MODEL STUDIES ON PLATE GIRDERS

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Tests on eighteen small scale models which simulate the elastic and post-buckling behaviour of plate girders when subjected to shear loading are reported and discussed. The models were fabricated of steel and Araldite; the major aim was to assess whether small scale models can be employed to study shear buckling problems. A secondary object was to examine whether araldite could be used for predicting the structural behaviour and ultimate loads of plate girders.

The strength and post-buckling characteristics exhibited by steel models were found to be similar to those observed by earlier investigators on full scale girders. The test results of steel models have been compared with the theoretical predictions obtained by using some ten design methods developed in different countries. Most of these methods are shown to give conservative but satisfactory predictions of the ultimate shear capacity of the model steel girders.

Tests on Araldite models demonstrated that post-buckling behaviour can be observed visually on account of the large elastic deformations which the material is capable of, before collapse. However, they were found to be unsuitable for the prediction of the ultimate shear capacity. As Araldite is brittle, collapse would occur prematurely by sudden fracture before the full development of the tension field.

1 INTRODUCTION

In the past, the ultimate strengths of plate girders used to be obtained by applying a factor to the critical shear buckling value of the webs. This practice was very conservative as the actual strengths were well in excess of the critical shear load, i.e., there was a significant post-buckling strength. Recent research has been directed towards predicting strengths of girders having large web slenderness (depth/thickness) values.

Many of the design methods developed recently in different countries are based on mathematical models which obtain the ultimate strengths of plate girders as the sum of the buckling strength and the post-buckling strength of the web together with the contribution made by the flanges **(1)**–**(10)**. These mathematical models are generally based on observed structural behaviour and serve as excellent bases for design. Obviously carefully controlled experimentation is necessary before a mathematical model can be adopted for design calculations.

Large scale model tests, although desirable for proof testing, are expensive and time consuming. For this reason, the amount of test data that could be obtained would necessarily be limited. It would be a great advantage if small-scale models could be used for simulating the behaviour of large-scale structures.

In this study, small-scale model plate girders made of steel and Araldite are reported. The object is to compare the strength and structural behaviour of these models with those observed by earlier investigators in large scale tests. (It should be noted that the models were completely made of Araldite or steel; Araldite was *not* merely used for jointing).

2 THE PATTERN OF FAILURE OF PLATE GIRDERS

The simple case of a web bounded by two flanges and two vertical stiffeners, loaded in shear will now be considered (Fig. 1(a)). To begin with, the shear load is restrained by the web plate up to the critical load, i.e., the onset of buckling. A further increase in the load, beyond the critical load, causes compressive buckling of the panel in a wave formation parallel to the principal tensile direction. A small band of web plate along the diagonal in tension commences to behave similar to a tension member of a corresponding truss bounded by the flanges and the two vertical stiffeners together with the web tension band (Fig. 1(b)). This action is denoted as the tension field action, and causes the web panel to have a shear capacity well in excess of the elastic critical load.

The tension field action stretches across the web panel in a diagonal direction. Being flexible, the flanges distort due to the tension field, but the stiffeners remain straight as the adjacent panels provide the restraint. So the flanges can be said to act as beams with fixed ends. When the load is increased further plastic hinges are formed in the flanges as they attempt to restrain the portion of the tension field connected to the flanges (Fig. 1(c)).

