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Ultimate Strength of Steel-Concrete Composite Sections under Biaxial Bending

Jun KAWAGUCHI
Research Associate
Mie University
Tsu, Mie, Japan

Shosuke MORINO
Professor
Mie University
Tsu, Mie, Japan

Mika UEDA
General Building Research Corporation
Suita, Osaka, Japan

Summary

Effects of loading path on the ultimate strength and the moment-curvature relation of cross sections subjected to axial load and biaxial bending were investigated by analyzing four kinds of cross section: wide flange, square tube, CFT and SRC sections. Three types of loading path were considered. It was found that an identical point on the ultimate strength interaction was reached regardless of the loading paths, and the maximum values of bending moments may be different from those at the ultimate strength point on the interaction.

1. Introduction

The ultimate strength interaction curves of reinforced concrete and steel reinforced concrete (SRC) cross sections were extensively investigated, and several mathematical formulas were proposed [1,2]. However, no research was found which mentioned the effect of loading procedure. The purpose of this investigation is to clarify the effect of the loading procedure on the ultimate strength by the numerical analysis of the moment-curvature relation considering the strain reversal and the local buckling of steel elements.

2. Moment-Curvature Relations

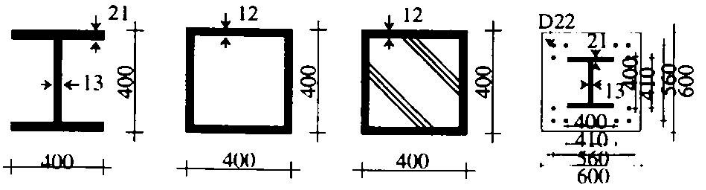
The moment-curvature relations of four cross sections were analyzed: wide flange, square tube, concrete-filled square tube (CFT) and SRC containing wide flange as shown in Fig. 1. The stress-strain relations of steel, reinforcements and concrete were assumed as shown in Fig. 2. The relation of the steel contains the stress reduction part caused by the local buckling [3]. Materials assumed here were SM490 class (σ_y = 323.4 N/mm^2) for steel, SD40 class (σ_y = 211.6 N/mm^2) for reinforcement, and the 300 kgf/cm^2 (F_c = 29.4 N/mm^2) for concrete. Parameters for the stress-strain relations are shown in Table 1.

Figure 3 shows the strain distribution in the cross section subjected to the axial load N and the biaxial bending moments M_x and M_y, which were calculated for given values of the curvature by the numerical integration, dividing the cross section into small elements and assuming the uniform stress distribution in each element. The following three loading procedures were considered:

A: M_y was kept constant, and M_x was gradually increased.

B: M_x and M_y were gradually increased so that the deformation angle ($\theta_f = tan^{-1}(\phi_y / \phi_x)$) was kept equal to the value which was already obtained at the ultimate stage in the procedure **A** .

C: Proportional loading with the bending angle ($\theta_m = tan^{-1}(M_y / M_x)$) kept equal to the value which was obtained at the ultimate stage in the procedure .



(a) Wide flange (b) Square tube (c) CFT (d) SRC

Fig. 1 Cross-sections

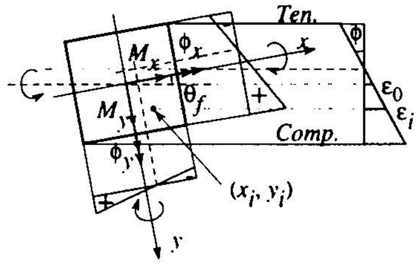
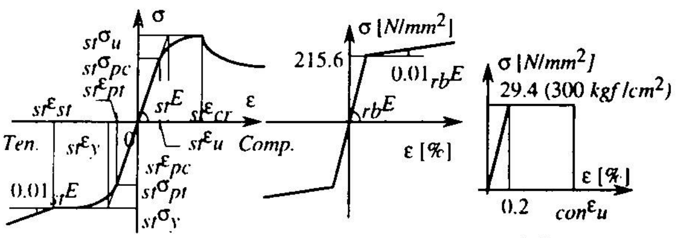


Fig. 3 Strain distribution

Table 1 Parameters

$_{st}E(kN/mm^2)$	205.8
$_{rb}E(kN/mm^2)$	205.8
$_{st}\sigma_{pc}(N/mm^2)$	196
$_{st}\sigma_{pt}(N/mm^2)$	-196
$_{st}\sigma_y(N/mm^2)$	-323.4
$_{st}\sigma_u(N/mm^2)$	288.6
$_{st}\varepsilon_u$ (%)	0.14
$_{st}\varepsilon_{cr}$ (%)	0.67
$_{con}\varepsilon_u$ (%)	0.6(CFT) 0.4(SRC)

