Development of a new long span composite floor system

Synopsis

This paper presents the results from the testing of a prototype for a new long-span composite floor system known as Tekdek. The tested prototype achieved a clear unpropped span of 10.5m, which is significantly greater than any other comparable composite system currently on the market. The system introduces a new concept of prestressing the steel deck prior to pouring the concrete. The prototype deck was tested to investigate its ultimate compressive prestressing capacity and its performance under pouring of the concrete to form the completed composite system. In addition, the constructed composite floor system was tested under vibration, cyclic and ultimate load. The performance when pouring the concrete, and the tests on the composite system, were repeated for two different decks (designated A and B) which were subjected to different prestressing loads. It was found that the composite system, based on a one unit width, had a fundamental frequency between 4.4 and 4.5Hz and achieved a maximum load carrying capacity between 9.0 and 9.87kN/m², which is acceptable for most buildings. An indicative fire test provided the temperature distribution through the cross-section, when subjected to the standard fire curve, allowing the fire resistance of the system to be estimated.

C. G. Bailey
BEng, PhD, CEng, FICE, MIStructE, MIFireE
School of Mechanical, Aerospace and Civil Engineering, The University of Manchester

P. M. Currie
BEng, PhD
School of Mechanical, Aerospace and Civil Engineering, The University of Manchester

F. R. Miller
BEng, CEng, MIStructE
Offshield Ltd, Wilmslow, Cheshire

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Introduction

TekDek, a new floor system invented by Ron Miller (Patent No. GB 2004/001949), introduces a totally new revolutionary approach to a metal decking flooring system. By prestressing the metal deck, prior to construction on site, unpropped spans in excess of 10.5m can be readily achieved, with a 225mm deep deck. Due to greater spans, compared to current comparable floor systems, the number of required secondary beams is significantly reduced saving costs on material, fabrication, fixing on site and fire protection. The steel deck is supported within the depth of the steel beams (Fig 1) keeping the overall construction thickness of the floor to a minimum, resulting in the possibility of extra floors within a given building height and possible savings on cladding costs.

Prior to delivery to site, a compressive prestressing force is applied to the metal deck. By curving the prestressed reinforcing tendon over the span, the capacity of the deck during construction and the final composite system is fully utilised. The basic components of the decking units are shown in Fig 2. The system consists of a cold-formed deck profile, manufactured using thin (1.25mm) gauge steel, which is prestressed via a single steel tendon and two end brackets transferring the prestressing load from the tendon to the deck. Due to the forced curvature of the tendon, which has a greater axis distance to the bottom of the deck at the supports compared to that at the centre, the applied prestressing force produces a camber to the steel deck prior to placing the concrete. The end brackets also serve to connect the units to the supporting steelwork. The floor system can be connected to the supporting steel beams by placing the end brackets in pre fabricated slots to the top flange of the beam and securing them via wedges (Fig 1), which also provides an effective composite union with the supporting beam. Alternatively, the end brackets can be shaped such that they can be shot fired to the top flange of the beam (Fig 2) and then shear studs added to enable composite action between the beam and supported floor slab. The units span one-way between steel beams with side-by-side units interlocking through a free edge joint, rather like a tongue and groove timber plank. Joints are then locked together with simple wire spring clips as shown in Fig 3, forming a cambered continuous permanent shutter on which *in situ* concrete is poured. The end brackets provide lateral and torsional restraint to the beam. A circular void is created in the deck, ranging from 110mm to 160mm diameter, as shown in Fig 3. The void can be used for routing services, or for heating or cooling the floor system. The reinforcing tendon and steel deck provide the required tensile resistance under normal loading, with the reinforcing tendon also acting under any possible fire load. Due to the small axis distance, between the reinforcing tendon and metal deck at mid-span, fire protection material is intro-

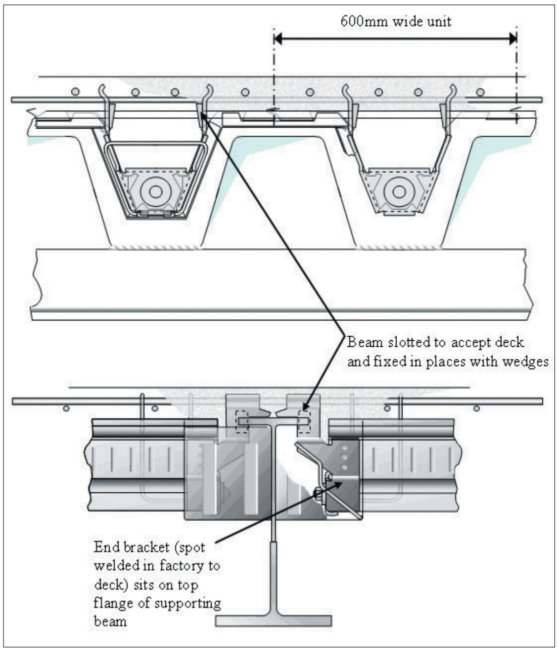


Fig 1. (above left) Proposed fixing of prestressed deck to top flange of supporting beam

