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## **A new ultra long-spanning, combined steel formwork and reinforcement hybrid decking system using cold-formed components**

**Mathias Gläsele<sup>1)</sup>, Mark Patrick<sup>2)</sup>, Ross Grey<sup>3)</sup>**

### **Abstract**

A new ultra long-spanning, combined steel formwork and reinforcement system designed to span up to 6 or 7 metres (20.7 or 23 feet) without propping has been under development for nearly a decade in Australia. The steel decking system includes a number of innovative features that make it adaptable to almost any type of building construction ranging from conventional, shallow one-way composite slabs in steel-frame buildings to deep two-way post-tensioned slabs in concrete-frame buildings. The hybrid system comprises a number of distinctly-different cold-formed steel components, which have efficient shapes and utilize different thicknesses, steel grades and coatings. The overall height of the main decking panel can vary from 90 to 260 mm (3.54 to 10.24 in) with an additional three intermediate sizes, despite the components being made from the same roll-forming equipment. The panel has a closed cellular section and is precambered to limit the final deflection under wet-concrete conditions. These panels can be interconnected for maximum spanning capability, or else different types of infill panels can be fitted between them to form composite slabs with a variety of cross-sectional shapes and features. Slabs of minimum overall depth or minimum weight can be constructed using the system. The cellular section can form a void in the concrete, or it can have holes pre-punched in its webs allowing it to be filled with concrete either partially or completely along its length. Prestressing cables and reinforcing bars can be fitted longitudinally and if need be transversely in the slab, even in the soffit region. Comprehensive sets of structural tests, some quite innovative, are currently being performed to

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thoroughly investigate the behaviour of the system for both the formwork and composite states. In particular, this involves developing a complete understanding of the complex behaviour of the main decking panel and its different cold-formed components under a variety of loading conditions that induce vertical and longitudinal shear, longitudinal and transverse bending, and torsion. The behaviour of the mechanical connection between the elements is being closely investigated experimentally, allowing the moment capacity of potentially critical cross-sections with partial shear connection to be accurately predicted and the spacing between connections optimised.

## **1 Introduction**

Cold-formed steel decks have been part of floor systems in buildings for over 70 years. In the early 1930's, the H. H. Robertson Co. laid the first cellular steel floor, called "keystone beam", at a warehouse in Pittsburgh, USA (Dallaire, 1971). This was the "keystone" of the development of structural steel decking. In about 1950, the first composite floor appeared, with the steel decking connected to the concrete by a grid of wires welded to the decking surface (Dallaire, 1971). Today, composite slabs incorporating cold-formed steel decking are the preferred type of floor in steel-frame buildings in most developed countries. Unpropped composite decks have become increasingly popular to promote rapid construction, but maximum spans of U.S. decks, for example, have for many years only been about 4.57 m (15 ft) and typically only in the range of 2.44 to 3.05 m (8 to 10 ft) (Widjaja and Easterling, 2000). Nevertheless, research in the area of long-span slabs has been carried out, resulting in some new long-span flooring systems (Ramsden and Segerlind, 1986), (Lawson and Mullet, 1993) and (Hillman and Murray, 1994). In 1999, an innovative (patented) long-spanning steel decking system, called TRUSSDEK<sup>®</sup>, was first released into the Australian construction market. In its initial form it was a lightweight, permanent, combined steel formwork and reinforcement system that allowed spans in excess of 5.5 m (18 ft) in the formwork stage without propping (Patrick, 2002). Reinforcing trusses are welded to the pans of high-tensile, galvanized profiled steel sheeting (Bondek II), which further enhanced the mechanical interlock between the decking and the concrete, ensuring that the combined truss and decking system acted as fully-effective reinforcement in the final slab or beam at critical cross-sections under maximum positive bending moment.

