Steel–concrete hybrid bridge decks: rational design models for connection regions

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Steel–concrete hybrid bridge decks have been used in several bridges for their added benefit in constructive operations and cost. They usually adopt reinforced concrete side-spans and all-steel or steel–concrete composite girders as main spans, typically with monolithic connections between them. Design codes address separately the design of the steel and concrete regions of the connection. However, a rational design, with an efficient transfer of stresses between the steel and the concrete parts, should be based on a global understanding of the connection region structural behaviour. Therefore, a rational model for the design of the connection region is proposed, which is based on web shear buckling models developed over recent decades for steel panels and stress field models for structural concrete elements. A hybrid bridge deck connection region is presented as the case study. Several solutions for the connection are introduced and shear resistances are evaluated based on the proposed models.

Notation

A_w	cross-sectional area of the web
a	longitudinal spacing between transverse stiffeners
d	height of web panel
F_{bb}	horizontal tension field anchorage force
F_h	horizontal deviation force
f_y	yield strength
g	width of diagonal tension field
k_τ	plate buckling coefficient
l	corbel length
l_{bd}	design anchorage length
M_{Ed}	design bending moment
$M_{pl,N}$	reduced plastic resistance of flange
N_i	normal force of flange at section i
N_s	end stiffener axial force
p	distributed force
s_c	anchorage length of tension field along compression flange
s_s	anchorage length of tension field along transverse end stiffener
s_t	anchorage length of tension field along tension flange
t	thickness of the stiffener
t_w	thickness of web
V_{cr}	pre-buckling component of shear strength of web panel
V_{pl}	web plastic shear resistance
V_{pos-cr}	post-buckling component of shear strength of web panel
V_R	web shear buckling resistance
θ	slope of the panel diagonal
σ_{bb}	strength of tension field of web panel
τ_{bb}	shear stress

τ_{cr}	elastic critical stress for shear buckling of web panel
ϕ	inclination of tension field

1. Introduction

In recent years hybrid bridges combining either steel or composite steel–concrete decks with those of prestressed concrete, have caught the attention of engineers owing to their clear advantages in constructive operations, structural behaviour and cost.

The basic concept of a hybrid structure is based on the rational use of the structural materials, and, in bridge design, a possible arrangement of these is the use of prestressed concrete decks in short side spans and steel or composite in long main spans. Prestressed concrete is usually cheaper and heavier than all-steel or composite solutions, which makes it particularly adaptable for the shorter side spans. However, all-steel or composite decks are much lighter, and easier to assemble and erect, when compared to prestressed concrete; therefore, they are more appropriate for long main spans.

Using this concept, hybrid steel–concrete decks have been used in very-long-span, cable-stayed and suspended bridges, such as the Normandy Bridge, the Stonecutters Bridge and, recently, the Third Bosphorus Bridge. Moreover, hybrid solutions are also proposed in short- and medium-span girder decks, with functional and constructive benefits.

Recent design codes (i.e. Aashto (Aashto, 2012); ACI 318 (ACI, 2008) and EN 1993 (CEN, 2005) address the specific design models to be adopted in steel and concrete regions and do so adequately but independently – steel and concrete are

treated separately. However, an important issue to be considered when designing hybrid structures is the study of steel-concrete connections in an integrated form. This is an essential step in understanding the structural behaviour of the connection and ensuring its rational design, to allow the efficient transfer of stresses between the steel and the concrete parts of the deck.

2. Connection in steel-concrete hybrid decks

The connection of steel-concrete hybrid decks can be designed to transmit only shear forces or also to allow bending moment to transfer. These monolithic connections can be executed by means of: (a) steel plates and shear connectors; (b) prestressed bars or cables. However, a combination of these two is often proposed.

In the first type, steel connectors allow the transfer of stresses, as is the case for the hybrid decks of the A3 highway overpasses in Portugal (Figure 1). Stresses are balanced by steel plates and shear connectors which are both welded along the internal sides of the webs and flanges of the steel box-girder. When used in girder bridges, these connections are generally placed at 0.15 to 0.25 of the length of the main span, where bending moments due to dead loads are low. This solution increases safety, by minimising on-site construction operations for the main span erection, over the carriageway of an operational highway. Although the material costs are greater, the hybrid deck, combining side spans in prestressed concrete with a main span in steel-concrete composite solution, often proves to be the best cost-effective option.

