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STRUCTURAL PERFORMANCE OF PFF PANELS

Report No. PFF-1

Prepared for

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By

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1. INTRODUCTION

The purpose of this report is to establish the structural properties and the design theory for composite foam panels produced by Precision Foam Fabricators. The structural performance parameters of these panels are to be developed through full scale load tests. Full scale ASTM E-72 vacuum chamber load tests were conducted under the supervision of this writer to define the structural parameters including shear modulus of panel core, shear strength of panel core, bending strength, and connection strength. The load tests were conducted at Farabaugh Engineering & Testing, Inc. of Turtle Creek, Pennsylvania. The design theory is developed for wind load condition.

2. PANEL PRODUCT

The panel is a composite structural member consisting of a structural shear core sandwiched between two structural steel skins. The steel skins have profiled side edges to form the panel side joint. The typical engaged panel side joint is shown on Figure 1.4(A) in Appendix A. The core and skin materials are defined as follows.

CORE : 1.0 lb/cu.ft. density Expanded polystyrene (EPS) with various foam depths.

SKIN : 26 gauge (0.018") G-90 galvanized steel confirming to ASTM A-446, minimum grade A.

PROCESS : The panels are produced by laminating process.

3. PRODUCT NOTATIONS

A typical product notation of EPS04.0-26/26 when punctuated by EPS, 04.0, 26/26 will read EPS Foam Core, 4.0" deep panel, 26 gauge facia skin and 26 gauge liner skin. All panels have a coverwidth of 45.625".

4. THEORETICAL SECTION PROPERTIES

In the composite design, the panel core is utilized to transfer the shear stress between the two facing skins. Due to the fact that Young's modulus of the panel core is very small as compared to that of the facing metal skins, the transformed panel core area can be ignored in the composite section property calculations. Due to the essentially flat skin profiles, the side joint profiles are ignored in the composite section property calcuations. The calculated full composite section properties are then divided by the panel coverwidth in feet to obtain the properties per foot width of panel coverage. The calculated composite section properties are presented in TABLE B-1A in Appendix B.

5. THEORETICAL ANALYSES

The shear deflection component of the composite panel can not be ignored. The effects of the shear deflection component change the reaction and moment distributions in a continuous span application. The theoretical analyses equations are presented in one paper published by this writer and included in Appendix C.

6. LOAD TESTING PROGRAM

Full scale load tests were conducted to empirically establish the following structural parameters.

A. Shear modulus of panel core.

B. Ultimate stress of the facing skins.

C. Shear strength of panel core.

All tests were conducted in accordance with the vacuum chamber method of ASTM E-72.

6.1. Bending Strength

All bending tests were conducted on a simple span condition with the most critical situation of foam core being butted at the midspan. For the tests failed in bending, the skin buckling strength is affected by the steel skin thickness, the skin profile, and the bond strength between the skin and the panel core. The bond strength may be affected by the skin orientation in production process. Therefore, both skins must be separately tested to fail compressively in order to establish the buckling stresses of the facia and the liner skins. The tests were conducted on a simple span condition with roller type of supports. A positive load is defined by the condition where the exterior skin is loaded in compression. A negative load is defined by the condition where the interior skin is loaded in compression. Midspan deflections were recorded throughout the tests for evaluating the shear modulus of panel core. Typical setup for the load test is shown in Appendix A.

6.2. Shear Strength

All shear strength tests were conducted on a simple span condition with the critical condition of a foam core butt joint being near the panel end (i.e. 18" from the panel end). For the tests failed in shear rather than in skin buckling, the shear strength of the foam core can be calculated. Due to the fact that the side joint profiles in the steel skins of all panels are minor, the stiffness of the steel skins alone can be ignored as compared to the stiffness of the composite section. Therefore, the analyses of all panels are assumed to have no non-composite action and all shear forces are taken by the foam core only.

7. SHEAR MODULUS OF PANEL CORE

The deflection of a composite panel under load consists of two components. The first component is bending deflection controlled by the profile, Young's modulus of the facing skin and the moment of inertia of the panel cross-section. The second component is shear deflection controlled by the effective shear modulus of panel core and the cross-sectional area of panel core. The effective shear modulus of panel core is affected by many factors and must be empirically determined. The bending deflection and the cross-sectional area of panel core can be theoretically determined. Therefore, in the theoretical load-deflection equation, the shear modulus of panel core is the only unknown parameter. Using the measured load-deflection data and the theoretical equations, the shear modulus of panel core can be empirically calculated. In order to achieve the best correlations among the various tests, the least square method is used to determine the effective shear modulus of panel core. Data trends indicated that the tested shear modulus of foam core is not a function of the panel depth (see TABLE B-2A in Appendix B). The regressed shear modulus is stated below.

