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Behavior of the MR Sandwich Panel in Flexure

by F.H. Fouad, J. Farrell, M. Heath, A. Shalaby, and A. Vichare

Synopsis: The housing industry is a critical component of the American economy representing about 4% of the economic activity of the nation. Light weight structural insulated panels (SIP) for walls and roofs are gaining wide acceptance in the construction industry because of the advantages they offer. Energy savings, sound abatement, disaster resistance and durability are just a few of the benefits of buildings constructed with SIP. A variation of these panels is a structural concrete insulated panel (SCIP), commercially referred to as the MetRock (MR) Panel system. The aim of this paper was to study the flexural behavior of the SCIP system and discuss the manufacturing and construction aspects of the SCIP system. An analytical method for estimating the panel's flexural strength was developed and a step-by-step design procedure is provided to predict the load carrying capacity of the panels and provide the engineer with a reliable tool for designing the panels. An experimental program was conducted and the results were compared to the analytical method for different size panels. The test results were in close agreement with the estimated values thus verifying the validity of the analytical approach.

Keywords: easy construction; energy saving; flexural behavior; sandwich panel; shotcrete

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INTRODUCTION

Structural insulated panels (SIPs) are a composite building material consisting of a sandwich of two layers of structural board with an insulating layer of foam in between. The board is usually plywood or oriented strand board and the foam either expanded polystyrene foam (EPS), extruded polystyrene foam (XPS) or polyurethane foam. The insulating core and the two skins of a SIP are nonstructural, but when pressure-laminated together under strictly controlled conditions, a composite material that has structural properties is created. SIPs are used primarily in residential and light commercial applications as exterior wall, roof, and floor systems. They replace components of the conventional building such as studs and joists, insulation, vapor barrier and air barrier. Arriving precut to the jobsite, the panels can be rapidly assembled by workers without extensive training. Overall, the SIP construction method allows for rapid erection of an exterior building envelope that is strong, airtight, and energy efficient.

Another variation of the SIP is a structural concrete insulated panel (SCIP), commercially referred to as the MetRock (MR) Panel system, which can be used for wall, floor, and roof applications. This SCIP system is a steel trussed sandwich concrete panel made of expanded polystyrene foam, flanked by galvanized wire mesh on both sides, and connected with galvanized vertical steel wire trusses spaced at 6 in. (150 mm). The assembly is then coated on site with 1 in. (25 mm) of Portland cement plaster by low-velocity shotcreting on both sides to form a composite panel. The advantages of the SCIP system over other types of sandwich panels that are currently in the market include the ease of manufacturing; the reduction of construction time and cost; the production of structural elements with greater structural integrity; and lower risk in fire and termite attack since the skins are not a wood product. A unique advantage of the SCIP system is its lower cost because it does not require any complicated machinery for construction. A sketch showing the reinforcement details is given in Figure 1, and Figure 2 shows a typical SCIP.

This paper discusses an experimental program that was conducted to study the flexural behavior of SCIP system. The results of the test program were used to refine the analytical procedures that were developed to predict the panel strength.

RESEARCH SIGNIFICANCE

For the MR structural concrete insulated panel (SCIP) to be widely used in the construction industry and accepted by the design engineer, a reliable design procedure for predicting the panel strength must be developed. A test program was planned and conducted to provide a basic understanding of the flexural behavior of the panel and to refine the analytical procedures that were developed to predict the panel's flexural strength. This paper provides a summary of the proposed design procedure and discusses manufacturing and construction aspects of the SCIP system.

CONSTRUCTION USING MR PANELS

The SCIP system offers a fast, economical and easy construction system that gives a finished concrete wall surface with improved structural properties and thermal insulation. The panels are delivered with only the foam and the galvanized steel reinforcement tied together on flat bed trucks. A standard panel width of 4 ft. (1.2 m) is commonly used for ease of transportation and handling, but wider panels are sometimes designed to accommodate door and window openings. They can be stored on a flat surface outdoors without protective covering for several weeks as shown in Figure 3.

A system of metal anchors is required to be placed in the wall footing or slab to secure the panel bases as seen in Figure 4. Usually #3 (10 mm) reinforcing bar 12 to 18 in. (300 to 450 mm) above the slab surface is adequate for anchoring the panel. When panels are to be placed on existing slabs, holes may be drilled on-line to house lengths of reinforcing bar dowels. The panels are usually placed so that the reinforcing bar is set between the mesh and the polystyrene. Adjacent panels are clamped together manually with wire ties. Strips of wire mesh are used to reinforce panel seams and corners. Bracing should be located on the same side of the wall opposite the side which will receive the shotcrete.