It can be concluded, therefore, that the three contribu-

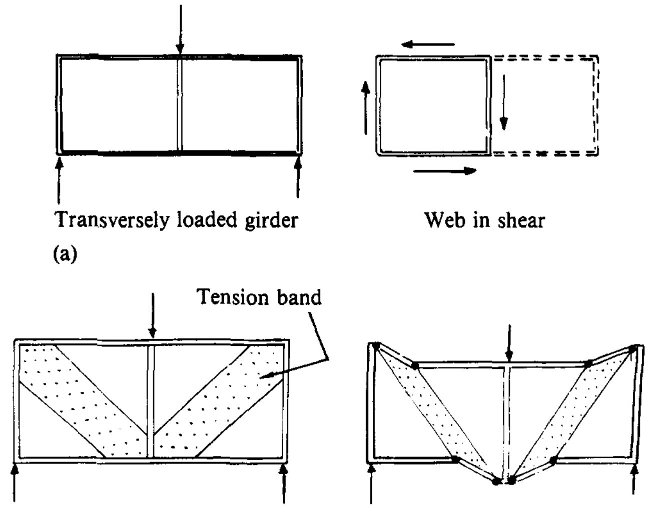

Fig. 1. (a) Loaded girder, (b) membrane stress in tension, (c) formation of hinges in flanges

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† *References are given in the Appendix.*

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tions which cause the failure of a plate girder subjected to shear are:

(i) the elastic critical load;
(ii) the tension field;
(iii) the contribution of the flanges.

A number of mathematical models incorporating the three contributions referred to above have been put forward recently by various investigators and these will be found in references (**1**)–(**10**).

3 MODEL ANALYSIS

Scale model representation is a well established method for the verification of existing and proposed mathematical theories. Experimental results on the structural performance (e.g., deflections, stresses, etc.) and modes of failure generally cover the principal aspects of modelling.

Plate girders are enormous with respect to sizes and load capacities and their proof-testing would be inconceivable in terms of economy. If small scale model tests were found to be satisfactory, they would facilitate efficient testing and assist in indicating the necessary modifications to the theory easily and economically.

Models are sometimes fabricated of another material, which would compare essential structural behaviour (**11**). In this study the use of Araldite to model steel plated structures is examined with a view to study the elastic post-buckled characteristics of plate girders. A major reservation, however, is that Araldite is not capable of any plastic deformations and in this respect is very different from steel. Thus, while the epoxy resin could be used to study the pattern of buckling and the position at which yield may commence, it cannot be used to study post-yield characteristics.

4 DESIGN OF MODELS

4.1 Steel models

The small-scale steel models reported in this paper consisted of varying parameters of web depth/thickness ratio (h/t) and aspect ratio (b/h), with all joints welded. Figure 2 and Table 1 give the design details of 9 steel models reported in this paper. It will be noted that two aspect ratios (*viz.* 1.0 and 1.5) and five web slenderness values (200, 250, 300, 350, and 400) were used.

The welding of components induces fusion areas, which create welding shrinkage stresses. Other uncertainties that exist in the fabrication are:

(i) the lack of homogeneity of the steel and the difficulty in detecting variations;
(ii) the initial curvature of panels;
(iii) the distortion caused by the relaxation of rolling stresses.

Notwithstanding these limitations, it is still possible to fabricate acceptable small scale steel specimens to represent full scale girders.

4.2 Araldite models

The design details of the Araldite models were exactly the same as the steel models listed in Table 1; i.e., they had the same aspect ratios and web slenderness values but were fabricated using an epoxy resin, Araldite 219, manufactured by Ciba–Geigy (**12**). The resin consisted of Araldite CY219, hardener HY219, and accelerator DY219, the former being a liquid of medium viscosity and the latter two of low viscosity.

The materials were mixed in fixed proportions, forming a low-to-medium viscosity liquid; the Araldite proportions used in this study were:

Araldite	CY219	100 g
Hardener	HY219	50 g
Accelerator	DY219	4 g

These were the quantities mixed at any one time; undoubtedly these seem small, but were necessary to avoid the hardening of the mixture, before it was placed into the moulds. By carefully controlling the mix proportions and fabrication procedures, a homogeneous model can be fabricated with very little distortion and no residual stresses (unlike welded steel models).

The method of fabrication of Araldite sheets is described in (**13**).