The second type of connection adopted in hybrid steel-concrete decks makes use of prestressing bars through the connection region. Placed at the top and bottom areas of the cross-section, these bars very effectively counteract the tension stresses due to live load bending moments. Often, the prestressing tendons of the concrete section are also extended to the

beginning of the steel section, where they are anchored, contributing to the efficiency of the connection. The widening of the PI275 deck in Portugal used this type of connection (Figure 2(a)). Rehabilitation and widening of concrete decks frequently makes use of hybrid structures. In fact, when using a new all-steel or steel-concrete composite deck, differential concrete time-dependent effects between the new and the existing structure can be minimised.

A combination of steel plates and connectors with post-tensioned bars and cables can be found in the Nagisa Bridge (Japan), among many others. This bridge was also designed with a hybrid structural system, using simultaneously the suspension and cable-stayed systems, adopting a 110 m long main span.

The deck connection of the Nagisa Bridge (Figure 2(b)) uses a precast steel box filled with self-compacting reinforced concrete. The transmission of bending and shear forces is ensured by headed stud connectors placed at the horizontal and vertical plates of the composite member, as well as by the compressive force induced by the longitudinal prestressing bars.

3. Modelling the connection region

The study of the connection region of a hybrid deck similar to that of Figure 2(a) is carried out as follows: (a) assessment of the shear strength of the steel-concrete composite cross-section at the connection region, and (b) evaluation of the strength of the concrete supporting dapped beam, by means of strut-and-tie models (STMs). Hence, a set of design models for the composite and reinforced concrete regions is considered.

3.1 Structural model for the shear strength of the end panel

The first issue with respect to the steel-concrete composite cross-section concerns the evaluation of the shear strength of the end panel. The particular geometry of the composite deck,

Figure 1. Connection used in the hybrid decks of the A3 highway overpasses, in Portugal (adapted from Cabral *et al.* (2011)) (units: m)

Figure 2. Connection used in the PI275 hybrid deck: (a) in Portugal (Rito and Loureiro, 2006) and (b) in the Nagisa Bridge (adapted from Sato *et al.* (2003)) (units: m)

at the connection interface, justifies a detailed analysis of this region.

Shear buckling of steel plate girders is not a recent issue. The shear behaviour of plate girders has been extensively studied in the past 50 years. Extensive laboratory work, analytical methods and numerical analyses have been used to further understand the behaviour of plate girders and have resulted in new design methods, confirmed by numerous research works, which were recently summarised by Hasan *et al.* (2015).

Despite the extensive experimental and analytical investigations of steel plate girders, only a limited number of studies on composite plate girders appear to have been carried out so far. Plate girders connected to reinforced concrete slab through shear studs welded on to the girder flange behave differently compared to the steel girder acting alone. In these girders, tension field action in the web is changed significantly in view of the composite action between the plate girder and the concrete slab. At the ultimate load level, the distance between the plastic hinges in the flange connected to the concrete slab is larger than that in the other flange. The behaviour, from elastic to ultimate state condition, is different and ultimate shear resistance of the composite girder becomes significantly larger than the corresponding value for a non-composite girder. The results available to date are not sufficient to understand completely the elastic and inelastic behaviour of such girders. Yet, studies by Allison *et al.* (1982), Porter and Cherif (1987), Narayanan *et al.* (1989), Roberts and Al-Amery (1991), Shanmugam and Baskar (2003), Darehshouri *et al.* (2011) and Yatim *et al.* (2015) include experiments on small-

and medium-scale composite plate girders and some simplified analytical methods to predict the shear resistance of composite plate girders.

As the web steel panels are typically very slender, with a height-to-thickness ratio of 150 or even more, the post-buckling shear strength should be considered. The tension field method (TFM) is adopted to assess the shear strength of the webs, because it can be adapted to the composite deck shear behaviour described above. This method, based on the Cardiff-Prague model, was included in the ENV 1993-1-1 European standard of 1992 (CEN, 1992), and should only be used in steel girders with transverse stiffeners spaced longitudinally by a distance a , within the range $d < a < 3d$, where d is the height of the web panel (Figure 3(a)).

