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Shear Connection for Composite Bridges, and Eurocode 4: Part 2

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Summary

The Eurocode for composite bridges will be completed to ENV stage during 1997. Although it is required by the rules of CEN to be supplementary to Eurocode 4:Part 1.1:1994, its provisions for design of shear connection are different in many respects. They include verifications for serviceability and fatigue, one of which will often govern the spacing of connectors. These provisions are summarised, with particular reference to the effects of cracking of concrete, local concentrations of longitudinal shear, inelastic bending of beams, and relevant research.

1. Introduction

The Third Draft of the Eurocode for composite bridges [1] was circulated for comment in January 1997, after four years of work. It is expected that the Final Draft will be approved during 1997 for publication by national standards bodies as ENV 1994-2. In accordance with the rule of CEN that prohibits duplication of material, it is presented as a supplementary document to ENV 1994-1-1:1992 [2], here denoted 'Part 1.1', and is referred to here as 'Part 2'.

Part 2 has the same numbering system for Sections and clauses as Part 1.1. Each clause of Part 1.1 applies unless stated otherwise, or unless a corresponding clause labelled 'modified' or 'replaced' appears in Part 2, which also has 'additional' clauses. The drafting of Part 2 is thus constrained by the clause structure and drafting of Part 1.1 (in 1983-4). It should be possible to improve the relationship between Part 1.1 and Part 2 when their EN versions are drafted.

A brief account is given here of the differences, especially in the treatments of shear connection, between Part 1.1 and Part 2. They arise from the differences between buildings and bridges, and their loadings and exposure. The main provisions of Part 1.1 for shear connection are given next, with comments; and in outline only, because they are widely known [3,4].

1.1. Comparison of EC4:Part 1.1 with draft Part 2

Most beams in buildings are designed for uniformly-distributed loading and ultimate limit states. Except where there are cross-sections in Class 3 or 4, the shear connectors are uniformly spaced between cross-sections of maximum sagging and hogging bending moment, and their number is calculated from the difference between the longitudinal forces in the concrete slab at those sections. The rules for partial shear connection enable designers to use a connector spacing that is compatible with the profiled steel sheeting used for the floor slab. Serviceability limit states have little influence on design, except where the degree of shear connection is so low that deflection governs; and fatigue is not within the scope of Part 1.1.

Nearly all continuous bridge beams are in Class 3 or 4. Both global analysis and the calculations of longitudinal shear per unit length use elastic theory, and design is based more on influence lines and envelopes of moment and shear, than on the simpler effects of uniformly-distributed loading. It may not be obvious which of three limit states - excessive slip in service, fatigue failure, and static failure - govern the spacing of shear connectors, so methods of verification are given in Part 2 for all three. Thin profiled steel sheeting is unlikely to be used in permanent bridges, because of the risk of corrosion. The use of partial shear connection increases slip, which in a long span may exceed the slip capacity of the connectors, so it is not within the scope of Part 2.

Beams that are of non-uniform section, or curved in plan, and sets of closely-spaced beams, perhaps on skewed supports, are common in bridges, but not in buildings. Provision has also to be made in Part 2 for structural systems such as tied arches, half-through bridges, and trusses, and for the use of prestress and double composite action.

Part 2 has to be consistent with Eurocode 2:Part 2 for concrete bridges [5], in which the clauses on fatigue and on creep, shrinkage, and cracking of concrete are more complex than in EC2:Part 1.1. The relevant provisions of EC4:Part 2 are simpler than in earlier drafts, especially in respect of tension stiffening in cracked concrete. Experience of its use in trial designs should lead to further simplifications, such as the definition of more situations where specific checks (e.g., on fatigue in reinforcement) are not required.

2. Properties of shear connectors

The types of shear connector covered in Part 2 are as in Part 1.1, except that welded reinforcing-bar anchors are excluded. The resistances to static load are unchanged, but for studs are limited to a maximum diameter of 22 mm. (More data are needed on properties of 25-mm studs). Where failure of a connector clearly occurs in a single material (e.g., steel, concrete, or a weld) the usual safety factor γ_M for that material is used. The failure of a stud connector arises from interaction between steel and concrete, so a single factor γ_M (= 1.25) is used, based on calibration. Thus, the ratio of design resistance at ultimate limit states to characteristic resistance P_{Rk} depends on the type of connector used.

For the serviceability limit state of maximum stress, the force per connector is limited to $0.6\ P_{Rk}$. The intention is to avoid inelastic behaviour and 'excessive' slip, for which there is no simple definition. The ratio 0.6 is based mainly on existing practice.

For all types of connector except studs, the resistance to fatigue is governed by welds that can be verified using Eurocode 3 for steel structures. There is no consensus in the literature on the fatigue resistance of welded studs. A recent study [6] of reports on 211 push tests found recommended values of the exponent m ranging from 5 to 11.5. At least seven different types of test had been used, and the results came from at least four different statistical populations.

