Bridges with Integral Abutments: Introduction

Integral Abutment Bridges are bridges with no expansion joints and no bearings. Their largest benefits are the lower construction and maintenance costs. Bridges of this type are widely spread in the United States but are also becoming more popular in Europe, but the technical solutions are very different in different countries. In some countries slender piles are used, minimizing the strains in the piles from thermal displacements of the bridge, while the approach in other countries is based upon heavy piles. Some countries also design for seismic loads which is not the case in other countries.

Structural Engineering International received a great response from around the world to its call for papers on the topic of Integral Abutment Bridges. The number of abstracts submitted, and subsequent high-quality papers received, prompted the extension of this Special Edition over two issues - the present issue, as well as the coming August issue. In this first issue, eight Scientific Papers on topics spanning from general papers describing the concept in different countries to results from the monitoring of integral abutment bridges are presented. The Scientific Papers are complemented by three Technical Reports showing the design and construction of three bridges with integral abutment. Our hope is that the papers will be of great value to researchers, bridge owners and bridge designers, and in the long run will contribute to more economical bridge solutions.

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Transition Slabs of Integral Abutment Bridges

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Abstract

Over the past decades, an increasing number of bridges with integral abutment have been built in Switzerland. This type of bridge offers various advantages over standard bridges with abutments, equipped with expansion joints and bearings that require regular inspection and maintenance. One main concern of integral abutment bridges is related to the soil–structure interaction, in particular between the transition slab and the embankment. To avoid any expansion joints, transition slabs are directly connected to the end of integral abutment bridges. They are therefore subject to large displacements of the bridge deck due to temperature effects and creep and shrinkage in concrete bridges. Consequently, detailing of transition slabs needs to be carefully considered. This paper investigates the behaviour of transition slabs, focusing on the settlement of the pavement at the end of the transition slab and on the cracking of the pavement between the bridge deck and transition slab. On that basis, a modified geometry of the transition slab and a new detail for the connection between the bridge deck and the transition slab are proposed. If these propositions are considered at an early stage in the design process, they will result in an improved long term performance of bridges with integral abutments without increasing the construction costs.

Keywords: integral abutment; semi-integral abutment; transition slab; soil–structure interaction; durability; serviceability limit state; conceptual design.

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Introduction

Over their service life, bridge decks expand or contract as a result of time and temperature-dependent effects. The displacement u at the bridge end is used to define the required type and opening of the expansion joints.[1] It can be estimated by Eq. (1) (*Fig. 1a*).

$$u = \varepsilon_{\mathrm{imp}} d_{\mathrm{fp}} = (\varepsilon_{\Delta T} + \varepsilon_{\mathrm{cr}} + \varepsilon_{\mathrm{c,sh}}) d_{\mathrm{fp}} \qquad (1)$$

where:

- $\varepsilon_{\mathrm{imp}} = (\varepsilon_{\Delta T} + \varepsilon_{\mathrm{cr}} + \varepsilon_{\mathrm{c,sh}})$ is the imposed deformation;
- $\varepsilon_{\Delta T} = \alpha_{\mathrm{T}}\ \Delta T$ is the temperature deformation;
- α_{T} is the thermal expansion coefficient;
- $\Delta T = T_0 \pm T_k$ is the variation of the uniform temperature of the bridge deck;
- T_0 is the temperature at the time of the bridge completion;
- T_k is the characteristic temperature of the bridge deck taking into account the effects of extreme daily and seasonal temperatures, generally specified in national codes[2];
- $\varepsilon_{\mathrm{cr}}$ is the creep deformations of concrete, which can for example be taken from national codes[3];
- $\varepsilon_{\mathrm{c,sh}}$ is the shrinkage deformations of concrete, which can for example be taken from national codes[3];
- d_{fp} is the distance between the fixed point and the end of the bridge.

