The Fire Resistance of a Shelf Angle Floor Construction, a BS476:
Part 8 Fire Test Carried out on 3rd November, 1982

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KEY WORDS

1. Fine Resistance
2. Concrete
3. Floorings
4. Deflection
5. Testing
6. Shelf Angle
7. Lab Reports

4th July, 1983
SYNOPSIS

The report presents the results of a BS476 : Part 8 fire test carried out on an unprotected BS476 : Grade 43A beam of serial size 406 x 178 mm x 54 kg/m which was used as part of a shelf angle floor construction. Precast concrete floor slabs 200 mm deep and 550 mm wide were supported on 125 x 75 x 12 mm angles bolted to the web of the beam. The construction was fully loaded during fire testing and the test was discontinued after 60 minutes when the deflection of the beam at its centre reached 150 mm which is the limiting deflection in BS476 : Part 8. This performance was better than expected and it is thought that composite action between the various parts of the construction with the colder ends of the beam may have contributed to the good results.

The use of partial protection of members is clearly beneficial to improve their fire resistance; however the present results of such tests need to be made more general and this could best be achieved by the development of computer-based models which can predict both the heating rate and stability of members which exhibit large temperature gradients. The cost benefits of shelf angle floors compared with conventionally protected members require careful evaluation.

1. INTRODUCTION

Attempts have been made in recent years to develop constructional methods utilising unprotected steel members, to achieve fire resistance period of 30 and 60 min. These studies have considered the effect of beam size, steel grade and loading system on the fire resistance and whilst periods approaching 60 min have been achieved the methods investigated to date have demonstrated that unprotected members can only reliably achieve fire resistance of 30 min and frequently necessitate 'end restraining moments' to be utilised to improve their inherent fire resistance.

A preliminary test on columns built into masonry walls has shown that this form of construction can achieve a considerable period of fire resistance. The masonry protection to one flange and part of the web, significantly reduces their heating rate and consequently an enhanced fire resistance was measured.

The concept of partial protection can be extended to horizontal members using shelf angle floor systems where precast concrete slabs rest on steel angles attached to the webs of the beam with the upper surface of the slabs below the upper flange of the beam (see Fig. 1). This form of construction has been widely used in the design of multistorey buildings and although expensive it is utilised to reduce the floor/ceiling services void depth which results in cost savings on the area of cladding and partitions which are required.
in this particular test a 406 x 178 mm 54 kg/m BS4360 : Grade 43A beam was utilised along with 125 x 75 x 12 mm BS4360 : Grade 50B angles and 200 mm thick precast concrete slabs to produce a shelf angle floor construction. The section was fully loaded and fire tested at the Warrington Research Centre on 3rd November 1982 and this report describes the construction of the specimen and observations made during and after the test. The report also discusses future testing requirements and the implications of this method of construction for the design of multistory steel framed buildings where periods of fire resistance up to 1 h are required.

2. THE CONSTRUCTION

2.1 Steel

The steel members used in the test were obtained from local stockholders. Their serial sizes and qualities were as follows:

- The beam: 406 x 178 mm x 54 kg/m, BS4160 : Grade 43A
- The angles: 125 x 75 mm x 12 mm, BS4360 : Grade 50B

The chemical composition and room temperature mechanical properties of these members are shown in Table 1 and 2.

The flange of the Grade 43A beam had a yield stress of 274 N/mm² and tensile strength of 439 N/mm² and satisfied both the mechanical and chemical requirements of BS4360.

The longer leg of the BS4160 Grade 50B angle had a yield stress of 384 N/mm² and tensile strength of 518 N/mm² which satisfied the requirement of BS4360 : Grade 50 B.

2.2 Design of the Construction

Steel

It is usual in shelf angle construction to support the concrete slab on an outer flange of the angle, the angle is then totally exposed to any fire which occurs underneath the concrete slab. However in this test the configuration was reversed in order to minimise the amount of angle exposed to fire attack.

The 75 mm leg of the angle was bolted to the web of the beam using M20 : Grade 4.6 bolts at 600 mm centres, the bolt holes being drilled at the centre of the 75 mm long leg. The angles were bolted in position at 8 places along their length. The beam used was 5.01 m long, the angles being of the same length. The angles were positioned to leave a 240 mm gap between the upper flange of the beam and the longer leg of the angle. A drawing illustrating the details of the construction is shown in Fig. 2.