An entirely new version of the TRUSSDEK<sup>®</sup> system has been developed (TRUSSDEK II) and is protected by a separate patent. The hybrid steel decking includes a number of innovative features that makes it adaptable to almost any

type of building construction ranging from conventional, shallow one-way composite slabs in steel-frame buildings to deep two-way post-tensioned slabs in concrete-frame buildings. It is designed to span up to 6 to 7 metres (20.7 to 23 feet) without propping, and makes it possible to form a variety of innovative solutions for each application being considered. By increasing the span of the decking, significant savings in construction cost and time can be expected. During the construction stage, the need for temporary falsework can be reduced or eliminated. Also the number of permanent supporting elements, such as beams or walls, can be reduced, which creates outstanding open-plan designs. A broad comparison is made in Figure 1 between TRUSSDEK II and some other prominent composite steel decks available in the United States, Australia and Europe, regarding overall slab thickness  $d$  versus maximum span  $L$  in a single, unpropped span. Data, based on the companies' own publications, were used to define each product's performance. The nominal base metal thicknesses (in mm) to form the upper and lower boundaries are shown in brackets underneath the product name. The following composite steel decks were examined: Condeck HP (Stramit Corporation Pty. Ltd., 2004), P-2432 (Canam Steel Corporation, 2004), CFD3 (Consolidated Systems Inc., 2004), ComFlor 100 and SD225, mainly used in the Slimdek<sup>®</sup> system (Corus - Panels and Profiles, 2004). Reinforced and prestressed concrete slabs are normally restricted to certain slenderness ratios, which form the outer borders for economy ( $L/d = 15$ ) and deflection control ( $L/d = 40$ ) within which TRUSSDEK II is normally used. TRUSSDEK II has been used in slabs with spans as short as 2.5 m (8.2 ft), while the longest panel length that can currently be made is 9.0 m (29.5 ft) (which would require propping, however).

## 2 Ultra long-spanning hybrid steel decking system

The ultra long-spanning hybrid steel decking system (TRUSSDEK II) is based on a modular approach, consisting of a number of complex cold-formed elements, which take on optimal shapes, thicknesses, steel grades and coatings. This approach gives the hybrid system great versatility compared to normal trapezoidal-shaped W-decks produced with a constant bare metal thickness around their perimeter. In the case of TRUSSDEK II, additional material can be efficiently concentrated in the flanges to strengthen and stiffen the panels to achieve longer spans during the formwork stage. The main decking panel comprises three basic components, viz.: a stiffened base panel with over-lapping joints; vertically-corrugated webs with or without holes; and a stiffened top plate. There are also diaphragms. The components are mechanically connected together after they have been roll-formed to produce a closed cellular section,

which can take on different heights  $h$  from 90 to 260 mm (3.54 to 10.24 in). The mechanical connection between the elements is established by press-joining. A typical section of the assembled main decking panel is shown in Figure 2.

## 2.1 Base panel

The base panel is specially designed to suit the unique demands of the system and is made of thin cold-formed high-tensile (500 to 550 MPa; 72.5 to 79.8 ksi) galvanized steel that provides significant tensile or compressive capacity acting as longitudinal reinforcement in the composite slab. The longitudinal stiffeners in the pan serve a number of purposes, viz: (i) they reduce the degree of waviness of the flat surfaces, known as oil canning, which can otherwise be the source of large imperfections and amplify local buckles as well as being a problem aesthetically; (ii) they improve the compressive capacity of the panel in negative moment regions; (iii) their outer edges are used to locate the webs; and (iv) they make the panel compact in negative or positive bending (noting that the panel can be used by itself as an infill element). The narrow, re-entrant open steel lapping ribs are only 20 mm (0.79 in) high, so unlike profiles with deep ribs, they do not impact on the behaviour of the slab in the transverse direction. The passage of reinforcing bars or prestressing cables in the transverse direction to provide two-way action in a solid slab is then possible. The base panel has a constant nominal width  $b = 250$  mm (9.84 in) for all section heights  $h$  and is available in different base metal thicknesses up to 1.2 mm (0.047 in). Prominent embossments on top of the steel ribs provide extra longitudinal shear resistance in the composite slab, which supplements the resistance provided by the mechanical connections along the pan due to grip of the webs in the concrete.