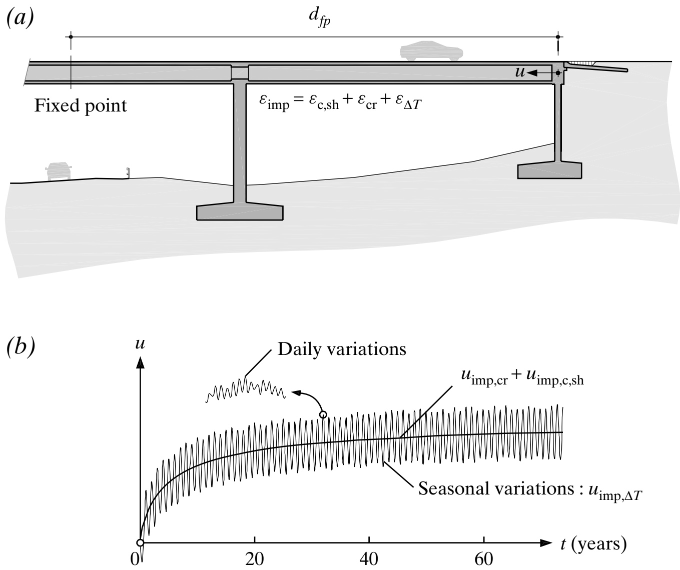

Fig. 1: Displacement u at the end of a concrete bridge: (a) definition of imposed deformation ε_{imp} and distance d_{fp} between the fixed point and the end of the bridge; (b) evolution of the imposed displacement u_{imp} at the end of the bridge with time t

For the estimation of the required opening of the expansion joint, the temperature T_0 to be considered is the concrete bridge deck temperature at the time t_0 of the final positioning of the expansion joint and for ε_{cr} and $\varepsilon_{c,sh}$ the increment between that time and the time t_k at which the characteristic temperature T_k prevails. The determination of the bridge fixed point is not always a simple task. Indeed, if the bridge under consideration is not fixed at one of its abutments, the location of the fixed point must be determined by considering the behaviour of columns comprising the intermediate supports of the bridge and their foundations. In integral bridges, the behaviour of the integral abutment (without expansion joint and bearings) also needs to be taken into account. A typical evolution of the displacement of a concrete bridge deck end u is given in *Fig. 1b*. The increase in u is particularly important during the first years after construction, because of creep ε_{cr} and shrinkage $\varepsilon_{c,sh}$ deformations.

In traditional bridge construction, the deck is longitudinally disconnected from the abutment by an expansion joint and bearings as shown in *Fig. 2a*. This disconnection prevents movements and thus any forces on the abutment and the embankment caused by displacement of the bridge deck u. However, abutments with joints have some serviceability and durability problems. Indeed, the degradation of the mechanical elements comprising the expansion joints and bearings occurs over time. These degradations are significant, especially in countries where deicing salt is intensely used, as in Switzerland,[4] and in bridges exposed to sea water. Consequently, in cold or coastal areas, these mechanical elements must be replaced approximately every 20 to 30 years, which leads to costly and complicated maintenance operations.

In Switzerland, standard abutments with joints are typically built with a transition slab. These slabs provide a smooth transition between the embankment and the bridge structure. They have another important function, namely to bridge over possible settlements in the vicinity of the abutment wall.[5] These settlements are mainly caused by mechanical compaction defaults, erosion or consolidation of natural soil due to embankment load.[6]

Semi-integral abutments (without expansion joint) or integral abutments (without expansion joint and bearings) are solutions to avoid the degradations related to expansion joints and bearings.[7,8] In these configurations, the bridge deck and the abutment are longitudinally connected. As a consequence, the displacement u of the bridge end is transmitted to the abutment.