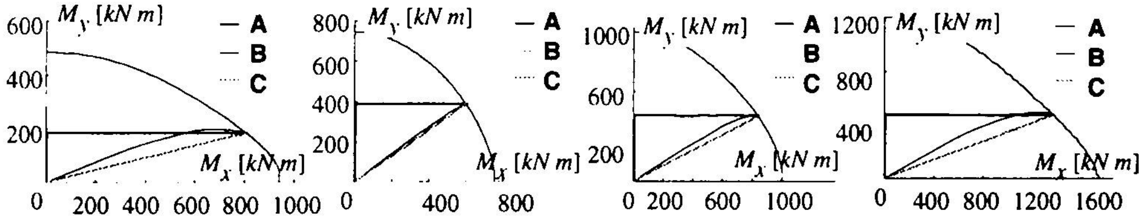


(a) Steel (b) Reinforcement (c) Concrete

Fig. 2 Stress-strain relations of material

3. Ultimate strength

Figure 4 shows the M_x-M_y paths of four cross sections together with the ultimate strength interaction curve which was calculated at the stage that the compressive strain of any element reached to the strain of the local buckling or the strain of the concrete crash. The procedure **A** in which M_y was kept constant, and the procedure **C** in which the bending direction was kept constant show the linear relation from the beginning to the ultimate stage on the interaction curve. On the other hand, procedure **B** shows the curved relation. However, M_x-M_y paths obtained from three analyses met at the exactly same point on the interaction curve for the ultimate strength of the cross section. The maximum values of M_x and M_y of the wide flange and the SRC sections are different from the strengths given by the ultimate point on the interaction curves. This is because these cross sections have unequal strengths about two major axes.



(a) Wide flange (b) Square tube (c) CFT (d) SRC

Fig. 4 Biaxial Ultimate Strength

4. Conclusion

1) An identical point on the interaction curve for the ultimate strength of the cross section subjected to the axial load and the biaxial bending moment was reached regardless of the loading procedure.
2) The ultimate strength point on the interaction curve was not the same as the maximum point of M_x and M_y in the case of the procedure **B** with the constant deformation angle for the section having unequal strength about two major axis.

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Long Term Performance of Timber Concrete Composite Structural Elements

Ulrich A. MEIERHOFER
Dipl.-Ing. ETH
EMPA
Dübendorf, Switzerland

Summary
Little information was available on the long term performance of timber concrete composite structural elements (TCCs) up to now. Recently finished long term shear and bending tests suggest design creep values.

1. General remarks

Composite construction originally meant the combination of structural steel and concrete. For many decades the combination of timber and concrete has not been considered as feasible for various reasons, one important being the distinctively different properties of the two materials. Together with a changing attitude of the engineers in combining various materials, the combination of concrete and timber - especially used as floors - also offered various advantages and became more acceptable. A vital point is the development of an efficient connection to transfer the shear forces between timber and concrete. A considerable amount of theoretical and practical research and development in this field resulted in a rapidly increasing knowledge and an in implementation in many building rehabilitations and new projects.

Newly acquired knowledge and experience on concrete composite structural elements (TCCs) are not restricted to structural properties such as strength and stiffness, but also encompass fire safety, acoustics, working condition during erection, economics and even aesthetics.

2. Long term deformation as governing criterion

The analysis of built up structural elements with semi-rigid connections requires the solution of differential equations and doesn't belong therefore to the everyday's problems of the structural engineer. Solutions have been developed, however, and designs aids are available. For the modelling of the structural behaviour various simplifying assumption must be made including behaviour and properties of materials and connections. To calibrate and verify this assumptions and to establish system behaviour not so far accessible by theoretical means, experimental work is indispensable. This refers particularly to the deformation behaviour of the connectors within the wood.

As had already been shown by early investigations, of all the factors necessary for a satisfactory performance of TCCs, the bending stiffness, particularly the long term bending stiffness or the long term deflection proved to be of special importance. The susceptibility of TCCs to long term deformations is a consequence of creep phenomena as well as relaxation of residual stresses caused mainly by the differential thermal and hygroscopic behaviour of concrete and timber. For a theoretical analysis of this complex phenomena the basics are still lacking and the experimental work - especially on long term deformation - is so far very limited.