Utility conduits can be run in the gap between the foam and the wire mesh; if more space is needed the foam can be removed. Openings for doors and windows can be cut out both before or after panel erection. Openings must be engineered to ensure that structural integrity is not jeopardized. Pieces of pressure treated lumber can act as a hidden frame and stapled to the surrounding mesh becoming a secure nailer for the door or window frame opening.

Shot-in-place columns or beams may be added in the SCIP panel system to strengthen a certain area. This is accomplished by removing the foam, providing additional steel reinforcement if required, and applying the low-velocity shotcrete.

The panels are then covered with shotcrete concrete as seen in Figure 5 on both sides making a composite panel. Concrete thickness varies according to design, but usually it is twice the distance between the face of the polystyrene and the wire mesh. Desired finishing can then be achieved by mortar if necessary.

EXPERIMENTAL PROGRAM

Ten full-scale specimens were tested to study the flexural behavior of SCIP system. The ten specimens were divided into four groups; each group had a different span-to-depth ratio. All the specimens tested were 24 in. (600 mm) wide. The first two groups consisted of three specimens each with a span-to-depth ratio of 8.73 and 16, respectively. The other two groups consisted of two specimens each with a span-to-depth ratio of 20 and 32, respectively. The test specimen dimensions are given in Table 1.

The shotcrete on each side of the panel was 1 in. (25 mm) thick and was reinforced with 14 gauge wire mesh of 1 x 1 in. (25 x 25 mm), centered in the cementitous wall, with ½ in. (13 mm) concrete cover. The steel trusses were made of 3/16 in. (5 mm) diameter wire spaced at 6 in. (150 mm) center to center longitudinally. The top and bottom chords of the steel trusses were embedded for about ½ in. (13 mm) into the shotcrete thickness. Steel truss wire used in the panels was tested in tension. Table 2 gives the results of the steel wire tensile tests.
Figure **1** shows a typical sketch of a SCIP depicting the steel reinforcing scheme.

Wet-mix shotcrete was used in general conformance to ACI 506R "Guide to Shotcrete"[1] to build the concrete of the SCIP system. Shotcrete control compression test specimens, 3 x 4 in. long cores (75 x 100 mm), were cut from 12 x 12 x 4 in. (300 x 300 x 100 mm) shotcrete samples made during construction of the test specimens and represented the shotcrete used in the test panels. The cores were cut and tested in compression in accordance to ASTM C 42 [2] to determine the actual compressive strength of the shotcrete used in the panels. The results of the control cores showed an average compressive strength at 28 days of 4405 psi (30 MPa). The shotcrete mixture proportions used are given in Table 3.

The test specimens were tested under 2-point loading according to ASTM E 72 [3], with load points located at quarter points of the panel span. The panels were oriented horizontally and seated on roller supports. The marked supports lines were aligned with the rollers. Bearing plates with rubber pads were used to prevent crushing of the material at the loading points and supports. A spreader beam was used to distribute the load into two point loads on the panel. A 10 kip (44 kN) load cell was used for measuring the applied load and a linear displacement potentiometer (LVDT) was used to measure the deflection at the centre of the panel. The weight of the spreader beam, roller supports and loading hardware acting on the panel was measured and accounted for in the calculations. The self weight of the panels was also measured and considered in the calculations. The span length of the panels, measured between the roller supports, was 4 in. (100 mm) shorter than the overall panel length. The test setup is shown in Figure 6. For some selected specimens, the concrete and the steel wire were strain gauged to obtain data that would further verify the load transfer mechanism. Figure 7 shows a schematic diagram of the strain gauge locations.

The load was applied to the panels in increments of about 1000 lbs (4.5 kN). There was a pause after each load increment application to allow time to check for the development of any cracks in concrete, measure crack widths, and inspect for any structural distress that might have occurred. The load deflection curve of panel 3R with a span-to-depth ratio of 8.73 is shown in Figure 8. Figure 9 shows the load deflection curve of panel 1R with a span-to-depth ratio of 16. It could be seen from the load deflection curves that the panels are ductile with a fairly large range of non-linear behavior. It could also be seen that the deflection is significantly affected by the span-to-depth ratio of the panels. The deflection at failure for panel 3R with span-to-depth ratio of 8.73 was 3.02 in. (77 mm), while for panel 1R with a span-to-depth ratio of 16, the deflection at failure was 1.38 in. (35 mm).