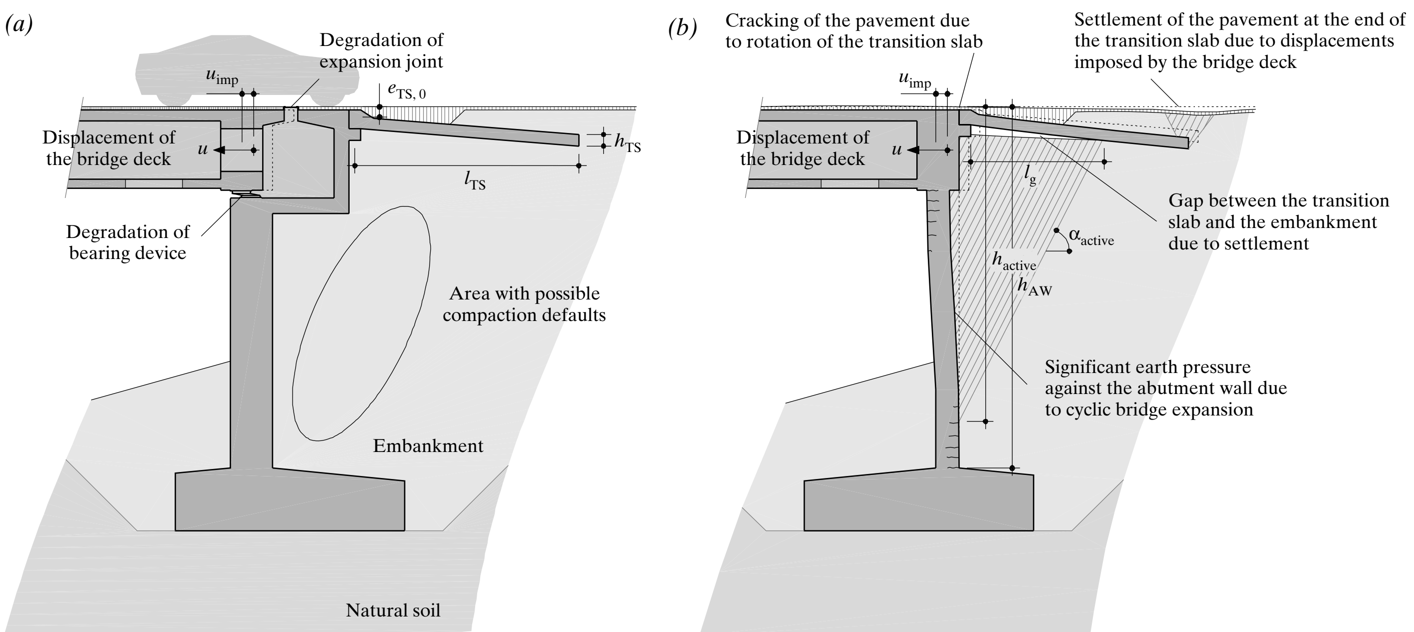

Fig. 2: Phenomena caused by the longitudinal displacement of the bridge deck; (a) standard abutment with joints; (b) integral abutment

Abutments of Integral Bridges

Abutments of integral bridges are subjected to imposed displacement $u_{imp} = u$ because of their longitudinal connection with the bridge deck. As shown in *Fig. 2b*, u_{imp} leads to phenomena which are generally negligible for standard abutment with joints.

The imposed displacement u_{imp}, transmitted to the head of the abutment wall, leads to an imposed deformation of the wall and consequently to internal forces in the wall. This displacement also causes changes in the distribution and intensity of the earth pressure σ_h behind the abutment wall. This complex soil–structure interaction needs to be taken into consideration to determine σ_h. The deformations of the abutment wall also lead to active soil mechanisms causing settlements under the transition slab (*Fig. 2b*).

Abutment Walls

As mentioned before, there is a complex soil–structure interaction between the abutment wall and the embankment behind it. Similar to a conventional retaining wall, Eq. (2) can be used to estimate the earth pressure σ_h against the wall of a standard abutment with joints as well as for the wall of an integral abutment.

$$\sigma_h = k\sigma_v \tag{2}$$

where k is the earth pressure coefficient and σ_v is the vertical stress due to the dead load of the backfill located above the considered vertical location.

For a standard abutment, the earth pressure coefficient k can be assumed as k_a for the ultimate limit state, where k_a is the active earth pressure coefficient. For integral abutments, the estimation of the earth pressure coefficient k is more difficult because the history of the displacement of the abutment wall has a significant influence. Cosgrove and Lehane[9] have experimentally shown the influence of cyclic wall displacements on k. In particular, they have shown that the value of k can become close to the passive earth pressure coefficient k_p even for small displacements of the abutment wall. This large value of k can be explained by the stiffening of the embankment subsequent to a cyclic compaction in the vicinity of the abutment wall. This increase in the stiffness of the embankment was also observed in field measurements.[10] Once the earth pressure σ_h against the abutment wall and the imposed wall deformations induced by u_{imp} are known, the internal forces in the abutment wall can be evaluated with conventional structural engineering methods. In the design of the abutment wall, particular attention should be given to the cracking of the wall. At the ultimate limit state, a flexible abutment wall is an elegant solution to prevent excessive internal forces due to imposed deformations. However, flexural and shear failure of the flexible abutment wall must be prevented even for k close to k_p.

In the same experimental study, Cosgrove and Lehane[9] have shown that the settlement in the vicinity of the abutment wall due to u_{imp} comes from the repeated active failure of the embankment subsequent to the cyclic displacement of the abutment wall. Dreier[11] has shown that the length l_g of this settlement can be reasonably estimated by Eq. (3). This estimation is based on an active plastic mechanism.

$$l_g = h_{active}/\tan(\alpha_{active}) \text{ with}$$
$$\alpha_{active} = 45^\circ + \varphi/2 \tag{3}$$

where h_{active} is the depth of the active plastic mechanism zone that can be approximated as the height of the abutment wall h_{AW} and φ is the friction angle of the backfill.

The transition slab, already used to bridge embankment compaction defaults, is also effective for bridging this cyclic settlement. Consequently, the design of the reinforcement of the transition slab must consider the bridging length l_g. Simply stating, the transition slab can be designed by considering the transition slab as supported at the end of the bridge deck and at the end of the settlement.[11]

Settlement at the End of the Transition Slab

To avoid any expansion joints, transition slabs are directly connected to the integral abutment. As a consequence, they are also subject to imposed displacement u_{imp}, with the main effect of localising deformations in the soil and surface settlements in the vicinity of the end of the transition slab (*Fig. 2b*).