5 TEST RIG AND INSTRUMENTATION

A photograph of the test rig with a specimen in position is seen in Fig. 3. The test frame was fabricated out of

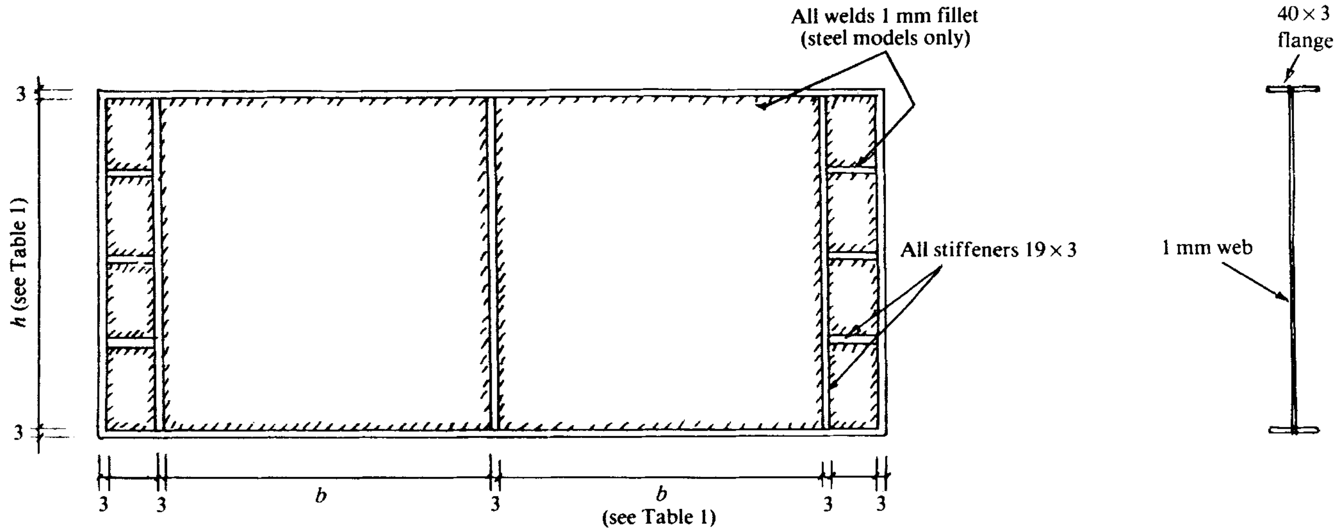

Fig. 2. Details of experimental models

Table 1. Design dimensions of Araldite and steel models

Specimen designation for Araldite models	Specimen designation for steel models	Nominal aspect ratio (breadth/depth) (b/h)	Nominal web slenderness (depth/thickness) (h/t)	Web dimensions (nominal) (mm) $b \times h \times t$	Flange dimensions (nominal) (mm)	Test span (mm)
A1/1	S1/1	1.0	200	200 × 200 × 1	80 × 3	400
A2/1	S2/1	1.0	250	250 × 250 × 1	80 × 3	500
A3/1	S3/1	1.0	300	300 × 300 × 1	80 × 3	600
A4/1	S4/1	1.0	350	350 × 350 × 1	80 × 3	700
A5/1	S5/1	1.0	400	400 × 400 × 1	80 × 3	800
A1/2	S1/2	1.5	200	300 × 200 × 1	80 × 3	600
A2/2	S2/2	1.5	250	375 × 250 × 1	80 × 3	750
A3/2	S3/2	1.5	300	450 × 300 × 1	80 × 3	900
A4/2	S4/2	1.5	350	525 × 350 × 1	80 × 3	1050

universal beams and box sections. The supports were suitably designed to ensure that the girders would not develop lateral instability; for this purpose, lateral roller supports were introduced to prevent torsional buckling. The load was applied hydraulically with an Ovec 200 kN jack, and an Ovec 50 kN jack, for steel and Araldite models, respectively. Load cells were used to monitor the applied loads using a Gemini digital voltmeter.

The deflections of the Araldite tests were obtained with dial gauges of 0.01 mm accuracy. The profiles of the flanges were obtained by using a low voltage, linear displacement transducer connected to an x–y plotter. The profile of the web was obtained by taking transducer measurements at various points for each load increment. The flange and web profiles of the steel models were recorded before and after the tests. The vertical deflections of the models were noted during the test at various load increments at three positions, viz., at the centre and at quarter points.