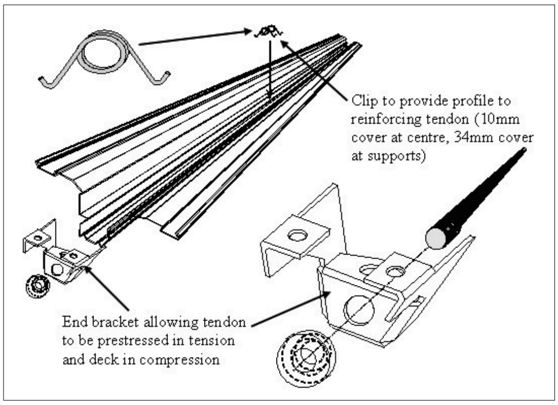


Fig 2. Deck, reinforcing tendon and end bracket

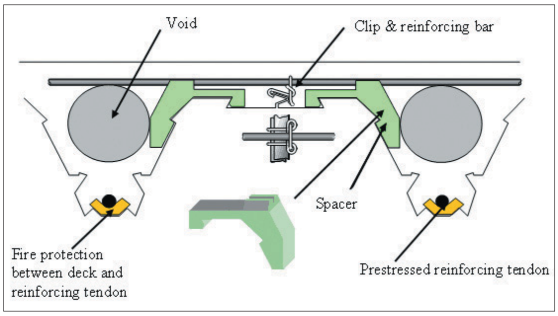


Fig 3. Cross-section of deck details

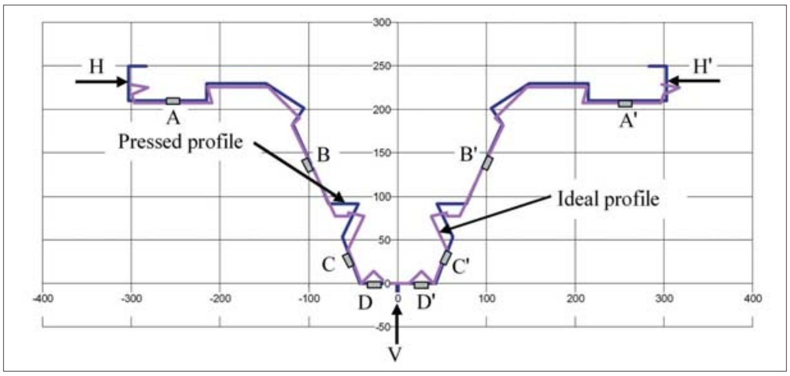


Fig 4. Comparison of pressed and ideal profiles (A, B, C, D, A', B', C' and D' are strain gauge locations, H, H' and V are displacement measurement locations)

duced between the tendon and deck, as shown in Fig 3, to achieve the necessary fire resistance. Away from mid-span, the axis distance increases sufficiently such that there is no need for the fire protection material.

Prototype testing

A testing programme was developed, at the University of Manchester, to investigate the principles of the proposed system. The ideal profile for the original 225mm deep, 600mm wide, TekDek design, comprising 1.25mm thick steel plate, is shown in Fig 4, which theoretically can achieve clear spans of over 10.5m. To obtain the ideal profile, the deck would need to be rolled. However, due to financial constraints it was impossible to roll the prototype at this stage of development. It was therefore decided that the deck would need to be press-formed. However, this raised two problems. The first was that there was not a press large enough within the UK to form a 10.5m length of deck. The second problem was the shape itself, which due to the number of acute angles, was not possible to press. Therefore the deck had to be pressed in six parts (two per cross-section and three lengths of 3.25, 4.0 and 3.25m) and welded together. In addition, the shape of the deck had to be modified, as shown in Fig 4, to allow the deck to be physically pressed. The edge lips of the prototype deck were increased slightly, as shown in Fig 4, to help with laterally restraining the deck during pouring of the concrete. The compromise of changing the deck's shape reduced its efficiency and the need to weld six parts of deck together induced imperfections, which due to the fact that the deck had to be prestressed, where not ideal. However, the prototype deck did allow the basic concept of the TekDek system to be investigated and demonstrated. The pressed prototype deck had a measured steel thickness of 1.2mm (instead of the ideal 1.25mm) and a measured yield strength ranging from 280N/mm^2 to 320N/mm^2.

In order to assess the overall performance of the prototype, and determine the accuracy of the analysis, six different tests were performed. In Test 2, two separate decks were cast (designated A and B) which were subsequently used in Tests 3, 4 and 5. The testing programme comprised:

Test 1: an ultimate prestressing test.
Test 2: two wet concrete pour tests (Decks A and B).
Test 3: two frequency tests (Decks A and B).
Test 4: two 5000 cyclic load tests (Decks A and B).
Test 5: two ultimate load tests (Decks A and B).
Test 6: an indicative fire test.

Test 1: Ultimate prestress test

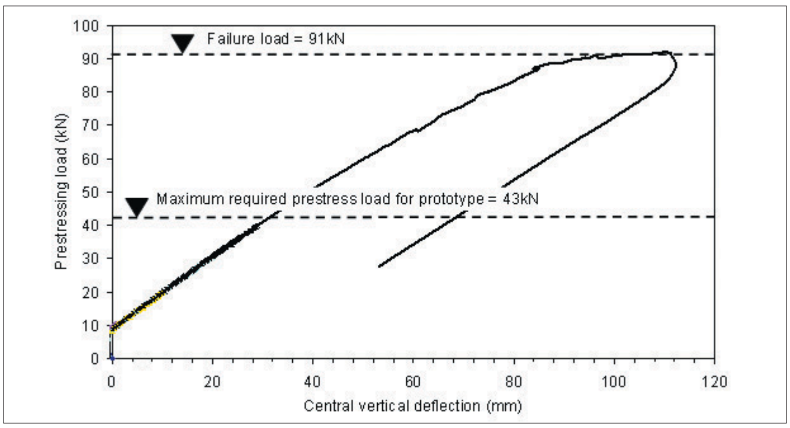
Following the theoretical concept of TekDek, the metal deck will eventually be prestressed in the factory and delivered to site as a unit. The first test on the prototype deck consisted of investigating the ultimate compressive prestress axial force. A number of steps had to be performed to prepare the deck for the first test. These included the manufacturing and fitting of the end brackets/shoes, the prestressing/reinforcing tendon, and the central tie clip, to mimic the final units as shown in Fig 2.

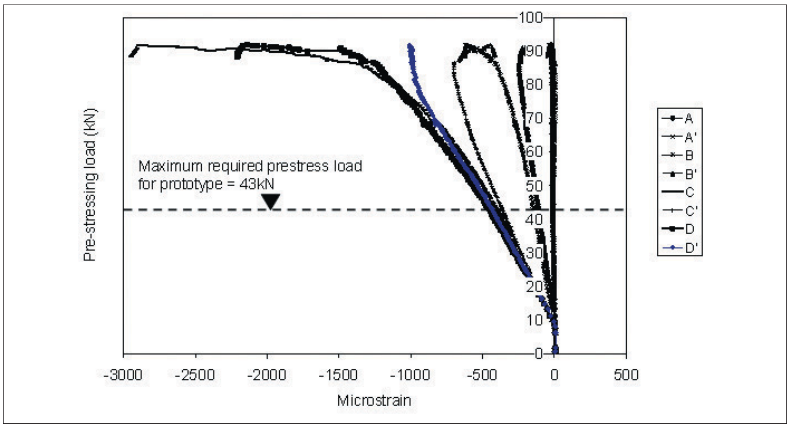
The end shoes, which were manufactured from 12mm plate and welded to either end of the deck, served the purpose of transferring the prestressing load from the tendon into the deck (Fig 5). The prestressing tendon consisted of a standard 20mm diameter ribbed reinforcing bar (measured yield strength of 525N/mm^2) cut to length and threaded at either end to enable locking nuts and a prestressing jack to be used. To represent the central tie clip, as shown in Fig 2, a 50mm × 4mm thick bar was bent around the diameter of the tendon and passed through two slots machined in the deck either side of the welded joint. The two elements of the clip passing through the slots were then bolted in place, with the tendon positioned at the correct location using a 10mm spacer placed between the tendon and the deck. The deck was supported on knife-edges creating a clear span of 10.5m with no lateral restraint provided to the deck. The test set-up represented the conditions in a factory where the units will be prestressed in the future, prior to delivery to site.