## 2.2 Top plate

The top plate is supplied with either one or two, deep longitudinal intermediate stiffeners, depending on the height of the panel, and a simple lip stiffener along each longitudinal edge to increase its compressive capacity, and also to allow the mechanical connections to possibly be more widely spaced. Embossments enhance the shear connection between the concrete and the top plate. The 350 MPa (50.7 ksi) plate is available in different base metal thicknesses (1.0 to 3.5 mm; 0.039 to 0.138 in) and produced in three widths up to 200 mm (7.87 in) such that each width is used for a pair of sequential section heights, i.e. 90 and 110 mm, 140 and 160 mm, and 210 and 260 mm (3.54 and 4.33 in, 5.51 and 6.30 in, and 8.27 and 10.24 in), where the higher of each pair is the more critical design case due to the greater length of outstand beyond the webs.

Finite strip analysis (THIN-WALL, 1994), supported by test results, has been used to find suitable geometries for the top plate in compression with the known

boundary conditions. The model incorporates the upper web flanges, which are mechanically connected to the top plate. The junction between the webs and their upper flanges are idealized as fixed supports, with the web flanges assumed to terminate at the connections with a simple joint. To account for the longitudinal spacing of the connections, an effective thickness of the web flanges,  $t_{\text{eff}}$ , has been calculated which takes into account the effective length of the web flange in the longitudinal direction of the panel when the flange is bent restraining the top plate from buckling. Based on test results, the following equation has been derived to calculate the effective thickness  $t_{\text{eff}}$  of the web flanges:

$$t_{\text{eff}} = \frac{4.121 \ t}{\sqrt[3]{s}} \quad (1)$$

where  $t$  is the base metal thickness of the web and  $s$  is the longitudinal spacing between the connections. The results of a finite strip analysis are shown in Figure 3 for a 200 mm (7.87 in) wide top plate with a base metal thickness of only 1 mm (0.039 in) and a web thickness of 0.7 mm (0.028 in). Three fundamental buckling modes can be identified, viz.: local (a); flange distortional (b); and overall distortional (c). It can be seen, that the buckling stress is over 350 MPa (50.7 ksi), the nominal yield stress of the top flange steel, up to a buckling half wavelength of 350 mm (13.78 in).

### 2.3 Web

A pair of webs is mechanically connected to the base panel and the top plate to form the cellular section. Grade 350 MPa (50.7 ksi) steel was chosen for economics, and also because the steel has high formability to make the corrugations. Holes, 90 mm long by 40 mm high (3.54 by 1.57 in), can be punched in the webs as close as at 190 mm (7.48 in) centres. This allows the cellular section to optionally be filled with concrete making a solid composite slab, and to possibly fit reinforcing bars or prestressing cables in ducts transversally through the holes close to the soffit of the slab. To improve the flow of concrete into the voids, the holes may be staggered on each side of the trusses when bars or cables are not to be passed through. Alternatively, the cellular section can be left closed to form a void in a one-way composite slab. The web is inclined at a fixed angle of 74 degrees to the horizontal irrespective of the height of the panel. Like the base panel and the top plate, the web is available in different base metal thicknesses, typically 0.7 or 0.9 mm (0.028 or 0.035 in). Thin-gauge diaphragms are essential at the ends of the panels where they fall on supports. These are needed to prevent premature buckling of the webs particularly in the lower portions where the support reactions are concentrated. Additional connections are also required in these regions to resist

transverse shear forces that act through the connection between the webs and the base panel. Vertical corrugations with a pitch of 35 mm (1.38 in) are pressed into the webs to increase their shear capacity and flexural stiffness. Significant shear distortion of the webs can occur at ultimate load if they are holed, but tests have shown that very high shear forces can be carried. The inner corners of the channel-shaped webs are regularly crimped, which increases the flexural stiffness of the web flanges by reducing the cantilever length, which in turn helps resist overall buckling of the top plate. A final feature of the webs is that they can be produced with a curve, which means that the main decking panel can be upwardly precambered a variable amount, this being an essential feature of a long-spanning formwork system to minimise self-weight.