It is well known that once the web plate, with thickness t_w , reaches the elastic shear buckling load τ_{cr} , it loses the capacity to equilibrate additional compressions. At that moment, a new force transmission mechanism is developed, allowing the equilibrium of additional shear stress through an inclined tensile membrane that is self-anchored on the top and bottom flanges of the girder and also on the transverse stiffeners along the web (Figure 3(b)). An analogy can be made between the post-buckling mechanism developed and the load transmission behaviour of a Pratt truss, matching the post-buckling strength available in the web plate with the resistance of the truss diagonal bar. The failure load is obtained by the superposition of the membrane stress effects in the pre-buckling phase with the diagonal tension field of width g and inclination ϕ , from the post-buckling phase.

Figure 3. Loading stages up to failure according to the TFM: (a) pre-buckling model; (b) post-buckling model; (c) failure mechanism

The total resistance to shear V_R of high, slender plates is, therefore, obtained by the sum of the elastic buckling contribution, V_{cr} , with the post-buckling strength, V_{pos-cr}

$$1. \quad V_R = V_{cr} + V_{pos-cr} = d t_w \tau_{cr} + 0.9 g t_w \sigma_{bb} \sin \phi$$

where σ_{bb} is the strength of the tension field, given by the von Mises–Hencky criterion, as follows

$$2. \quad \sigma_{bb} = \sqrt{f_y^2 - 3\tau_{cr}^2 + (1.5\tau_{cr} \sin 2\phi)^2} - 1.5\tau_{cr} \sin 2\phi$$

The maximum shear capacity is attained when the diagonal tension stress in the web equals the steel yielding stress f_y , and a set of four plastic hinges are formed on the flanges (Figures 3(c) and 4(a)). The value of g is given by

$$3. \quad g = d \cos \phi - (a - s_c - s_t) \sin \phi$$

and depends on the anchorage lengths of the tension field along the upper and lower flanges, s_c and s_t . Using the principle of virtual work, applying a unit load for example on the upper flange of the beam (Figure 4(b)) gives

$$4. \quad \sigma_{bb} t_w s_c \sin^2 \phi \frac{s_c}{2} = M_{pl,N_1} + M_{pl,N_2} \rightarrow s_c = \frac{2}{\sin \phi} \sqrt{\frac{M_{pl,N_1} + M_{pl,N_2}}{2\sigma_{bb} t_w}} \leq a$$

where M_{pl,N_1} and M_{pl,N_2} are the plastic moments of the upper flange where hinges 1 and 2 of Figure 3(c) are formed (regarding the lower flange, s_t is evaluated in the same form).

The presence of the axial force N_i at each section, due to the global bending moment, reduces these local plastic moments which, for a flange with width b and thickness t_f , are given by

$$5. \quad M_{pl,N_i} = 0.25 b t_f^2 f_y [1 - N_i / (b t_f f_y)]^2$$

In accordance with Equation 4 the global bending moment decreases the anchorage lengths of the tension field on the flanges and, as a result of Equations 1 and 3, the width and the post-buckling strength of the webs are also reduced. Also τ_{cr} is affected by normal stresses installed in the web due to the global bending moments. For these cases τ_{cr} can be assessed numerically taking into account shear and normal stresses simultaneously in the web panel.

Equations 1–4 depend on the inclination of the tension field, ϕ , that should be in the range of $\theta/2 \leq \phi \leq \theta$, where θ is the slope of the panel diagonal (Figure 3(c)) given by $\arctan(d/a)$. An iterative process, rapidly convergent, gives the maximum value of the shear post-buckling strength V_{pos-cr} . However, adopting $\phi = 2\theta/3$ it is possible to expect an accuracy that is satisfactory.