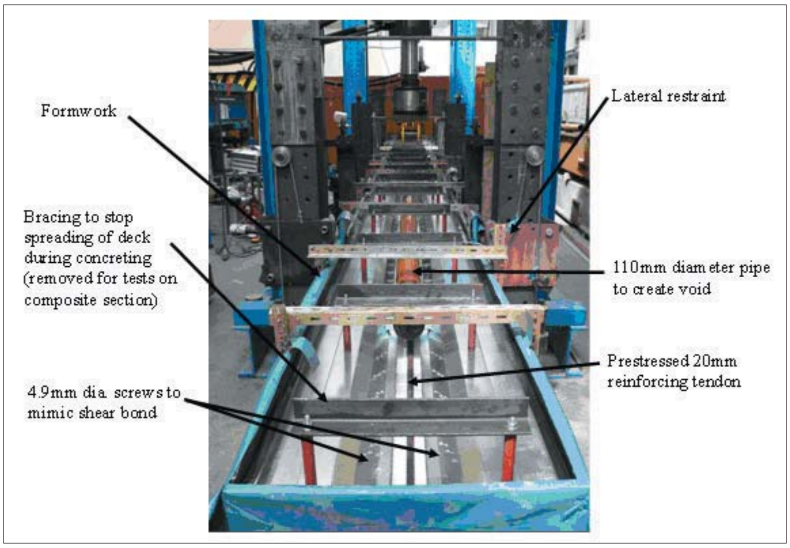
The deck was monitored using strain gauges and displacement transducers, with the prestressing load applied to the tendon monitored using a 250kN load-cell. A total of 40 strain gauges were positioned on the outside surface of the deck (as shown in Fig 4) at mid-span, and at third points along the deck. With reference to Fig 4, gauges A and A' at $^1/_3$, mid, and $^2/_3$ span, and gauges D and D' at mid-span, were all uni-axial strain gauges orientated longitudinally along the deck. All other gauges on the deck were bi-axial gauges with strains

Fig 5. Failure of deck under prestressed load

Fig 6. (below) Prestressing/deflection response of prototype deck (deflection upwards)







measured both longitudinally along and transversely across the deck. In addition, the reinforcing tendon was also strain gauged top and bottom at mid-span and third-points using uni-axial gauges orientated longitudinally along the tendon. The vertical and horizontal displacements were measured at mid-span and quarter points, at the locations shown in Fig 4.

The prestressing load was applied to the reinforcing tendon and deck using a standard prestressing setup as shown in Fig 5. The deck was loaded in several steps. Each step involved gradually increasing the prestressing load to a specified load before gradually releasing the load. The cyclic loading steps were 5kN, 10kN, 20kN, 30kN and 40kN. Following these steps the deck was then tested to failure. The maximum load obtained was 91kN, which is significantly higher than the 43kN prestressed load required for the prototype. A photograph of the test specimen showing buckling failure is shown in Fig 5.

Fig 6 shows the load-displacement response of the deck, with Fig 7 showing the recorded strains in the deck at mid-span. The upward displacement of the deck reached a maximum of 111mm at the failure load of 91kN, and a displacement of 32mm at the required design prestress of 43kN. The strains at a prestress of 43kN did not exceed 469 microstrain (Fig 7), equating to a stress of 99 N/mm^2. The test showed that the required prestress of the prototype deck had a factor of safety against failure of 2.1.

Test 2: Casting of specimen Decks A and B

Two separate decks (designated A and B) were cast in order to monitor the prestressing, casting, and curing phases, and to provide test specimens for the subsequent frequency, cyclic, and ultimate load tests. Different prestressing loads were used in Decks A and B. The initial preparation of the decks was similar to that of the ultimate prestressing test with the

Fig 7. (top) Prestressing/strain response of prototype deck
Fig 8. (above) Deck prior to casting concrete

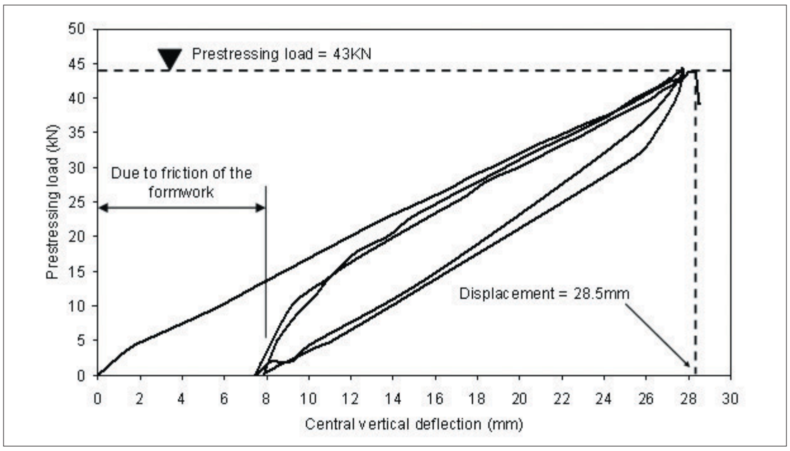


Fig 9. Prestress-load relationship for Deck A

end shoes, prestressing tendon and central clip manufactured in the laboratory. To represent the void a plastic pipe of 110mm diameter was used together with spacers and tying wire. Since the deck was pressed, no indentations or embossments where included to enhance the bond between the deck and concrete. To mimic the bond, which will be present in a rolled deck, self-tapping screws (4.9 × 16mm) at 65mm centres were used. Details of the deck before casting the concrete are shown in Fig 8.