During testing the panels, the truss diagonals nearest the supports buckled in compression at an early stage of the test, but due to the internal redundancy of the shear transfer mechanism, redistribution of shear occurred allowing the panel to continue resisting load. Shear cracks were observed in the foam in panels 8R and 9R, with a span-to-depth ratio of 32, which indicates that the foam failed in shear. In panels 3R and 4R and 11R, with a span-to-depth ratio of 8.73, there were no shear cracks observed in the foam. The foam was seen to be compressed at the ends which lead to distortion of the structure and a highly localized failure was observed. Also, it was observed that the top concrete flange was relatively straight as compared with the bottom flange. From these observations, it could be concluded that the span-to-depth ratio significantly affects the failure mode of the panels with the same concrete flange thickness. Examples of the failure mode of the panels are shown in Figure 10 and Figure 11. The peak (failure) test loads resisted by the panels are given in Table 4. The peak load includes the weight of the loading hardware. The actual test loads were recorded and the weight of the loading hardware given in Table 4 was added to get the load carrying capacity of the panels.

The strains were recorded from the strain gauges attached to panel 3R with a span-to-depth ratio of 8.73. The maximum compressive strain in concrete was recorded as 0.000178. This value conforms to the visual observation that there was no crushing of concrete observed at the top of the panel. The maximum tensile strain in the truss bottom chord was 0.00048. The maximum tensile and compressive strains in the truss diagonals were 0.0013 and 0.0017, respectively. These high stresses conform to the visual observation that the truss diagonals were buckling in compression.

ANALYTICAL APPROACH

Sandwich panels in general have a different structural behavior than conventional solid reinforced concrete panels. The behavior can be broadly classified into three possible types: fully composite, semi-composite and non-composite panel behavior.

For the fully composite behavior, the flexural design of the panel is similar to that of a solid panel that has the same cross-sectional dimensions. Sandwich panels whose wythes (concrete layers) are connected in such a way that both wythes resist applied flexural loads as if they were an integral section are said to be fully composite panels. In this case the connectors must transfer the required longitudinal shear (horizontal shear) so that the elastic bending stress distribution on the cross-section of the panel is as shown in Figure 12. This theory is the same as that applied for regular solid reinforced concrete beams.

Sandwich panels in which the two wythes are connected with elements (connectors) that have no capacity for longitudinal shear transfer are said to be non-composite panels. The flexural design of non-composite sandwich panels is identical to that of solid panels that have the same cross-sectional dimensions as the structural wythes of the non-composite panels. One additional consideration is the distribution of loads between the wythes for a panel that has two structural wythes. The distribution of loads is based on the relative flexural stiffness of each wythe. Once this distribution is made, each wythe is then individually designed as a solid panel. In the case of SCIP system, the two layers of concrete have the same flexural stiffness. If the two concrete wythes are of equal stiffness and reinforcement, each wythe resists 50% of the load and the elastic bending stress distribution is as shown in Figure 13.

Sandwich panels in which the connectors can transfer between zero and 100 percent of the longitudinal shear required for a fully composite panel are said to be semi-composite panels. The elastic bending stress distribution in this type of panel is shown in Figure 14.

The PCI approach [4] for the semi-composite sandwich panel assumes that such panels behave both as a fully composite panel and a non-composite panel at different stages in the life of the panel. In one design approach, the panel is considered to act fully composite during construction phase. The assumption of initial fully composite action is based on past observations of the panel during stripping, handling and erection. After the panel is erected, a conservative approach of not relying on horizontal shear transfer to resist service loads is taken. The initial composite action (horizontal shear transfer) is attributed to bond between the concrete and insulation plus any contribution of the wythe connectors. Another approach is to assume something less than full composite action for stripping, handling and/or for the full life of the panel.

The maximum flexural capacity of the tested specimens was calculated using the ACI 318-05 [5] assuming both, fully composite behavior, and non-composite behavior. The test results were compared with the composite and non-composite capacity calculations. Table 5 shows the composite and non-composite capacities for the different groups of test specimens and the average test results. A graph showing the load-carrying capacity of the composite, non-composite, and actual test result for the four panel groups is shown in Figure 15. As seen from the test data, the actual behavior falls in between the composite and non-composite capacities, which indicates a semi-composite behavior of the panel under ultimate loads.