## 2.4 Mechanical connections

The mechanical connections between the components of the main decking panel are formed by press-joining, also referred to as clinching. Press-joining is a relatively old technique, the first patent being granted as early as 1897 (Thies, 1897). However, its potential was not fully appreciated until the 1980's, and it is now widely used in the automotive and white goods industries. The advantages of press-joining compared to traditional screwing or welding are now well recognized in the building industry, especially in steel-frame housing. A simple punch presses the materials to be joined into the die cavity. As the force continues to increase, the punch side material is forced to spread outwards within the die side material (TOX Pressotechnik, 2002). Some of the main advantages of press-joining are that:

- consumables such as bolts or rivets are not needed;
- pre-processing work such as drilling or surface cleaning is not needed;
- protective coatings such as galvanising and paint can be present and are undamaged;
- the connections can be formed rapidly, taking only seconds to form a joint and move to the next position;
- multiple connections can be made simultaneously (TRUSSDEK II is made with four at a time); and
- it is environmentally friendly with no fumes.

Although a significant amount of research has been undertaken into the strength and behaviour of press-joints, little has been published relating to the use of high strength structural steels (Liebig and Bober, 1986), in particular for round joints (Varis, 2002). Therefore testing of the joints is required and a series of shear tests with various combinations of base metal thickness and steel grade has already been performed. Some typical results from the tests are shown in Figure

4, where the punch and die side material is indicated with a P and D, respectively.

## **2.5 Features**

The TRUSSDEK II system comprises some innovative features that make it adaptable to many different applications.

The main hybrid decking panels can be pre-cambered to allow for deflection during the formwork stage, immediately after the concrete is poured before composite action develops between the steel and the concrete. This feature allows good visual appearance and general alignment of the slab soffit to be achieved. This is especially important when the residual deflections would otherwise be large, for example in long spans. Considerable savings can be achieved in the amount of steel used in the panels, and by a voiding excess, ponded concrete.

By using internal formers, the cellular sections can be partially filled with concrete, such as at their ends where welded studs can be fixed or over internal walls, where a solid concrete slab is advantageous for a variety of reasons. As already mentioned, the voids can also be completely filled with concrete, to make a solid composite slab throughout.

Prestressing cables can be fitted longitudinally between the main decking panels to achieve very long spans of the finished composite slabs by reducing the immediate vertical deflection if propping is used.

Infill panels of various shapes can be fitted alternately between decking panels. Currently the infill panel is very similar in shape to the base panel except that the lap joints have been modified to allow the panel to simply "hang" between two cellular panels. It can also include a snap-in voided unit (see Figure 5). The voids are non-structural and serve to reduce the volume of concrete and weight of the floor. By non-structural it is meant that the voids are not longitudinally connected to the infill panels and therefore do not increase the flexural stiffness or strength of the panels. Voiding units can be deep resemble a deep W-deck, or very shallow (in which case it is called a "pan plate") such that they do not interfere with reinforcing bars or prestressing cables but transfer the weight of the wet concrete to the lapping ribs which avoids excessive transverse deflection of the infill panels.

## **3 Test program**

Comprehensive series of innovative tests are currently being conducted to thoroughly investigate the behaviour of the system in both the formwork and composite states. This will give rise to a complete understanding of the complex



behaviour of the main decking panel and its different cold-formed components. The behaviour of the mechanical connection between the components is being closely studied experimentally, allowing the moment capacity of potentially critical cross-sections with partial shear connection to be accurately predicted. Full-scale flexural and/or shear tests on single and double span specimens are being performed to investigate the behaviour in positive and negative bending, and a variety of failure modes have already been identified. Detailed test results and design methods will be published in the future. As an example of an innovative loading system, a well-articulated loading frame has been built to conduct tests under distributed loading that accurately determine the flexural stiffness and the strength of the hybrid steel deck. The load is introduced into sets of loading fingers via crossbeams and then transmitted onto the top side of the base panel through loading brackets. A roller bearing at the end of each loading finger minimises the longitudinal frictional restraint that can develop. Several of the test series currently being undertaken are briefly described below, noting that a number of other types of tests will be performed.

