IDEA

Innovations Deserving
Exploratory Analysis Programs

National Cooperative Highway Research Program

An All Composite Bridge Sidewalk

Final Report for NCHRP-IDEA Project 67

T. Campbell, Foster-Miller, Inc.

September 2001

TRANSPORTATION RESEARCH BOARD • NATIONAL RESEARCH COUNCIL

INNOVATIONS DESERVING EXPLORATORY ANALYSIS (IDEA) PROGRAMS
MANAGED BY THE TRANSPORTATION RESEARCH BOARD (TRB)

This NCHRP-IDEA investigation was completed as part of the National Cooperative Highway Research Program (NCHRP). The NCHRP-IDEA program is one of the four IDEA programs managed by the Transportation Research Board (TRB) to foster innovations in highway and intermodal surface transportation systems. The other three IDEA program areas are Transit-IDEA, which focuses on products and results for transit practice, in support of the Transit Cooperative Research Program (TCRP), Safety-IDEA, which focuses on motor carrier safety practice, in support of the Federal Motor Carrier Safety Administration and Federal Railroad Administration, and High Speed Rail-IDEA (HSR), which focuses on products and results for high speed rail practice, in support of the Federal Railroad Administration. The four IDEA program areas are integrated to promote the development and testing of nontraditional and innovative concepts, methods, and technologies for surface transportation systems.

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ILLUSTRATIONS

1. Executive Summary

The objective of this effort was to design and develop a lightweight, corrosion free, cantilevered composite sidewalk system for roadway bridges. The composite sidewalk system consists entirely of fiber composite materials and will have a great impact on the current practice of installing bridge sidewalk systems by lowering installation and maintenance costs, improving worker safety, and by eliminating corrosion and the environmental impact of painting. The system has a single molded component for cantilevered support. The cantilever support, which consists of carbon fabric and epoxy resin, is a constant cross section I-beam with an overall height of 18 in. The flange width, flange thickness and web thickness is 12-3/4, 1/2, and 1/4 in., respectively. The weight of each cantilever support is approximately 125 lb. The length of the cantilever support is 11 ft. The width of the walkway portion of the sidewalk system is 6 ft. Foster-Miller worked closely with the composite pedestrian bridge firm, E.T. Techtonics, to complete the design and development of the composite sidewalk system. The composite sidewalk system was sized for a minimum factor of safety (FS) of 3. Validation of the design was performed through the use of static and creep tests, which were performed at the University of New Hampshire (UNH). Finally, plans are currently in progress to implement the composite sidewalk system on a bridge in Colchester, VT.

2. Performance Specification

One of the specifications used by Foster-Miller to design the composite sidewalk system was the American Association of State Highway and Transportation Officials (AASHTO) document, "Guide Specifications for Design of Pedestrian Bridges," which was most recently published in August 1997. In addition, Foster-Miller also received additional strength and deflection requirements from the Vermont Agency of Transportation (VAOT). The VAOT requested that the sidewalk system have a width of 6 ft. According to the VAOT and aforementioned AASHTO specification, the pedestrian live load requirement for the sidewalk system is 85 lb/ft^2 (psf). In addition, according to the VAOT, the dead weight (i.e., snow) load requirement is 100 psf. In accordance with a VAOT request, the sidewalk system was designed for strength at a combined loading of 185 psf (i.e., pedestrian and snow loads combined). The deflection requirement of the cantilever support component of the sidewalk system, according to the AASHTO specification mentioned above, is L/300 for pedestrian live loads, where L is the length of the cantilever support. A copy of the pedestrian bridge guide AASHTO specification is found in Appendix A.

3. Implementation Sites

Foster-Miller contacted several state agencies to solicit interest in the project, receive design requirements input and identify potential implementation sites for the composite sidewalk system. Among those agencies contacted were the Vermont Agency of Transportation (VAOT), Massachusetts Highway Department, New York State Department of Transportation, Rhode Island Department of Transportation, New Hampshire Department of Transportation and the

Maine Department of Transportation. Principal interest, as discussed in the initial program proposal, came from VAOT and MassHighway. NYDOT has expressed interest in the project and the potential for installation sites once the system has been field demonstrated. NHDOT attended the static testing conducted at the University of New Hampshire and expressed interest in the system.

A site has been selected for the first demonstration installation of the all-composite sidewalk/ bicycle pathway system. VAOT has identified a need for a 260 ft long sidewalk on Blakely Road Bridge that crosses I-89 in Colchester, VT, just north of Burlington, VT. The bridge, shown in Figure 1, is 24 ft wide, curb-to-curb, and is too narrow to allow for lane alterations, installation of a bicycle lane or installation of a sidewalk. Nonetheless, it has seen a substantial growth in pedestrian and bicycle traffic due to residential development on both sides of the Interstate. In addition, new secondary schools were constructed within a mile of the bridge on the west side of I-89. It is further notable that the bridge rises significantly in the center, as shown in Figure 2, severely limiting sight distance.