The actual capacity of the semi-composite panels was calculated as a percentage of the fully composite capacity. A calibration factor was developed to take this into account. It was observed that the calibration factor varies according to the span-to-depth ratio of the panels. Therefore the graph was plotted as span-to-depth ratio versus the load (Figure 15). Regression analysis was performed to find out the best fitting curve for the plotted values, and it was found that the power regression model is the best fit for the available data with a coefficient of determination of about 99% and 91% for the fully composite capacity and the test results, respectively. The strength calibration factor (SCF) equation was calculated by dividing the test results equation with the composite capacity equation. The resulting equation is given as follows:

$$\text{Strength Calibration Factor (SCF)} = 0.182\ (\text{span-to-depth ratio})^{0.1515}$$

A plot of the SCF versus the span-to-depth ratio is shown in Figure 16. From this figure, it could be seen that the SCF varies from 0.24 to 0.32 for the panels tested, and the ratio increases as the span-to-depth ratio increases. Using the SCF equation, a factor can be calculated for any given panel of a specific span and thickness. This factor can be used to predict the actual flexural strength of the panel by multiplying the composite capacity of the panel by the SCF.

To account for materials variability, a flexural capacity reduction factor of 0.85 is used to further reduce the estimated flexural strength. Table 6 gives the flexural strength calculated for the panels, the strength calibration factor, and the test results.

Figure 17 shows a comparison of the theoretical calculations and actual test results. It is evident that the calculated (estimated) flexural capacity is conservative as compared to the actual test results.

PROPOSED DESIGN APPROACH FOR MR PANELS

Based on the study results, a proposed step-by-step design approach for the SCIP system in flexure may be summarized as follows:

Step 1 – The fully composite flexural capacity of the panel is calculated assuming a regular reinforced concrete solid panel. The steel wire reinforcement and the truss bottom chords are assumed to be in tension and have yielded.
Step 2 – The strength calibration factor (SCF) is calculated for the specific span-to-depth (S/d) ratio of the panel using the following equation:

$$SCF = 0.182\ (S/d)^{0.1515}$$

Step 3 – The SCF factor is multiplied by a material safety factor of 0.85.

Step 4 – The design flexural strength of the panel is calculated by multiplying the values in Steps 1 and 3.

LOAD CARRYING CAPACITY

The maximum service uniform distributed live load was calculated for the tested panels from the ultimate failure load, and by using load factors of 1.6 and 1.2 for live and dead loads respectively according to the ACI 318-05 [5]. A flooring load of about 15 psf (0.72 kPa) was also assumed. The calculated service uniform live loads for the panels are shown in Table 7. ASCE –7 [6] indicates that the minimum uniform distributed live loads for residential buildings vary from 10 to 40 psf (0.50 to 2.00 kPa) except stairs and balconies. Table 7 shows that the tested panels have service live load capacity in excess of 40 psf (2.00 kPa), except for panels 8R and 9R with span-to-depth ratio of 32. It should be noted that the latter span-to-depth ratio is quite large for typical reinforced concrete structures, and the lower service live load capacity should be expected. Therefore, it is recommended to use a panel span-to-depth ratio of about 20 or less to ensure a reasonable amount of service live load capacity.

SUMMARY AND CONCLUSIONS

The SCIP system presented in this paper offers numerous advantages over other types of SIPs. The advantages include the ease of manufacturing; the reduction of construction time and cost; and the production of structural elements with greater structural integrity. A unique advantage of the SCIP system is its lower cost because it does not require any complicated machinery for construction. An experimental program was performed to study the flexural behavior of the SCIP system. Analytical procedures were developed and compared to the experimental test results. The results of the study indicated that the SCIP system exhibits a semi-composite behavior under ultimate loads. A strength calibration factor (SCF) was derived by comparing the test data to theoretical calculations based on a fully composite behavior according to ACI 318 Code. This factor, which accounts for the partial composite behavior, may be used to estimate the ultimate flexural strength of the SCIP system.

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Table 1 —– Test specimen dimensions

Group No.	Span-to-depth ratio	Specimen ID	Width (ft.) [mm]	Span (ft.) [mm]	Thickness (in.) [mm]
Group 1	8.73	3R	2 [600]	8 [2400]	11 [280]
		4R	2 [600]	8 [2400]	11 [280]
		11R	2 [600]	8 [2400]	11 [280]
Group 2	16	1R	2 [600]	8 [2400]	6 [150]
		2R	2 [600]	8 [2400]	6 [150]
		13R	2 [600]	8 [2400]	6 [150]
Group 3	20	5R	2 [600]	10 [3000]	6 [150]
		6R	2 [600]	10 [3000]	6 [150]
Group 4	32	8R	2 [600]	16 [4800]	6 [150]
		9R	2 [600]	16 [4800]	6 [150]