3. Research and development at EMPA

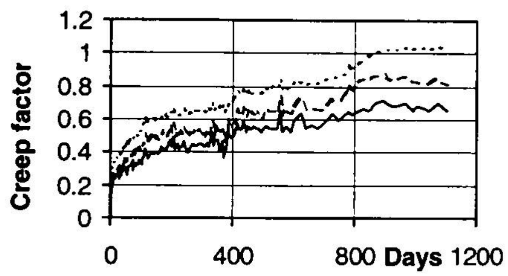
The Wood Department of the Swiss Federal Laboratories for Materials Testing and Research (EMPA) has undertaken extensive research in this field with a special screw like connector developed by SFS Stadler, Heerbrugg, Switzerland. This connector is characterised by its high slenderness which allows a very efficient installation without predrilling. The lack of bending stiffness of this connector is compensated by a mounting under an angle of ±45° which allows transfer of the shear forces between the concrete and the timber by tension and compression within the connectors (instead of the more common bending and bearing).

The tests performed encompassed basically withdrawal tests from timber and concrete, shear tests with symmetrical timber/concrete/timber specimens and bending tests with TCCs having spans of nearly 4 m. This work has been performed mainly in the years 1990 to 1993 and was reported in [1,2,3]. Within this period appropriate long term shear and bending tests were started too. Test parameters were the (geometrical) arrangement of the contors, load level and climatic conditions. The maximum load level was fixed in such a way that it superseded to some extent which was considered a reasonable operational level deducted from the results of the short term tests. The shear creep tests were run under constant climatic conditions (23°/50% r. h.) whereas the bending creep specimens were installed outside under roof, i. e. they were subjected to the natural variation of temperature and humiditiy. Even though the investigated TCCs had been developed and earmarked for interior use, this severe test configuration was selected to obtain conservative results.

After an extended period of creep (about 5 years for the bending tests and about 3 years for the shear tests) the specimens were unloaded, creep relieve recorded and then tested to failure in a short term test. The results are reported in [4] and some brief information follows.

4. Long term shear tests

Fig.1 Development of creep factor of shear tests with different connection stiffnesses



The main interest was the long time deformations which are shown in *Fig. 1* as the development of the creep factors over a period of about three years.The recovery behaviour after unloading (not shown here) proved to be satisfatory and indicated that the long term high loads did not cause any damage to the structure, resp. connection system. The creep curves may be fitted particularly well by potential curves of the type y = a * x exp b (correlation factors between 0.966 and 0.981), where 'a' variied between 0.10 and 0.16 and the exponent between 0.243 and 0.276. Extrapolated to a period of about 20 years, this implies creep factors between 1 and 1.7.

5. Long term bending tests

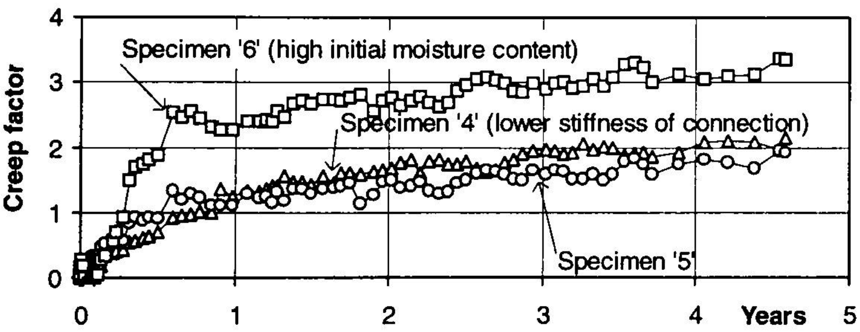


Fig. 2. Creep of three different TCC-specimens

The bending specimens, composed of a timber beam 12 cm x 18 cm and a concrete slab 8 cm x 75 cm cross-section were equipped with dial gauges to measure the sag at mid span and the end slip between timber and concrete. Strain at various points at mid span were recorded also by deformeter. One of the main results is the increase of sag shown in *Fig. 2* as creep factor. Due to the strong influence of climatic residual stresses (changes of humidity and temperature), the sag creep factors are considerably higher than those of the shear tests. Extrapolations of the fitted potential curves suggest a twenty years creep factor between 2.5 for normal and 4.5 for especially unfavourable conditions.

5. Conclusion

Long term deflections of TCCs under higher loads can't be neglected even in indoor climatic conditions. A creep factor of at least 2.5 should be applied until further research results suggest more differenciated values.