The bridge, which is a steel girder, reinforced concrete deck construction, is shown in Figure 3. The steel fascia beams are 33 in. WF in the two side spans and 36 in. WF in the two main spans over I-89. The sidewalk mounting location is shown in Figure 4. The bridge overhangs the fascia beams by approximately 3 ft, adding significant length to the cantilever beam requirements. A standard steel and concrete cantilevered sidewalk system was estimated

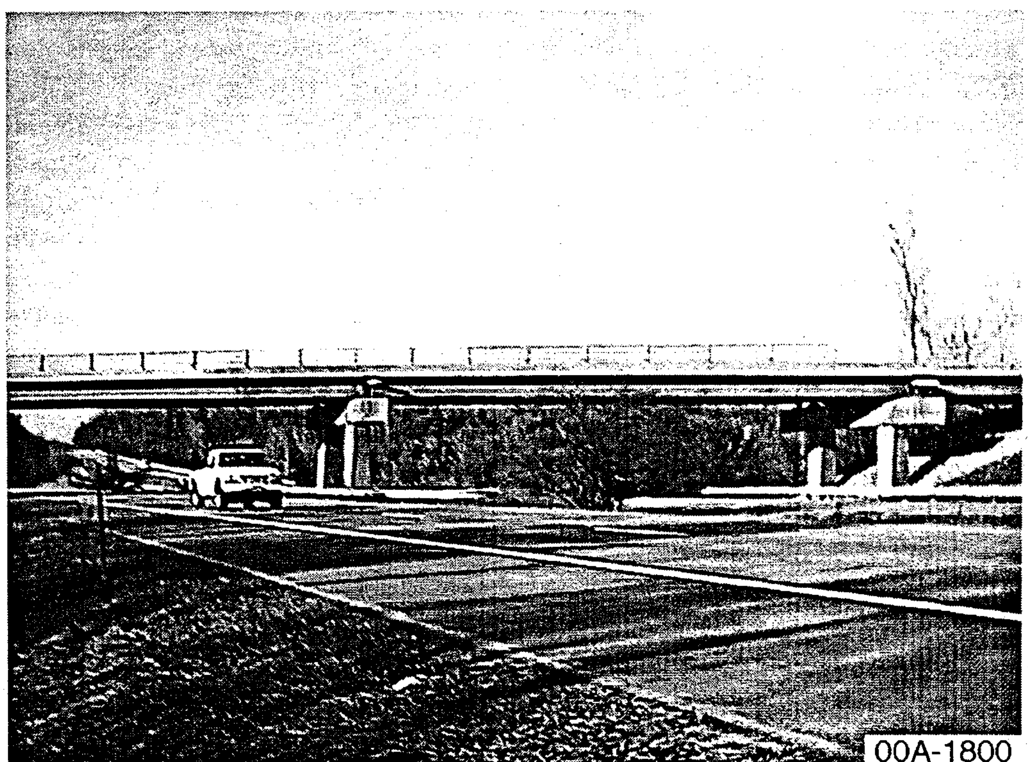


Figure 1. Blakely Road Bridge

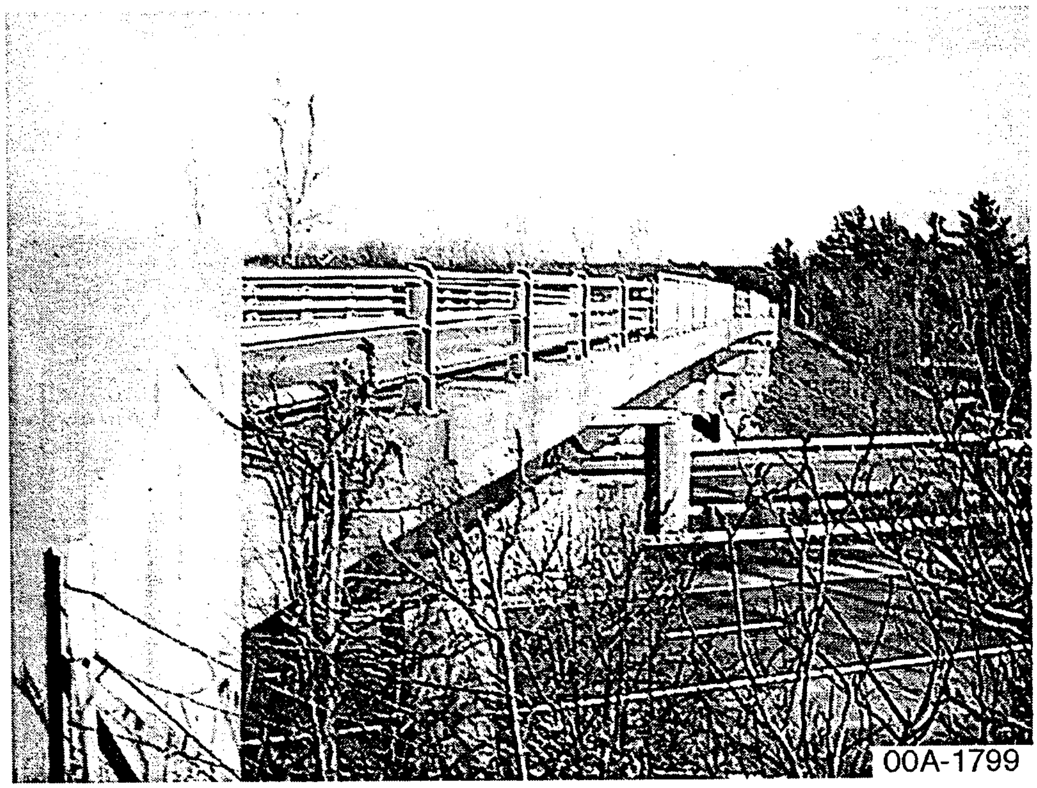
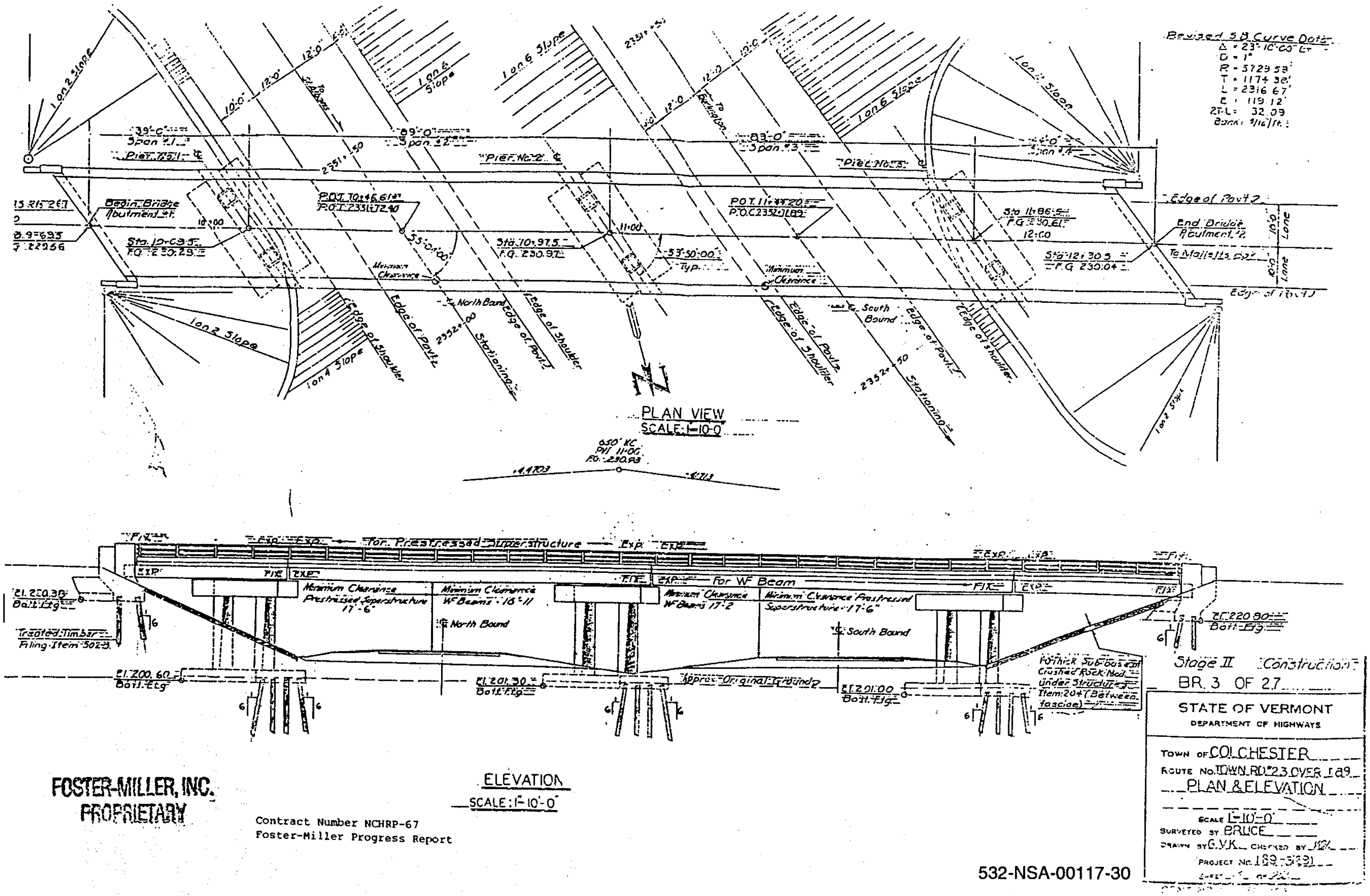


Figure 2. Close-up of bridge

to weigh 300 lb per running foot, approximately three times the weight of the Foster-Miller all-composite system.

The demonstration system will utilize the carbon fiber epoxy cantilever beams, discussed in this report, on an average spacing of 8 ft. Some variation will be required to match the existing fascia beam reinforcing diaphragms. The carbon beams will be attached to the steel fascia beam using the steel angle iron and bolted connections discussed in this report. Superstructure sections will be delivered to the site in pre-assembled sections (up to 20 ft in length) to match the beam spacing and other design requirements. Pre-assembled sections significantly reduce installation costs and provide much higher quality control, particularly in a first time demonstration installation. The system is projected to weigh 100 lb per running foot.

VAOT has contracted for the approach work and reinforcement, as required, of the bridge fascia beam in order to accommodate the addition of a sidewalk/bicycle pathway. If VAOT chooses to install a conventional steel and concrete sidewalk/bicycle pathway, major reinforcement of the bridge superstructure will be required to carry the increased dead load of concrete and steel. Thus, the composite sidewalk/bicycle pathway provides a highly desirable alternative that is lightweight, lower-cost, and which will eliminate corrosion related maintenance requirements. Consequently, VAOT has specified the Foster-Miller all-composite system and the lead option for installation at this site in the late summer of 2001.