It became clear that in design to Part 2, fatigue loading would not cause loads per connector exceeding $0.6\ P_{Rk}$, and that the influence of strength of concrete on fatigue failure could then be neglected. (This is not done in the UK Bridge Code, BS 5400, where fatigue resistance is a function of static resistance, which for many connectors is influenced by the strength of the concrete). This assumption enabled the fatigue resistance of studs to be defined by an S-N line, as for other welded details. Based partly on [6], the fatigue resistance of a stud of diameter d has been defined, in N, mm units, by

$$N\,[\,\Delta P_R\,/\,(\,\pi d^2/\,4)\,]^8 = 10^{22.123}, \qquad (1)$$

where N is the number of cycles to failure, for shear force range ΔP_R per stud. This is a characteristic value, for use with $\gamma_M = 1.0$. It gives the fatigue strength in shear as $\Delta\tau_c = 95$ N/mm^2, for the reference value $N = 2 \times 10^6$. For lightweight concrete, this value is scaled down in proportion to the density. A tri-linear interaction expression is given for fatigue verification where a shear connector is welded to a steel flange in tension; and in this situation, the diameter of a stud connector is limited to 1.5 times the thickness of the flange. It has not been possible to implement recent research on the influence of cumulative fatigue damage on static resistance [7].

During construction, shear connectors may be subjected to shear force before the surrounding concrete has reached its design strength. It is recommended in Part 2 that the placing of fresh concrete should be so planned that this does not occur where the cylinder strength of existing concrete is less than 20 N/mm^2, at which strength connectors should be asssumed to become effective.

3. Determination of longitudinal shear

In principle, shear connection is designed for a shear flow, v, that is the rate of change of the longitudinal force in either the steel or the concrete element of a composite member, using the maximum value at each cross-section, and considering all design combinations and arrangements of actions. The usual basis is the design envelope of transverse shear (i.e., vertical shear V in a horizontal member) and use of the well-known elastic theory that gives $v = VA\bar{y}/I$. To apply this principle, approximations are necessary. Some of these are now given.

Where account is taken in global analysis both of the effects of cracking of concrete and of tension stiffening, the second moment of area I and the associated section properties A and $\bar{y}$ in the equation above may, in cracked regions, be calculated for the cracked concrete section, including the effects of tension stiffening. In all other situations, the 'uncracked' properties should be used. The 'cracked' properties neglecting tension stiffening may not be used, because this underestimates the forces on connectors, especially in structures such as tied arches where the deck is not prestressed. (This option of using the cracked section, which reduces the number of connectors required, is not given in the UK Bridge Code).

Generally, longitudinal slip is ignored, but there are many situations where it is necessary to take advantage of the effects of slip on local concentrations of longitudinal shear. Design rules, mostly based on finite-element analyses, are given in Part 2 for the following situations:

- sudden change of cross-section of a composite member (empirical),
- isostatic effects of temperature and shrinkage near an end of a composite member (based on the UK Bridge Code),
- application of a concentrated longitudinal force; e.g., by a prestressing tendon or a web member in a truss or frame (based on reference [8]).

This last rule is illustrated in Figure 1(a), which shows a prestressing force with design value F_d applied to the slab at lateral distance e_d from the shear connection. The proportion of F_d that is transferred to the steel section, V_ℓ, is determined by conventional elastic analysis of the composite section. The distribution of design longitudinal shear v_d may be assumed to be trapezoidal, as shown, with a maximum value

$$v_{d,max} = V_\ell / (e_d + b_{eff}/ 2)$$

where b_{eff} is the effective width of the concrete flange, as used in the global analysis. For stud connectors, this distribution may be replaced by a rectangular one over the length L_v.

For the trapezoidal distribution, the compressive force in the slab caused by the prestress increases from zero to $F_d - V_\ell$ over a length L_v $(= e_d + b_{eff})$ as shown in Figure 1(b). The maximum tensile force in the slab is given by

$$\Delta N = v_{d,max}\, b_{eff} / 4.$$

The distribution of longitudinal shear caused by several forces F_d is obtained by summation. Similar rules are given for the longitudinal force applied to the concrete element of a flange by a web member in a truss or frame.