Concrete

The concrete slabs used in the test were specially cast in the middle of May 1982, and were stored indoors until the day of the test. During storage the slabs were spaced 75 mm apart to allow a free flow of air between the stacked slabs to aid their drying out. The slabs were 550 mm wide, 200 mm thick and 1550 mm long, this length being selected to permit a gap of 50 mm between the nose of the slab and the web of the beam without interfering with the loading system of the furnace.

The slabs were specially designed to withstand the loading force anticipated during the test and details of the calculations used are shown in Appendix 1. Shown in Fig. 3 shows the reinforcement positions within the slab.

The concrete quality used was Grade 30 and the results of cube tests made on similar samples on 4th November 1982 are shown in Table 3.

2
2.3 Instrumentation

A total of 35 Pyrotenax thermocouples (chromel/alumel with insulated hot junctions) were used to monitor the heating rate and temperature of the steel during the test. The thermocouples were located at the positions shown in Fig. 4 and in summary 5 thermocouples were attached to the exposed lower flange, 4 thermocouples to the exposed part of the web, 4 thermocouples were attached to the protected part of the web and 4 were attached to the upper flange of the section. These thermocouples were located around the central part of the beam. An additional set was positioned (upper flange 2 cm webs and lower flange) 180 mm away from the furnace wall. Two additional thermocouples were also located at the flange/web junction on the lower flange of the beam.

The remaining 12 thermocouples were attached to the shelf angles with 4 thermocouples being mounted on the exposed flange, 2 on the unexposed flange and 2 on the root of the angle. Figure 1 shows a photograph of some of the thermocouples in position demonstrating the technique used. Three thermocouples were also used to monitor the temperature rise in the concrete slab, i.e. fourth segment at 1/4, 1/2 and 3/4 depths positions, 110 mm away from the web of the beam.

The final six thermocouples were installed once the assembly was constructed to monitor surface atmosphere temperatures at the locations shown in Fig. 6.

2.4 Assembly

The beam was located in the furnace at the appropriate position and the individual concrete, floor slabs were slotted into the space between the shelf angle and the upper flange to give a load bearing width of 75 mm. A wall was constructed along the long edges of the furnace to support the free ends of the slabs. At the short end of the furnace a small wall was constructed but the slabs did not rest on this wall, the gap being filled using compressible insulating ceramic blanket.

Once all the slabs were located in position the gap between the slab nose and web of the beam was filled with sand to represent the thermal characteristics of a scribed. The top flange of the beam was also covered with 25 mm of sand to also represent a scribed as used in practice. Photographs showing the construction during assembly are shown in Figs. 7 and 8.

2.5 Loading

In an attempt to represent the type of loading which would be encountered in practice it was considered essential to apply all the load to the beam and angle through the concrete slabs. High point loads under the ram of the hydraulic loading jacks were avoided by applying the load through 4 sections, each 1 m long, of 112 x 112 mm universal column placed above each side of the beam. These lengths of column acted as load spreaders between the slabs. Partially factored loads applied were validated to the equivalent of loads carried by the wall along the long edge of the flange. A total load of 1.6 kN was applied at 8 points and details of the loading calculations are shown in Appendix 2. Photographs of the specimen immediately before the test are shown in Fig. 9.

3. THE TEST

The fire resistance of the unit of construction was 68 min, the test being discontinued when the deflection at the centre of the beam reached 150 mm, i.e. the L/30 failure criterion stated in BS476 : Part 8.

3.1 Deflection Measurements

The results of deflection measurements made on the beam and also on the concrete slabs at the centre of the construction are shown in Fig. 10, from which it can be seen that the two curves followed similar patterns. These curves were slightly different from those typically obtained from unprotected steel beams, in that the rate of deflection increased steadily during the test, whereas, normally, the deflection rate is slow at the beginning of the test and then increases rapidly during the final minutes of the test.
3.2 Temperature Measurements

All instrumentation operated satisfactorily and the results of temperature measurement are shown:

- Fig. 12 shows temperature data collected from the lower flange.
- Fig. 13 shows temperature data collected from the exposed web.
- Fig. 14 shows temperature data collected from the concealed web.
- Fig. 15 shows temperature data collected from the exposed leg of the shelf angle.
- Fig. 16 shows temperature data collected from the concealed leg of the shelf angle.
- Fig. 17 shows temperature data collected from the root of the angle.
- Fig. 18 shows temperature data collected 100 mm away from the furnace wall.