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Foster-Miller Progress Report

532-NSA-00117-30

Figure 3. Bridge design

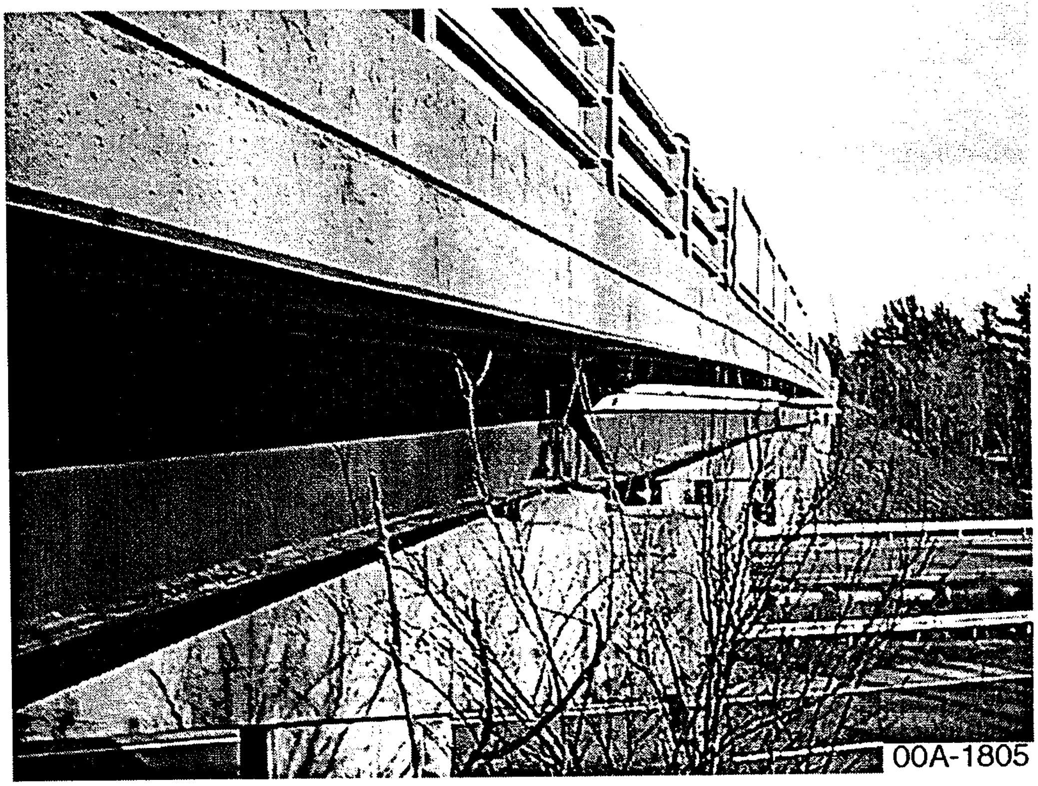

Figure 4. Sidewalk mounting location

The Foster-Miller team is working actively with VAOT to secure funding for installation, on-site testing and long-term monitoring of the sidewalk system for the Blakely Road bridge. VAOT has identified several additional bridge sites, which would benefit from a similar system. The team has also identified three potential future sites in Massachusetts and one in Maine. Discussions will continue with interested personnel at NHDOT and NYDOT to identify additional sites.

4. Cantilever Support Design

The cantilever support beam component of the composite sidewalk system was designed exclusively by Foster-Miller. All schematics and background analysis are presented herein.

The cantilever support beam of the composite sidewalk system attaches to the fascia beam of the existing bridge. It was recommended by the Vermont Agency of Transportation (VAOT) that support diaphragms, which connect to the fascia beam and adjacent support beam, be added to the underside of the bridge in the areas where the cantilever support beams are attached. In order to provide greater adaptability to other bridge designs, Foster-Miller set the depth of the cantilever support beam at approximately 18 in. The flange width, flange thickness and web thickness are approximately 12-3/4, 1/2, and 1/4 in., respectively. The walkway of the sidewalk system begins 3 ft from where the cantilever support beam attaches to the bridge fascia beam, the width of the walkway is 6 ft and an additional 2 ft is required for mounting handrail supports for the sidewalk system. The total length of the cantilever support beam is 11 ft. The spacing of the cantilever support beams was designed to be 5 ft on center. The design of the beam consisted of woven carbon fabric and epoxy resin with a fiber volume fraction of 40 percent. The fibers were oriented along the beam axis with the exception of a few 45 deg bias layers in the region where the steel attachment fitting is bolted to the composite. The steel attachment fitting connects the cantilever support beams to the existing bridge. A schematic of the cantilever support beam with the steel attachment fittings is shown in Figure 5. The cantilever support beam was sized for ultimate strength (live and snow loads combined, 185 psf) with a minimum factor of safety (FS) of 3. Also, the beam was sized for deflection according to the AASHTO specification for pedestrian bridges mentioned previously. The weight of the cantilever support beam is approximately 125 lb. Schematics of the cantilever support beam and attachment fitting, shown separately, are presented in Figures 6 and 7, respectively.

The flange of the cantilever support beam was sized for bearing, crippling, shear-out and local compression. In addition, the "twist-bend" buckling stability of the beam as well as buckling and bearing in the web was examined.

