text{"j": pin-jointed piers} \\ \rightarrow \text{always destabilising} \\ (N_{d,j}<0)}}} \geq 0$$

Longitudinal forces applied to the piers (schematic)

$$\Delta H_d(q_h) = \frac{3qh}{8} \qquad \Delta H_d(q_h) = \frac{qh}{2} \qquad \Delta H_d(q_h) = \frac{qh}{8}\left(3 + \left[1 + \frac{4EI_d}{kh}\right]^{-1}\right) \qquad \Delta H_d(q_h) = \frac{qh}{2}$$



Equilibrium of longitudinal forces acting on girder



Note that in the approximation shown in the slide, the *rigid body displacement of the superstructure* is determined under the applied horizontal load alone, neglecting geometrical imperfections and imposed deformations (girder expansion and contraction). The latter are merely accounted for in the dimensioning of the piers by *superimposing the rigid body displacement of the superstructure* with the displacements due to *girder expansion and contraction*, as well as *geometrical imperfections*, see next slide.

Strictly speaking, the rigid body displacement u_h would have to account for the geometrical imperfections and imposed deformations, since in a second-order analysis, they also cause horizontal forces at the pier top, resulting in larger rigid body displacements of the girder. The respective 'exact' equations are:

$$\sum_i F_{h,i} + \sum_j F_{h,j} = H_d + \Delta H_d(q_h)$$

$$\sum_i F_{h,i} = u_h \cdot \left(\frac{12}{\pi^2} \sum_i \frac{N_{d,i} - N_{cr,d,i}}{h_i} \right) + \frac{12}{\pi^2} \sum_i \left(\frac{N_{d,i}}{h_i} \cdot \left(\pm u_{h,0d,i} \pm u_{h,i}(\varepsilon) \right) \right) \qquad \sum_j F_{h,j} = u_h \cdot \left(\sum_j \frac{N_{d,j}}{h_j} \right) + \sum_j \left(\frac{N_{d,j}}{h_j} \cdot \left(\pm u_{h,0d,j} \pm u_{h,j}(\varepsilon) \right) \right)$$

$$u_h = \frac{H_d + \Delta H_d(q_h) - \frac{12}{\pi^2} \sum_i \left(\frac{N_{d,i}}{h_i} \cdot \left(\pm u_{h,0d,i} \pm u_{h,i}(\varepsilon) \right) \right) - \sum_j \left(\frac{N_{d,j}}{h_j} \cdot \left(\pm u_{h,0d,j} \pm u_{h,j}(\varepsilon) \right) \right)}{\frac{12}{\pi^2} \sum_i \underbrace{\frac{N_{d,i} - N_{cr,d,i}}{h_i}}_{\substack{\text{"i": all non pin-jointed piers} \\ \geq 0 \text{ for } |N_{cr,d,i}| \geq |N_{d,i}| \\ <0 \text{ else (slender piers)}}} + \sum_j \underbrace{\frac{N_{d,j}}{h_j}}_{\substack{\text{"j": pin-jointed piers} \\ \rightarrow \text{always destabilising} \\ (N_{d,j}<0)}}} \geq 0$$

(compression negative, i.e. $N_{d,i} < 0$ and $N_{d,j} < 0$)

However, second-order effects on girder displacements due to geometrical imperfections and imposed deformations are small for realistic cases with substantial horizontal forces and moderate pier slenderness. On the other hand, the determination of u_h accounting for imposed deformations and imperfections requires studying many different cases. Hence, in preliminary design (hand calculations), the approximation is more useful.

Additional remark: In the figure, apart from the pin-jointed P1, the slender P2 (with hinge at top) is also shown as destabilising; this depends on the level of normal force (this cannot be generalised).

