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# Limit States of Composite Bridges

## Etats-limites des ponts mixtes

## Grenzzustände von Verbundbrücken

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**SUMMARY**

Limit state design has been in use for composite bridges in the UK since 1982, and is being adopted in most European countries. Design is normally governed by the ultimate limit state. Evidence from research shows that real ultimate strengths usually lie well above the design values, partly because of the difficulty of drafting design codes that fully exploit existing knowledge. It is foreseen that this margin of resistance may increasingly be eroded, as time passes, by undetected deterioration, which may eventually become a greater threat to the safety of some bridges than any foreseeable degree of overloading. To improve existing codes, it is better to compare them with research data and experience, than with each other.

**RÉSUMÉ**

Le calcul aux états-limites est en usage depuis 1982 dans le Royaume-Uni et est en passe d'être adopté dans la plupart des pays européens. L'état limite ultime représente en général le critère déterminant pour le dimensionnement. La résistance ultime effective des structures est nettement supérieure à la résistance calculée, comme le prouve les résultats de recherche; ceci provient en partie de la difficulté à introduire dans les codes la totalité des connaissances existantes. Il est probable que cette marge soit de plus en plus érodée avec le temps à cause d'une détérioration non détectée de la structure. Ceci représente, à long terme, une menace pour la sécurité des ponts plus importante que tout degré de surcharge prévisible. Pour améliorer les codes existants, il est préférable de les comparer avec les résultats de recherches et avec l'expérience que de faire une comparaison entre codes.

**ZUSAMMENFASSUNG**

Die Bemessung mit Grenzzuständen wird in England für den Verbundbrückenbau seit 1982 angewendet und wird nun auch von den meisten anderen europäischen Ländern eingeführt. Die Bemessung wird normalerweise durch den Bruchzustand bestimmt. Versuchsresultate zeigen, dass die tatsächliche Tragfähigkeit wesentlich über den Bemessungswerten liegt. Dies ist zum Teil bedingt durch die Schwierigkeit, alle vorhandenen Kenntnisse bei der Ausarbeitung einer neuen Norm vollständig zu berücksichtigen. Es wird angenommen, dass diese zusätzliche Reserve der Tragfähigkeit mit zunehmendem Alter des Bauwerks infolge versteckter Schädigung des Tragwerks abnimmt. Solche versteckte Schadenseinflüsse können sogar zu einer grösseren Gefährdung der Tragsicherheit gewisser Brücken führen, als alle vorhersehbaren Überbelastungen. Um bestehende Normen zu verbessern ist es vorteilhafter, diese mit Versuchsresultaten und Erfahrungen zu vergleichen als einfach einen Vergleich zwischen Normen anzustellen.

# 1. INTRODUCTION

Composite structures of steel and concrete are competitive for most highway and railway bridges with spans between 20 m and 160 m or more. Problems of skew (up to 70°) and curvature have been solved, and decks over 40 m wide have been built.

A recent design method for such bridges, BS 5400 [1], is based on limit state design philosophy. Many bridges have now been designed and built to that code, which is known in the U.K. as 'the Bridge Code'. The draft Eurocodes 2, 3, and 4 [2,3,4] are also intended to be used for composite bridges. They are at present less comprehensive than the Bridge Code, but provide a basis for the design of composite superstructures using rolled steel sections or plate girders, in accordance with national loading specifications.

Ideally, a bridge designed to BS 5400 or the Eurocodes should not reach a limit state before the end of its design life, which in the United Kingdom is normally 120 years. The question of which limit states are most likely to be reached in such bridges is the main subject of this paper. It is relevant to the design of future bridges, the revision of design codes, the choice of values of partial safety factors, the planning of new research, and the maintenance of existing bridges.

Experience shows that imposed loading on highway bridges usually increases during their life, and may after only 20 years be different in character, as well as in magnitude, from that for which the bridge was designed. This process increases the probability that a limit state will be reached, and creates a need for methods of assessing the overload capacity of existing bridges.

The scope of this paper is generally limited to the superstructures of highway bridges with spans not exceeding 100 m. No account is taken of exceptional events such as earthquakes, fires, explosions, impact from aircraft, and serious errors in the design, construction, or management of bridges. The main sections of the paper are as follows:

- review of existing knowledge, especially that relating to failure;
- review and discussion of design loadings in relation to conceivable modes of failure;
- study of the relationship between existing knowledge and methods used by designers;
- discussion of the overload capacities of existing bridges, and prediction of situations in which limit states seem most likely to be reached; and
- some comparisons between recent design procedures for continuous beams.

# 2. REVIEW AND DISCUSSION OF LITERATURE

## 2.1 Limit states and methods of analysis

A summary is given below of the results of a recent detailed study [5] of failure modes of composite bridges. In the current practice of limit state design, bridge decks are proportioned and detailed primarily for the ultimate limit state, so the emphasis here is on the inelastic behaviour that invariably precedes failure, and on the results of tests.