This imposed displacement u_{imp} causes a local settlement of the pavement at the end of the transition slab, induced by an active plastic mechanical development in the embankment. This settlement at the end of the transition slab can become problematic for the serviceability limit state, as it reduces the planarity of the road pavement and degrades the comfort of the road users. This phenomenon has not been investigated so far. The following text of the paper presents and explains this phenomenon and shows possible ways to minimise it.

Numerical Model

A finite element software[12] was used for the simulation of the soil–structure interaction in the vicinity of the abutment of integral bridges. This software's results have been thoroughly compared with experimental values and are quite reliable.[13,14]. It has the ability to reproduce the complex behaviour of gravel backfills and to simulate the interaction between the embankment and the structural elements of the bridge end, which were the main parameters of this investigation. The geometry of a semi-integral abutment shown in *Fig. 3a* was investigated. The main variables used to define the geometry of the studied transition slab were its length l_{TS}, its slope α_{TS}, its thickness h_{TS} and the buried depth of the transition slab at the connection with the bridge deck $e_{TS,0}$ (*Fig. 3a*). For this study, the differences between the semi-integral abutment of the model and an integral abutment are minor. The system is composed of three materials. The first material is the granular material (gravel backfill) of the embankment. The second material is the reinforced concrete used for the transition slab, the abutment wall and the bridge deck. The third material is the bituminous road surfacing. The mechanical characteristics of these materials are extremely different. The Hujeux[15,16] mechanical model was used to simulate the behaviour of the backfill of the embankment. It uses an elastic–plastic approach with multiple plastic mechanisms. This approach covers all types of soils, from a perfectly granular material to clay. This model includes in its formulation the entire range from well-known elastic–plastic models with a Mohr–Coulomb failure criterion to the Cam–Clay model.[17] The behaviour of the reinforced concrete structural elements was assumed to be elastic, with a reduced stiffness accounting for the flexural cracking state of the transition slab. A soil–structure interface element was used between the reinforced concrete elements and the embankment to reproduce the adherence and to allow a relative displacement between the soil and concrete elements. A rigid–plastic mechanical model was chosen for the interface with a Mohr–Coulomb failure criterion. The behaviour of the bituminous road surfacing was modelled with elastic elements instead of its actual viscous behaviour. Indeed, the velocity of the imposed displacement u_{imp} is really low, such that time-dependent effects could be neglected even at low temperatures. Its assumed modulus of elasticity was consequently really low. In this study, the wing walls of the abutment are assumed to be structurally disconnected from the rest of the bridge and were thus not considered. The zone behind the abutment was investigated with a plane strain two-dimensional (2D) model. To avoid edge effects, the mesh was extended to include a significant portion of the embankment (length × height = 17,2 × 7,5 m while the structures occupies 7 × 2 m). The size of the elements composing the mesh was progressively decreased, from 0,9 × 0,9 m at the edges of the mesh to 0,03 × 0,03 m in the most deformed area. A total of more than 3000 elements were used, of which approximately 2000 elements are concentrated in the vicinity of the end of the transition slab.