At the end of the test the following average temperatures were recorded around the centre of the beam:

- Lower flange (beam): 815°C
- Exposed web (beam): 889°C
- Concealed web (beam): 1580°C
- Exposed leg of angle: 824°C
- Concealed leg of angle: 611°C
- Root of angle: 722°C

The furnace atmosphere heating curve is compared with the international time-temperature curve in Fig. 14, which shows that the heating rate was in accordance with the standard curve throughout the test. A summary of steel temperatures and furnace atmosphere temperature at various stages during the test is given in the data sheet.

The temperature rises which were monitored in the fourth concrete cover slab approximately 150 mm from the flange tip at the quarter, half and three-quarter depth positions are shown in Fig. 30. The temperature rose steadily after 12 min into the test from 23°C to 53, 106 and 107°C at the 50, 106 and 150 mm depths respectively.

After cooling the shelf angle floor test arrangement was satisfactorily reloaded before being dismantled and removed from the furnace.

3.3 Observations

3.3.1 During The Test

As soon as the test commenced light but dense fumes started to escape from the concrete cover slabs. These fumes (see Fig. 21) were continually emitted throughout the duration of the test. After about 25 min some hairline cracks appeared in the top surface of the concrete cover slabs in close proximity to the lifting rings. There was also some evidence of spalling in areas along the edges of the slabs.

As the beam deflected the concrete cover slabs developed a stepwise pattern which became more exaggerated towards the ends of the shelf angle arrangement as the test progressed as shown in Fig. 22.

The beam and angle appeared to be deflecting in a uniform manner - in fact very similar to that experienced in a simply supported beam test.

3.3.2 After The Test

Prior to dismantling, inspection of the unit of construction from within the furnace confirmed that the deflection on both the beam and angle was fairly uniform (see Fig. 23) and that there was no evidence of cracking in the underside of the concrete cover slabs.
Figure 24 shows that the slabs had not moved significantly towards the web of the beam. Some of the slabs exhibited cracks which were contained in an area of the slab supported on the shelf angle. There were two types of crack patterns which ran through the slab thickness, one type vertical and the other at an angle of about 45°, as shown in Fig. 25. A closer examination of the steel angle, once all the concrete slabs had been removed, revealed a slight wave pattern (see Fig. 26) probably caused by the side edge action of the slabs.

Two of the end bolts had sheared, one which was completely outside with the furnace while the other was protected behind the first pair of concrete slabs. It is not known whether this occurred during or after the test but it was noticed during the reload test.

4. DISCUSSION

4.1 Behaviour Of The Construction In The Present Test

The fire resistance time obtained with this construction using the maximum permissible design loads was 68 minutes, easily exceeding a 1 h requirement. Furthermore at the end of the test when the deflection of the assembly had reached the L/30 limit of deflection its rate of deflection of the specimen was very low at 2 mm/min and it is possible that an additional period of fire resistance could have been realised if the test had been extended and a 'rate of deflection' failure criterion utilised. This period of fire resistance is much greater than that observed for fully loaded but totally exposed simply supported beams and clearly the partial protection afforded by the concrete floor slabs had made a considerable contribution towards it fire resistance. This was demonstrated in two ways:

1. The concrete slabs resisted heat flow to the upper flange of the section and this part of the section would therefore retain almost all its original load bearing capacity.

2. There was some evidence that the concrete slabs made a partial contribution to the load bearing capacity of the assembly. (This aspect will be considered in more detail later in this report).