Where non–compact sections are present, as is usual in bridges, design is almost always based on elastic global analysis, as the alternatives are impracticable. Elastic analysis tells us little about the attainment of limit states (for yielding of steel is not itself a limit state), except in relation to cracking – of concrete in tension and of steel in fatigue.

Analysis of non–compact cross–sections at the ultimate limit state is no longer based solely on elastic theory. Some recent codes allow extensive redistribution of longitudinal bending stresses from webs to steel flanges, and from steel tension flanges in hogging bending to reinforcement in the adjacent deck slab. Interaction between bending moment and vertical shear is treated using action effects, rather than stresses; so are the effects of unpropped construction; and inelastic action is exploited in the design of slender webs for vertical shear.

The design of bridges to carry high–speed trains may be governed by limits on deflection; and dynamic behaviour has to be considered in the design of footbridges. Elastic theory is relevant to these subjects. The design procedures are well understood [6], and these limit states are rarely reached, so they are not discussed further.

The concrete in the members studied is invariably normal–density, not prestressed unless stated; haunches are small or absent; and the shear connectors are usually welded studs. Where reference is made to design to BS 5400, this means the version in use in 1986, taking account of the Departmental Standards of the Department of Transport.

## 2.2 Laboratory tests

Reports have been studied on all known relevant laboratory tests (about 100) on hogging moment regions of continuous composite beams; but fewer than ten of these cross–sections had webs with depth/thickness ratios exceeding 100, though the use of higher values in stiffened webs is common in practice. Some of the tests gave information on rotation capacity and longitudinal redistribution for particular loadings, but none used travelling loads, or studied shakedown and incremental collapse. No comparisons exist between the behaviour of these members and their resistances predicted in accordance with BS 5400 or the Eurocodes, because none were designed or analysed to those codes.

Reports were found on about 20 tests on models of multi–beam composite decks at scales ranging from one–third to one–twelfth of full size. In many of these, the steelwork was less slender than it would be in practice. The results of the more realistic tests are summarised below.

A model of a deck with two open–top trapezoidal box girders, of prototype span 30.5 m, was tested to failure [7] under loading that represented the wheel pattern of a standard 6–axle truck. The structure carried nine times the scaled weight of the truck without local failure of the slab. This is typical of many tests that have shown [8,9,10] that reinforced concrete decks of thickness between 190 mm and 250 mm have a far higher resistance to local loading than is allowed for in design, due to the effects of membrane action.

Two series of tests [11,12] on decks with rolled I–beams of compact section, at (prototype) spacing not exceeding 3.5 m, have shown that the lateral distribution of concentrated loads rises significantly above the level predicted by elastic theory, as yielding of steel develops. It has been found that yield–line theory gives excellent agreement with test results, in both failure load and failure mode, showing that the radical simplifications of membrane effects, geometry, and properties of concrete made in the theory do not introduce significant error. In Reference 12, the decks tested were found to be about 30 per cent stronger than would be predicted by BS 5400, even though that method uses plastic analysis for cross–sections.

Many elaborate tests were done on a realistic model of a bifurcated box girder highway

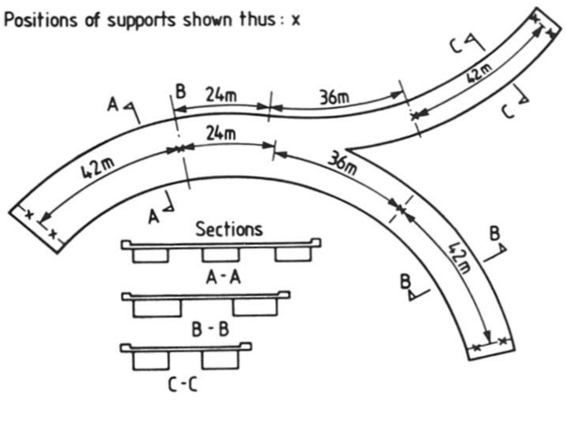


Fig. 1 Bifurcated box girder highway viaduct

viaduct, curved in plan, in which a deck composite with three open–top boxes split into two decks, each composite with two boxes (Fig. 1). Geometrical imperfections and sequential construction were simulated. The principal results were as follows [13].

(1) It is difficult to predict in the inelastic range the actions to which transverse members, such as crossheads at piers and inter–box bracings, are subjected; for these actions are sensitive to the *relative* stiffnesses of two or more longitudinal members. It follows that if the margin of safety for crossheads were to match that of the longitudinal members (which is at present unduly high), they would have to be designed for the most adverse of the actions determined from two or more global analyses using different sets of assumptions.

(2) Inter–box truss or frame bracings within spans typically have high stiffness and low strength. They can be subjected in service to very high stresses, and so are prone to fatigue damage. Their contribution to transverse distribution can be dispensed with, so it is better not to provide them. Open–top boxes are better braced internally.