Substructure – Piers: Design / Flexible systems

ULS and SLS design of individual piers (1)

The design pier head displacements, used for the dimensioning of the individual piers, follow by superimposing

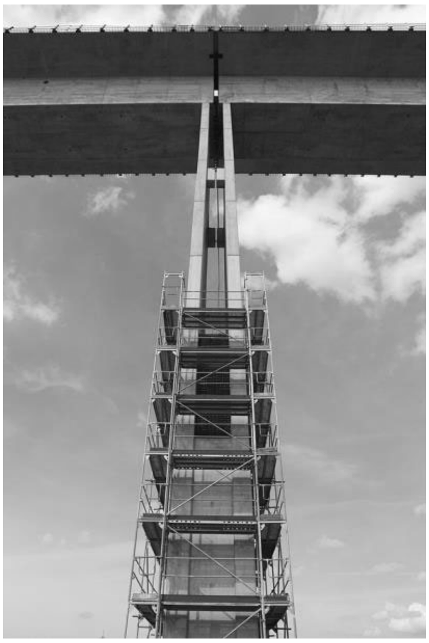
- the girder displacement $u_h(H_d)$, see previous slide (when using 'exact' formulas: $u_h(H_d; u_{h,0}; u(\varepsilon))$)
- the displacements $u_h(\varepsilon)$ due to girder expansion and contraction
- an additional displacement $u_{h,0d}$ accounting for geometric imperfections, e.g. according to SIA 262

$$u_{h,0d} = \alpha_i \cdot h_{\max}, \quad \frac{1}{200} \geq \alpha_i = \frac{0.01}{\sqrt{h_{\max}[m]}} \geq \frac{1}{300} \quad \rightarrow \frac{h_{\max}}{200} \geq u_{h,0d} = \frac{\sqrt{h_{\max}[m]}}{100} \geq \frac{h_{\max}}{300}$$

These displacements must be superimposed in the most unfavourable combination, considering different cases (expansion / contraction of girder, positive / negative longitudinal forces on girder, …):

$$u_{h,tot,i} = \pm u_{h,0d,i} \pm u_{h,i}(\varepsilon) \pm u_{h,i}(H_d)$$

and accounting for the fact that the fixed point position is not exactly known but depends on the stiffness of the foundations (see behind) and the piers. As approximation for the latter, the design stiffness $EI_d \approx 230\ M_{Rd}\ (d\text{-}d')$ may be used with an estimated reinforcement content in preliminary design).



Note that the first two terms of the total horizontal displacement at the pier top are the 1st order displacements, but do of course cause 2nd order effects in the pier. The third term includes 1st and 2nd order effects due to the horizontal forces (and 2nd order effects of geometric imperfections and imposed deformations on the rigid body displacement of the girder if the 'exact' equations for u_h given in the notes of the previous slide are used).

Photo: Separating pier of Unstruttalbrücke © Wikiwand creative commons (separating pier)

Substructure – Piers: Design / Flexible systems

ULS and SLS design of individual piers (2)

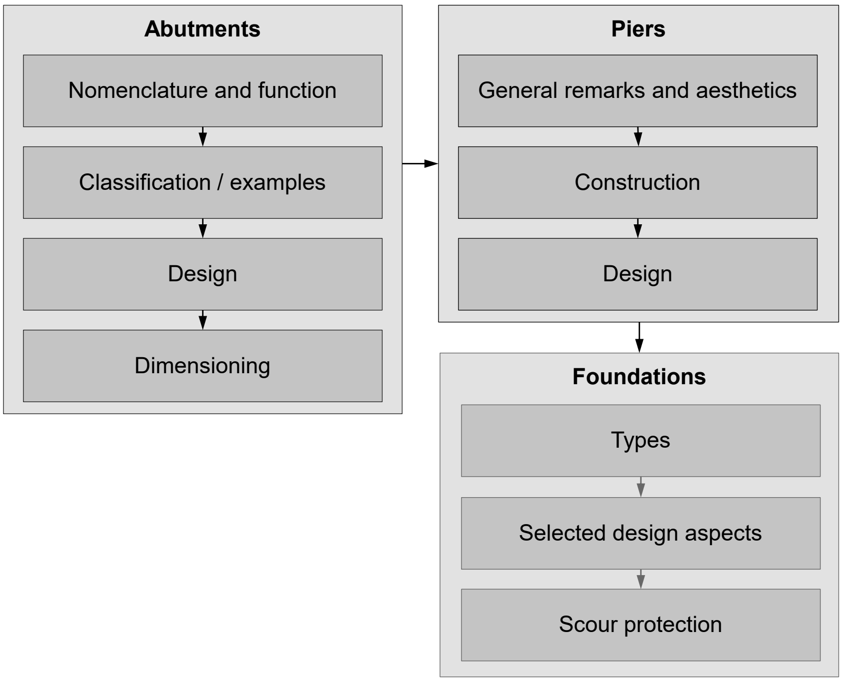
Each individual pier is then dimensioned for its governing pier head displacements $u_{h,tot}$ as outlined for piers fixed to a girder stabilised longitudinally at an abutment:

- in ULS using lower-bound values of pier stiffnesses EI_d (using $EI_d \approx 230\ M_{Rd}\,(d\text{-}d')$ for all piers is sufficient except in slender piers/systems, where it may be overly conservative)
- in SLS using characteristic values of pier stiffnesses EI_k (accounting for cracking, which reduces bending moments and minimum reinforcement demand)

The procedure outlined on the previous slides is applicable in cases where the linear relationship between pier head displacement and horizontal loads is reasonably accurate (no extremely slender piers), and as long as the assumption of a conservative design stiffness EI_d for all piers is not overly conservative.

In other cases, a second order calculation of the entire system – following similar lines as in the approximation, but using less conservative pier stiffnesses but accounting for geometric imperfections of the individual piers – is recommended.

Photo: Puente sobre el río Tajo en el Embalse de Alcantara en la LAV Madrid-Extremadura tramo Cañaveral-Embalse Alcántara (2016), Carlos Fernández Casado SL


Abutments
Nomenclature and function
Classification / examples
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Substructure

Foundations

Substructure

Foundations – Types

Substructure – Foundations: Types

Spread footings

Spread footings are usual for abutments due to their large dimensions.

On relatively stiff soil at shallow depth, spread footings may also be used for piers. Since pier reactions are often high, correspondingly large dimensions are required except in solid rock.

If soft soil layers extend several meters from the surface, excavation pits become large and expensive → pile foundation more economical.

Jonentobelbrücke, dsp Ingenieure+Planer AG, ACS Partner AG

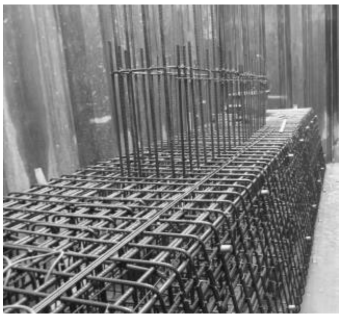
raft foundation of pier / preparation of abutment foundation

Substructure – Foundations: Types

Pile foundations – Driven piles

In soft soil, driven piles (“Rammpfähle”) of small-medium diameter (40...60 cm) are economical, since skin friction carries most of the load. Driven piles may be prefabricated (e.g. spun concrete / Schleuderbetonpfähle) or cast in situ (Ortbetonrammpfähle).

Several driven piles are required per pier foundation. These piles are commonly connected through a pile cap accommodating tolerances and carrying the pier.



Steinbachviadukt. Pile driving equipment on barge (90t, falling mass 9t), driven piles (spun concrete Ø45 cm) and pile cap in sheet pile caisson.

Substructure – Foundations: Types

Pile foundations – Bored piles (aka “drilled pier foundations”)

In stiffer soil, bored piles (“Bohrpfähle”) are used, with larger diameter than driven piles. Bored piles carry more load and are also better suited to transfer horizontal loads.

Bored piles may be cased (verrohrt) or uncased (unverrohrt), depending on borehole stability. In piles reaching below the groundwater level, water ingress must be prevented (fill casing with water or bentonite suspension). Concrete is cast using hoses (tremie pipes) (Contractorverfahren) to prevent segregation.

While pile diameters of 1.20 m were considered as very large few decades ago, diameters of 1.50 m or even 1.80 m are common today. As a prerequisite, large machinery must be able to access the site.