Tension tests were carried out on coupons cut from the steel flange and web materials using an Avery Denison machine (See Table 2).

Tensile specimens were also cut from the Araldite sheets and machined to suitable sizes. The tests were carried out with a very low straining rate with the Avery Denison machine and the load–extension curves were drawn by the x–y plotter direct from the testing machine. It was found that the yield zone is very small indeed and the failure occurred very soon after the peak load.

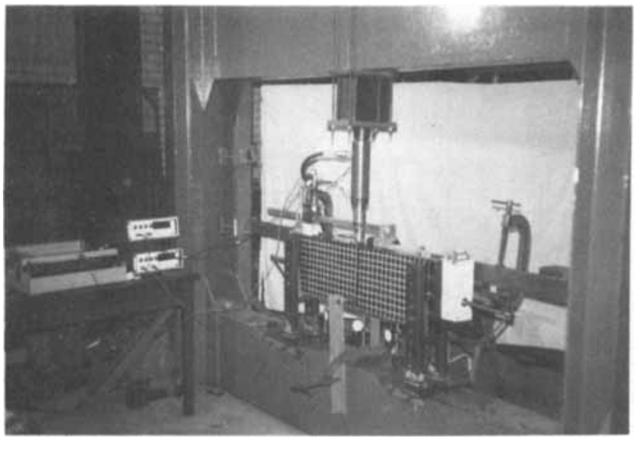

Fig. 3.

Table 2. Material properties

Material tested		Static yield stress (N/mm^2)	Dynamic yield stress (N/mm^2)	Ultimate stress (N/mm^2)	Modulus of elasticity (kN/mm^2)
Steel models	Coupons from flange material (Tension test)	295	305	444	197
	Coupons from web material	169	175	311	200
Araldite models	Coupons from web and flange (tension test)	—	—	49	2.25
	Cylinders tested in compression	—	—	73	2.30

Compression tests on Araldite were carried out using cylindrical specimens of length to diameter ratio 2 : 1. These specimens were made simultaneously with the casting of Araldite sheets so that the proportions of the Araldite mixture, and the curing procedure, were the same as for the sheets. The compression tests were also carried out in the Avery Denison machine with an x–y plotter. From the load–axial shortening relationship, the compressive yield strengths and modulus of elasticity for Araldite were calculated.

The properties of the materials used in the models are reported in Table 2.

6 TESTS ON ARALDITE MODELS

The test girders were mounted on roller supports and the lateral roller supports adjusted to prevent torsional instability.

Using an Ovec 50 kN jack and a 50 kN load cell, the load was applied at the centre of the test girder and monitored by a Gemini digital voltmeter. The lateral distortions of the web panel and the profile of the top flange were recorded after each load increment. In addition, three dial gauges placed at the mid-points of the web panels and directly under the central stiffener were also read at every increment of load. The *transducers* were removed at the sounds of the Araldite 'crackling'. Thereafter the load increment was continued until failure and the failure load was recorded.

Figure 4 shows a typical load–deflection relation observed at the three locations for an Araldite model. In all