Table 2 — Steel wire tensile tests

Steel type	Sample number	Average diameter (in.) [mm]	Tensile strength (psi) [MPa]	Average tensile strength (psi) [MPa]
Steel wire mesh	Sample # 1	0.1000 [2.5]	93,500 [645]	92,333 [635]
	Sample # 2	0.1000 [2.5]	91,000 [625]	
	Sample # 3	0.1000 [2.5]	92,500 [640]	
Steel truss wire	Sample # 1	0.1820 [4.55]	105,937 [730]	112,965 [780]
	Sample # 2	0.1700 [4.25]	122,478 [845]	
	Sample # 3	0.1735 [4.34]	110,480 [760]	

Table 3 — Wet shotcrete mixture proportions

Water cement ratio w/cm ratio	Portland cement Type I/II (weight %)	Concrete sand (weight %)
0.45	0.23	0.77

Table 4 — Failure loads and ultimate moments of the tested specimens

Panel	Span-to-depth ratio	Weight of load hardware (lb) [kN]	Self weight of panel (lb) [kN]	Peak Applied Load (lb) [kN]	Ultimate mid-span moment (lb.ft) [kN.m]	Failure mode
3R	8.73	129 [0.60]	453 [2.00]	6053 [27]	5801 [7.90]	Horizontal shear, End Region
4R	8.73	129 [0.60]	470 [2.10]	6043 [27]	5791 [7.85]	Conc. Load at Support
11R	8.73	127 [0.60]	544 [2.40]	6417 [29]	6673 [9.05]	Horizontal shear, End Region
1R	16	120 [0.50]	446 [2.00]	4003 [18]	3836 [5.20]	Horizontal shear, End Region
2R	16	104 [0.50]	467 [2.10]	4998 [22]	4790 [6.50]	Horizontal shear, End Region
13R	16	127 [0.60]	500 [2.20]	4525 [20]	4820 [6.55]	Horizontal shear, End Region
5R	20	130 [0.60]	578 [2.60]	2534 [11]	3062 [4.15]	Horizontal shear, End Region
6R	20	130 [0.60]	595 [2.70]	2754 [12]	3328 [4.50]	Horizontal shear, End Region
8R	32	155 [0.70]	1027 [4.60]	2267 [10]	4440 [6.00]	Horizontal shear, End Region
9R	32	155 [0.70]	974 [4.30]	2183 [10]	4275 [5.80]	Horizontal shear, End Region

Table 5 — Average test results of the SCIP system with their composite and non-composite capacities

Group No.	Panel length (ft) [mm]	Panel depth (in.) [mm]	Span-to-depth ratio	Non-composite capacity (lb) [kN]	Composite capacity (lb) [kN]	Average test result (lb) [kN]
Group 1	8 [2400]	11 [280]	8.73	1454 [6.50]	26107 [116]	6171 [28]
Group 2	8 [2400]	6 [150]	16	1459 [6.50]	13342 [59]	4509 [20]
Group 3	10 [3000]	6 [150]	20	950 [4.20]	10360 [46]	2644 [12]
Group 4	16 [4800]	6 [150]	32	-31.04 [-0.15]	7375 [33]	2225 [10]

Table 6 — Maximum theoretical flexural capacity of SCIP system calculated using SCF

Group No.	Span-to-depth ratio	Average test results (lb) [kN]	Composite nominal capacity (lb) [kN]	SCF	SCF x 0.85	Design capacity* (lb) [kN]	Design / test
Group 1	8.73	6171 [28]	26107 [116]	0.252	0.215	5613 [25]	0.91
Group 2	16	4509 [20]	13342 [59]	0.277	0.235	3135.5 [14]	0.70
Group 3	20	2644 [12]	10360 [46]	0.287	0.244	2528 [11]	0.96
Group 4	32	2225 [10]	7375 [33]	0.308	0.262	1932 [8.60]	0.87

* Design capacity = Composite capacity x SCF x 0.85

Table 7 — Service uniform distributed load of the tested specimens

Group No.	Span-to-depth ratio	Ultimate moment (lb.ft) [kN.m]	Average weight of panel (lb) [kN]	Factored weight (psf) [kPa]	Flooring (psf) [kPa]	Service load (psf) [kPa]
Group 1	8.73	6088 [8.25]	489 [2.18]	38 [1.82]	15 [0.72]	223 [11]
Group 2	16	4482 [6.08]	471 [2.09]	37 [1.77]	15 [0.72]	156 [7.5]
Group 3	20	3195 [4.33]	587 [2.61]	36 [1.72]	15 [0.72]	51 [2.4]
Group 4	32	4358 [5.91]	1000 [4.45]	38 [1.82]	15 [0.72]	9 [0.4]