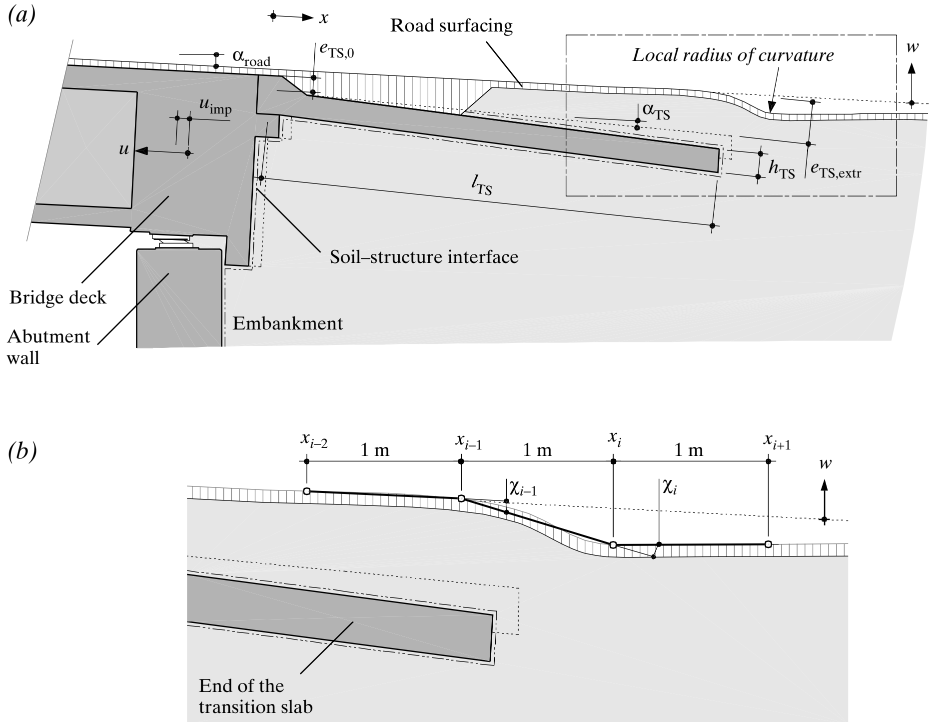

Fig. 3: Model for the numerical study of the settlement at the end of the transition slab; (a) geometry and materials; (b) definition of the slope variation criterion χ according to Swiss codes[18,19]

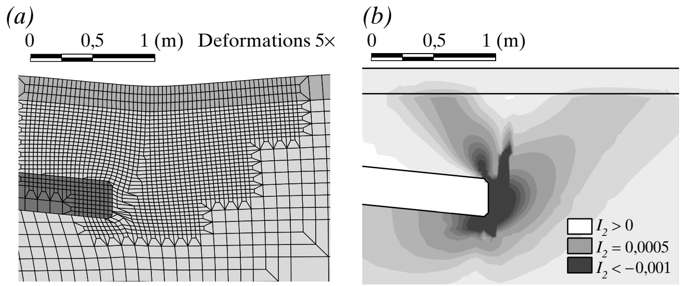

Fig. 4: Results of numerical study for u_{imp} = 50 mm; (a) mesh deformation; (b) second strain invariant I_2

Road Pavement Planarity Limit State

To quantify the planarity of the road pavement, a slope variation criterion χ according to the Swiss code[18] was used. This criterion (*Fig. 3b*) provides an efficient evaluation of the local curvature, related to the comfort of road users. It takes into account the depth and the length of the settlement. The slope variation χ, defined mathematically in Eq. (4), must always be less than the limit value χ_{adm}. According to the Swiss code,[19] χ_{adm} is equal to 28‰ for normal roads and 20‰ for highways.

$$\chi(x) = \frac{w(x) - w(x - 1\,\text{m})}{1\,\text{m}} - \frac{w(x + 1\,\text{m}) - w(x)}{1\,\text{m}} = \frac{2w(x) - w(x - 1\,\text{m}) - w(x + 1\,\text{m})}{1\,\text{m}} \le \chi_{adm}(x) \quad (4)$$

Results for a Standard Geometry of the Transition Slab

This section presents the main results of the numerical study. The assumed geometry of the transition slab is consistent with the Swiss recommendations for transition slabs of integral bridges,[5] (*Fig. 3a*). Thus, l_{TS} is equal to 6 m, α_{TS} = 10%, h_{TS} = 0,3 m and $e_{TS,0}$ = 0,1 m. The thickness of the bituminous road pavement is 0,07 m. The slope of the road α_{road} is assumed equal to zero. This last assumption has no influence on the results, as α_{TS} is defined with respect to α_{road}.

The main parameters of the embankment correspond to a well graded gravel backfill with a limited proportion of fines. Its specific weight γ is equal to 19,4 kN/m^3 and its initial void ratio e_0 is 0,32. Its bulk modulus K_{ref} is 47,1 MPa and its shear modulus G_{ref} is 21,7 MPa at the mean reference effective pressure p_{ref} of –0,1 MPa. Its friction angle φ is assumed equal to 36°. Additional soil parameters for the model were defined according to the work on the Hujeux model by Lassoudière and Meimon.[20] More details about the choice of these mechanical parameters can be found in Ref. [11].

For this study, the time t_0 to be considered is the time when the transition slab is completed and longitudinally connected to the concrete bridge deck. The characteristic time t_k is the time when creep and shrinkage of concrete are fully developed and the temperature T_k is minimal.

Figure 4a shows the mesh deformation with an amplification factor of 5 and *Fig. 4b* shows the second strain invariant I_2. In both cases, the imposed displacement u_{imp} of the transition slab was chosen equal to 50 mm.

Figure 4 highlights the relative movement between the soil and the end of the transition slab. In particular, the figure shows the localisation of strains between the soil located over the transition slab and the embankment after the end of the transition slab (*Fig. 4b*). This localisation leads to a local settlement at the end of the transition slab which reduces the planarity of the road pavement (*Fig. 4a*).