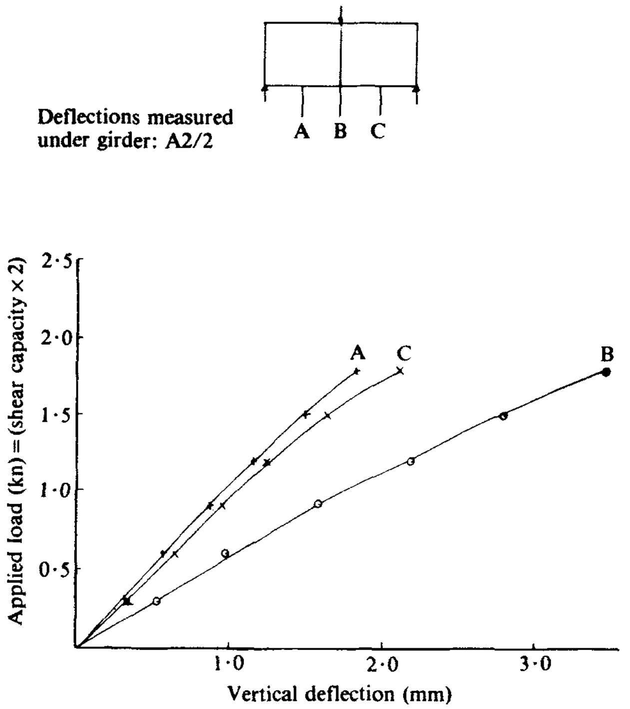

Fig. 4. Load/vertical deflection relationship

the tests, the load–central deflection was linear until collapse. The webs of girders developed very large deflections before collapsing abruptly. No visible hinge formation was observed before collapse, although the pattern of buckles was clearly visible.

7 TESTS ON STEEL MODELS

The rig used to apply loads to the Araldite models was also used for tests on steel models but an Ovec 200 kN jack and an appropriate load cell were employed. Only one half of the girder was tested, by stiffening the web which was not under test. (The reason for stiffening the panel not under test was to enable another test to be carried out on that panel subsequently; this procedure could not be followed with the Araldite models since they shattered at failure).

A load increment of 250 N was used when testing steel models; measurements of deflections were made at midspan and quarter span of each girder. After the hinge development, indicating the commencement of failure, further loading was continued to develop the hinges fully. The final transducing of the web distortions and the upper and lower flanges were carried out at the end of the test.

Figure 5 shows a typical load–central deflection relation observed at the two locations referred above for a steel model. In all these tests, the load–central deflection reached a peak value asymptotically, as the tests were conducted in the load control mode. The relationships were found to be linearly elastic for a substantial part; near the peak load, the vertical deflections increased rapidly.

The girders did *not* exhibit any noticeable difference in behaviour at loads corresponding to the elastic critical stresses of the webs in shear; in any case, these loads were very low in comparison with the ultimate loads.

The deflections at the mid points of the webs were also observed to increase linearly for a major part of the load

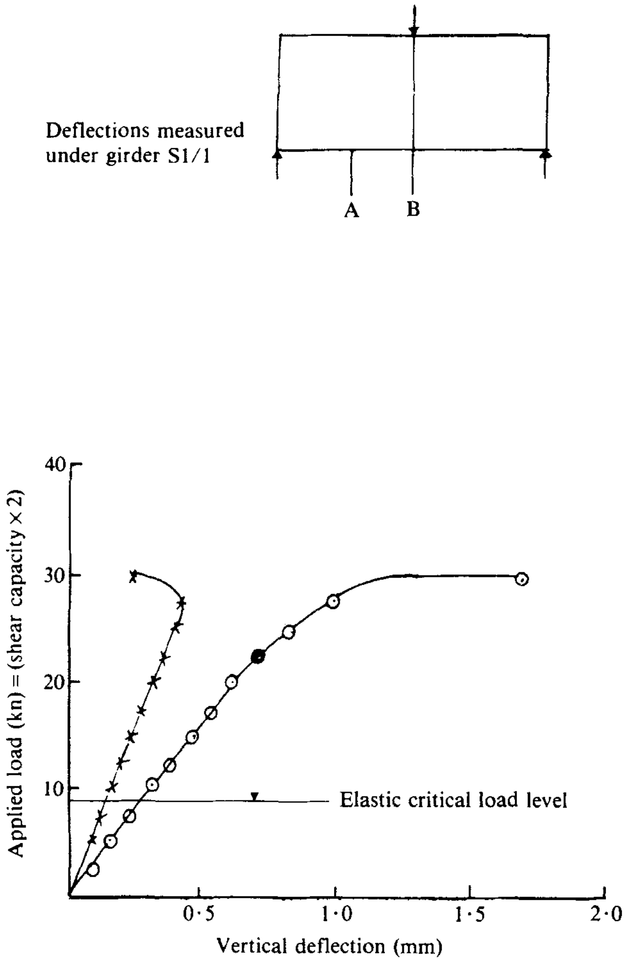

Fig. 5. Deflections measured on S1/1

increment and when the plastic hinges started developing in the flanges, these deflections started to reduce.