To mimic the behaviour of a continuous floor slab the prototype deck was restrained laterally at five locations. The restraints (Fig 8) were designed to allow the deck to deflect freely in the vertical plane, and also allow the deck to rotate along its length due to bending, whilst restricting lateral movement. The restraints consisted of a rectangular plate with two small angle sections connected in a manner that allowed the angle directly bolted to the deck to rotate freely. Each plate was then attached to a vertical column section using four guide rollers. The guide rollers enabled the plate to run vertically up and down the flange of the vertical column, whilst restricting both lateral deflections and twist. The plates were counterbalanced, as shown in Fig 8, so that no additional weight was added to the deck.

During the concrete placement, a series of cross bracing elements (Fig 8) were also included to prevent the deck from spreading due to the hydrostatic pressure of the wet concrete. In practice the interlocking of the decks will prevent any spreading. To create a level top surface to the slab, horizontal formwork was constructed to provide a minimum depth of 280mm at the supports.

The behaviour of the prototype decks was monitored using a load-cell (for the prestressing), strain gauges and displacement transducers. The decks were strain gauged at similar locations to the deck used in the prestress test (Fig 4). Deck A was only strain gauged at mid-span, whilst Deck B was strain gauged at mid-span and at third points. For Deck A, strain gauges A, A', D and D' were all uni-axial strain gauges orientated longitudinally along the deck. All other gauges on the deck were bi-axial gauges with strains measured both longitudinally along and transversely across the deck. In addition, the reinforcing tendon was also strain gauged top and bottom at mid-span using gauges orientated longitudinally along the tendon. For Deck B, gauges A and A' at $^1/_3$, mid and $^2/_3$ span and gauges D and D' at mid-span were all uni-axial strain gauges orientated longitudinally along the deck. All other gauges on the deck were bi-axial gauges with strains measured both longitudinally along and transversely across the deck. The reinforcing tendon was strain gauged top and bottom at mid-span and third points using gauges orientated longitudinally to the tendon.

Deck A was constructed and supported to give a clear span of 10.5m. With the formwork in place, to allow a level top surface to the completed composite floor to be achieved, it was possible to measure the geometric imperfection of the steel deck prior to prestressing. In this case the geometric imperfection (due to welding) was an upward deflection of 12mm

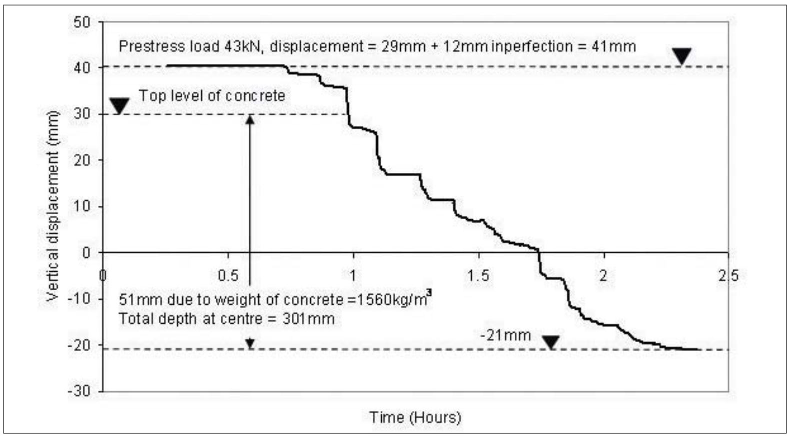


Fig 10. Vertical displacement at mid-span of Deck A during pouring of concrete

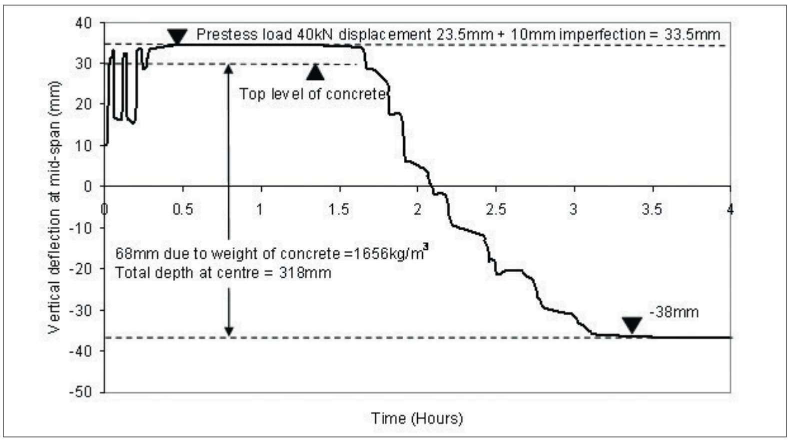


Fig 11. Vertical displacement at mid-span of Deck B during pouring of concrete

at mid-span. Once in place within the formwork and lateral restraints, the deck was prestressed to 43kN. The prestress applied to the deck was cycled at this load a number of times, as shown in Fig 9, with a vertical upward displacement of 28.5mm recorded under the prestress load of 43kN. It can be seen that the deck did not return to zero when the prestress load was removed. By loosening the fixings to the formwork it was found that the deck returned to its original position when the load was removed. This suggests that friction between the formwork and deck was restraining the deck when the load was removed, however under a prestressed load the deck returned to the maximum upward displacement of 28.5mm. Together with the initial 12mm imperfection, the prestress load resulted in an overall upward deflection of 41mm relative to the supports (Fig 10). Similar behaviour was observed for Deck B, except in this case the initial geometrical imperfection was 10mm upwards at mid-span and the prestress load was reduced to 40kN. The measured vertical upward displacement due to the prestress was 23.5mm, which together with the 10mm imperfection resulted in an overall upward displacement of 33.5mm, relative to the supports.