For the particular case of an end panel of a simply supported beam, a suitable end post should be designed avoiding a premature plastic mechanism, with the formation of plastic hinges at the vertical end post (Figure 3(c)), created by the effect of the tension field anchorage force, F_{bb} , and given by

$$6. \quad F_{bb} = t_w s_s \sigma_{bb} \cos^2 \phi$$

Figure 4. (a) Flange plastic resistance at the end support to anchor the tensioned diagonal of the post-buckling model (from Lee and Yoo (1999)) and (b) failure mode of the flange

where $s_s = d - (a - s_t) \tan \phi$. Once again, resorting to the principle of virtual work applied now to the vertical end post of the beam, this mechanism is prevented if

$$7. \quad M_{pl,2,min} + M_{pl,3} > F_{bb} s_s / 2$$

In any case, V_{pos-cr} , F_{bb} or N_s should be used to evaluate $M_{pl,3}$ and $M_{pl,2,min}$ by Equation 4, at hinge 2 of Figure 3(c), which can occur either at the flange or at the top section of the end post.

As shown above, the TFM clearly identifies the contribution of each part of the steel structure to the shear strength of the web. Moreover, the TFM treats consistently web end panels

with rigid and non-rigid end posts, which is advantageous when compared with the rotate stress field method, used in the current version of the Eurocode 3, Parts 1–5 (CEN, 2006) on plated structural elements. Finally, it can be customised to the case of composite girders, including the contribution of the slab to the shear resistance of the webs in a comprehensible form.

3.2 Structural concrete design model

Strut-and-tie models have been used to design special regions like corbels, deep beams or dapped-end beams of structural concrete members, and accurately cover the design of B and D regions of structural concrete members (Muttoni *et al.*, 1998). Fundamental work was also done by Lourenço and Almeida, (2013), emphasising stress field models (SFMs). More recently, the adaptive stress field approach (fib Model Code (fib, 2013)) extended the application of stress-field-based models for the non-linear analysis of structural concrete discontinuity regions, allowing a consistent study of service behaviour, ductility and, more generally, model assessment topics.

For all these D regions, STM/SFM models are very practical design tools, and are already included in the more recent technical and normative documents (ACI SP-273 (ACI, 2010); EN 1992-1-1 (CEN, 2004); fib Model Code 2010 (fib, 2013)).

In fact, the STM provides a simple, yet reliable tool to accurately evaluate structural concrete members. It revives the understanding of the flow of forces within the members, and it provides an unprecedented awareness of structural behaviour. Some of the recognised advantages of this method are (Reineck *et al.*, 2011) listed below.

- It gives a clear view of the load path throughout any region, providing an insight into the behaviour of the structure under the applied loads, prior to any detailed analysis.
- It complies with the lower bound of the theory of plasticity; this is particularly useful for design purposes, as the ultimate load obtained is a lower bound of the actual failure load.
- It provides improved consistency in design concepts by allowing the treatment of discontinuity regions (D regions) and adjacent B regions with comparable accuracy.
- It accounts for the behaviour of concrete after cracking.
- It emphasises the essential role of detailing in the design process, clearly defining the necessary amount and correct distribution of reinforcement, as well as the geometry of nodes and the anchorage lengths for the local design of the support widths.

These advantages make the use of STMs of particular interest when studying the concrete discontinuity region of hybrid steel–concrete decks.

Figure 5. Layout of the post-buckling tension field and compliance with the geometry of the connection region

Figure 6. Geometry of the case study simple supported connection (units: mm)

3.3 Geometry of the support region

The function of the steel corbel, supported by the concrete dapped beam end, is exclusively to transmit the vertical reaction of the steel girder. Therefore, there is every advantage in adopting a reduced span l for the corbel, provided that it allows safe assembly and support of the deck at the

construction phase and effective shear transmission during service conditions.

The geometry of the interface is also an important aspect for the analysis, and has an essential role to ensure that the full post-buckling strength is achieved (Figure 5). In fact, the

Figure 7. Design model 0 for assessment of the elastic buckling strength of an end web panel (units: mm)

post-buckling width of the tension field model has to comply with the connection geometry. This is an additional reason for the use of the Cardiff–Prague TFM, as it proposes a clear and simple path of stresses throughout the support, with the possibility of adjusting the inclination ϕ , and thus the width g . Likewise, if the end post does not satisfy the resistance criterion in Equation 7, an increased value of ϕ may be adopted such that the anchorage length s_s is reduced sufficiently for the criterion to be satisfied. The reduced shear buckling is then determined for the values (ϕ' , g') that comply with the geometry and end post resistance requirements.