After the test, the distances between the hinges formed at the top and bottom flanges were measured from the flange profiles. A photograph of a typical steel girder after test is seen in Fig. 6.

The clearly visible formation of large web buckles with the simultaneous development of hinges in the top and bottom flanges, followed by the ultimate collapse of the girder, were characteristics which were similar to those observed by the earlier investigators in large scale tests (2). In other words, the steel models under test were observed to behave in exactly the same manner as full scale girders.

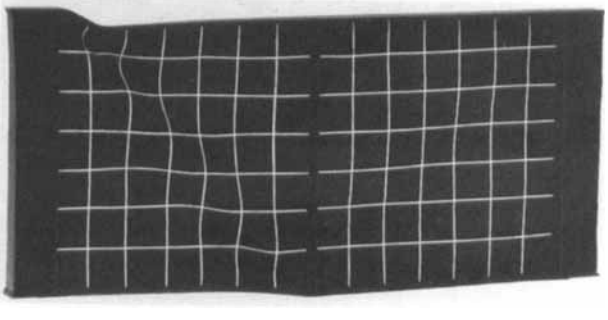

Fig. 6.

8 COMPARISON OF PREDICTED STRENGTHS WITH THE OBSERVED VALUES IN STEEL MODELS

For each of the steel models tested, strengths were predicted using the different methods, given in references **(1)–(10)**, and these have been compared with the experimentally observed values in Table 3. The hinge distances predicted by each of these methods are compared with the values observed in the test specimens in Table 4. In some of these methods, the plastic hinges are assumed to form at standard locations. The Table incorporates these distances and compares with the experimentally observed values.

Of the models studied, the Aarau method **(1)** was seen to be moderately conservative in predicting the ultimate capacities. The predicted values using the Prague–Cardiff method **(7)** and the Zurich Method **(10)** when compared with the observed results exhibited a scatter over a wide band and the standard deviation was very high in both cases. The Karlsruhe method **(4)** was unconservative and also had a wide scatter band. The Lehigh method **(5)** gave consistently higher predicted strengths compared with the observed values, although only by a small margin. There is little difference in the accuracy of prediction obtained by using any of the five methods, viz., the Cardiff **(2)**, Gothenburg **(3)**, Osaka **(6)**, Stockholm **(7)**, and Tokyo **(9)** methods; when compared with the experimental results, all the predictions were found to be within a small scatter band. Thus, as far as consistency is concerned, there appears to be no reason for preference between these five methods.

A similar calibration of the design methods against available full-scale tests is reported in reference **(14)** and reproduced in Table 3. It will be seen that the calibration of full-scale tests gave results which are very similar to those obtained on models.

From Table 4 it can be seen that the Gothenburg method **(3)** appears to give closer estimates of hinge distances compared with the observed values. Once again it will be seen that the Cardiff **(2)**, Gothenburg **(3)** and Osaka methods **(6)** give solutions of acceptable accuracy.

9 COMPARISON OF PREDICTED STRENGTHS WITH OBSERVED VALUES IN ARALDITE MODELS

Similar comparisons of predicted and measured values of ultimate capacities for Araldite specimens will be found in Table 5. No hinges were observed in the tests on Araldite specimens and as stated before, these specimens shattered at peak loads; hence no comparisons of hinge distances are tabulated.

With Araldite models a small applied load gives large deflections, thus giving a visual indication of the displacement patterns. The Araldite girders tended to deflect laterally, so careful positioning of the lateral supports at the central stiffeners was particularly important; otherwise the girders would become torsionally unstable.