Figure 5 shows the evolution of the vertical deformation w of the road surface and the corresponding slope variation χ for various values of the imposed displacement u_{imp}. *Figure 5a* reveals a global settlement of the transition slab starting at $x/l_{TS} = 0{,}3$ due to the active soil mechanism subsequent to the displacement of the end of the bridge deck and a significant local settlement at the end of the transition slab, approximately between $x/l_{TS} = 0{,}8$ and 1,3. The location of the local settlement is independent of u_{imp} studied. However its depth continuously increases with increases of u_{imp}. This observation is even more pronounced in *Fig. 5b* for the slope variation χ. *Figure 5b* shows four different areas with localisations of the slope variation χ. The first one is at $x/l_{TS} = 0$ where the transition slab is connected to the bridge deck. It is caused by the rotation of the transition slab due to u_{imp}. The other three are located in the vicinity of the local settlement. At the edges of the local settlement, positive slope variations develop two "bumps" and a negative slope variation at the bottom of the local settlement leads to a "hole".

To determine if the imposed displacement u_{imp} is acceptable for road users, that is, if it satisfies the admissible slope variation χ_{adm}, the graph shown in *Fig. 6* can be used. It was prepared by plotting the maximal and minimal values of χ in the vicinity of the transition slab as a function of u_{imp}. This graph can also be used to determine the maximal admissible displacement $u_{imp,adm}$ for a given χ_{adm}. In all the cases studied, the "hole" was controlling. For a standard geometry of the transition slab, this gives 43 mm as the admissible imposed displacement $u_{imp,adm}$ for highways. Consequently, in accordance to national codes,[2,3] if a total imposed deformation $\varepsilon_{imp} = -0{,}8$ mm/m for a concrete bridge deck is assumed (sum of $\varepsilon_{cr} \approx -0{,}2$, $\varepsilon_{c,sh} \approx -0{,}35$ mm/m and $\varepsilon_{\Delta T} \approx -0{,}25$ mm/m due to $\Delta T = -25$°C), the maximal distance between the investigated abutment and the fixed point of the bridge d_{fp} is 54 m. In this case, the largest possible length for an integral concrete bridge with a fixed point at its midpoint is 108 m. An interesting possibility for concrete bridges is, at the time of a major retrofitting of the bridge ends, to transform a conventional bridge into a bridge with integral or semi-integral abutments. In this case, only the residual creep and shrinkage deformations need to be taken into account. Thus, for the admissible imposed displacement $u_{imp,adm} = 43$ mm and assuming that the residual deformation ε_{imp} is $-0{,}4$ mm/m, the maximal possible distance between the new integral abutment and the fixed point of the bridge d_{fp} is 108 m. Thus, the largest possible length for a bridge with two retrofitted abutments and a fixed point at its midpoint is 216 m. For steel bridges ($\Delta T = -35$°C, $\varepsilon_{imp} = \varepsilon_{\Delta T} \approx -0{,}35$ mm/m) the largest possible length can even reach 246 m.

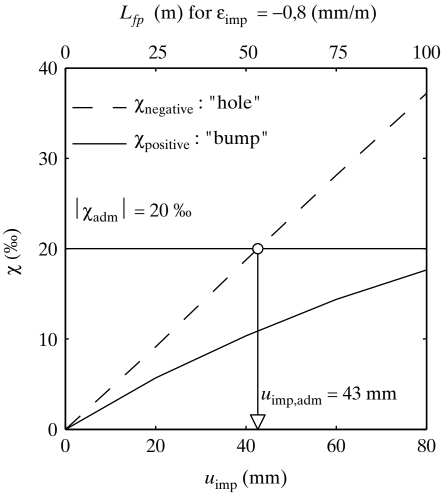

Fig. 6: Determination of the admissible imposed displacement $u_{imp,adm}$ for $\chi_{adm} = 20$‰ according to Swiss codes[18,19]

The controlling situation is usually for a movement away from the abutment (active soil failure). Investigations in the passive direction have shown that the settlements for this case are significantly less problematic. This is due to the fact that the displacement needed to activate the passive plastic mechanism is quite larger than for the active mechanism. In addition, for concrete bridges, ε_{imp} includes components that cause large deformations in the active direction due to creep and shrinkage.