(3) The design rules of BS 5400 (e.g., for stiffeners and plate panels) do not take full account of the inelastic interaction that ocurs between elements. They significantly underestimated the capacity of this structure for redistribution, both within each box and between neighbouring boxes.

Six one–eighth scale composite box girders have been tested in hogging bending and torsion [14]. The three closed–top boxes behaved much as expected, but the torsional resistance of the open–top boxes was significantly reduced by the effects of flexural tension in the micro–concrete slab, which was 30 mm thick. It was deduced from theoretical analyses that at full scale, cracking of the slab could lead to a loss of torsional rigidity of from 20% to 40%, depending on the type of loading. The distributions of imposed loading for maximum torsion and for maximum hogging bending are so different that this adverse interaction may be irrelevant to design; but further study of this subject is needed, focussed on realistic cross sections and loadings.

## 2.3 Large–scale tests

There have been many tests on completed bridge decks; but for obvious reasons, most of them had the limited objective of checking elastic response to service loading. Their results were much as would be expected. Only tests to failure are reviewed here.

A two–lane four–girder continuous highway bridge in the U.S.A., designed in 1963, was tested to failure when it was less than ten years old [10]. The longest span was 27.5 m, and the steel beams were rolled sections. Eight point loads, simulating a heavy truck in each lane, were applied near midspan of an internal span. The bridge behaved essentially as predicted by plastic hinge theory for a continuous beam. The maximum load was 5620 kN; the deflection under the load was then 610 mm; and the deck withstood severe local overload without failure.

A detailed proposal was made [15] for the testing to failure by incremental plastic collapse of a two–lane beam–and–slab bridge in the U.S.A. with a maximum span of 28.4 m. No suitable method of loading could be found at acceptable cost, so the fatigue behaviour of the deck was studied [16] by exciting it in its fundamental mode of flexural vibration for a total of 471 000 cycles. The range of dynamic strain in critical regions of the steelwork was typically $780 \times 10^{-6}$ and the range of vertical displacement at midspan was about 115 mm.

Fatigue failures occurred in welds at the ends of cover–plates, as predicted. By the end of the test, their propagation had completely severed the girder flanges at five different locations. Unexpected fatigue cracks in unwelded steel were initiated by the heat numbers stamped on the faces of flanges. The extensive cracking caused very little change in the natural frequencies, and only a small increase in the damping ratio. During testing, periodic searches for fatigue damage were made by eight methods, of which ultrasonic inspection was found to be the most reliable.

## 2.4 Tests on the local effects of wheel loads

In Canada, about forty tests in punching shear were done, mostly at full scale on the Conestogo River Bridge, as part of a thorough study of membrane and arching action [8]. It was found that deck reinforcement could be reduced to about one−third of that used previously. Account was taken of these results in drafting the Ontario Bridge Code [17], which in many regions allows local reinforcement to be as little as 0.3% each way near each face of the deck slab.

Tests in the U.K. showed [18] that in−plane biaxial tension had no effect on the punching shear resistance of slabs 90 mm thick with 1.8% and 1.0% of reinforcement in orthogonal directions, a result that is consistent with theoretical work [19]. The prestress in the deck of the Conestogo Bridge is therefore unlikely to have enhanced the Canadian results; but it would be imprudent to apply them, without confirmation from tests, in non−prestressed regions of biaxial tension. Such regions occur near internal supports of continuous viaducts with cantilever cross−girders.

## 2.5 Failures of bridges in service

In the paper 'Bridge collapses in Europe', Badoux states [20] that the common features of the major failures in 1969 to 1971 (Vienna, Milford Haven, Koblenz) included:

(1) reliance on simple design codes and theories falsely applied in complex situations; and

(2) insufficient allowance being made for fabrication defects and tolerances.

Lessons from these and other failures influenced the drafting of BS 5400; but the first feature (with 'simple' deleted) should still cause concern, for misuse of design codes was found to be 'a foremost failing ..., virtually international'. The paper also draws attention to the steady erosion of safety margins that begins as soon as a bridge enters service, for the following reasons.

(1) Increase in live loading: '.... the demands of modern society frequently complicate and augment natural processes by changing the rules after the game has started'.

(2) Deterioration of materials, exacerbated by the use of road salt and detailing that creates regions inaccessible to inspection.

## 2.6 Shear connection

A parametric study of the effects of slip in fifty bridge girders of span 10 m to 50 m has shown [21] that the normal design practice of neglecting the effects of slip causes negligible error in the predicted resistance of beams in sagging bending. There is, however, uncertainty about some other aspects of the design of shear connection.

In continuous beams, it is essential to provide in hogging moment regions shear connection appropriate for the longitudinal reinforcement in the deck slab. Questions arise concerning the effects of cracking of the slab on the behaviour of shear connectors, and of biaxial tension and shear in the plane of the slab (e.g., near crossheads at internal supports). A simple but conservative design method that provides a margin for such uncertainties is to calculate the longitudinal shear assuming that the concrete slab is uncracked. This method is given in BS 5400.