### 3.1 Flexural tests

Single span tests under distributed loading (see Figure 6) are used to gain essential information about the behaviour of the main decking panel in positive bending, when the base panel is in tension and the top plate in compression. The principal mode of failure depends on the degree of longitudinal shear connection existing between the components, i.e. between the corrugated web and the top plate, or the web and the base panel. A state of complete shear connection can be assumed to have existed if a bending failure occurs at a critical cross-section and its moment capacity was not governed by the strength of the longitudinal shear connection. This condition can be achieved when the number of connections  $n$  between the end of the panel and the critical cross-section equals or exceeds the number required for complete shear connection, i.e.  $n \geq n_c$ . The strength of the shear connection between the top plate and the webs, and the webs and the bottom panel, must be determined separately to see which governs. In this case, either the base panel yields and possibly fractures in tension or the top plate yields or buckles in compression. If the nominal moment capacity is governed by the strength of the connections, the critical cross-section can be deemed to exhibit partial shear connection, i.e.  $n < n_c$ . Depending on the spacing and strength of the mechanical connections, the material properties and thicknesses of the top plate, web and base panel, different failure modes can occur and different degrees of shear connection can be exhibited. The connections between the base panel and the webs normally fail well before the top connections, because they are normally weaker with the thin base panel

sheeting (see Figure 7) and also, because of the equipment used, equal numbers of connections are made in the top and the bottom of the main decking panel. If the connections in the top plate fail in the mid-span region by pull-out, the top plate can buckle distortionally before yielding. In this case the longitudinal spacing of the connections becomes critical. To improve manufacturing productivity, the number and distribution of the connections must be optimised and can be programmed for production.

During the experimental studies, careful attention will be given to the behaviour of the cold-formed components and their connections in transverse bending. Prying can occur in the connections, for example when the webs are holed and the wet concrete fills the cellular deck. In particular, the weight of concrete on the base panel can cause significant transverse bending within the closed section and relatively large bending moments to occur at the connections. Prying causes additional tensile forces to develop in the connectors. On the other hand, the transverse deflection of the base panel is greatly reduced due to the rotational stiffness of the web, which is even further enhanced by the crimps located at the junction of the web with its lower flange. Calculations have shown that the additional tensile forces on the connections are normally relatively small. Other forces can also act on the connections, e.g. hanging forces from the infill panel. A detailed investigation of these effects is being undertaken, with the connections being systematically subjected to different combinations of tension and shear in a purpose-built test rig.

### **3.2 Shear tests**

A series of vertical shear tests, including end shear tests, is currently being undertaken to investigate the shear behaviour of the hybrid steel deck. The web is stiffened by vertical corrugations to increase the shear capacity. Shear distortion of the webs occurs particularly if the webs are holed (see Figure 8). Specimens with the unreinforced, rectangular holes are being studied with single and multiple holes located within the shear span. Vierendeel action over the length of the penetration takes place and causes additional secondary bending moments to develop in the upper and lower parts of the webs across the penetration, followed by local buckling at diagonally opposite corners of the hole. Over supports where there are concentrated reaction forces, the web can fail due to out-of-plane buckling or a combined shear/pull-out failure of the mechanical connections between the web and the base panel. Diaphragms located over the support region largely prevent web buckling and significantly increases the load-carrying capacity of the panel. An investigation is being made into the mechanisms by which shear is transferred in the end regions, with a view to developing open-ended types of diaphragms that will allow access

through the ends of the panels for fitting services as well as the flow of concrete when required. The effects of varying the degree of shear connection of critical sections under combined bending and shear are also being studied.