Gc = 280.1 -- (1)

where Gc = shear modulus of foam core (psi)

The data correlations using the above equation for shear modulus are presented in Tables B-2 in Appendix B.

8. SHEAR STRENGTH OF PANEL CORE

The bonding between the metal skin and the panel core includes a layer of interior primer bonding to the metal skin and the adhesive bond bonding the panel core to the primer. Therefore, the panel shear strength includes the following four possible failure modes.

Failure Mode No. 1 : Primer failure due to horizontal shear. This mode of failure was not observed in the full scale load tests indicating good primer adhesion. Therefore, this failure mode is eliminated from the design parameters.

Failure Mode No. 2 : Adhesive bond failure between the primer and the panel core due to horizontal shear.
This mode of failure was not observed in the full scale load tests indicating adeqate strength of the adhesive bond developed in the laminating process. Therefore, this failure mode is eliminated from the design parameters.

Failure Mode No. 3 : Shear failure within the foam core. This mode of failure was not observed in the full scale load tests. Therefore, this failure mode is eliminated from the design parameters.

Failure Mode No. 4 : Off-set Shear failure at the panel core butt joint.

This failure mode was observed in all shear strength tests. Correlating the results of tests failed in this mode, the ultimate shear strength of the panel core is given by the following equation.

$$V_u = 40.5 / D \quad \text{---} (2)$$

where V_u = ultimate shear strength (psi)
D = panel core depth (")

9. BENDING STRENGTH OF PANEL

The panel bending strength includes the following four possible failure modes.

Failure Mode No. 1 : Tensile or compressive yielding of the skin.

This mode of failure can be detected by comparing the bending stress at the failure load to the yielding strength of the skin.

Failure Mode No. 2 : Compressive skin buckling away from the panel core initiated by tensile bond failure.

Failure Mode No. 3 : Compressive skin buckling into the panel core initiated by compressive failure of the panel core.

Failure Mode No. 4 : Skin buckling at the midspan where the foam core butt joint is purposely located.

The data trend indicated that the skin buckling strength is a constant for the 26 gauge steel skins regardless of skin orientation in the production process or the panel depth and is given by the following equation.

$$Fu = 15.85 \text{ ksi} \quad \text{-------------------------------------} \quad (3)$$

The data correlations using the above equation is shown on Tables B-5 of Appendix B.

10. CONNECTION STRENGTH

Serveral connection strength tests were conducted with the original system of using connection clip with 1/8" x 3" steel plate joint spline and 3/8" bolt. The test results indicated that the connection strength is inadequate for practical applications and thus, the conncetion system was abandoned.

The following six connection systems are evaluated in this report through full scale load tests. The details of the test set-up are shown in Appendix A. The connection systems are identified in the following table.

Connection System	Descriptions
S1	Intermediate support location; One clip; 3/4" x 3.5" x 24" wooden spline; Four #12 screws per clip; Discontinuous Liner.
S2	Intermediate support location; One clip; 3/4" x 3.5" x 24" wooden spline; Four #12 screws per clip; Continuous Liner.
S3	Intermediate support location; Two clips; 3/4" x 3.5" x 24" wooden spline; Four #12 screws per clip; Discontinuous Liner.
S4	Intermediate support location; Two clips; 3/4" x 3.5" x 24" wooden spline; Four #12 screws per clip; Continuous Liner.
S5	End support location; Two clips; One 3/8" thru-bolt per clip.
S6	End support location; Three clips; One 3/8" thru-bolt per clip.

Two typical failure modes were observed in tests of Systems S1, S2, S3 & S4. The first mode of failure was due to the #12 screws being pulled out from the wooden spline. The second mode of failure was due to the foam bearing & shear failure around the wooden spline. The following empirical equations are developed through data analyses and data correlation.