Design of the shear connection for the ultimate limit state also tends to be conservative when it is based on envelopes of longitudinal shear, calculated without allowance for plastic redistribution; and also because the stiffness and strength of studs with weld collars are usually higher in beams than in the standard push test, as defined in BS 5400 and draft Eurocode 4. The slabs used in this test are too narrow, and their transverse reinforcement is not fully anchored, so that premature failure tends to occur by splitting of a slab.

As for other welded details, design of shear connection for fatigue is usually based on Miner's cumulative damage rule, and a relation between endurance $N$ cycles and stress range $\sigma_r$ of the type $N\sigma_r{}^m$ = constant. In BS 5400, $m$ is given as 8. More recent work [22,23] has shown that $m$ should be about 5, the value given in draft Eurocode 3 [3]. For highway loading in the U.K., fatigue normally governs spacing of connectors in midspan regions.

2.7 Composite plates and diaphragms

Slabs that are composite with steel plates occur in closed–top box girders. Research has led to a good understanding of their elastic behaviour [24], and of forces on their shear connectors due to wheel loads [25]. The limited evidence on their ultimate strength has been reviewed [24]. Composite plates are also suitable for use as diaphragms at piers [26].

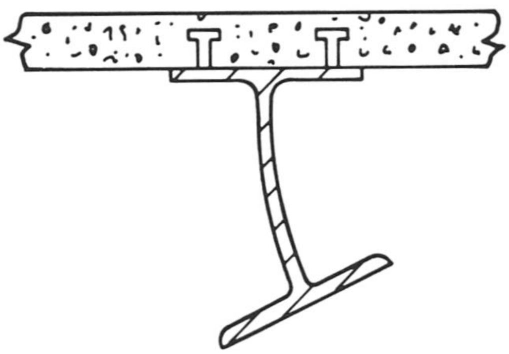

Fig. 2 Distortional lateral buckling of composite beam in hogging bending

2.8 Lateral–torsional buckling

Care must be taken to prevent lateral buckling in plate girders and open–top box girders during construction; but once the deck slab has been cast, buckling is possible only for plate girders near internal supports, and then only in a distortional mode (Fig. 2). Current design methods for this type of buckling underestimate the beneficial effects of moment gradient along the member, and of the torsional and warping stiffnesses of the bottom flange.

The method of BS 5400 is typically conservative by a factor of two [27], in comparison with a new method, so far developed only for beams without vertical stiffeners [28]. It is based on numerical analyses of distortional buckling, taking account of its interaction with inelastic local buckling of both the bottom flange and the web.

3. REVIEW AND DISCUSSION OF LOADINGS, IN RELATION TO CONCEIVABLE MODES OF FAILURE

In studying the ultimate limit state, it is necessary to envisage specific situations. A 'situation' is a combination of a particular structure at a particular instant in its life with a particular loading, where 'loading' may include extremes of temperature, wind, earthquake, fire, differential settlement, impact, and so on.

In several of the tests referred to above, collapse occurred under design (factored) dead load plus $x$ times the unfactored nominal imposed (live) loading, where $x > 5$. If no situation can be envisaged in which , say, six times the nominal live loading could occur, then a test in which $x$ was found to be six is not a realistic example of a failure situation.

In Reference 5, an attempt was made to identify loading situations that could be accepted as possible, even though highly improbable. No statistical study was attempted, through lack of relevant data on extreme loadings and their distributions, particularly in respect of traffic loading 100 years or more in the future. The loadings and load factors ($\gamma_f$) of BS 5400: Part 2 [1] were taken as a basis, and only highway bridges were considered.

The preceding review of literature shows clearly that for most types of bridge deck, an ultimate limit state can be reached only when one or more major components of the total loading are significantly in excess of their *factored* specified values.

The least improbable of such situations was found to be excessive imposed loading on the deck in the presence of one or more secondary load–related causes and some deterioration of the structure.

Excessive imposed loading can arise from a combination of several of the following events.

(a) Overloading of individual vehicles.

(b) An increase in the number of traffic lanes; for example, during repair of one carriageway.

(c) Bunching of heavy vehicles at certain times of day, made worse by a traffic accident.

(d) Changes in vehicle design or loading regulations during the life of the bridge.

(e) Military emergency; for example, a convoy of tank transporters, failing to observe the rules on vehicle spacing.

(f) An accident; for example, one that displaces a heavy vehicle onto the edge of a cantilever footway.

(g) Deliberate passage of an over-load, coupled with an error in the prior check on the safety of the operation.

Secondary load-related causes include:

(a) local fatigue damage, and

(b) excessive secondary (hyperstatic) effects of temperature.