Parametric Study

A parametric study was performed to investigate the influence of the geometry of the transition slab and the type of backfill on the admissible imposed displacement $u_{imp,adm}$. The results of the parametric study of the geometry are shown in *Fig. 7*. The admissible

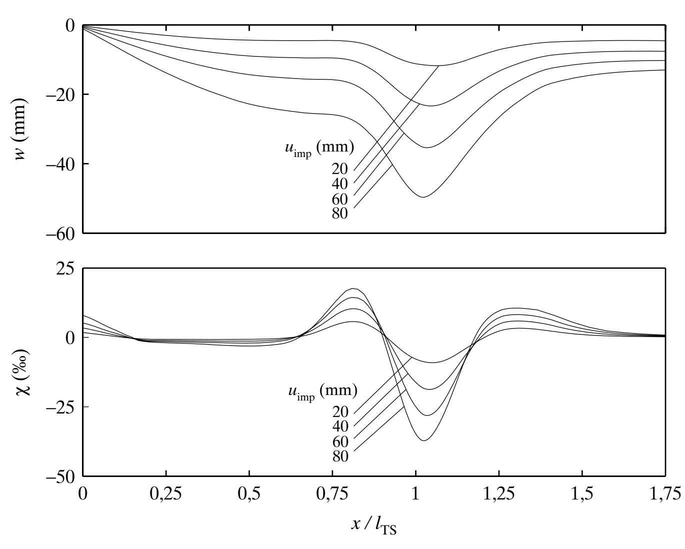

Fig. 5 (a, b): Evolution of the vertical settlement w and the slope variation χ of the road pavement as a function of the imposed displacement u_{imp}

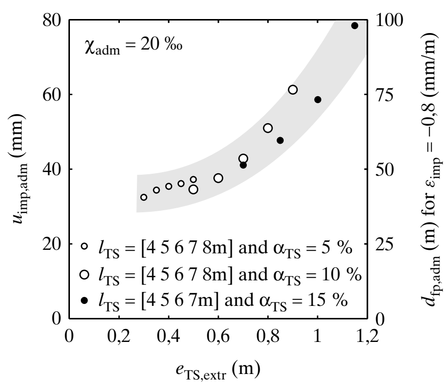

Fig. 7: Influence of the buried depth at the end of the transition slab $e_{TS,extr}$ on the admissible imposed displacement $u_{imp,adm}$

imposed displacements $u_{imp,adm}$ are plotted as a function of the buried depth of the end of the transition slab $e_{TS,extr}$, defined in Eq. (5) and shown in *Fig. 3b*, for the highways planarity limit.

$$e_{TS,extr} = e_{TS,0} + \alpha_{TS} l_{TS} \quad (5)$$

The grey area in *Fig. 7* clearly shows the beneficial effect of increasing $e_{TS,extr}$ on the admissible imposed displacement $u_{imp,adm}$. This effect is particularly significant if $e_{TS,extr}$ is larger than 0,6 m. For bridges with integral or semi-integral abutments and imposed displacement u_{imp} larger than 43 mm, the geometry of the transition slab ($e_{TS,extr}$) can be adjusted according to *Fig. 7* to comply with the required planarity value of the road surface. This can be done by either increasing l_{TS} or α_{TS} (see Eq. (5)). Limits for α_{TS} are in the range of 5 to 20%. The lower value is required to have a favourable transition from the embankment to the bridge while the larger value is given to avoid a general sliding of the backfill over the transition slab that could lead to significant serviceability problems of the road surfacing.

The results of the parametric study of the embankment material are not given in this paper. This study has shown that the results caused by movements of the transition slab are quite insensitive to the range of gravel backfills typically used for embankments in Switzerland as for track ballast or blasted rock. More details about this study can be found in Ref. [11].

The results presented in *Fig. 7* should be considered with some caution. So far, neither laboratory test results nor *in situ* measurements are available in the literature (an experimental campaign is planned at the Ecole Polytechnique Fédérale de Lausanne/EPFL). Moreover, neither the probable increase of the slope variation χ due to the cyclic displacement of the transition slab due to daily temperature variations (*Fig. 1a*), nor the increase of χ due to the repeated passing of trucks over the local settlement has been considered. Although some uncertainty about the exact value of the admissible imposed displacement $u_{imp,adm}$ for a given buried depth of the end of the transition slab $e_{TS,extr}$ remain, the tendency of the beneficial effect of $e_{TS,extr}$ on $u_{imp,adm}$ is clear.

Cracking of the Road Pavement at the Connection Between the Transition Slab and the Bridge Deck

The imposed displacement u_{imp} of the transition slab can lead to cracking of the road pavement at the connection between the transition slab and the bridge deck,[8] as shown in *Figs. 2b and 8*. These cracks typically appear during the winter season when the road pavement is brittle as a result of low ambient temperatures. They also occur occasionally in standard abutments with joints. They are caused by the rotation of the transition slab around the connection due to the general settlement behind the abutment wall induced by u_{imp} and the consecutive flexural deflections of the transition slab due to the passing of trucks. A new connection detail, avoiding the occurrence of these cracks, is presented in the following text.