When designing the concrete dapped supporting beams, a reduced l/h_1 ratio will result in a more direct load transmission to the concrete. The length l of the corbel is also important to prevent buckling of the small end web panel when a double end stiffener is used. But, in that case, the distance between the two stiffeners should guarantee the required plastic resistance of the vertical end post.

4. Case study assessment

The case study corresponds to a typical hybrid deck with a prestressed concrete girder section at the side spans and a composite steel–concrete box-girder at the main span. The overall deck dimensions, type of connection, anchorage bars and steel plate geometry follow that of an already existing bridge (Rito, 2010).

Therefore, the connection studied adopts a similar geometry (Figure 6) and explores the combination of steel plates grade S355 J2 and a reinforced concrete slab using C35/45 and reinforcement B500B. First, four design models are proposed

for the simply supported connection type and second, a monolithic connection is analysed. The thickness of all transverse stiffeners is defined for each model, so that they do not decrease the shear strength of the steel end panel.

4.1 Design model 0 – elastic buckling resistance

Model 0 assumes that the maximum shear resistance is achieved when the plate reaches its elastic critical stress for shear buckling, τ_{cr} . At this point the principal stresses occur at 45° to the horizontal and take the value of the pure shear stress installed (Figure 3(a)).

Considering a vertical web panel with $a = 3000$ mm, $d = 1440$ mm and thickness, $t_w = 10$ mm, simply supported at the four edges, and disregarding the post-buckling strength, $k_\tau = 6.26$, $\tau_{cr} = 56.8$ MPa, the maximum shear resistance of the web is $V_{R,0} = V_{cr} = 818$ kN (Figure 7). Taking into consideration the existence of an indirect support, a lower end stiffener plate 10 mm thick is adopted, for supporting the web panel and to suspend the end shear force to the concrete corbel.

4.2 Design model 1 – single stiff end stiffener

It is clear that a stiff transverse end stiffener is useful, making it possible to anchor the diagonal tension field developed after the web buckling occurs. The post-buckling strength induced by this behaviour is added to the elastic web strength, evaluated in Section 4.1.

Model 1A makes use of a single stiff end stiffener 400 mm wide (Figure 8(a)), and disregards the presence of the concrete slab in the analysis of shear resistance, which is usually done in the design of plate girders. The global failure mode is

Figure 8. (a) Design model 1A for post-buckling strength assessment of a steel girder with a single end stiffener and (b) global and local failure modes studied (units: mm)

attained with the yielding of the upper and lower flanges, with effective cross-sections 400×35 mm² and 300×25 mm² (Figure 8(b)), with the diagonal tension field geometry in accordance with Figure 8(a). The correspondent shear resistance of the web is $V_{R,1A} = 818 + 611 = 1429$ kN, which corresponds to an increase of 75% with respect to its elastic buckling resistance.

An early failure of the end post, with the formation of three plastic hinges, should be prevented. Using the plastic model from Figure 8(b), and locating the span hinge to maximise the virtual work produced by the horizontal load p , it is concluded that a vertical end stiffener with $t_s \geq 65$ mm should be adopted.

Model 1B also adopts a single stiff end stiffener (Figure 9(a)) and considers the contribution of the steel-concrete composite cross-section formed by the concrete slab and the top flange (Figure 9(b)) for the evaluation of post-buckling shear strength. In fact, laboratory tests (Allison *et al.*, 1982; Gomes, 1999; Lebet and Lääne, 2005) have already attested to the significant increase of post-buckling shear strength of simply supported steel-concrete composite girders.

As before, the global failure mode is achieved with yielding of both flanges, but the presence of the concrete slab greatly increases the plastic bending resistance of the upper flange. By equilibrium, the internal force distributions used to define the reduced plastic bending moments of the flange are shown

Figure 9. (a) Design model 1B for post-buckling strength assessment of a composite girder with a single end stiffener; (b) adopting the contribution of the reinforced concrete slab; (c) with the internal forces due to the diagonal tension field (units: mm)

in Figure 9(c). With the diagonal tension field geometry given in Figure 9(a), the shear resistance of the web is $V_{R,1B} = 818 + 778 = 1596$ kN (an increase of 95% with respect to its elastic buckling resistance).