'Deterioration' here means changes in the structure not specifically allowed for in its initial design. These include:

(a) adverse changes in the properties or integrity of the concrete due, for example, to alkali-silica reaction, carbonation, or free-thaw cycles, leading to corrosion of reinforcement and further damage due to bursting pressure; and

(b) excessive cracking or spalling due to previous local overloading.

Extreme wind load could be a significant cause of failure in special circumstances, for example, when combined with the presence during erection of unforeseen loads from plant and materials.

## 4. THE INFLUENCE ON FAILURE OF THE METHOD OF CONSTRUCTION OF THE CONCRETE DECK

In composite members in bridges, imposed loading is the predominant source of stress. They carry it, whatever may be the method of construction. The method of construction influences only the way in which dead loads are carried. Its influence on design is small, and simplifying assumptions are both possible and necessary. The following situations can occur.

(1) *Propped construction.* All dead load is resisted by composite members. This is costly, and uncommon in practice. Analysis and design are straightforward. Existing design methods that allow for creep by doubling the short-term modular ratio (e.g. Ref.[1]) certainly underestimate its long-term effects and so are conservative for the deck and the shear connection, but not for the structural steel.

(2) *Mixed construction.* The steelwork is unpropped and the deck is cast in stages. For dead load, the first areas to be cast are almost fully composite, and the last areas are non-composite. This is the most common situation. Except in the largest and most slender structures, it is accurate enough to assume in design that all the dead load is carried by the steelwork alone. Any resulting overstress in concrete will certainly be relieved by creep before an ultimate limit slab is reached. This assumption would be unconservative in relation to cracking of concrete if regions above internal supports were cast first. In practice they are cast last, to minimise cracking.

(3) *Fully unpropped construction.* This is achieved when the deck slab is completed before it is attached to the steelwork; for example, when the sliding ribbon method or precast deck units are

used. The method of analysis is as in (2) above, and the results are more accurate than are those in situations (1) or (2).

(4) *Prestressing*. This can occur in conjunction with any of situations (1) to (3). It is rarely used in the U.K. or North America, and its use in Continental Europe is declining. The main problem in design is to predict accurately the loss of prestress due to creep [24].

## 5. THE RELATIONSHIP BETWEEN DESIGN PRACTICE AND EXISTING KNOWLEDGE

Reference is made in this Section to the results of a study [5] of the designs by Cass, Hayward and Partners of three composite bridges for highway loading, in accordance with BS 5400. These designs were all successful in competition with alternative non-composite designs, and all three bridges were under construction in 1985.

The first is a pair of continuous viaducts, with spans of up to 50 m, each consisting of four box girders typically 1.7 m deep, with integral composite crossheads and an *in situ* deck 18.4 m wide, cast using precast concrete planks as formwork.

The second bridge has three continuous spans, the longest being 37 m, each consisting of ten plate girders, an *in situ* deck 36 m wide with a skew of 9°, and composite crossheads.

The third bridge is a jetty with continuous spans of 24 m, each consisting of two rolled steel beams supporting precast deck slabs 4.75 m wide, which act as permanent formwork for concrete topping 100 mm thick.

These designs are representative samples of the best of current practice in the U.K., where 'best' relates both to cost-effectiveness and to the use of *that part of* existing knowledge that appears in BS 5400. Two of the designs were for a government department, and were also subject to checking by an independent consultant. This common situation effectively prevents the designer from using any recent advances in knowledge not explicitly set out in the Code that he is required to follow.

The question is therefore not whether the designs exploit existing knowledge; there are two questions:

- do the designs fully exploit the relevant Code in force, and
- does that Code fully exploit existing knowledge?

*The full exploitation of the Code.* This question needs discussion at three levels. In relation to overall concept, it is unlikely that these designers, or any others, would have found optimal solutions within a year of the publication of the first code for limit state design of bridges in the U.K. Only the lapse of time will show this.

At the second level, types and sizes of members are much influenced by factors not within the scope of codes, such as available sizes and grades of steel plate and section, types of permanent formwork, and facilities for fabrication, transport, and erection.

Finally, detailing of the structure is much influenced by factors such as estimated costs of welding and inspection, available techniques for the control of distortion, and available sizes of reinforcing bars, etc.

Detailed study of the three designs showed them to be strongly influenced by these non-code factors, which require judgement. To establish partial exploitation, one therefore has to show not only that a partially stressed member could have been given a smaller cross-section, and that with the smaller member the bridge would still have the specified serviceability, durability, and fatigue life; but also that the bridge would have been cheaper. It is rarely possible to do this.

*The exploitation of existing knowledge in codes of practice.* Before any new design procedure can come into general use, it has to be determined that:

(a) it is founded on established knowledge;

(b) it is practicable to incorporate it within the relevant code of practice; and

(c) if that had been done, the design of these (or some other) bridges could have been so modified that their cost would have been reduced, or their durability increased.