Compared to driven piles, fewer piles are required per pier foundation. Still, they are commonly connected through a pile cap accommodating tolerances and carrying the pier.

Ultrasonic pile integrity checks (using tubes installed before casting) are common today. Alternatively, impact echo testing is also used, but is less reliable.

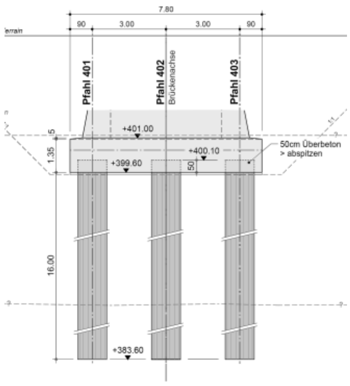
Brücke Schönenwerd, Dietikon. Bored piles D= 150 cm. Photo © dsp Ingenieure + Planer

Substructure – Foundations: Types

Rotary drill

Concreting

Pile cap



Pile heads

Pile cap formwork

Brücke Schönenwerd, Dietikon. Bored piles D= 150 cm. Photo © dsp Ingenieure + Planer

Brücke Alter Rhein (Zollbrücke) St. Margrethen, Bored piles D=120 cm. Photo © dsp Ingenieure + Planer

Substructure – Foundations: Types

Pile foundations – pile tests

The dimensioning of pile foundations is often conservative, since the soil properties at large depths are uncertain.

In-situ static pile tests allow accounting for higher bearing capacities. Due to the high cost of such tests, and lack of time in most projects, they are only rarely carried out.

In the example shown in the photos, roughly 20% of the planned total pile length of 9.6 km could be saved, making the tests worthwhile.

Alternatively, dynamic pile tests are also being used. They are less expensive, but yield less direct information on the bearing capacity (→ higher safety margin required)

Steinbachviadukt, Pile tests.

Substructure – Foundations: Types

Shaft foundations (aka “excavated pier foundations”)

Shaft foundations are a viable alternative to transfer loads to stiff soil (rock) in moderate depth. The shafts are excavated to the required depth resp. the desired soil layer (typically rock) and filled with concrete reinforced in the upper part.

An advantage of shafts is that the soil properties at foundation level can be examined 1:1.

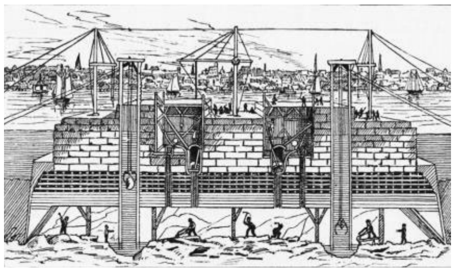
Versamertobelbrücke, Shaft foundation.

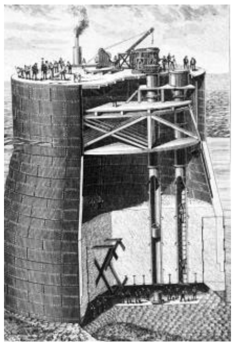
Substructure – Foundations: Types

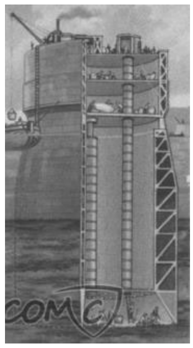
Caisson foundations

Caissons are watertight prismatic hollow foundation structures built above the ground level and sunk to the required depth for foundations under the water level. The following types can be distinguished:

- Pneumatic caissons are bottomless boxes, filled with compressed air to keep the water out and provide a dry working chamber where excavation can be carried out. They were used where open caissons could not be sunk due to obstacles (boulders), or would be unstable.







Top: Pneumatic caisson of Brooklyn Bridge (Source: A complete history on the New York and Brooklyn Bridge: from its conception in 1866 to its completion in 1883 / compiled by S. W. Green; ETH-Bibliothek Zürich, Rar 2156, https://doi.org/10.3931/e-rara-15277 / Public Domain). This was one of the first projects using pneumatic caissons, and the need for controlled decompression of workers was unknown at the time It resulted in numerous workers being either killed or permanently injured by “caisson disease” (decompression sickness) during its construction.