The patterns of lateral deflections of the Araldite girders were much better developed visually compared with the steel model girders. The reason for this is that large strains are possible in Araldite (i.e., 35 000 microstrains compared with about 1500 microstrains in steel); hence, the formation of the large buckles could be observed clearly. The actual formation of the hinges in the flanges could not be seen as the material has very little plasticity; however, the likely locations of the hinges could be inferred from the large deflections which were apparent in the profiles of the flanges.

All the models exhibited the same pattern of behaviour, viz, large elastic deflections of the girder showing well developed buckles in the web under applied loading. The profiles of flanges indicated the pulling *down* of the upper flanges and the pulling *up* of the lower flanges by the tension field. All these characteristics were elastic and repeatable after unloading. Increased loading resulted in buckles of increased magnitude and indicated the potential for the development of hinges in the flanges, if the material had plastic characteristics. However, due to the lack of plasticity, the hinges could not develop and sudden failure occurred by brittle fracture.

The predicted failure loads using the various theories presented in Table 5 demonstrate that these theories cannot be used to compute the ultimate shear for models made of Araldite. This is because these methods do not accommodate the brittle behaviour (such as the one exhibited by Araldite) in terms of material properties. These models continue to be in the elastic range until collapse on account of the large range of elastic strain.

10 CONCLUSIONS

The tests demonstrate clearly that small scale steel models can be employed successfully to simulate the post-buckling behaviour of plate girders and to obtain an estimate of the collapse load. The pattern of collapse of the models was the same as that observed by earlier investigators on large scale girders. Many prevailing theories for the prediction of ultimate strengths of full scale girders are also found to be applicable to the steel model girders.

Small scale steel models are shown to be quite suitable for simulating plate behaviour in spite of the limitations imposed by welding shrinkage stresses. Hence there is a wide scope for experimental studies on plates containing holes, plates with reinforcement and plates subjected to complex system of loads. These would provide experimental verification for a range of problems for which only theoretical solutions are available currently, and assist in the formulation of new theories based on the experimental observations.

Even though models made of Araldite (or similar brittle materials) cannot be employed to obtain an estimate of the collapse load, they develop large elastic deformations under relatively small loads. This characteristic can be successfully employed to simulate visually the post buckling behaviour and the patterns of buckling. This could be an advantage when investigating a complex arrangement of stiffeners.

ACKNOWLEDGEMENT

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Table 3. Comparison of strength predictions by various methods

Girder No.	Observed shear at collapse (kN)	$\frac{\text{Predicted load}}{\text{Observed load}}$ using the various design methods: Aarau method (1)	Cardiff method (2)	Gothenburg method (3)	Karlsruhe method (4)	Lehigh method (5)	Osaka method (6)	Prague–Cardiff method (7)	Stockholm method (8)	Tokyo method (9)	Zurich method (10)	Theoretical value for elastic critical load (kN)
S1/1	15.0	0.772	1.025	0.951	1.180	1.061	1.136	1.053	1.076	1.033	1.181	8.45
S2/1	18.5	0.744	0.905	0.878	1.151	0.992	0.976	0.801	0.907	0.932	0.949	6.76
S3/1	19.0	0.767	0.930	0.930	1.322	1.063	0.958	0.739	0.918	0.910	0.916	5.63
S4/1	21.0	0.745	0.899	0.916	1.339	1.062	0.907	0.65	0.861	0.870	0.821	4.83
S5/1	23.0	0.725	0.871	0.963	1.200	1.065	0.872	0.56	0.786	0.838	0.737	4.22
S1/2	12.5	0.844	1.004	0.960	1.402	1.178	1.178	1.12	1.100	1.180	1.234	6.44
S2/2	15.5	0.831	0.853	0.929	1.365	1.037	0.950	1.11	0.962	0.973	0.975	5.16
S3/2	16.0	0.867	0.858	0.990	1.537	1.086	0.925	0.79	0.969	0.918	0.927	4.30
S4/2	13.0	0.859	0.805	0.966	1.559	1.052	0.844	0.67	0.867	0.828	0.811	3.68
Mean $\left(\frac{\text{predicted load}}{\text{observed load}}\right)$		0.795	0.906	0.943	1.339	1.066	0.972	0.833	0.938	0.942	0.950	
Standard deviation		0.055	0.072	0.033	0.146	0.049	0.113	0.210	0.101	0.110	0.165	
Mean value of $\frac{\text{predicted load}}{\text{observed load}}$ for large scale tests		0.99	1.02	0.99	0.97	1.02	1.07	0.98	0.96	0.95	0.79	
Standard deviation		0.20	0.06	0.12	0.14	0.12	0.17	0.08	0.11	0.15	0.25	