When specific examples are studied, it is found that some fail test (a), because what has been considered by a code drafting committee to be 'established knowledge' is not accepted as such by a government department that commissions bridges. (This problem will certainly delay, and could even prevent, the full implementation of the Eurocodes for bridge design.)

Another common situation is that some beneficial effect clearly exists in relation to a *particular* design decision, but no advantage can be taken of it because no *general* method of codifying it can be found, so that it fails test (b). To a much greater extent than a decade ago, any designer who departs from the Code in such a situation, in the belief that the checker and the client will accept his judgement, takes a grave risk; for if they do not, the cost of ensuing delays to the project may far exceed the potential saving, may lose him the commission, or may lead to litigation.

In practice, test (c) is not important, because if a new procedure brings no benefit, designers do not use it.

## 6. LIMIT STATES FOR COMPOSITE BRIDGES

Section 2 of this paper provides evidence of many favourable aspects of the behaviour of composite bridge decks at the ultimate limit state that are not fully exploited in design; usually because they fail test (a) or test (b), and so are not in codes. In other words; most superstructures of composite bridges are *initially* much stronger than we assume them to be, in relation to short–term overload.
It is significant that the experimental evidence for this conclusion comes almost entirely from tests on structures less than 15 years old, in which deterioration was negligible.

It has also been concluded (Section 3) that the degree of overloading needed to reach an ultimate limit state in a young structure is so great as to be implausible; there has to be deterioration as well. Obviously, the greater the deterioration, the less the overloading need be. There is a tendency for design loadings, and probably overloading, to increase with time; but recent experience with concrete bridges suggests that progressive deterioration is by far the greater danger.

The most significant characteristic of a given type of deterioration is, therefore, how soon its effects become evident, under the type and frequency of inspection that it is feasible to provide in practice. This notion is not quantified in our current system of partial safety factors. For example, corrosion of poorly grouted prestressing tendons is more serious than rusting of a prominent surface of a steel beam, a fact not clearly recognised in limit state design. The former has already caused an ultimate limit state to be reached in at least one (non–composite) bridge; in contrast, the latter can be (and regularly is) treated as a mild form of unserviceability.

Giving due weight to detectability, the most vulnerable regions of a composite bridge appear to be its shear–connector welds, which are at risk in respect of

(a) poor workmanship,

(b) fatigue failure, and

(c) corrosion caused by salt–laden water, either passing through cracks in the deck slab, or seeping horizontally across the steel–concrete interface, after being deposited near a flange tip as spray from a roadway below.

The only relevant evidence known to the writer in fact points the other way. In 1978 he inspected a partially−demolished beam−and−slab composite bridge in Japan, then 25 years old, which was being removed to make way for a larger bridge. The corrosion of the steel top flange in contact with the concrete soffit, and of the shear connectors, was negligible; even though there was severe corrosion of *exposed* steelwork on some much younger nearby bridges, due probably to the adverse industrial and marine atmosphere of Osaka. But this bridge was over water, not over a road; and 25 years is only one−fifth of the design life for some composite bridges. As yet, we know little of the extent to which weakened or fractured studs modify the stiffness and dynamic characteristics of a bridge deck. The evidence from Reference 16 suggests that quite severe deterioration in the ultimate strength of a shear connection could go un−noticed.

Any engineer who has the opportunity to expose the shear connection of an old bridge is therefore urged to look for fatigue cracks and signs of corrosion, and to publicize the findings, good or bad.

Great care is taken in design to establish that the fatigue lives of welded details are sufficient; but the methods used are far from exact. Miner's rule is a convenient approximation; stress ranges at connectors caused by passing wheels are not accurately known; and there is uncertainty in the $N-\sigma_r$ relation for stud connectors, discussed above. Even with a fifth−power law, a 10% increase in wheel or axle loadings, at constant traffic levels, may cause a 38% reduction in fatigue life. The experience so far with composite bridges is very good − but they need to be watched.

## 7. COMPARISONS BETWEEN SOME LIMIT−STATE DESIGN METHODS

It will be shown in this section that superficial comparisons between codes can be misleading, and that the many differences between practice in different countries make in−depth comparisons very difficult. Even the costly process of preparing design calculations in accordance with two or more codes for a given crossing (not a given *bridge*, because the resulting bridges will differ in many respects), will provide data relevant only to bridges suitable for that crossing, and to a few of the many possible limit−state situations.

We consider the Autostress method of design for bridges using braced compact rolled or welded steel I−section beams. It is a development [29] of the Load Factor Design method of AASHTO [30]. Design calculations are available [31] for a bridge with the cross section shown in Fig. 3. It carries a two−lane highway and is continuous over two 30.5−m spans.