It should be noticed that the contribution of the slab considerably increases the width g of the post-buckling stress diagonal. If a high bending resistance of the composite flange–slab section is attained, g would be equal to the length a of the panel, but the girder vertical deflection would be very high (Esteves, 2015). Moreover, to ensure this composite behaviour between the flange and the concrete slab, an adequate detailing

of the connectors should be provided to guarantee the composite action up to its point of failure.

4.3 Design model 2 – double end stiffeners

Model 2 adopts double end stiffeners formed by two transverse plates with a width of 400 mm, and benefits from the existence of the slab. The global failure mode is attained with the yielding of the upper and lower flanges, as shown in Figure 10(a), together with the formation of a plastic hinge at the top of the end post. With the diagonal tension field geometry given in Figure 10(a), the shear resistance of the web is

Figure 10. (a) Design model 2 for post-buckling strength assessment of a composite girder with a double end stiffener; (b) local failure mode check (units: mm)

$V_{R,2} = 818 + 894 = 1712$ kN (an increase of 110% with respect to its elastic buckling resistance).

The solution with two end stiffeners enhanced the capacity to prevent the early failure of the end post, by using an upper H stiffener with flanges $t_s = 13$ mm and a lower T stiffener (formed by the end plate and the effective part of the web with 300 mm). As shown in Figure 10(b), the local mechanism involves a top hinge at the end post and a new hinge at the bottom stiffener. By equilibrium, a deviation horizontal force F_h arises. Applying the principle of virtual work to the vertical beam, clamped at the top flange and simply supported at the bottom flange (Figure 10(b)), the minimum necessary thickness is 40 mm for the lower end stiffener.

4.4 Design model 3 – monolithic connection

As in the proposed models for vertical shear transmission, model 3 also considers the existence of a bending moment in the connection and the transverse stiffeners are placed according to model 2. The bending transmission to the concrete section is achieved primarily through the two elements of Figure 11: (a) two layers of six prestressing bars and (b) the upper and bottom front plates. The front plates, positioned at both ends of the composite box-girder, allow the compression stresses to be transmitted from the composite deck to the reinforced concrete region. A 1.0 m long reinforced concrete block is cast inside the composite cross-section, improving the rigidity and efficiency of the monolithic connection and providing a simple mode of anchoring of the prestressing bars.

Figure 11. Geometry of the case study monolithic connection (units: mm)

From the structural point of view, the existence of a concrete block compressed against the front plates and transverse end stiffeners prevents the latter from bending due to the diagonal tension field in the web, rendering it unnecessary to verify the end transverse stiffeners. Thus, the thickness of the end stiffeners can be significantly reduced, because their main function is now to support the end web panel and to transmit the vertical shear force to the corbel. Model 3 also considers that the upper plastic hinges have to be formed at the composite steel–concrete flange–slab section. Therefore, for the concrete slab, the bending resistance is evaluated taking into account: (a) the axial forces transmitted by the tensioned diagonal; and (b) the compression/tension due to the global bending moment acting in the connection cross-section.

Considering the design bending moment $+7250 \text{ kNm}$ (such that a stress of $0.4f_y$ at the bottom flange is considered) equilibrated by the composite section, the elastic buckling strength of the web panel defined in Section 4.1 is numerically assessed, taking into consideration the shear and normal stresses due to the sagging bending moment: $\tau_{cr} = 72.92 \text{ MPa}$, $V_{cr} = 1050 \text{ kN}$. The post-buckling shear failure mode is attained with the yielding of the upper and lower flanges. With the diagonal tension field geometry given in Figure 12(a), the shear resistance of the web is finally given by: $V_{R,3+} = 1050 + 799 = 1849 \text{ kN}$.