In both tests, lightweight self-compacting concrete was used. The measured wet density for Deck A was 1560kg/m^3 and 1656kg/m^3 for Deck B. The concrete was placed starting from one end of the deck and working towards the other end. The measured vertical displacements at mid-span for Deck A and Deck B, during placing of the concrete, are shown in Figs 10 and 11. With reference to Fig 10, the prestress load of 43kN, together with a 12mm imperfection, gave an upward mid-span deflection, measured from the top surface of Deck A, of 41mm. The concrete was poured such that a level surface was achieved which was a minimum 30mm above the deck measured at the support. The deck at mid-span deflected 51mm, below the required top level surface of the concrete, creating an overall depth at mid-span of 301mm. For Deck B, Fig 11 shows that the depth of the slab, at mid-span, was 318mm after pouring the concrete. The average strains in the prestressed tendon and the strains in the deck, for Deck B, are shown in Fig 12. Fig 13 shows the deck following casting the concrete indicating the different depths at the centre compared to the depth at the supports. The formwork and cross bracing (Fig 8) were removed, as shown in Fig 13, to allow testing of the deck at serviceability and ultimate loads. The five lateral restraints remained in place, except for the impact tests discussed in the next section.

Test 3: Natural frequency

Impact tests were performed on Decks A and B to determine their natural frequencies. For accurate results to be obtained it was necessary to create a clear span by removing all of the lateral restraints attached to the deck. This was necessary to stop any possible damping effects from the restraints. Steel plates were then attached to the top surface of the composite floor deck using a bed of dental plaster at a third point and mid-span. A structural analyser Di-2203 was used to record and analyse the acceleration time-histories and frequency spectra of the prestressed composite system. The structural analyser monitored the deck using a probe attached magnetically to one of the steel plates. A rubber mallet was then used to hit the deck and the acceleration time-histories and frequency spectra were recorded.

From the tests it was found that Deck A had a fundamental frequency of 4.5Hz, with Deck B having a fundamental frequency of 4.4Hz. For a continuous floor plate the fundamental frequency is expected to be higher, and acceptable, for most building types.

Test 4: Cyclic load test

A loading system consisting of two load trees, each attached to its own hydraulic actuator and load cell, was manufactured to enable an eight point loading to be applied to the deck to simulate a uniformly distributed load (Fig 14). To stop

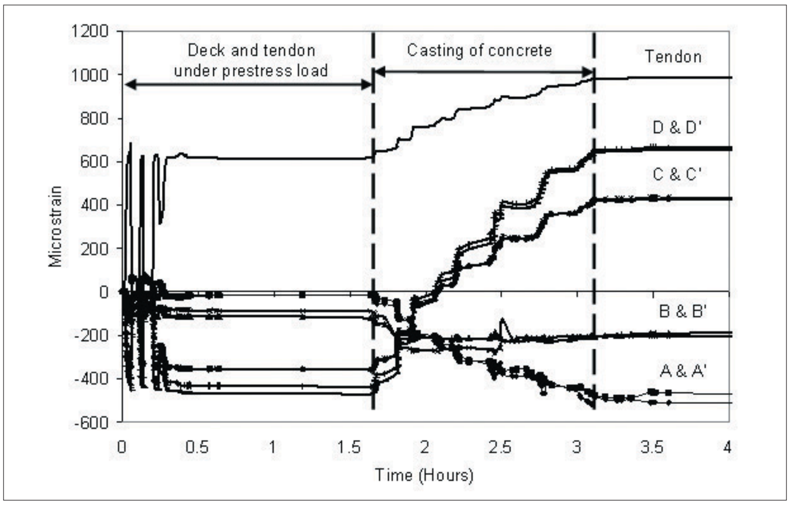


Fig 12. (above) Strains in deck and tendon during prestressing and casting of concrete for Deck B (refer to Fig 4 for location)

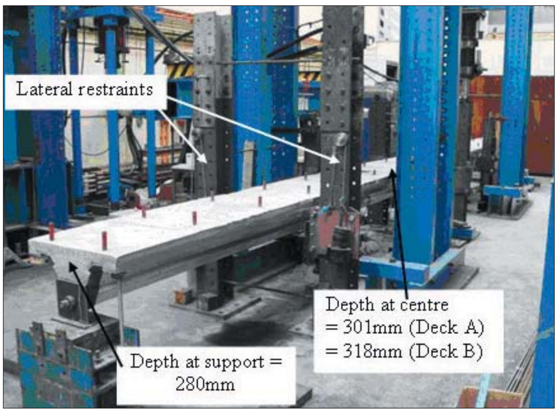


Fig 13. (right) Deck after removal of formwork and cross-bracing

localised crushing on the surface of the deck, plates were bonded to the deck directly under the load points using dental plaster. Strains on both decks were measured at the same locations as discussed for Test 2, and shown in Fig 4.

The cyclic test consisted of loading and unloading the deck between an upper and lower limit of 0.48kN/m^2 and 2.42kN/m^2 a total of 5000 times at a frequency of 0.5Hz to test the bond between the deck and concrete. These loads were the actual loads on the deck as the weight of the load tree was suspended from the load cells, which were then initialised at the start of the test. As it was not possible to accurately monitor the strains and deflections during the load cycling process, the 5000 cycles were split up into five steps of 1000. Before and after each set of cycles, the deck was slowly loaded and unloaded between 0.16kN/m^2 and 2.42kN/m^2 and the strains, deflections and loads were monitored. This enabled any changes in the deck's performance that occurred during the cycling steps to be identified.

During the 3840th cycle of Deck A, there was a loud cracking noise and the deck deflected considerably more than usual, which triggered the deflection sensitive automatic stop on the controller and the load cycling process was halted. Once the load was removed the deck retained a permanent central deflection of 9-10mm. It is thought that the permanent set was due to a break-down in the bond provided by the screws. However, there was no definitive evidence to fully support this theory. The deck was then loaded and unloaded slowly and no serious deterioration in performance was noted. Therefore the cycling process was completed.

When testing Deck B, cracking noises were noticed in the initial loading steps prior to starting the cyclic loading, suggesting again that the bond provided by the screws was breaking down. However, after this initial loading, no further serious deterioration in the deck's performance was noted during the full 5000 cycles. The load-displacement response from the cyclic loading is shown in Fig 15, which shows a permanent vertical displacement of 5mm after the first load cycle due to the bond breaking. Fig 16 shows the strains recorded in the reinforcement, which were initialised at the start of the test. Similar to the displacements, a permanent strain was recorded after the first loading step.