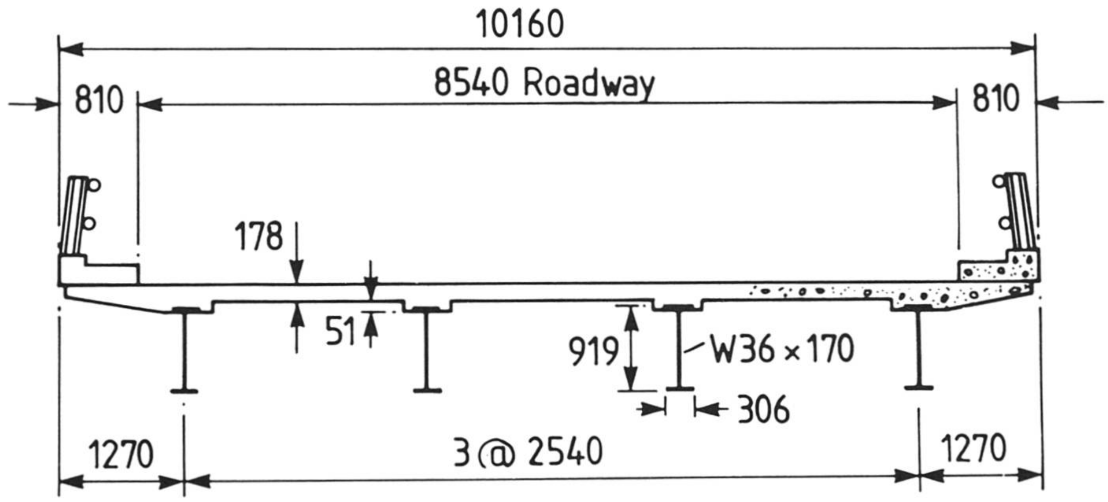


Fig. 3 Typical section of bridge designed by the Autostress method

*Loading*. The imposed load, including impact, is denoted by $I$. It consists of two alternatives: the HS20−44 lane and knife−edge loading, and the HS20−44 three−axle truck, of total weight 320 kN. Three loading levels are considered:

| | |
|---|---|
| Service | $D + I$ |
| Overload | $D + 1.67\ I$ |
| Maximum | $1.3\ (D + 1.67I)$ |

where $D$ represents the dead load. The ratio $D{:}I$ for this bridge is 1.5:1, so the ratios of the three load levels are 0.61:0.77:1.0. Service loads are those used in the Working Stress design method. The Overload is related to permanent deformations of the members, caused by yielding of steel, and is used when calculating camber. Maximum load is expected to cause "severe permanent deformation"; but the structure is expected to survive "a few passages" of loads at that level [31]. There is no obvious equivalence between these criteria and the Serviceability and Ultimate loads used in the Eurocodes and the Bridge Code[1]. Eurocodes do not yet give a highway loading, or any partial safety factors for one, so we consider the HA highway loading of the Bridge Code. The global safety factor is the product $\gamma_m \gamma_{fL} \gamma_{f3}$. At the ultimate limit state $\gamma_m$ depends on the mode of failure, and ranges from 1.05 to 1.2. Taking a mean value of 1.12, and assuming the same $D{:}I$ ratio of 1.5:1, the products of the factors are 1.80 and 2.62 for the Serviceability and Ultimate limit states, respectively.

To make progress we now assume that the Maximum load (Autostress) and the Ultimate load (Bridge code) are equivalent. The Serviceability load is 0.69 times the Ultimate load, and so falls between the Service and Overload values of the Autostress method.

*Global analysis, and moments of resistance.* The Autostress method allows plastic hinge analysis of individual main girders, associated with effective values of moments of resistance, $M_u$ and $M'_u$, say, that are reduced below the full plastic values, $M_p$ and $M'_p$, say, for the composite sections in sagging and hogging bending, respectively. The reduction depends on the slendernesses of the steel compression flange ($b_o/t$ = outstand/thickness) and the web ($d_c/w$ = depth in compression/thickness). It also depends on the yield strength of the steel, which is assumed from here onwards to be 345 N/mm² (50 000 lb/in²). The maximum slendernesses for zero reduction (i.e., when $M_p$ and $M'_p$ can be used) and those in Eurocodes 3 and 4 for Class 1 sections are given in Table 1.

TABLE 1. Limiting slendernesses for steel flanges and webs.

| Design method | $b_o/t$ | $d_c/w$ |
|---|---|---|
| Autostress, plastic/plastic | 7.0 | 27.7 |
| Eurocode 4, Class 1 | 6.2 | 24.7 |
| Eurocode 4, Class 2 / Eurocode 3 Class 1 | 7.4 | 27.2 |
| BS 5400, compact | 7.1 | 28.4 |

A detailed survey of the evidence relating to slenderness limits for full plastic design of composite beams is given in the Commentary on Eurocode 4 [32]. It is concluded that $b_o/t$ should be between the values given in Eurocodes 3 and 4, and that $d_c/w$ should be as in Eurocode 3. The Table shows that this agrees closely with the values of the Autostress method.