Assuming the design bending moment is -6700 kNm (such that a stress of $-0.4f_y$ at the bottom flange is considered) applied at the steel section and reinforcement of the slab (Figure 9(b)), the elastic buckling strength of the web panel is again assessed numerically, taking into consideration the normal stresses due to the hogging bending moment: $\tau_{cr} = 54.02 \text{ MPa}$, $V_{cr} = 778 \text{ kN}$. The shear failure mode is still attained with the yielding of the upper and lower flanges, and the diagonal tension field geometry given in Figure 12(b). The corresponding shear resistance of the web is finally given by: $V_{R,3-} = 778 + 867 = 1645 \text{ kN}$.

These results confirm that the shear resistance of the webs is not much affected by the existence of moderate bending moments, which is usually the case for the connection regions located frequently at 0.15 to 0.25 of the main span length.

4.5 Design of the structural concrete connection region

The design of the structural concrete region for the most typical model 3 monolithic connection requires the assessment of the $V_{R,3-}$ – vertical force distribution along the cross-section of the concrete dapped. Using elastic finite-element analysis, the end of the composite region near the connection interface is modelled (Figure 13). It is found that the transverse distribution of the vertical reaction on the dapped beam is

Figure 14(a) displays the STM proposed for the connection to the concrete deck, assuming the vertical reactions occur mainly along the web supports (400 mm of width) of the steel–concrete composite box-girder. The addition of the prestressing bars to execute the monolithic connection corresponds to the superposition of a second model, as presented in Figure 14(b).

A consistent detailing of the steel reinforcement is proposed in Figures 15(a)–15(c), as well as the definition of the node geometry for the local check of the web support (Figure 15(d)), with the anchorage length and the concrete strut compression.

An approach entirely based on stress field models, using the TFM and STMs, respectively, for steel and concrete regions of

the connection, provides a simpler, yet reliable framework to safely assess the connection region of a hybrid steel–concrete bridge deck. The proposed models reproduce the compression and tension fields due to the load path deviation within the connection region, both for the steel and concrete parts, wherein every element has a clear physical meaning. The main conclusions of this case study are as follows.

- Rational design models should be used for the design of the connection region of a hybrid deck; when a simply supported connection is adopted the most structurally efficient design is achieved with a rigid double end post; for the case study presented, the shear ultimate resistance

Figure 13. Finite-element shell model for the study of the concrete force distribution

- can be increased up to 110% of the elastic shear buckling strength, with the contribution of the steel webs' post-buckling behaviour and of the slab bending resistance, connected to the upper flange of the girder;
- In fact, the assessment of the shear web's resistance, taking into consideration the deck flange–slab, steel–concrete composite behaviour, allows a greater anchorage length of the diagonal tension field along the upper steel flange, and thus the ultimate shear resistance of the steel component of the structure is increased by about 20% for the case study presented;
- Special attention must be given to ensure an efficient design of the end post, with sufficient rigidity and strength, for the full development of the tension field action in the end panel.
- Alternatively, a monolithic connection can be designed and evaluated using model 3; in such a case, prestress bars anchoring should be performed in a concrete block cast inside the composite box-girder end, increasing the rigidity of the connection and preventing local bending of the front plates.
- The shear resistance of the webs is not much affected by the existence of moderate bending moments at the monolithic connection, usually located at 0.15 to 0.25 of the main span length; thus, this connection seems to be the best solution, also considering the overall behaviour of the structure.
- A three-dimensional (3D) STM is the best model for the design of the concrete part of the connection, proving to be essential for the reinforcement detailing of the concrete discontinuity region; it clearly defines the necessary amount and correct distribution of reinforcement, as well as the geometry of nodes and the anchorage lengths for the local design of the support widths.

Figure 14. Strut-and-tie model for the concrete region of the connection: (a) three-dimensional (3D) view of the simply supported model; (b) 3D view of monolithic model; and results of the model – (c) longitudinal section; (d) front view; (e) plan view

Figure 15. Reinforcement detailing of the concrete connection region: (a) longitudinal section 1–1; (b) cross-section 2–2; (c) plan view 3–3; and (d) local node A–A of the support web reaction region (units: mm)

- Ultimately, the proposed methodology and models allow a clear understanding of the flow of forces within the connection region of hybrid structures, providing an excellent awareness of structural behaviour and high-quality design and detailing.

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