Test 5: Ultimate load test

The aim of the ultimate load test was to check the general overall performance of the prototype deck and determine its maximum load capacity. The load on the deck was increased gradually and the loads, strains and deflections were monitored at each stage. In between each step, the deck was allowed to settle before the load was increased further. The measured cube strengths and age of concrete, on the day of the test, were 45N/mm^2 and 108 days for Deck A, and 38N/mm^2 and 43 days for Deck B.

For Deck A, the load was increased to a maximum of 9kN/m^2. At this load the deck had deflected 337mm (equivalent to span/31) at mid-span. Under this load the deck had not failed or collapsed, however, the hydraulic actuators had reached the limit of their travel. The load was released and then cycled up to the maximum load a couple of times. For Deck B the load on the deck was increased to a maximum of 9.87kN/m^2. As with Deck A, the test was terminated due to the hydraulic actuators reaching the limit of their travel. The load-displacement response for Decks A and B is shown in Fig 17, with the deflection shape of Deck B shown in Fig 18. At a deflection of span/360 (29mm) Deck A supported a load of 2.75kN/m^2 and Deck B supported a load of 2.79kN/m^2.

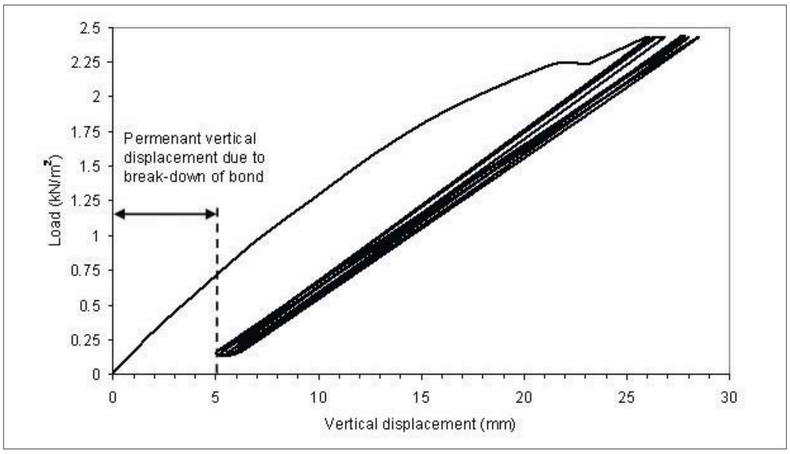
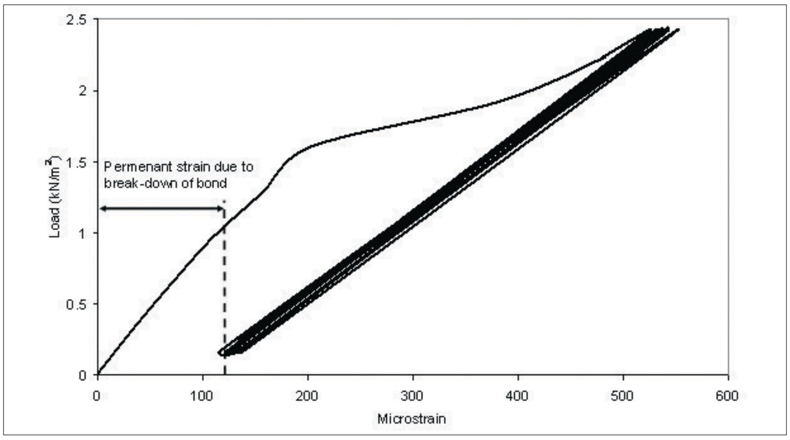


Fig 14. (left) Loading tree for cyclic and ultimate loading
Fig 15. (above) Load-displacement response during cyclic loading (0.48 to 2.42kN/m2) for Deck B
Fig 16. (below) Strain measurements in reinforcing tendon during cyclic loading for Deck B (note strains initialised prior to test)
Fig 17. (bottom) Load-displacement response for Decks A and B



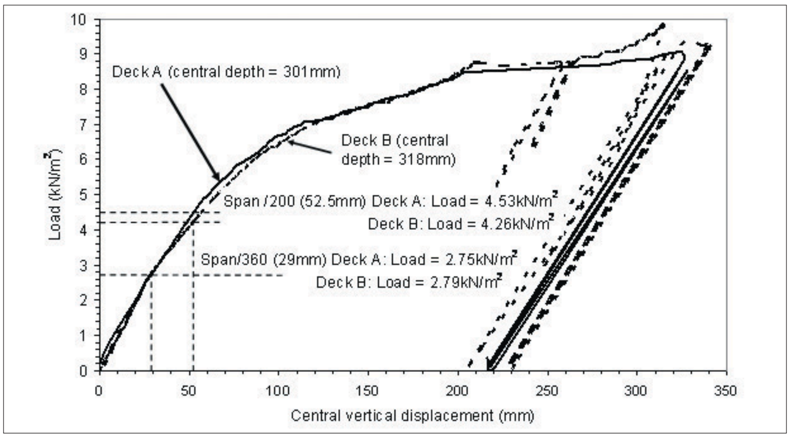
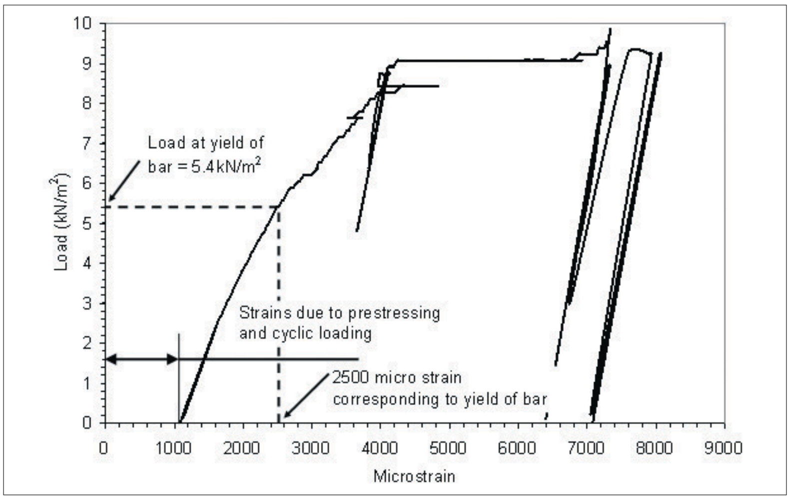


Fig 18. (left) Deck tested to maximum displacement capacity of jacks.