$P_o = 259$ # per screw ------------------------------(4)

$P_j = 722.7\ (D)^{0.25}$ ----------------------------------(5)

$P_c = 1002\ (D)^{0.25}$ -----------------------------------(6)

where P_o = ultimate #12 screw pull-out load (#/screw)
P_j = ultimate clip load governed by foam shear around wooden spline for discontinuous liner (#/clip).
P_c = ultimate clip load governed by foam shear around wooden spline for continuous liner (#/clip).
D = panel core depth (")

The connection failure of Systems S5 and S6 was typified by clip disengagement due to tearing of liner skin around the 3/8" bolt. The following empirical equation is developed from data analyses of this failure mode.

Pe = 542 # per clip ----------------------------------(7)

where Pe = ultimate clip load for Systems S5 & S6 (#/clip)

11. DESIGN THEORY FOR WIND LOAD CONDITION

The determination of the allowable load includes two major considerations, stiffness and strength. The design safety factors for the strength parameters are evaluated as follows.

According to AISC Specification or AISI Manual for Cold-Formed Steel Members, a one third increase in the allowable stress is permitted for wind load design. This increase of allowable stress is equivalent to a quarter reduction in the design safety factor. Based on ICBO-ES requirement, a safety factor of 3.0 against shear failure should be used for wind load design and the safety factor reduction against bending and panel connection failures are permitted.

The design safety factors are summarized below.

Parameter	Wind Load Design	Live Load Design
Bending	1.875	2.500
Shear	3.000	3.000
Connection (Side Joint)	1.875	2.500
Screw Pull-out	2.250	3.000

12. DESIGN DATA

Since the connection system can be improved or enhanced in the future, the load-span data are generated in two groups. The first group is governed by the connection stength based on the tested connection systems only. Using the empirical equations, the design theory and "Tributable Loading Area Method", the design equations governed by connection strength are derived as follows.

$$Qi = WL \times 45.625/12 = C\ (D)^{0.25}/1.875 \text{ -----------------(8)}$$

where Qi = tributable connection clip load at intermediate support (#)
W = design wind load (psf)
L = allowable panel span (ft)
C = empirical constant ; 722.7 for discontinued liner; 1002.0 for continuous liner.
D = panel depth (")

$$Ns = Qi/(259/2.25) \text{ rounded up \& min.} = 4 \text{ ---------------(9)}$$

$$Nb = Qi/(2 \times 542/1.875) \text{ rounded up \& min.} = 2 \text{ ---------(10)}$$

Each clip can have four #12 screws, therefore, if Ns is between 5 and 8, double clip system must be used.

The Load-Span data generated from the above equations are shown in Tables 1 & 2.

The second group of load-span tables are generated in accordance with the structural parameters and the design theory developed in this report without the consideration of the connection strength parameter. This group of load-span data is the upper limit for the panel connection design.

13. EXAMPLE DESIGN PROBLEM

[Known] : Design Wind Load = ± 25 psf

Panel : EPS04.0-26/26 with discontinuous liner

Allowable Deflection = L/180

Find the allowable span and the corresponding fastener design.

[Solution]:

From Table 1, L = 5.734'; Ns = 5; Nb = 2.

Since the panel has discontinuous liner, the panel structure must be treated as a simple span structure. From the second group Load-Span Tables, the allowable span is listed below.

Governing Parameter	Allowable Span (ft)
Panel Strength	12.96
Deflection (L/180)	13.64

Comparing the above, the connection strength governs the design and the design parameters are listed below.

Allowable Span = 5.734'

Use two clips at each intermediate support with three #12 screws per clip.

Use two 3/8" thru-bolts per panel at panel end.

14. CONCLUSIONS AND RECOMMENDATIONS

(1). Observing the design data, the following conclusions can be made.

a. The panel end connection using thru-bolts is adequate and will not become the structural limiting parameter.

b. The connection system at the intermediate support will be the structural limiting parameter in all design conditions.

(2). Further research aimed at improving the connection system at the intermediate support is recommended. The following methods of improvement are recommended.

a. Use longer and/or deeper wooden spline to increase the side joint strength.

b. Use Fablok fasteners between side joints on the liner side to supplement the wooden spline connection system.

c. Based on this writer's experiences, the connection strength behavior is a very complicated matter affected by the overall distortions of the panel under load. Therefore, the connection strength can only be evaluated by full scale load tests.

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