The use of plastic hinge analysis with reduced moments of resistance in the Autostress method is equivalent to the design of beam with Class 2 sections in Eurocode 4. The limiting slendernesses

(Table 1) are 7.4 and 27.2. These take account of the method of global analysis, which is elastic, using "uncracked" flexural stiffnesses, with up to 30% redistribution of moments at internal supports to adjacent midspan regions. For Class 2 sections, $M_p$ and $M'_p$ can be used; so in this range of slendernesses, the Eurocodes (and the Bridge Code) use a global analysis more conservative than the plastic hinge method, whereas the Autostress method penalises the moments of resistance.

Calculations for a fixed–ended beam with slendernesses 7.4 and 27.2, and uniformly–distributed load, show that which code is the more conservative depends on the ratio $M_p/M'_p$ for the beam. The Autostress method gives an ultimate load 5% lower when $M_p/M'_p \leqslant 1.16$; but 7% higher when $M_p/M'_p = 1.4$.

The design to BS 5400 for bending of beams with compact sections is similar to that of Eurocode 4, except that only 10% redistribution of moments is allowed. The limiting slenderness for webs, Table 1, appears to agree quite well with the Class 2 limit of Eurocode 4. In fact it does not, because in BS 5400, unlike the other codes listed, $d_c$ is calculated using the elastic (rather than the plastic) neutral axis. In hogging regions and for a steel section with equal flanges, this gives lower slendernesses, by amounts up to over 30%, depending on the area and yield strength of the slab reinforcement [32].

*Other differences.* In the preceding comparisons, other differences between the Autostress method and the other codes have been glossed over. They include the following.

(1) An empirical and quite conservative method of transverse load distribution is used in the Autostress method. In Europe, an elastic grillage analysis would normally be used. Even this is very conservative when compared with results of tests [5].
(2) The Autostress slenderness limits, in Table 1, are modified when the slendernesses of both the web and the flange exceed 75% of the tabulated value. This is not done in the European codes.

(3) Rules for effective breadth of a concrete flange differ quite widely from code to code.

(4) The Autostress method applies only to beams braced against lateral buckling. The criteria for satisfying this condition differ from code to code.

Many more differences could be listed. Because rules in codes are often step functions, the differences affect design in an almost capricious way. Their combined effects tend to cancel out, because all codes are derived essentially from experienced and research.

Comparison of one code with others is in fact an unrewarding activity. The state of the art would benefit more if most of the effort devoted to it were diverted to the recording and distillation of experience and the broadening of the research base which, as shown earlier in this paper, is in places very narrow.

## 8. CONCLUSIONS

### 8.1 Scope

These conclusions relate to superstructures of composite highway bridges of span less than 100 m, in regions where risk of earthquake damage is considered to be negligible. Their applicability is limited by the fact that in every country there is a unique relationship between current actual highway loadings and the design (factored) loadings for the ultimate limit state, which changes with time. The conclusions are appropriate to the relationship that exists in the U.K., which is similar to those in many other European countries.

### 8.2 Ultimate limit state

The probability of failure of a complete span is *initially* so low (in the absence of major human error in design or construction) that failure is inconceivable. This probability increases gradually with time, due to the tendency for imposed loads to increase and structures to deteriorate. Whether the risk of failure reaches a level, during the design life of the bridge, that should give cause for concern, leading to action, depends on many factors. The risk becomes less:

- as the number of main girders in a span increases,
- as the breadth of the deck increases (because there is more scope for transverse distribution),
- as the degree of continuity between spans increases,
- when the steel plates in compression are less slender,
- as the number of alternative load paths increases,
- as the quality and frequency of routine inspections increases, and
- when the structure is made easy to inspect.

At present, we probably pay too much attention to design for ultimate limit states that can never be reached unless the structure has deteriorated, and not enough to design to minimise corrosion, carbonation, exposure to salt, and fatigue damage. One cannot be certain, for few composite bridges have yet completed the first quarter of their design life; but in future, their resistance to overload may increasingly depend on the extent of undetected deterioration, and so on the last two factors in the list above.

### 8.3 Serviceability limit state

The important types of unserviceability all lead to deterioration. Its effects should ideally always cause a serviceability limit state to be reached and detected before the consequent loss of resistance is such that occurrence of an ultimate limit state becomes conceivable. This issue is not formally treated within limit state design philosophy as used in the recent codes, probably because of shortage of data. It is one where research is still needed, on questions such as: what proportion of the resistance of a composite beam to longitudinal shear is it possible to lose (due to corrosion or fatigue cracking) before the weakness becomes evident during routine inspection?

### 8.4 Codes of practice

Even though all the limit state codes for composite bridges known to the author were published since 1976, the differences between them are so numerous and varied, that detailed comparison of one with another is a difficult and unrewarding task, which was not attempted during the drafting of Eurocode 4.

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