Bottom: Pneumatic Caisson of the Forth Bridge (Source: Will’s Engineering Wonders, 1927, https://www.amazon.com/Caisson-REVIEWED-Wills-Engineering-Wonders/dp/B00C104AYE?tag=pops02-20

Substructure – Foundations: Types

Caisson foundations

Caissons are watertight prismatic hollow foundation structures built above the ground level and sunk to the required depth for foundations under the water level. The following types can be distinguished:

- Pneumatic caissons are bottomless boxes, filled with compressed air to keep the water out and provide a dry working chamber where excavation can be carried out. They were used where open caissons could not be sunk due to obstacles (boulders), or would be unstable.
- Open caissons have neither top nor bottom cover. They were used mainly for foundations in sandy soil and shallow water, typically using the “sand island method” and underwater excavation using clamshells.
- Box caissons are closed on the bottom and lowered through water onto a prepared foundation layer, typically consisting of a sand bed. Alternatively, steel caissons serving as formwork for underwater concrete are also used.

While they were widely used in the past, caissons have become largely obsolete through the development of other methods, particularly large diameter bored piles.

Top: Open steel caisson of a pier of the Changtai Yangtze River bridge, Jiangsu, China, 2019. This was the world’s largest open steel caisson at the time of construction.

Bottom: Steel caisson (formwork for underwater concrete) of the Akashi Kaikyo Bridge, 1997. Source: Scientific American.

Substructure

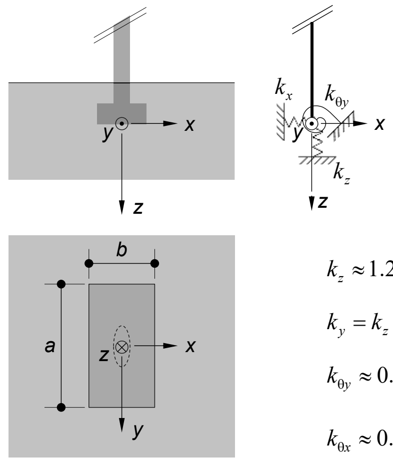
Foundations – Selected design aspects

Substructure – Foundations: Selected design aspects

General remarks and stiffness of spread footings

- Foundations are an important part of bridges, and often high vertical loads need to be transferred to the subsoil
  - → strength (“bearing capacity”) of foundation highly relevant
  - → pile foundations frequent
- If large horizontal forces need to be transferred to the subsoil, combine with large vertical reactions whenever possible (steep inclination of resultant force)
- Bridge piers are often stiff compared to the foundation
  - → modelling foundations as infinitely stiff is inadequate
- Appropriately modelling the stiffness of foundations is particularly relevant for
  - → design of slender piers (buckling length)
  - → (semi-)integral bridges (quantify restraint, position of movement centre, …)
  - → seismic design
- The stiffness of spread footings may be modelled using elastic springs at the pier base, see figure on this slide

Elastic stiffness of spread footings
(for global structural analysis of bridge)



$$k_z \approx 1.25 \frac{M_e}{1-\nu^2} \sqrt{ab}$$

$$k_y = k_z \approx M_e \sqrt{ab}$$

$$k_{\theta y} \approx 0.25 \frac{M_e}{1-\nu^2} ab^2$$

$$k_{\theta x} \approx 0.25 \frac{M_e}{1-\nu^2} ba^2$$

Illustrations and formulas: W. Kaufmann: AGB Report 629: Integrale Brücken – Sachstandsbericht.