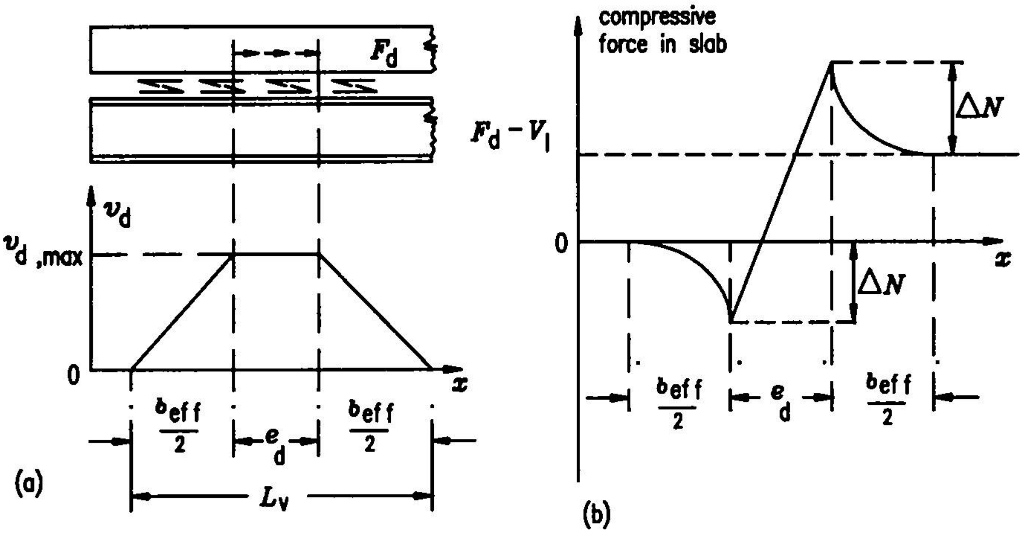

Fig. 1 Distribution of longitudinal shear force along the interface

The preceding elasticity-based methods are applicable for all limit states, but are unconservative for longitudinal shear near a region of a member in slenderness Class 1 or 2, where resistance to bending is based on plastic section analysis, and can exceed the elastic resistance by 30% or more. After many trials, a design rule for this situation was found, based on the treatment of partial shear connection in Part 1.1. In a midspan region, its use may not lead to the provision of more connectors, because each check is for a specific bending-moment distribution, rather than for an envelope, and also because the separate check for fatigue may govern.

4. Fatigue of shear connectors

Application rules are given in Part 2 only for fatigue design using nominal stress ranges based on fatigue load models (FLM) 3 for road bridges and 71 for railway bridges, as defined in Eurocode 1:Part 3 [9]. For road bridges, the FLM3 vehicle applies eight wheel loads of 60 kN within an area 8.8 m long and 2.4 m wide. For stud connectors, the range of shear stress per stud is determined for its passage across the bridge in the most critical lane, and multiplied by a damage equivalence factor $\lambda_{v1}\, \lambda_2\, \lambda_3\, \lambda_4$, using tabulated λ factors that are functions of the length of the influence area, the specified traffic and design life of the bridge, and the slope of the *S-N* line for fatigue strength. These factors are as given in Eurocode 3, for both highway and railway loading, except for λ_{v1}, which differs from λ_1 in Eurocode 3, and is given in Part 2. This is to allow for the exponent 8 in equation (1), which is higher than for the welded details covered by Eurocode 3.

The result is $\Delta\tau_E$, the damage-equivalent constant-amplitude shear stress range for 2×10^6 cycles, and the verification is

$$\gamma_{Ff}\, \Delta\tau_E \leq \Delta\tau_c \,/\, \gamma_{Mf},$$

where $\Delta\tau_c$ is given above, and γ_{Ff} and γ_{Mf} are the partial safety factors for fatigue.

5. Which limit state governs the spacing of shear connectors ?

For simplicity, the methods of analysis given in Part 2 are, wherever possible, the same for all limit states. Where alternatives are given, as for the treatment of cracking of concrete, the simpler one is usually the more conservative, and is likely to be used in the design of small and simple bridges.

This similarity of methods enables a rough comparison to be made, for highway bridges, between the numbers of stud connectors required for the serviceability and ultimate limit states, n_s and n_u, respectively. For most permanent and traffic actions, $\gamma_F = 1.35$, so the ratio of action effects (SLS/ULS) is $1/1.35 = 0.741$. For studs, the ratio of limiting force per connector is $0.6\, P_{Rk}$ to $P_{Rk} \,/\, 1.25$, which is 0.75. On this basis, $n_s \,/\, n_u = 0.741/0.75 = 0.99$.

It is not as simple as this, for many reasons, such as the differences between the load combinations for the two types of limit state; but the conclusion that ultimate and serviceability criteria give similar numbers of connectors is supported by the design exercises done so far. If fatigue governs, it is most likely to do so in a region near midspan.

5. Miscellaneous

Reference is made here to some aspects of Part 2 that do not belong under earlier headings.

There is an application rule that, adjacent to cross frames and vertical stiffeners, design should be such that no significant uplift forces are applied to the shear connection by rotation of the deck slab about a longitudinal axis. It has not been possible to give specific guidance on a problem that often leads to discussion between designers and checkers but, it is believed, has never been implicated in a failure.

For the design of transverse reinforcement near shear connectors, the only significant change from Part 1.1 is that the design shear strength of the concrete, τ_{Rd}, is taken as zero in regions where the calculation of longitudinal shear is based on a cracked cross-section (and so is much lower than if the uncracked section had been used).

The preceding rule is an example of many clauses where there is an implied assumption that composite action occurs in one flange only of the member. Where there are two composite flanges, the rule on τ_{Rd} is intended to apply only to the flange in tension, but that is not stated.

6. Closure

The new design methods summarised above refer to and are consistent with the ENV (preliminary) versions of Eurocode 1:Part 3, Eurocode 2:Part 2, and Eurocode 3:Part 2. All these codes need extensive trial use in practice, over the 3-year ENV period that should begin in 1998. This should enable improved EN versions to be drafted and, it is believed, some methods to be further simplified.

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