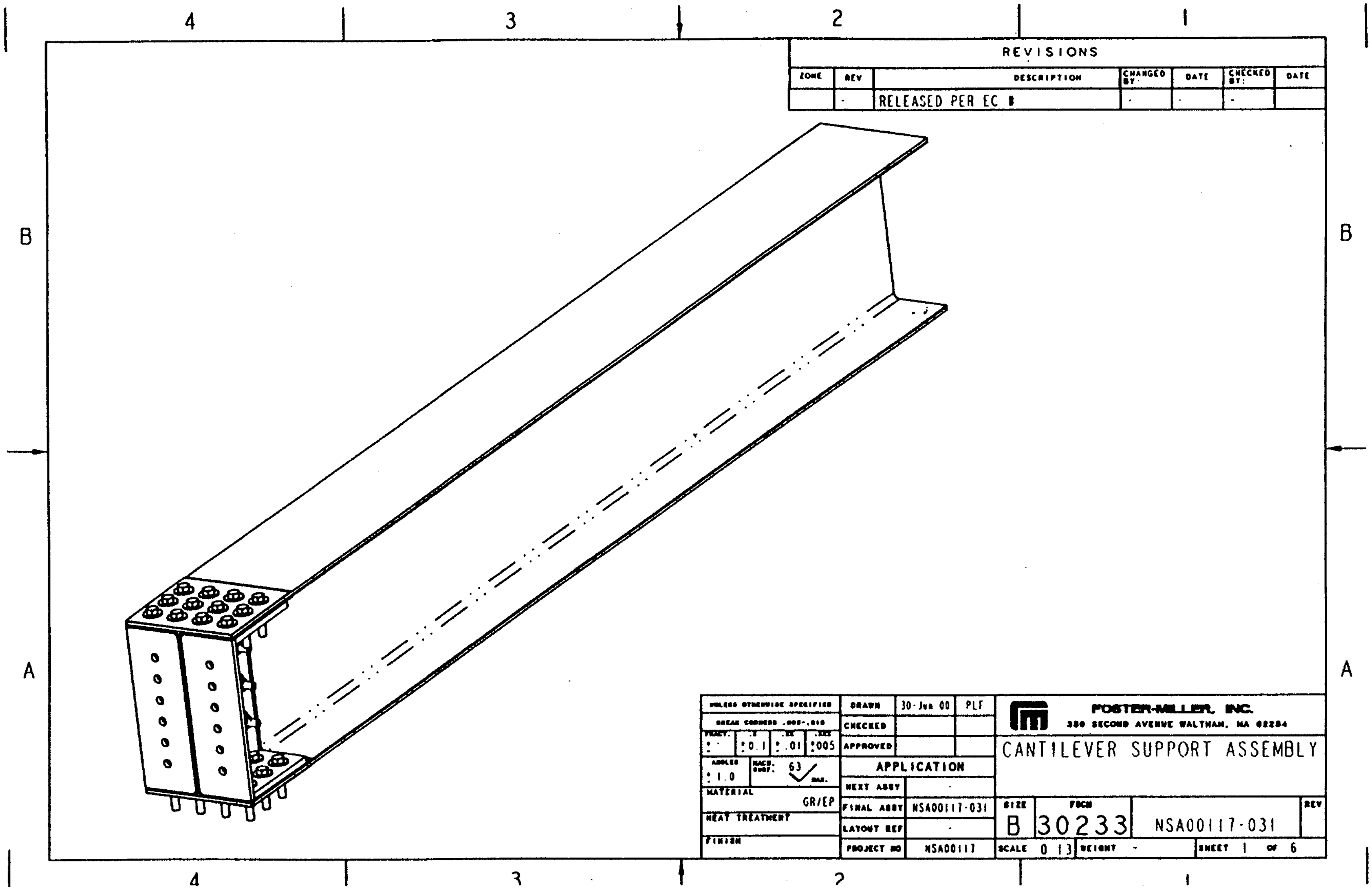

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Figure 5. Cantilever support assembly