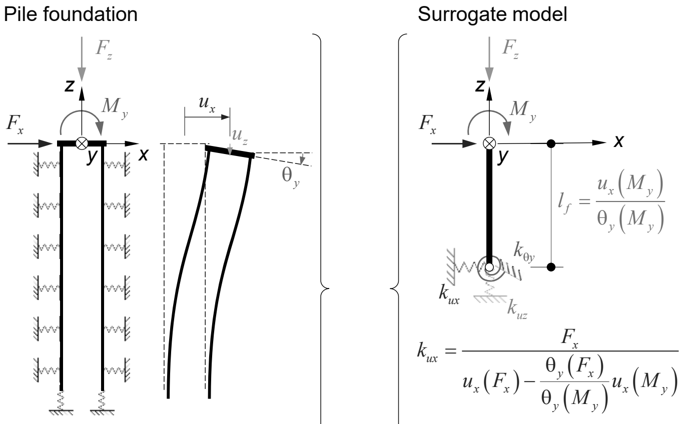
Substructure – Foundations: Selected design aspects

General remarks and stiffness of pile foundations

- The modelling of pile foundations with elastic springs at the pier foot is not straightforward, as lateral movement and rotation are coupled
  → include pile foundations in global analysis model (piles with lateral and vertical elastic springs)
- Alternatively, the model illustrated in the figure can be used. The length l_f of the rigid bar and the stiffnesses are determined such that the surrogate model shown on the right side has the same global response as the pile foundation (displacements and rotations used to define l_f and the stiffnesses are determined from a separate model of the pile foundation).

For details on geotechnical design see lectures of IGT

Elastic springs+rigid bar model for stiffness of pile foundation (for global structural analysis of bridge)



$$\begin{Bmatrix} F_x \\ M_y \\ F_z \end{Bmatrix} \rightarrow \begin{Bmatrix} u_x \\ \theta_y \\ u_z \end{Bmatrix} = \begin{Bmatrix} u_x(F_x) + u_x(M_y) \\ \theta_y(M_y) + \theta_y(F_x) \\ u_z(F) \end{Bmatrix}$$

$$l_f = \frac{u_x(M_y)}{\theta_y(M_y)}$$

$$k_{ux} = \frac{F_x}{u_x(F_x) - \frac{\theta_y(F_x)}{\theta_y(M_y)} u_x(M_y)}$$

$$k_{\theta y} = \frac{M_y}{\theta_y(M_y)}$$

$$k_{uz} = \frac{F_z}{u_z}$$

W. Kaufmann: AGB Report 629: Integrale Brücken – Sachstandsbericht.

Substructure

Foundations – Scour protection

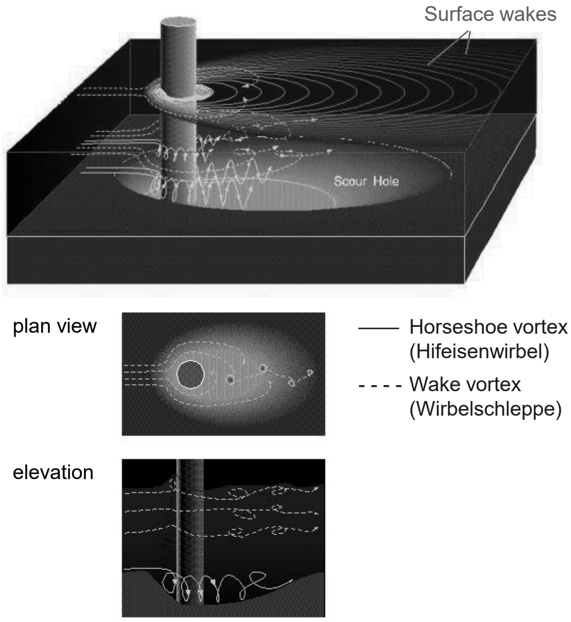
Substructure – Foundations: Scour protection

Scour protection

If foundations are located in or near streams (rivers, creeks, tidal channels), erosion of the channel bed, known as scour, must be considered when designing the foundation.

Erosion may occur due to turbulence caused by the bridge piers and foundation (local scour, see figure), or larger scale effects.

Deep foundations, using shafts or piles, are effective measures to prevent scour damage.



Hydrodynamic scour is the removal of sediment such as silt, sand and gravel from around the base of obstructions to the flow in the sea, rivers and canals. Scour, caused by swiftly moving water, can scoop out scour holes, compromising the integrity of a structure. It is an interaction between the hydrodynamics and the geotechnical properties of the substrate. It is a notable cause of bridge failure and a problem with most marine structures supported by the seabed in areas of significant tidal and ocean current.