The thermal protection provided by the concrete slabs was very significant in that when the lower flange had been heated to 915°C at the end of the test, the upper flange of the section was only at 94°C. The temperature measurements made indicate that a considerable portion of the beam was maintained at a temperature below 400°C, and hence exhibited load bearing capabilities not significantly reduced from its original load bearing capacity. This considerable temperature gradient did, however, contribute to the deflection of the beam particularly in the early stages of the test due to differential thermal expansion. The lower flange being hotter expands to a greater extent than the upper flange, and hence this differential expansion causes the beam to bow. For instance, the calculations shown in the Appendix 3 model the situation which existed 30 min into the test when the lower flange was heated to 728°C and the upper flange was still at 28°C. This gradient of 700°C would cause bending of the beam and the simple analysis described predicts a central deflection of 52 mm compared with the observed deflection at this time of 79 mm. The underestimate given by these calculations can be explained in part by a reduction in the value of elastic modulus at elevated temperature. The very slow rate of deflection at the end of the fire test is difficult to explain for a simply supported member. One would expect the rate of deflection to increase rapidly towards the end of the test and hence it is thought that other factors such as composite action from the concrete slabs restricted the deflection of the steel member. During the test as the beam deflected the concrete slabs moved independently in a stepwise manner and hence there was some tendency for the slabs to jam-up against each other. This interaction could provide a load bearing member in itself provided that the edges of the 'raft' are adequately supported. In this test the ends of the shelf angles were outside the furnace and the upper flange of the beam and the angle leg provided a load bearing surface to prevent movement of the edge slabs. Hence the loads generated by wedging on these edge slabs were high and subsequently caused bolts which were outside the furnace to fracture by shear. Hence it is
concluded that the interaction of the slabs coupled with the cold periphery of the shelf angle provided an additional load bearing member and improved the overall fire resistance of the construction. The significance of this composite action in the application of these results to real structures is difficult to consider. In a fire in a real structure all the shelf angle could be heated and then the degree of restraint which would be provided at the periphery could be significantly lower and hence it is possible that the beam floor construction would not perform in the same way in real structures in real fires.

4.2 Future Work

The results obtained from the present tests only apply to this particular combination of beam size and concrete slab depth, but to extend the commercial utilisation of this construction method system it is necessary to predict the behaviour of other similar floors. This would be best achieved by the development of two models which would simulate:

1. The temperature of the exposed and concealed parts of the section. This could be done using finite element analysis techniques and some work has been completed by James on the development of a model to predict the temperature rise of a column built into a masonry wall.

2. The structural stability of an element which exhibits temperature gradients. Conrado have already developed a preliminary version of a model which can predict the load bearing capacity of a beam which exhibits temperature gradients. Clearly there is scope to utilise and develop both models so that various situations can be examined without the need to perform costly and time consuming fire tests. The first stage would be to establish confidence in the accuracy of these models by performing a detailed analysis on the data generated from the present test to determine the ability of the models to predict for this situation.

4.3 Commercial

The impetus to develop methods of achieving fire resistance in unprotected steel members arises because of the need to reduce or eliminate the costs of applied fire protection. Clearly the cost aspects of this shelf angle floor construction need to be carefully evaluated before extensive further test work is pursued.

The use of shelf angles increases the weight of steel considerably; for instance these 125 x 75 x 1.5 mm angles weigh 17.8 kg/m adding 35.6 kg/m to a beam weighing 54 kg/m.

Using a fabrication cost of £500/t this 'extra steel' would add £77.8/m to the overall cost of the framework. Alternatively the 406 x 178 mm beam would require 1 m² of board per extra run to fire protect it and the use of board for fire protection could easily achieve the same level of fire resistance (1 h) at the same cost most probably lower cost. The spray applied materials would be even less expensive. However the use of shelf angle floors also reduces the surface area of the facade and this factor must be borne in mind in any cost evaluation.

A preliminary exercise, carried out by the Mackay Development Unit of BBC Sections, has indicated that the use of a shelf-angle floor type of construction could increase the tonnage of steel in a multi-storey building by 20% whilst reducing the total building steel cost by 7%.

5. CONCLUSIONS

A shelf angle construction involving a 406 x 178 mm x 54 kg/m beam and 125 x 75 x 1.5 mm angle supporting 200 mm precast concrete floor slabs has been fire tested following the BS476 : Part 8 requirements and using the maximum design load.
The unit of construction achieved a fire resistance time of 60 min, easily satisfying any Building Regulations requirement for 1 h.

The deflection time graph recorded during the test was different from those of other tests performed on simply supported steel members in that the deflection increased more rapidly in the early stages of the test due to differential thermal expansion and in the later stages the rate of deflection remained relatively constant and 'runaway' did not occur. It is thought that the interaction of the concrete slabs may have produced a load bearing raft and this could have contributed to the enhanced fire resistance observed. Some evidence for this 'raft theory' was observed due to the shear fracture of bolts holding the shelf angle which were outside the furnace. All other bolts remained intact during the test.