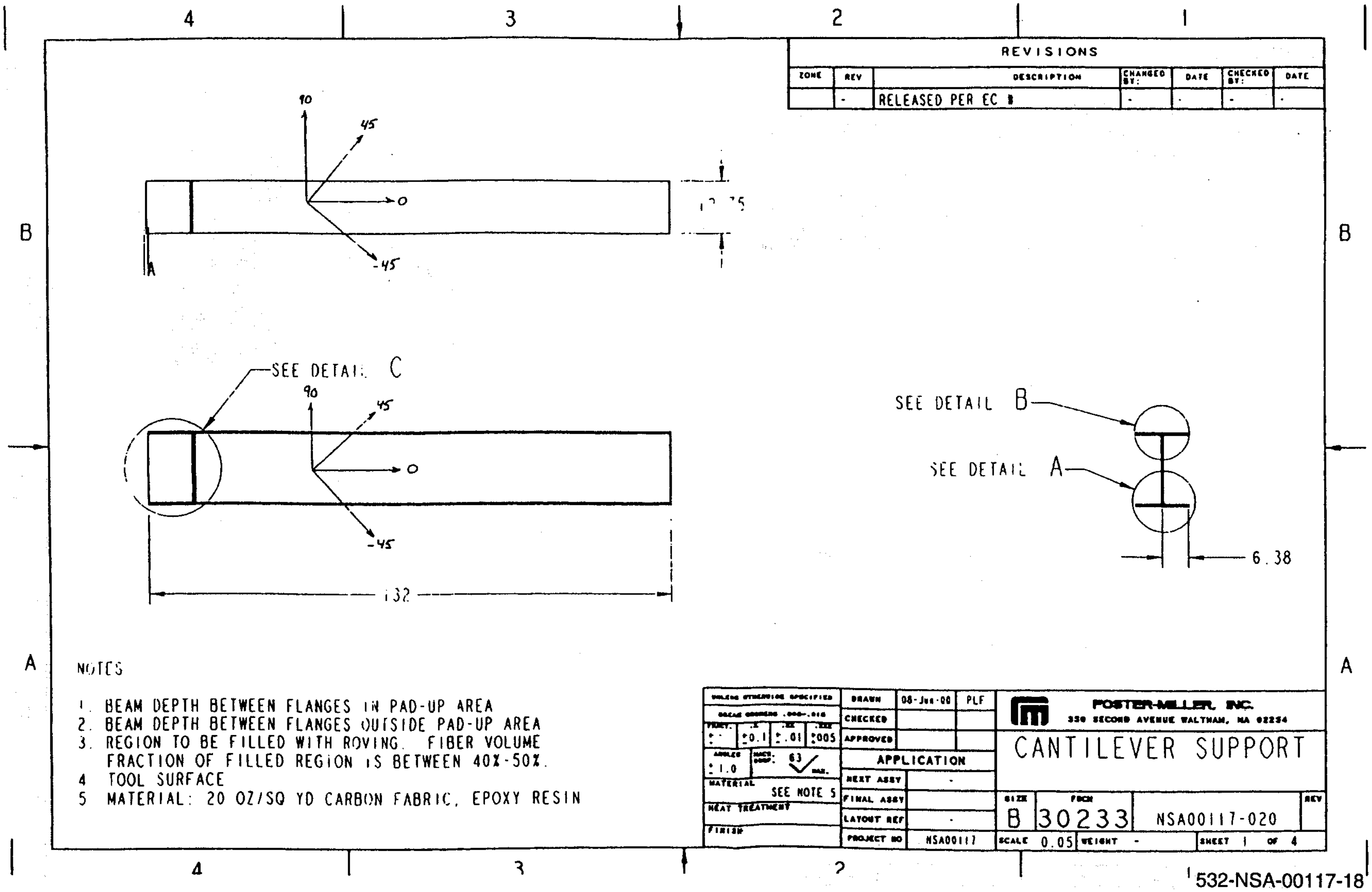
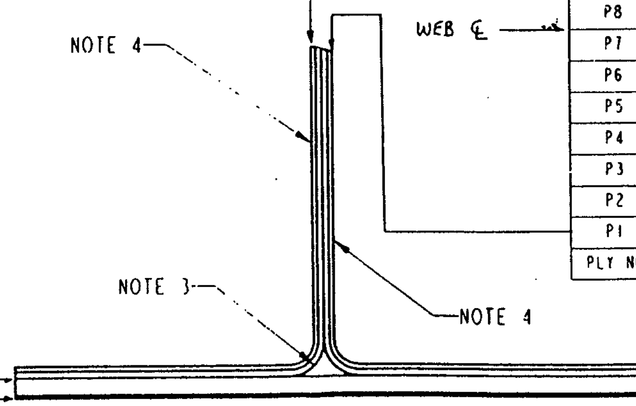


Figure 6a. Cantilever support

REVISIONS

ZONE	REV	DESCRIPTION	CHANGED BY:	DATE	CHECKED BY:	DATE
	-	RELEASED PER EC #	-	-	-	-

PLY NO	ORIENT	MATERIAL	START	END
P14	0	(GR/EP)	DATUM A	132
P13	45	(GR/EP)	DATUM A	11.25
P12	0	(GR/EP)	DATUM A	132
P11	45	(GR/EP)	DATUM A	11.5
P10	0	(GR/EP)	DATUM A	132
P9	45	(GR/EP)	DATUM A	11.75
P8	0	(GR/EP)	DATUM A	132
P7	0	(GR/EP)	DATUM A	132
P6	45	(GR/EP)	DATUM A	11.75
P5	0	(GR/EP)	DATUM A	132
P4	45	(GR/EP)	DATUM A	11.5
P3	0	(GR/EP)	DATUM A	132
P2	45	(GR/EP)	DATUM A	11.25
P1	0	(GR/EP)	DATUM A	132



DETAIL A
SCALE 0.500

PLY NO	ORIENT	MATERIAL	START	END
P15	0	(GR/EP)	DATUM A	132
P16	0	(GR/EP)	DATUM A	132
P17	0	(GR/EP)	DATUM A	132
P18	0	(GR/EP)	DATUM A	132
P19	0	(GR/EP)	DATUM A	132
P20	0	(GR/EP)	DATUM A	132
P21	0	(GR/EP)	DATUM A	132
P22	45	(GR/EP)	DATUM A	11.5
P23	0	(GR/EP)	DATUM A	132
P24	0	(GR/EP)	DATUM A	132
P25	45	(GR/EP)	DATUM A	11.25
P26	0	(GR/EP)	DATUM A	132
P27	0	(GR/EP)	DATUM A	132

UNLESS OTHERWISE SPECIFIED
BREAK EDGES .005-.015
TOLERANCES: ±0.1 ±.01 ±.005
ANGLES ±1.0
MACH. 63 MAX.
MATERIAL SEE NOTE 5
HEAT TREATMENT
FINISH