Figures © U.S. Geological Survey, www.usgs.gov

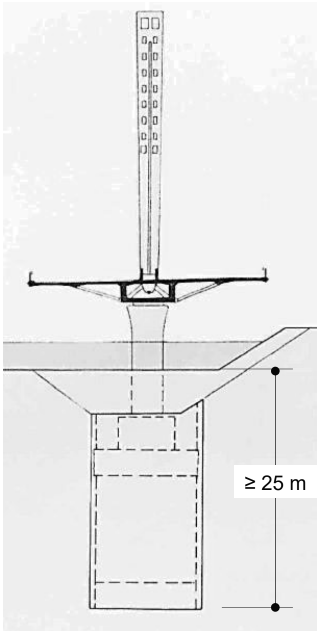
Substructure – Foundations: Scour protection

Scour protection

In spite of the general awareness of the issues related to scour, it remains a notable cause of bridge failures.

This may be due to the fact that the foundation depths required for an effective scour protection may often appear excessive at first glance. As an example, in the Chandoline bridge (figures), the foundations had to be located 25 m below the riverbed.

However, deep scour has been observed in many bridges, which justifies deep foundations in or near streams to avoid scour – as observed in the cases illustrated on the following slides.



Pont de Chandoline, C. Menn / KBM Ingenieurs (1989). Spans 72+140+72 m, width 27 m. Foundation 25 below riverbed requested by client for scour protection.

Figure from C. Menn, “An Approach to Bridge Design”, Eng Struct Vol. 13, April 1991, pp. 106-112

Photo © christian-menn.ch

Substructure – Foundations: Scour protection

Scour protection

Flood events, particularly in alpine regions, where rivers and creeks change their beds, are a typical cause of damage due to scour.

The Reussbrücke Wassen (right), built 1972, was severely damaged due to scour in a flood event on the 24./25.8 1987. It could be repaired and is still used oday, see lecture Stahlbeton I (rotation capacity)

A more recent example is the Hüscherabachbrücke near Splügen GR (below), damaged by scour in a flood event in June 2019.

Substructure – Foundations: Scour protection

Scour protection

A famous bridge affected by scour is the Sunnibergbrücke, whose foundations suffered severe scour in a flood event in 2005, while still closed to the public.

Since the large diameter pile foundation ensured sufficient resistance, only limited repair was required and the bridge could be opened as planned in 2006,

Foundation of Sunnibergbrücke (Christian Menn, Bänziger Partner AG, 1998/2006), scour after the flood event on 21.-23.8.2005

T. Vogel, K. Schellenberg: “The Impact of the Sunniberg Bridge on Structural Engineering, Switzerland”, Structural Engineering International, IABSE, 04/2015, pp. 381-388

Overview © Tiefbauamt Graubünden

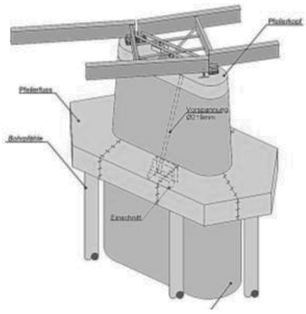
Substructure – Foundations: Scour protection

Scour protection

Historical bridges are often subject to scour, since the technology at the time of their construction did not allow as deep foundations as today (particularly no large diameter piles available). In such bridges, the riverbed must be inspected regularly.

As an example, in the Aarebrücke Koblenz, divers detected up to 9 m deep scour in the riverbed after a flood event in 1999. A «concrete block carpet» was installed immediately for protection, and freight trains must not pass the bridge at speeds higher than 30 km/h since.

Currently, each pier is secured with a new pile cap sitting on four large diameter piles drilled to the solid rock (10…18 m below ground). At the same time, the piers are strengthened for horizontal forces.



Aarebrücke Koblenz (1892): Rehabilitation 2019-2012, Staubli, Kurath & Partner)

Photos: © Georg Aerni / Figure © SBB / Staubli, Kurath+Partner AG