Fig 19. (below) Strain measurements (using post-yield gauges) in reinforcement tendon during ultimate load test on Deck B



For a deflection of span/200 (53mm), Deck A supported a load of $4.53kN/m^2$ and Deck B supported a load of $4.26kN/m^2$.

The measured strains, using post-yield gauges, in the reinforcing tendon are shown in Fig 19. Before the deck was loaded to failure the reinforcing tendon had an average value of 1095 microstrain due to prestressing and casting of the concrete (Fig 12) and due to loss of bond during the cyclic loading (Fig 16). The tendon had a measured yield strength of $525kN/m^2$ which corresponds to 2500 microstrain. From Fig 19 it can be seen that the tendon reached its yield stress under a load of $5.4kN/m^2$.

The measured strains on the deck are shown in Fig 20. The initial strains shown in Fig 20 are the strains due to prestressing, pouring of the concrete, and cyclic loading. The deck had a measured coupon yield strength ranging from $280N/mm^2$ to $320N/mm^2$, corresponding to a microstrain of 1333 to 1523. Based on a microstrain of 1333 it can be seen that at locations D and D' the deck reached yield between an applied load of 2.2 and $2.7kN/m^2$. At location C and C' the deck reached yield at an applied load of $4.46kN/m^2$, based again on a microstrain of 1333. If the yield strength is based on the upper measured value of $320N/mm^2$ then the applied load, before yield occurs, is increased, as shown in Fig 20. At strain locations A, A', B and B', the deck did not yield up to a load of $9kN/m^2$.

Test 6: Indicative fire test

An unloaded indicative 1m length of floor slab was tested to provide a preliminary indication of the temperature distribution through the deck when subjected to a standard fire test. Hot and cold face temperatures and temperatures at various locations within the sample were monitored, including the temperature of the prestressed reinforcing tendon.

The 1m long test sample was cast so that the overall depth was 280mm, which represents the depth of floor at the supports but is thinner than the depth found at mid-span, as shown in Fig 13. A 10mm thick rockwool strip was placed along the length of the section in order to act as fire protection to the tendon, which would be placed in the final floor system, as shown in Fig 3. The void was formed using a 160mm diameter pipe. The use of an overall depth of 280mm and a 160mm diameter void represents the worst possible geometry, in terms of heat transfer, for the cross-section of the deck.

The fire test was performed using the small gas fired furnace, with a working volume of $1.5m^3$, in the Fire Laboratory of the University of Manchester. The fire test specimen was fixed vertically in the front $1m^2$ aperture of the furnace with the steel section on the inside and then sealed in place using fire insulation board and carbo-wool. The sample was then tested with the temperature inside the furnace following the standard BS 476-20[1] cellulosic fire curve. The recorded time-temperature relationships of the furnace, together with the recorded temperatures through the cross-section, are shown in Fig 21. The average temperature of the reinforcing tendon after 30min and 60min was 144ºC and 338ºC respectively. At these temperatures the tendon, which is a standard high-yield bar, retains 98% of its cold yield strength. The cold face temperature was 100ºC at 60min. Based on the results from the indicative fire test it is possible to design the deck, using the principles in BS 5950-8[2] or EN 1994-1-2[3], to withstand a given fire resistance period.

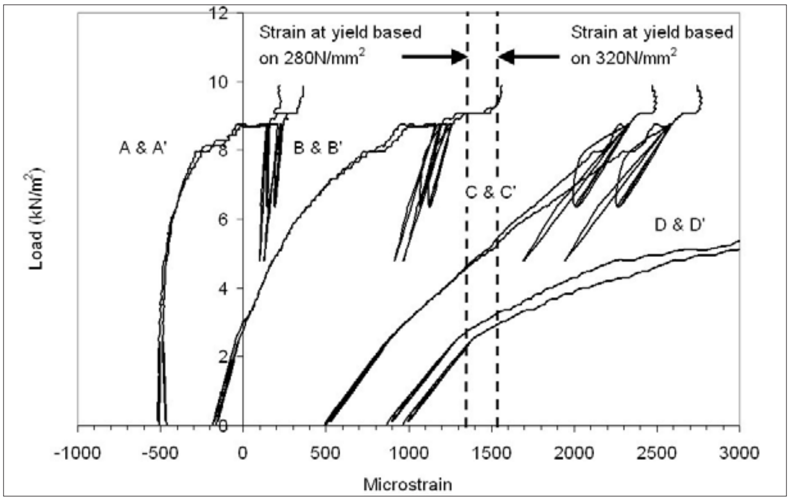
Inspection of the post-test specimen showed that in places the concrete had separated from the steel deck and that large vertical cracks had formed in the concrete between the void and the steel deck in numerous places, as shown in Fig 22.

Conclusions from testing the prototype

The testing programme of the prototype covering prestressing of the deck, concreting, serviceability and ultimate limit states, together with an indicative fire test, has shown the capabilities of the Tekdek system. Even with the compromise of using six pressed sheet parts, welded together to form the steel deck, a clear span of 10.5m was achievable, based on a 225mm deep deck.