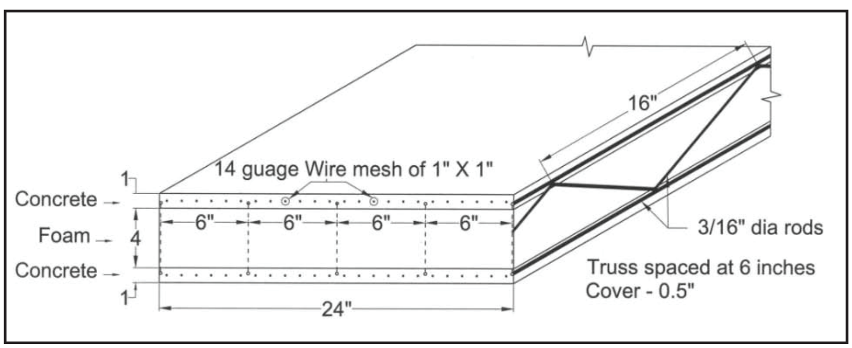

Figure 1 — Dimensions and reinforcement details of typical SCIP system (1 in = 25.4 mm).

Figure 2 — Typical SCIP system.

Figure 3 — Storage of SCIP system on the job site.

Figure 4 — SCIP system tied to the foundation with metal anchors.


Figure 5 — SCIP system being covered with shotcrete.

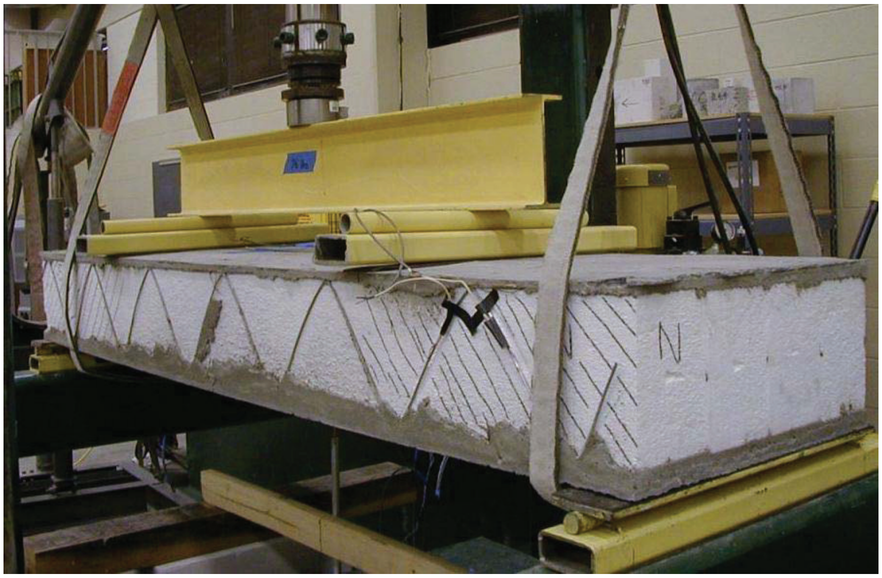

Figure 6 — Flexural test setup.