Table 4. Predicted distances between plastic hinges compared with measured distances

Specimen No.	Measured distance between the hinges (mm)	Ratio of $\frac{\text{Predicted hinge distance}}{\text{Observed hinge distance}}$ using the various design methods: Aarau method (1)	Cardiff method (2)	Gothenburg method (3)	Karlsruhe method (4)	Lehigh method (5)	Osaka method (6)	Prague–Cardiff method (7)	Stockholm method (8)	Tokyo method (9)	Zurich method (10)
S1/1	88	0.80	0.75	0.66	Hinges	Hinges	0.44	0.70	0.61	Hinges are	1.16
S2/1	65	1.16	0.94	1.02	are	are	0.80	0.75	1.68	assumed	1.85
S3/1	72	0.89	0.76	0.89	assumed	assumed	0.80	0.58	1.61	to form	1.38
S4/1	68	0.81	0.76	0.98	to	to	0.97	0.61	1.29	at the	1.45
S5/1	60	0.99	0.87	1.15	occur	occur	1.26	0.66	1.67	middle	1.65
S1/2	84	0.75	0.94	0.76	at the	at the	0.85	1.39	0.96	of the	1.62
S2/2	92	0.55	0.76	0.73	ends	ends	0.78	0.93	1.66	panel and	1.41
S3/2	96	0.41	0.72	0.74	of the	of the	0.89	0.89	1.20	ends of	1.35
S4/2	80	0.41	0.84	0.92	panels	panels	1.24	0.97	1.66	girder	1.61
Mean ratio		0.72	0.82	0.98			0.89	0.83	1.17		1.48
Standard deviation		0.23	0.08	0.15			0.24	0.25	0.33		0.18

Table 5. Ultimate shear capacity of Araldite models

Girder No.	Predicted ultimate shear capacity (kN)*										Observed shear at collapse* (kN)	Theoretical value for elastic critical load (kN)
	Aarau method	Cardiff method	Gothenburg method	Karlsruhe method	Lehigh method	Osaka method	Prague–Cardiff method	Stockholm method	Tokyo method	Zurich method		
A1/1	3.26	2.86	3.33	0.31	3.19	2.72	3.14	1.34	2.45	1.60	0.80	0.095
A2/1	3.50	3.29	3.83	0.32	3.92	3.11	2.13	1.28	2.90	1.57	0.95	0.076
A3/1	3.76	3.73	4.34	0.36	4.65	3.55	2.11	1.23	3.34	1.54	1.80	0.063
A4/1	4.67	4.17	4.85	0.39	5.39	4.00	2.15	1.21	3.81	1.52	0.95	0.054
A5/1	4.43	4.61	5.36	0.37	6.14	4.48	2.05	1.18	4.28	1.51	1.05	0.047
A1/2	1.95	2.74	2.91	0.29	2.59	1.99	2.16	1.11	1.82	1.30	0.75	0.072
A2/2	2.16	2.46	3.41	0.30	3.18	2.28	2.06	1.06	2.12	1.27	1.00	0.058
A3/2	2.43	2.78	3.93	0.34	3.78	2.60	2.10	1.03	2.45	1.25	0.90	0.048
A4/2	2.75	3.10	4.45	0.37	4.36	2.94	2.08	1.01	2.79	1.24	0.90	0.041

* Ultimate shear capacity = 0.5 × peak load

APPENDIX

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