## 4 Conclusions

Some details about a new ultra long-spanning hybrid decking system developed in Australia have been presented. The main decking panel comprises three basic components, viz. a base panel, a pair of vertically-corrugated webs and a top plate, all mechanically connected to form a very stiff and strong, closed cellular section. The system has numerous innovative features such as: pre-cambering for deflection control; pre-punched holes in webs to allow the cellular section to be filled with concrete and reinforcement to be fitted transversally through the webs; infill panels of various shapes to reduce the volume of concrete and weight of the floor as well as improve economy. Comprehensive sets of full-scale structural tests are currently being performed to investigate the many different types of behaviour that can be exhibited. The experimental work is being supported by analytical studies including finite-element modelling, and will allow the development of simplified behavioural models and design rules.

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## Appendix – Notation

$b$	=	Nominal width of base panel
$d$	=	Overall slab thickness
$h$	=	Nominal height of hybrid deck
$L$	=	Maximum unpropped single span
$s$	=	Nominal spacing between the connections
$t$	=	Base metal thickness of the web
$t_{eff}$	=	Effective thickness of the web flanges

## Appendix– Figures

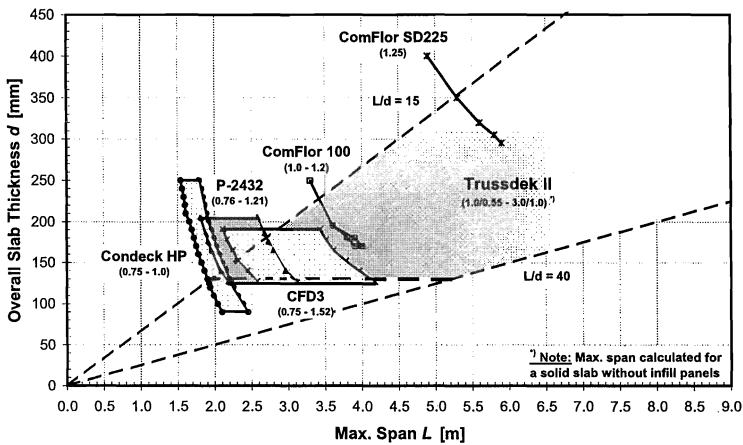
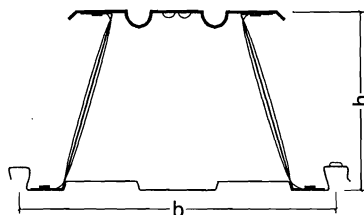
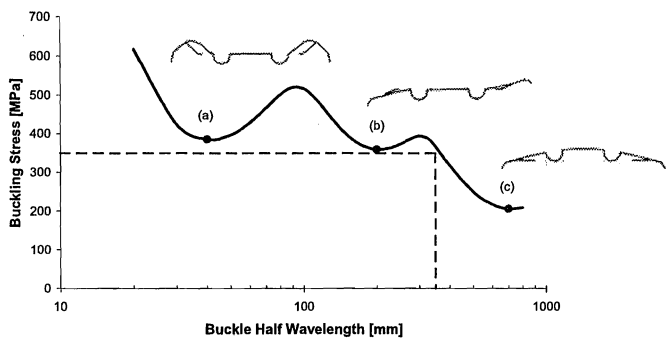


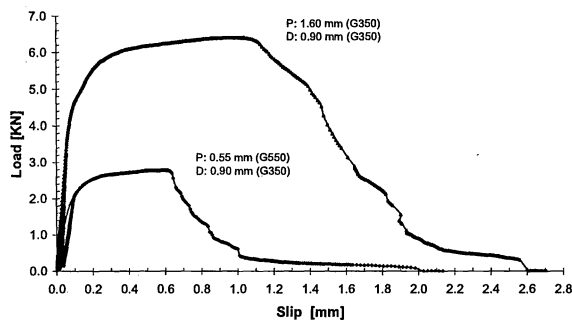
Figure 1 Range of application of different composite steel decks