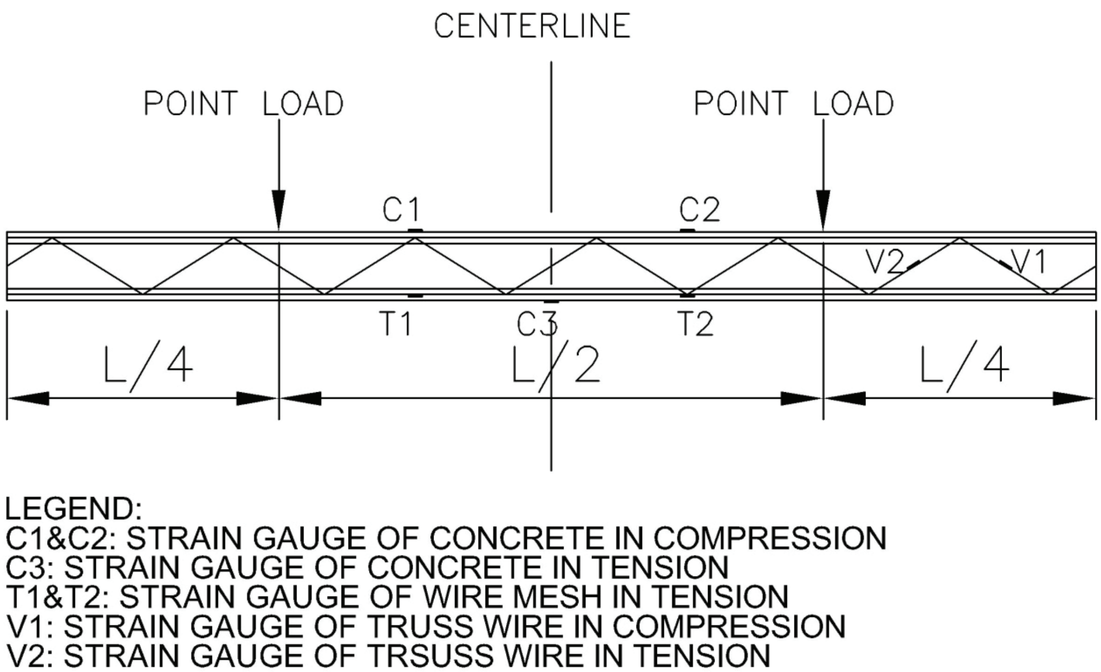

Figure 7 — Schematic diagram of the strain gauge locations.

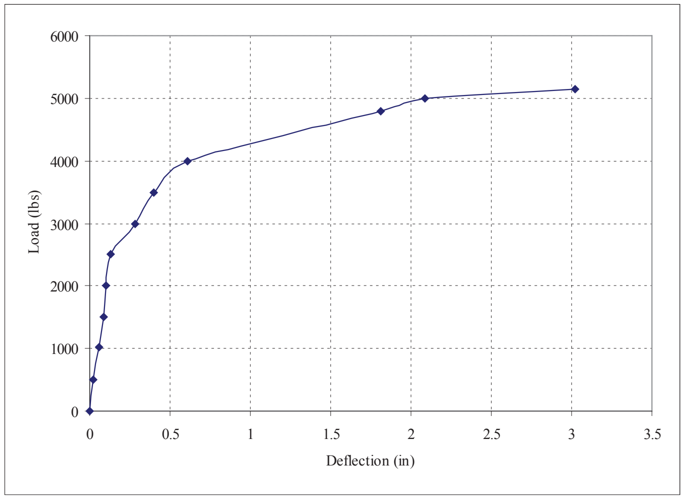

Figure 8 — Load deflection curve of panel number 3R (1 in = 25.4 mm, 1000 lbs = 4.5 kN).

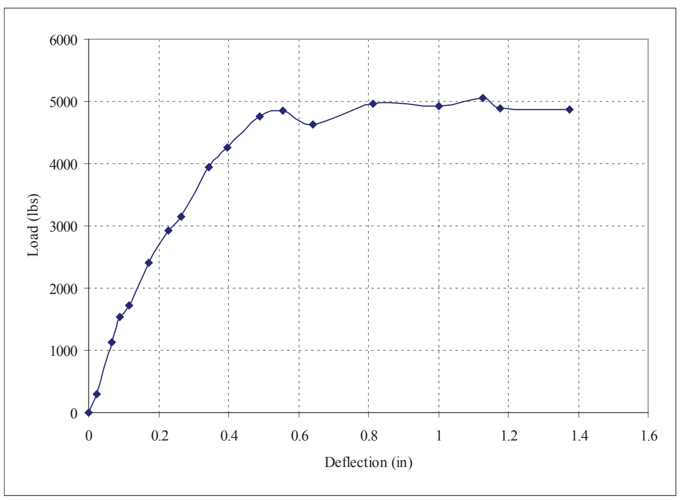

Figure 9 — Load deflection curve of panel number 1R (1 in = 25.4 mm, 1000 lbs = 4.5 kN).

Figure 10 — Failure of panel number 4R.

Figure 11 — Failure of panel number 9R.

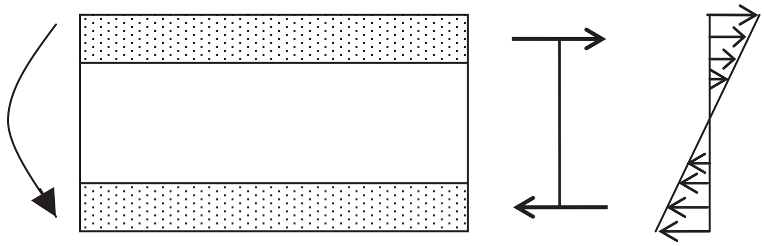

Figure 12 — Fully composite panel behavior.

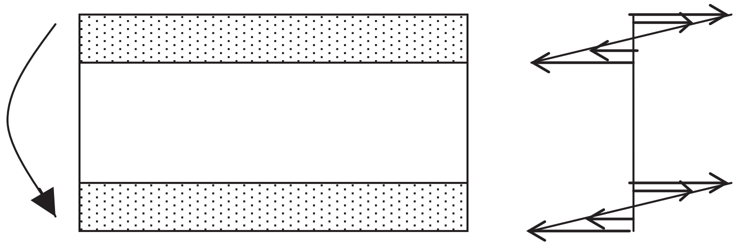

Figure 13 — Non-composite panel behavior.