DRAWN 08-Jun-00 PLF
CHECKED
APPROVED
APPLICATION
NEXT ASSY -
FINAL ASSY
LAYOUT REF -
PROJECT NO NSA00117

FOSTER-MILLER, INC.
350 SECOND AVENUE WALTHAM, MA 02254

CANTILEVER SUPPORT

SIZE B FSCM 30233 NSA00117-020 REV
SCALE 0.05 WEIGHT - SHEET 2 OF 4

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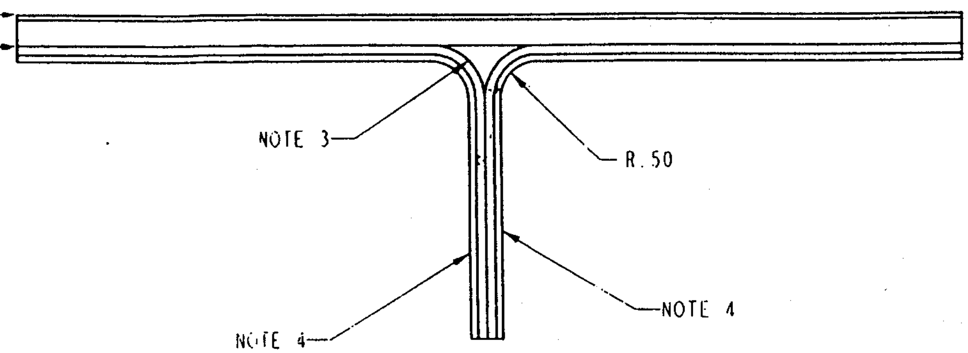
Figure 6b. Cantilever support (continued)

REVISIONS						
ZONE	REV	DESCRIPTION	CHANGED BY:	DATE	CHECKED BY:	DATE
	-	RELEASED PER EC #	-	-	-	-

PLY NO	ORIENT	MATERIAL	START	END
P40	0	(GR/EP)	DATUM A	132
P39	0	(GR/EP)	DATUM A	132
P38	0	(GR/EP)	DATUM A	11.25
P37	0	(GR/EP)	DATUM A	132
P36	0	(GR/EP)	DATUM A	132
P35	0	(GR/EP)	DATUM A	11.5
P34	0	(GR/EP)	DATUM A	132
P33	45	(GR/EP)	DATUM A	132
P32	0	(GR/EP)	DATUM A	132
P31	0	(GR/EP)	DATUM A	132
P30	45	(GR/EP)	DATUM A	132
P29	0	(GR/EP)	DATUM A	132
P28	0	(GR/EP)	DATUM A	132



DETAIL B
SCALE 0.750

UNLESS OTHERWISE SPECIFIED
BREAK CORNERS .005-.010
TOLERANCES: ±0.1 ±.01 ±.005
ANGLES ±1.0
MACH. FINISH 63 MAX.
MATERIAL SEE NOTE 5
HEAT TREATMENT
FINISH

DRAWN 08-Jun-88 PLF
CHECKED
APPROVED

APPLICATION
NEXT ASSY -
FINAL ASSY
LAYOUT REF -
PROJECT NO NSA00117

FOSTER-MILLER, INC.
350 SECOND AVENUE WALTHAM, MA 02254

CANTILEVER SUPPORT

SIZE B FSCM 30233 NSA00117-020 REV
SCALE 0.05 WEIGHT - SHEET 3 OF 4

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Figure 6c. Cantilever support (continued)