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Vertical Shear Resistance Models for a Deltabeam

Matti V. LESKELÄ
Ph.D. (Civ.Eng.)
University of Oulu
Oulu, FINLAND

Matti Leskelä, born 1945, received his Ph.D. in 1986 and has been carrying out research into composite structures from the early 1980's. His latest work has concerned problems of partial interaction and various shear connections in composite structures such as slim floors, composite slabs and concrete filled steel tubes.

Summary

The Deltabeam is an innovative structural form for beams for use in slim floor construction fabricated by Deltatek in Finland. The principles employed in developing vertical shear resistance in the composite state and in the initial state of steel construction are discussed. Changes in behaviour due to exposure to fire from the underside are introduced, and methods of reinforcing the system are reviewed.

1. Introduction

The Deltabeam consists of a boxed steel member in which circular web holes are spaced at constant distances along the span so as to make it possible to fill the box section with concrete and to make the structure behave compositely after solidification of the concrete. In the initial state, as a steel construction, the web holes, extending maximally to 60 % of the web depth, cause an obvious reduction in the vertical shear resistance of the beam, but the resistance is nevertheless adequate for the applications for which the beams are intended. After the solidification of the concrete the vertical shear resistance is enhanced greatly and can cope with all loads introduced in the ultimate limit state.

2. Mechanisms for shear resistance

Mechanisms for the evaluation of vertical shear resistance are defined for three states of behaviour: (1) steel construction, (2) composite construction and (3) composite construction when exposed to fire. The principal difference in the effective structure between normal conditions and fire temperatures is that the unprotected bottom flange is normally lost in fire exposure and cannot contribute to the shear resistance.

2.1 Steel construction

The resistance of the steel member to vertical shear forces should be considered with respect to stresses formed due to the combined effects of local and global actions in the sections through the web holes. It is scarcely possible to derive reasonable formulae directly for the maximum stresses around circular holes, however, and therefore an approximation with respect to a beam having square web holes of the same depth is used.

In German research into I-beams with circular and square web holes [1, 2] it has been shown that the ratio of the ultimate resistances in beams with square (resistance $V_{\square.R}$) and circular web holes (resistance $V_{\phi.R}$), $V_{\square.R}/V_{\phi.R}$, can satisfactorily be defined as a function of the relative depth of the holes, ϕ/h, ϕ being the hole depth and h the depth of the beam. Since it is possible to evaluate the stress state in the case of square holes, the known ratio of the resistances may be applied to convert $V_{\square.R}$ into $V_{\phi.R}$. The principle was verified by means of loading tests and was observed to work well when the resistance was determined based on the first yielding in the edges of the holes.` This is also justified, considering the state of construction.

2.2 Composite construction

The concrete infill inside the boxed section suggests that a system of compression struts and tension ties will develop in a truss form when the load is increased in steps to failure, i.e. compression struts will be formed between inclined cracks in the concrete contained in the boxed section and the web sections between the holes work as vertical tension ties. The system bears some resemblance to that observed in concrete beams reinforced with vertical stirrups, but it must be noted that the contributions of the concrete and steel to the shear resistance are not additive. This is explained by the considerably higher yield capacity of the web sections as compared to the normal density of stirrups in the reinforced concrete structures.

2.3 Composite construction exposed to fire

If no thermal insulation is applied in the bottom flange of the steel section, fire exposure from below will normally make it inefficient, not only for allowing excessive bending of the beam, but also for maintaining shear resistance. To ensure the bending resistance, reinforcing bars are used inside the box section, but these do not contribute to the vertical shear resistance of the structure unless it is ensured that the maximum force in the diagonal compression struts, formed in the same manner as at normal temperatures, is able to anchor to the webs of the steel section. It was observed in loading tests that the diagonal compression struts in beams with no bottom flange were finally pushed out, causing then anchorage failure in the reinforcing bars, which were not able to develop any yield. The majority of the diagonal concrete forces are anchored by the web holes, and the rest of them can be anchored to additional devices such as stirrups and shear plates welded to the inside of the top flange.

2.4 Interaction with bending

The vertical shear resistance described in section 2.2 above is such that very thick flanges in the beam are required to prevent the flanges from yielding before reaching maximum resistance. When concentrated loads are used in testing, there are normally high bending moments in the sections where the shear resistance is to be reached, and some interaction with bending is to be expected. The prediction of the vertical shear resistance by the strut and tie models has proved to be satisfactory, however, although the models developed for the shear resistance consider the bending effects only in the case of fire design.

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