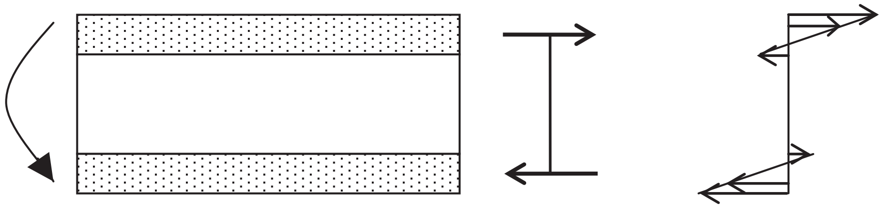

Figure 14 — Semi-composite panel behavior.

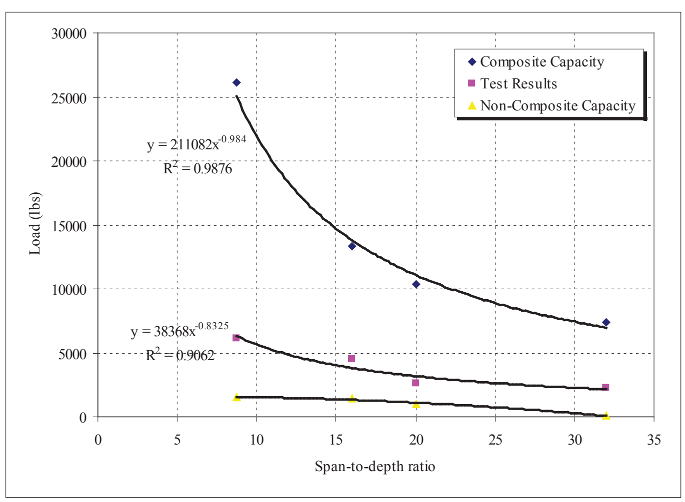

Figure 15 — Span-to-depth ratio versus load carrying capacity of the panels (1000 lbs = 4.5 kN).

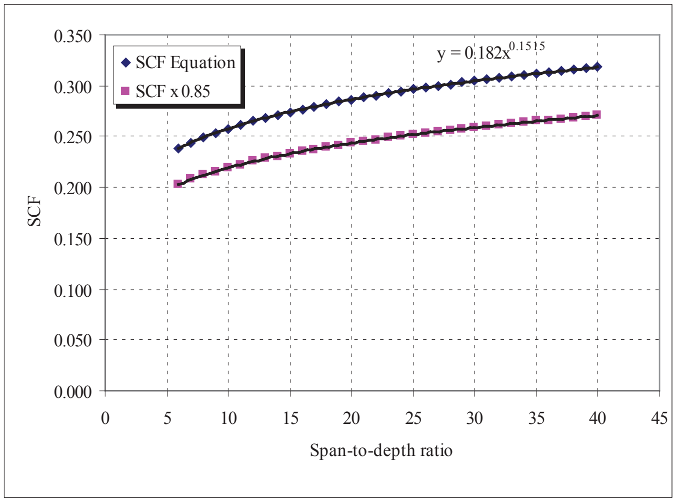

Figure 16 — Span-to-depth ratio versus the strength calibration factor.

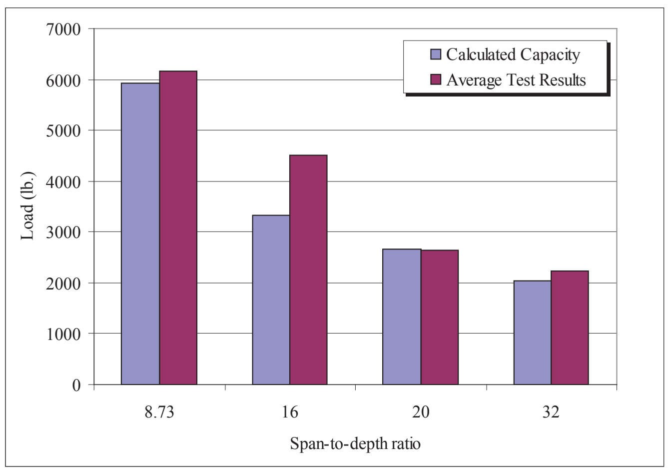

Figure 17 — Comparison of calculated capacities versus test results (1000 lbs = 4.5 kN).