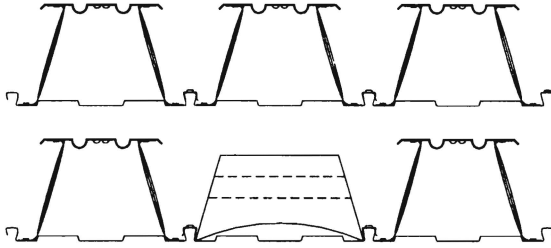
**Figure 2** Typical cross-section of main TRUSSDEK II decking panel (note: only one longitudinal stiffener fits in the top plate in the highest sections)



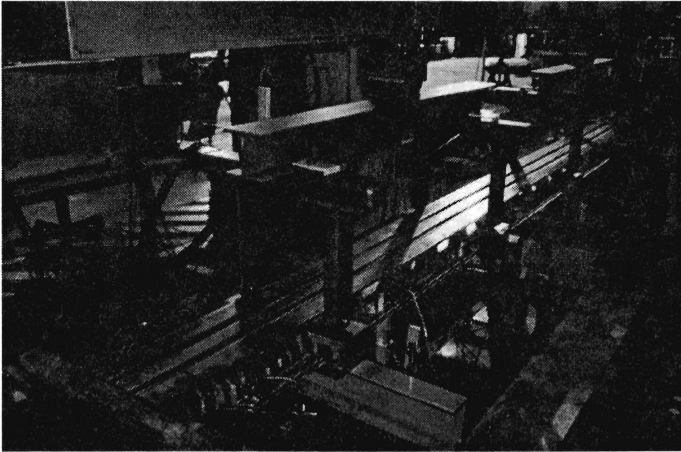
**Figure 3** Typical buckling analysis of top plate



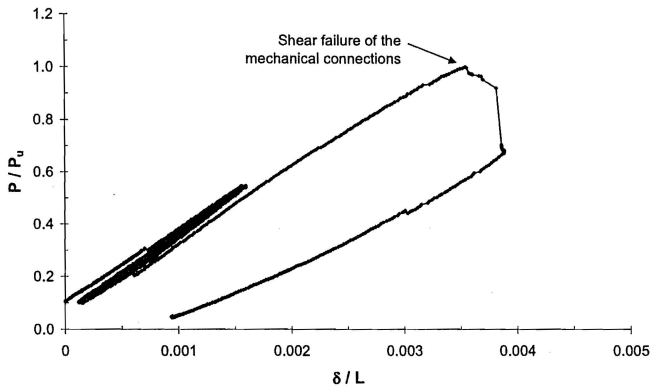
**Figure 4** Typical deformation behaviour of mechanical connections



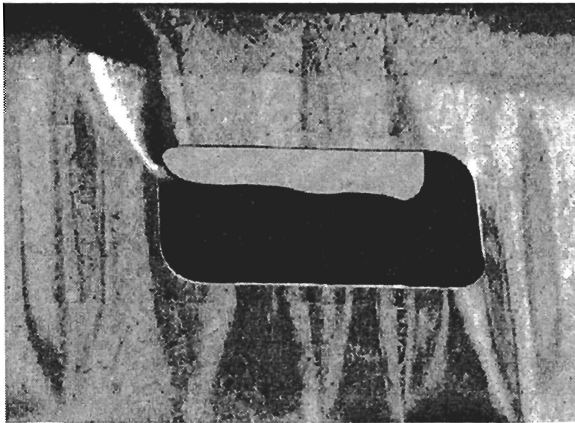
**Figure 5** TRUSSDEK II without (top) and with (bottom) infill panel using different snap-in voiding units



**Figure 6** Single-span test under simulated distributed loading



**Figure 7** Result from a single-span test under simulated distributed loading



**Figure 8** Shear distortion of the holed web