The first test comprised of prestressing the prototype, without any lateral restraints, to failure. This test mimics the prestressing process which will be carried out in the factory prior to delivery to site. The failure was shown to be buckling at a load 2.1 times the actual maximum prestressing load required for the prototype. For the vibration, cyclic and ultimate load tests, two decks with varying prestressed force and different depths at mid-span were tested. Deck A had the largest applied prestress load (43kN) resulting in a depth at mid-span of 301mm, whereas Deck B had a slightly lower applied prestress load (40kN) giving a depth at mid-span of 318mm. The measured fundamental frequency was 4.5Hz and 4.4Hz for Deck A and Deck B respectively. Although these fundamental frequencies are acceptable in most buildings, these values are expected to slightly increase when the whole orthotropic floor plate and continuity over supporting

Fig 20. Strain measurements on Deck B during ultimate load test



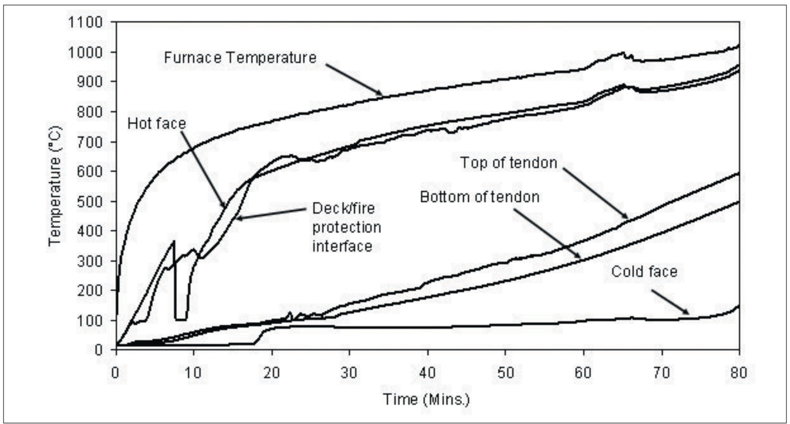


Fig 21. (above) Recorded temperatures through the deck

Fig 22. (left) Picture of composite deck following the fire test (note the debonding of the deck)

beams are considered.

During the 5000 cyclic loading tests there was evidence of breakdown in the bond provided by the screws. For the final rolled section, the bond between the deck and concrete should be greater due to the capability of rolling in indentations, embossments and sharper re-entrants. Even though the bond was reduced, during the cyclic loading, the decks still achieved maximum load capacities of 9kN/m² (Deck A) and 9.87kN/m² (Deck B). There was no evidence of collapse at the maximum load applied to the decks, with the tests being terminated due to the jacks running out of travel.

The indicative fire test provided the temperature distribution through the cross section of the composite deck using a worst-case geometry, for heat transfer, in terms of minimum depth and maximum diameter of the void. As with the tests at ambient temperature, lightweight concrete was used resulting in lower temperatures through the cross-section compared to normal-weight concrete. The cold face temperature remained below an average increase in temperature of 140ºC for 80min meeting the insulation criterion for 60min fire resistance. For higher fire resistance periods the diameter of the void would require reduction or a greater overall depth would need to be specified. The reinforcing tendon reached a maximum average temperature of 338ºC after 60min which, for a high yield bar, retains 98% of its cold strength. Based on the recorded temperatures, the floor system can be designed at the fire limit state using the principles of BS 5950-8 or BS EN 1994-1-2.

Future work

The testing of the prototype, presented in this paper, shows the feasibility and potential of prestressing the steel deck prior to pouring the concrete. The prototype section achieved a clear span of 10.5m and supported a maximum load of 9kN/m² for Deck A and 9.87kN/m² for Deck B. All the tests comprising prestressing of the deck, concreting, cyclic loading, ultimate loading, and indicative fire test, produced satisfactory results. The only slight disconcertment is the low load at which the bottom of the deck reached yield (Fig 20). However, this can easily be addressed by using the correct designed profile for the deck (Fig 4), or increasing the compressive prestress in the deck prior to pouring the concrete, or by using a higher grade steel (S350) for the deck.

With the prototype showing such promising results the next stage is to carry out a full-scale test using the correct rolled profile shown in Fig 4. The full-scale test will investigate buildability issues and the structural response under serviceability loads. It is proposed that this test will be carried out on two 10.5 × 10.5m bays giving an overall floor plate size of 21m × 10.5m. Once complete, the system will be ready for market.

Although this paper, and the prototype testing, has concentrated on a 225mm deep deck, a further two different depths for the Tekdek system have theoretically been developed to cover a range of different spans. These decks consist of a 325mm deep by 700mm wide deck, capable of spanning 14m, and a 400mm deep by 800mm wide deck, capable of spanning 16m. The theoretical deck spans for the proposed profiles are shown in Fig 23, together with typical span capacities of existing composite and precast floor systems. The 10.5m deck span achieved with the prototype profile is also shown in Fig 23.

Considering the cost-savings, due to less secondary beams when compared to other composite deck systems, the reduced crane demand and reduced transport costs when compared to pre-cast options and the speed of construction when compared to *in situ* options, it is felt that the Tekdek system will make a significant impact within the Built Environment. Leading property developers, engineers, architects and contractors have provided support for the new system and negotiations are currently in progress to fund a full-scale assembly test to provide complete confidence in its use. It is currently proposed that the full-scale test will be carried out at the University of Manchester, with the results published in due course.

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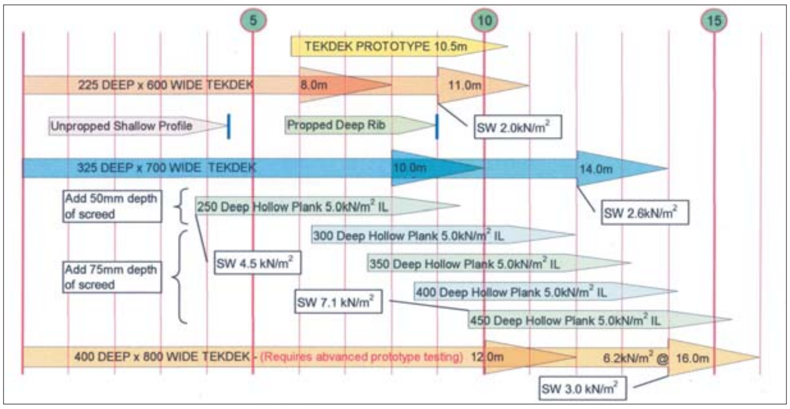

Fig 23. Proposed spans of Tekdek systems compared to existing composite and precast systems

REFERENCES

1. BS 476-20: *Method for determination of the fire resistance of elements of construction: Part 20*, London, British Standards Institution, 2001
2. BS 5950-8: *Structural use of steelwork in buildings: Part 8: Code of practice for fire resistant design*, London, British Standards Institution, 2003
3. EN 1994-1-2: *Eurocode 4. Design of composite steel and concrete structures. Part 1.2. General rules. Structural fire design.* Final Draft European Committee for Standardization, Brussels, 2003