The results obtained from the present test apply only to this construction and cannot be made more general without the development of computer based models which should predict both the temperature rise in various parts of the partially protected member and also its stability. Preliminary work has already been performed to develop such models.

The cost benefits of utilizing shelf angle floor construction require careful evaluation to show that it does not prove more expensive than conventional fire protection systems.

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Investigator
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Structural Advisory Engineer
C.I. Smith
Principal Investigator

Mr. J. Lessells
Research Manager
General Steel Products

LC
### Table 1
Chemical Composition of the 406 x 178 mm x 14 mm Universal Beam and 135 x 76 x 12 mm Angle Used in the Test Arrangement

<table>
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<th>Code No.</th>
<th>406 x 178 mm x 14 mm</th>
<th>135 x 76 x 12 mm</th>
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<tr>
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<td>0.24</td>
<td>0.24</td>
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<tr>
<td>Mn</td>
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<td>1.20</td>
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<tr>
<td>P</td>
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<td>0.013</td>
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<tr>
<td>S</td>
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<td>0.024</td>
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<tr>
<td>Cu</td>
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<td>0.001</td>
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<td>Ni</td>
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<td>V</td>
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<tr>
<td>Ti</td>
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### Table 2
Tensile Test Data from the 406 x 178 mm x 14 mm Universal Beam and 135 x 76 x 12 mm Angle Used in the Test Arrangement

<table>
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<tr>
<th>Code No.</th>
<th>Section</th>
<th>Position</th>
<th>Quality</th>
<th>Yield Stress (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Elongation (%)</th>
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<tr>
<td>10360F</td>
<td>406 x 178 mm x 14 mm</td>
<td>Flange 1</td>
<td>BS4360 Grade 43A</td>
<td>274.7</td>
<td>439.7</td>
<td>26</td>
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<tr>
<td>10360H</td>
<td>406 x 178 mm x 14 mm</td>
<td>Web</td>
<td>BS4360 Grade 43A</td>
<td>308.4</td>
<td>47.7</td>
<td>22</td>
</tr>
<tr>
<td>10370F</td>
<td>406 x 178 mm x 14 mm</td>
<td>Flange 1</td>
<td>BS4360 Grade 43A</td>
<td>315.5</td>
<td>43.9/44.9</td>
<td>20</td>
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<tr>
<td>10370H</td>
<td>406 x 178 mm x 14 mm</td>
<td>Flange 1</td>
<td>BS4360 Grade 43A</td>
<td>395.5</td>
<td>35.4/32.0</td>
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<td>10380F</td>
<td>125 x 75 x 12 mm</td>
<td>Web</td>
<td>BS4360 Grade 43A</td>
<td>385.5</td>
<td>49.2/40.3</td>
<td>10</td>
</tr>
<tr>
<td>10380H</td>
<td>125 x 75 x 12 mm</td>
<td>Web</td>
<td>BS4360 Grade 43A</td>
<td>395.5</td>
<td>35.4/32.0</td>
<td>21</td>
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### Table 3
Concrete Compression Test Results

<table>
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<tr>
<th>Test Date</th>
<th>Age Days</th>
<th>Compressive Strength (MPa)</th>
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</tr>
<tr>
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<td>11/14/92</td>
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**Note:** These readings are for illustrative purposes only.
FIG. 1

SHELF NUTS FLOOR DESIGN BEING USED IN THE CONSTRUCTION OF A MULTI-STORY BUILDING
Beam size: 406 x 178 mm x 14 kg/m
Angle size: 125 x 75 x 12 mm
Bolts: M20 Grade 4.6

Dimensions in mm

SCHEMATIC ILLUSTRATION OF TEST ARRANGEMENT

FIG. 2
(R1/8766)
All dimensions in mm

Lifting hooks as necessary

25 Cover

A 98 Mesh

20 Cover

25 Cover

200

Distribution bars Y10-200

40 Cover

A 98 Mesh

40 Cover

Y10 Bars

BS4466 Shape Code 35

1. Concrete crushing strength 25 N/mm² at 28 days
2. Reinforcement cold worked high yield to BS4461

FIG. J
(P1/8767)
PHOTOGRAPH OF TEST BEAM OF FURNACE SHOWING POSITIONS OF THE ATTACHED THERMOCOUPLES
POSITION OF FURNACE ATMOSPHERE THERMOCOUPLES