Improved Connection Detail Between the Transition Slab and the Abutment

Figure 9a shows the connection detail between abutments with joints and transition slabs according to the Graubünden/CH Department of Civil Engineering.[21] The transition slab rotates around the steel stud. This rotation gets propagated to the surface through the road pavement, which can lead to localisation of cracks. The advantage of this solution is that the water sealing layer is protected against tearing by the mass concrete that covers it. This detail is not suitable for integral or semi-integral bridges, because the stud connection is too weak to transfer the longitudinal forces due to u_{imp}.

Fig. 8: Cracking of the road pavement at the connection between the transition slab and the bridge deck, semi-integral bridge of 68 m length built in 1986 in Reichenau/CH[8]

Figure 9b shows the detail currently recommended in Switzerland for integral abutments.[5] The connection reinforcement can carry the internal forces due to u_{imp}. Moreover, it is favourable with respect to cracking because the centre of rotation of the transition slab is at the level of the connection reinforcement and consequently in the direct vicinity of the road surfacing. However, the construction of this detail is difficult because the connection reinforcement must be placed before the bituminous layer and the sliding bituminous layer. The new improved detail shown in *Fig. 9c* is a concrete hinge

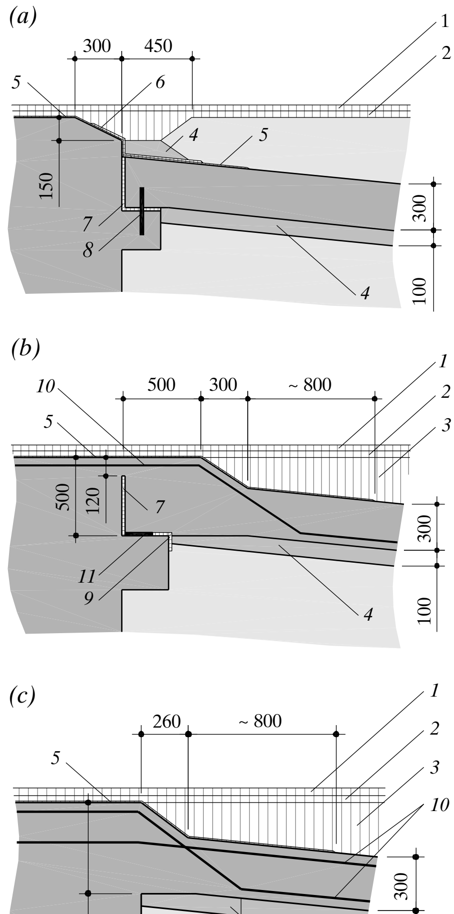

Fig. 9: Details of connection between abutment and transition slab, units in mm; (a) detail for standard abutment with joints according to the Graubünden/CH Department of Civil Engineering[21]; (b) detail for integral abutment according to Swiss recommendations[5]; (c) new improved detail for standard and integral abutments. 1: surface layer; 2: support layer; 3: additional support layer; 4: mass concrete; 5: first water sealing layer; 6: second water sealing layer; 7: bituminous layer; 8: steel stud; 9: synthetic foam; 10: connection reinforcement (other reinforcement not shown); 11: sliding bituminous layer

reinforced against shear failure by the diagonal connection reinforcement. The construction of this detail is easier than that shown in *Fig. 9b* and leads to a distribution of the rotation of the transition slab over the entire length of the concrete hinge l_{ch}. The resulting cracks have only small openings that cannot propagate to the road surfacing.

An experimental validation of this detail has been performed (see appendix of Ref. [11]). The experimental results have shown that the required rotation capacity of the concrete hinge is always reached if the connection has a reinforcement ratio ρ around 0,3 %. This relatively low reinforcement ratio also ensures a good distribution of the cracks.

Conclusions

Transition slabs are an important element for the long term performance and the serviceability behaviour of semi-integral and integral abutments. Imposed displacements from the bridge deck are transferred to the abutment and the transition slab, which leads to a strong soil–structure interaction.

The paper shows the effects of various geometric and material parameters on the planarity of the road pavement at the end of the transition slab. The beneficial effect of increasing the buried depth of the end of the transition slab $e_{TS,extr}$ is demonstrated.

A new improved connection detail between the abutment and the transition slab has been proposed. It allows avoiding cracks in the road pavement at this critical location. This detail can also be used in conventional bridges. The application of these considerations in the early design stages can ensure that integral or semi-integral abutments will be durable and perform satisfactorily at the serviceability limit state.

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