FIG. 6
(#1/9769)
FIG. 8

VOID FILLED WITH SAND TO SIMULATE THE THERMAL CHARACTERISTICS OF THE SCREEN USED IN SITE PRACTICES
Furnace top side

Inside furnace

COMPLETE ARRANGEMENT JUST PRIOR TO TESTING

FIG. 9

18
Deflection, mm

- Steel beam
- - - Concrete slab

Time, min

Central vertical deflection of beam and concrete slab measured throughout the test

FIG. 10
(R1/8777)
Temperature, °C

0 10 20 30 40 50 60 70

TEMPERATURES RECORDED ON THE LOWER FLANGE OF THE TEST BEAM DURING THE TEST

FIG. 11
(81/0771)
Temperature, °C

0  20  40  60  80  100

0  10  20  30  40  50  60  70

Web 1
Web 2
Web 3
Web 4

FIG. 12
TEMPERATURES RECORD ON THE EXPOSED WEB OF THE TEST BEAM AT THE QUARTER-WIDTH POSITION
(83/8772)
Figure 13: Temperatures recorded on the unexposed web of the test beam at the three-quarter-wind position.
Temperatures recorded on the upper flange of the test beam during the test.

FIG. 14

(81/8774)
Fig. 15

Temperatures recorded on the exposed flange of the angle during the test.
TEMPERATURES RECORD ON THE UNEXPOSED FLANGE OF THE ANGLE DURING THE TEST

FIG. 16

(81/B776)
Temperature, °C

Temperature recorded at the root of the angle during the test

Fig. 17
(S1/8777)
FIG. 18
ADDITIONAL THERMOCOUPLES ON TEST ARRANGEMENT
POSITIONED 100 mm AWAY FROM THE SURFACE WALL
Furnace atmosphere temperatures, °C

- - - - - - Furnace atmosphere 1

- - - - - - Furnace atmosphere 2

- - - - - - Furnace atmosphere 3

- - - - - - Furnace atmosphere 4

- - - - - - Furnace atmosphere 5

- - - - - - Furnace atmosphere 6

- - - - - - International temperature/time curve

**Furnace atmosphere temperatures recorded during the test**

*FIG. 19 (R1/8779)*

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TEMPERATURES RECORDED AT DEPTHS OF 50, 100 AND 150 mm IN THE FOURTH CONCRETE SLAB

FIG. 20
(R1/8786)
STEPWISE PATTERN FORMED BY THE CONCRETE SLABS AS THE DEFLECTION ON THE BEAN INCREASED
Figure 22

Underside of the test arrangement after testing showing the beam and angle had deformed uniformly.
CENTRAL CONCRETE SLAB AFTER TEST SHOWING THAT THERE WAS NO SIGNIFICANT MOVEMENT OF THE SLAB DURING THE TEST

FIG. 24
Types of cracks exhibited in some of the concrete slab employed within the load-bearing area on the bridge abutment.
APPENDIX 1  DESIGN CALCULATIONS FOR PRECAST CONCRETE FLOOR SLABS

Floor Slabs

Consider a concrete strength at 28 days = 25 N/mm².

Let slab depth = 200 mm
slab width = 1000 mm

Concrete cover to steel = 40 mm for 4 h resistance.

\[ R_L = 30 \text{ kN} \]

\[ R_W = 1.1 \text{ m} \]

\[ 1.6 \text{ m} \]

Consider a load from the loading frame = 30 kN
Self weight of slab = 24 x 0.2 = 4.8 kN/m
Total distributed load = 4.8 x 1.6 = 7.7 kN
Reaction from the shelf angle \( R_s = 20.6 = 3.85 = 24.65 \text{ kN} \)
Reaction from the wall \( R_w = 9.4 x 3.85 = 36.25 \text{ kN} \)
Bending moment at load point \( = (24.45 x 0.5) - (4.8 x 0.5^2) \)
\[ = 11.63 \text{ kN m} \]

Effective depth of tensile reinforcement = \( d_1 = 155 \text{ mm} \)
slab width = \( b = 1000 \text{ mm} \)

Resistance moment of section \( m = k bd^2 \)

\[ \frac{M}{bd^2} = \frac{11.63}{1 x 0.155^2} = 484 \]

From graphs of lever arm ratio, \( j \) against \( \frac{M}{bd^2} \) for the maximum permissible steel stress, then \( j = 0.90 \)

\[ \text{Leaver arm, } la = 0.90 x 155 = 139.5 \text{ mm} \]

From CP 114 : Part 2 1969

Based on the tensile reinforcement \( M = A_{ST} F_{ST} la \)

where \( A_{ST} \) = area of tensile reinforcement

\[ F_{ST} \] = permissible tensile stress in reinforcement = 236 N/mm² from Table 11.

\[ la = 139.5 \text{ mm} \]

\[ A_{ST} = \frac{11.63 x 10^6}{362.5 \text{ mm}²} = 32.5 \text{ mm}² \]

Provide 5 off Y10 = 200 bars as tensile reinforcement.

Shear Stress across reinforced concrete slab \( = \frac{R_L}{b la} = \frac{24.45 x 10^3}{10 x 1.1 x 139.5} = 0.18 \text{ N/mm}² \) (permissible shear \( 0.3 \text{ N/mm}² \))

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Bond Stress = \( \frac{R_l}{I_a \times 0} \), where \( I_a \) = sum of moments of tensile reinforcement

\[ = \frac{24.45 \times 10^3}{(139.5) \times 5 \times 3.4} = 1.11 \text{ N/mm}^2 \] (Permissible local bond stress < 1.5 N/mm²)

Bearing Stress, Assume 75 mm of bearing

\[ = \frac{24.45 \times 10^3}{10^3 \times 75} = 0.326 \text{ N/mm}^2 \] (Permissible bearing stress < 6.5 N/mm²)
APPENDIX 2  SHELF ANGLE : CONCRETE FLOOR TEST

Beam 406 x 178 x 54 kg/m
Shelf angles 125 x 75 x 10 (15 kg/ea) - 2 off
Distance between beam end supports = 4.5 m
Safe working load uniformly distributed = 271 kN (Assume working stress of 165 N/mm²)

Load spreader beams
Hydraulic loading jacks (4-off each side)
Concrete cover slabs

Shelf angles

0.5 m
1.1 m
1.6 m
Wall

(i) Self weight of cover slabs and spreader beams = 36 kN
Reaction on each shelf angle due to (i) above \( \frac{36}{2} \times 1.6 = 9 \) kN

(ii) Total force required on each shelf angle to produce working stress in test beam = \( \frac{221}{2} = 135.5 \) kN - 9 kN (Self Wt. cover slabs)

= 126.5 kN

Force required by each set of jacks to produce (ii) above

\( \frac{126.5 \times 1.6}{1.1} = 184 \) kN

... Force required at each loading jack = \( \frac{184}{4} = 46 \) kN

Total hydraulic forces applied = 46 x 8 368 kN (36.9 tonf)

TO CALCULATE STRESS IN SHELF ANGLE

Bending Moment/m = \( \frac{135}{4.5} \times 73 \)

Section modulus 'Z' of angle leg/m = \( \frac{1000 \times 12}{6} \)

BM stress = \( \frac{135 \times 73 \times 6 \times 1000}{4.5 \times 1000 \times 12} = 91 \) N/mm²

Shear stress = \( 4.5 \times 12 \times 1000 = 2.5 \) N/mm²

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i.e. \( R_b = 114.592 \)

Substituting this value in Equation A3.2 we have:

\[
\frac{0.042 \times 360}{2} = 0.046
\]

\[
0.406 \times 0 = 0.042 \times 360 - 114.59
\]

\[
0.406 \times 0 = 115.795 - 114.592
\]

\[
= 1.203^\circ
\]

\[
\therefore \quad = 2.963^\circ
\]

And using Equation A3.1:

\[
R = \frac{4 \times 360}{4 + 7}
\]

\[
= 38.673 \text{ m}
\]

Using the values of \( R \) and \( \theta \) the value of \( d \) = deflection can be determined as follows:

\[
\text{Now } \cos \theta = \frac{d}{R} \quad \text{or} \quad x = R \cos \theta
\]

\[
\therefore \quad x = 38.673 \times 0.9966631
\]

\[
= 38.6213
\]

\[
\therefore \quad d = 38.673 - 38.621
\]

\[
= 0.52 \text{ m} = 52 \text{ mm}
\]

Hence a deflection of 52 mm could arise from differential thermal expansion.
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