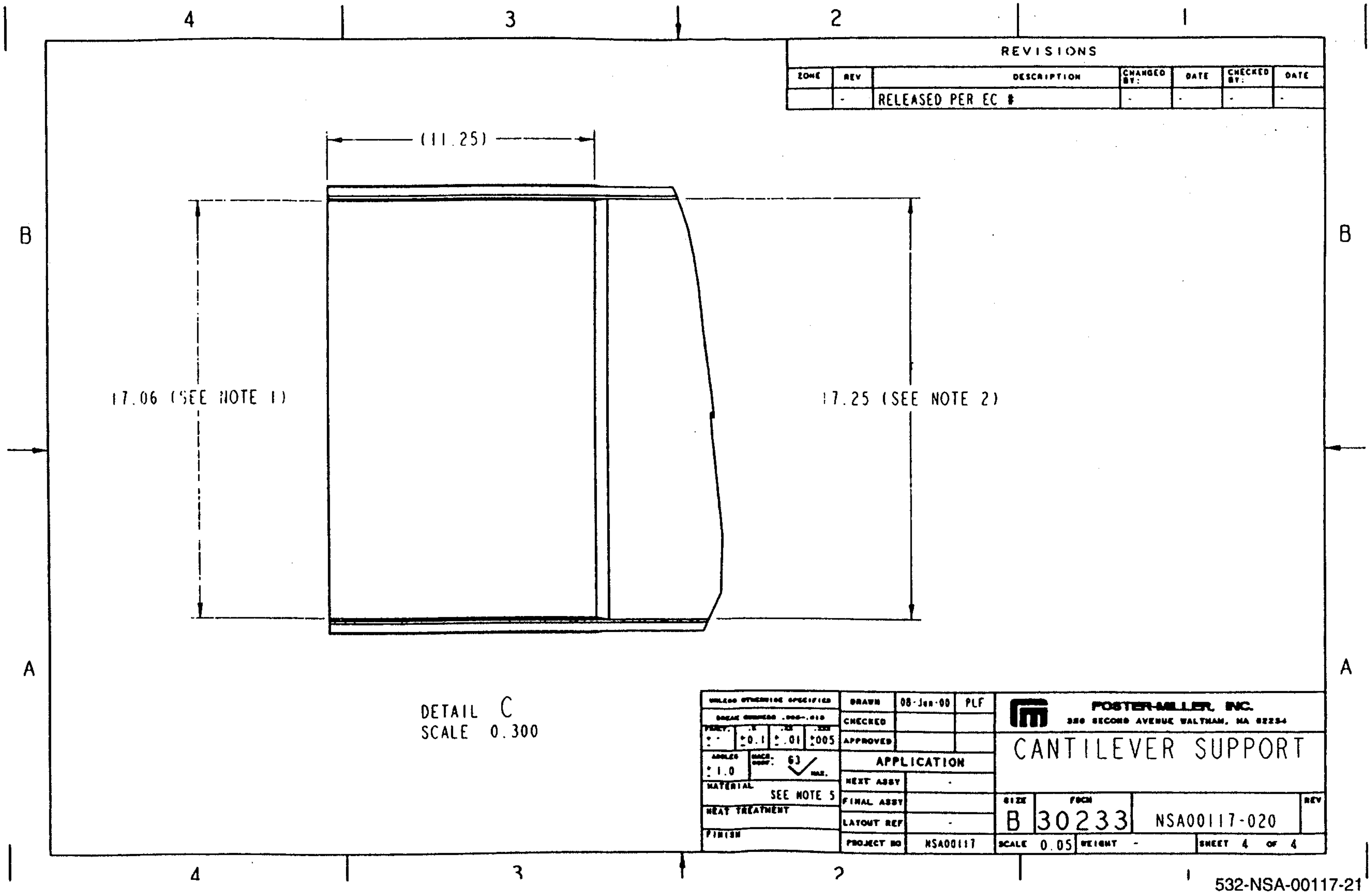


Figure 6d. Cantilever support (continued)

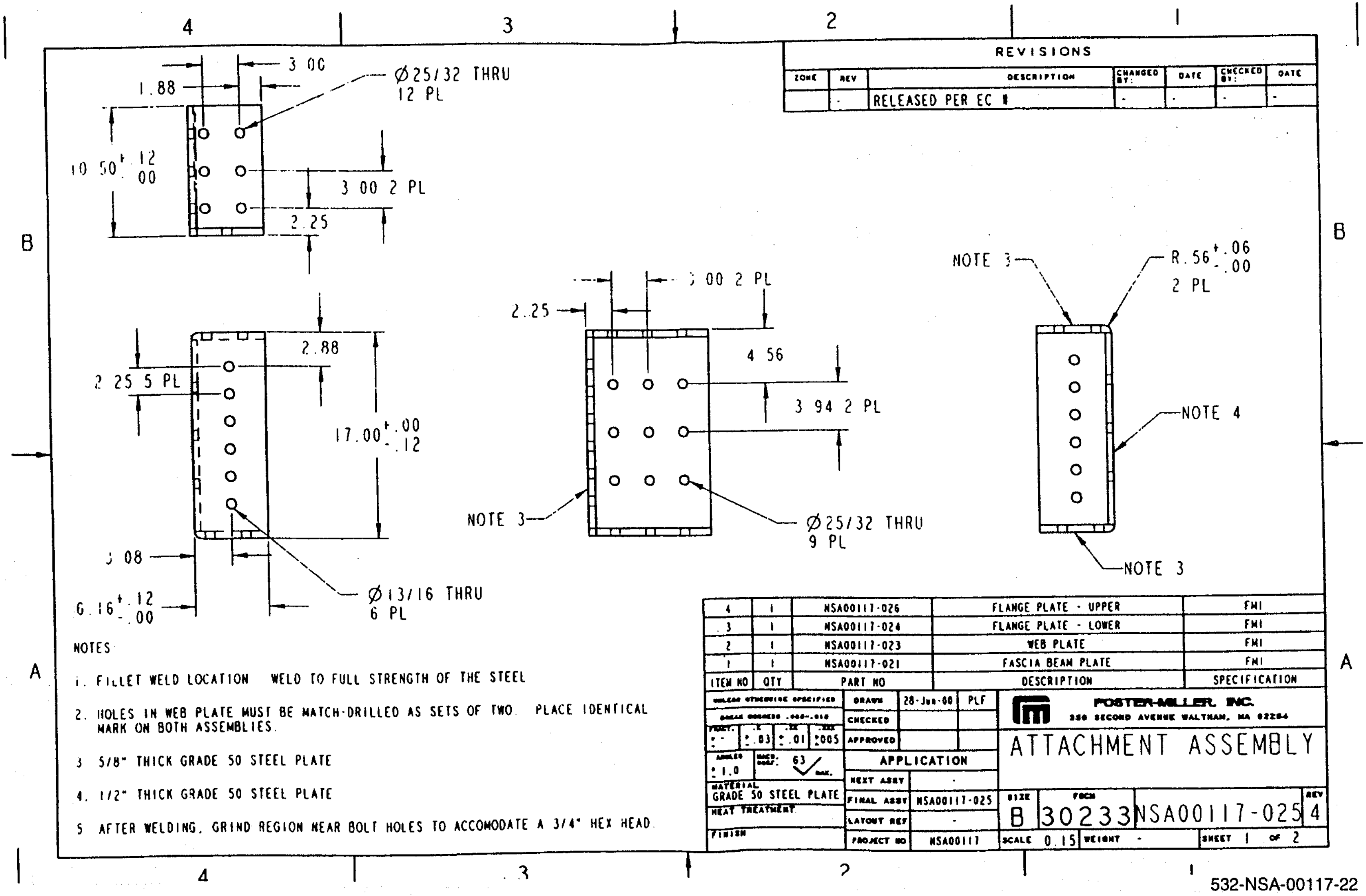


Figure 7a. Attachment assembly

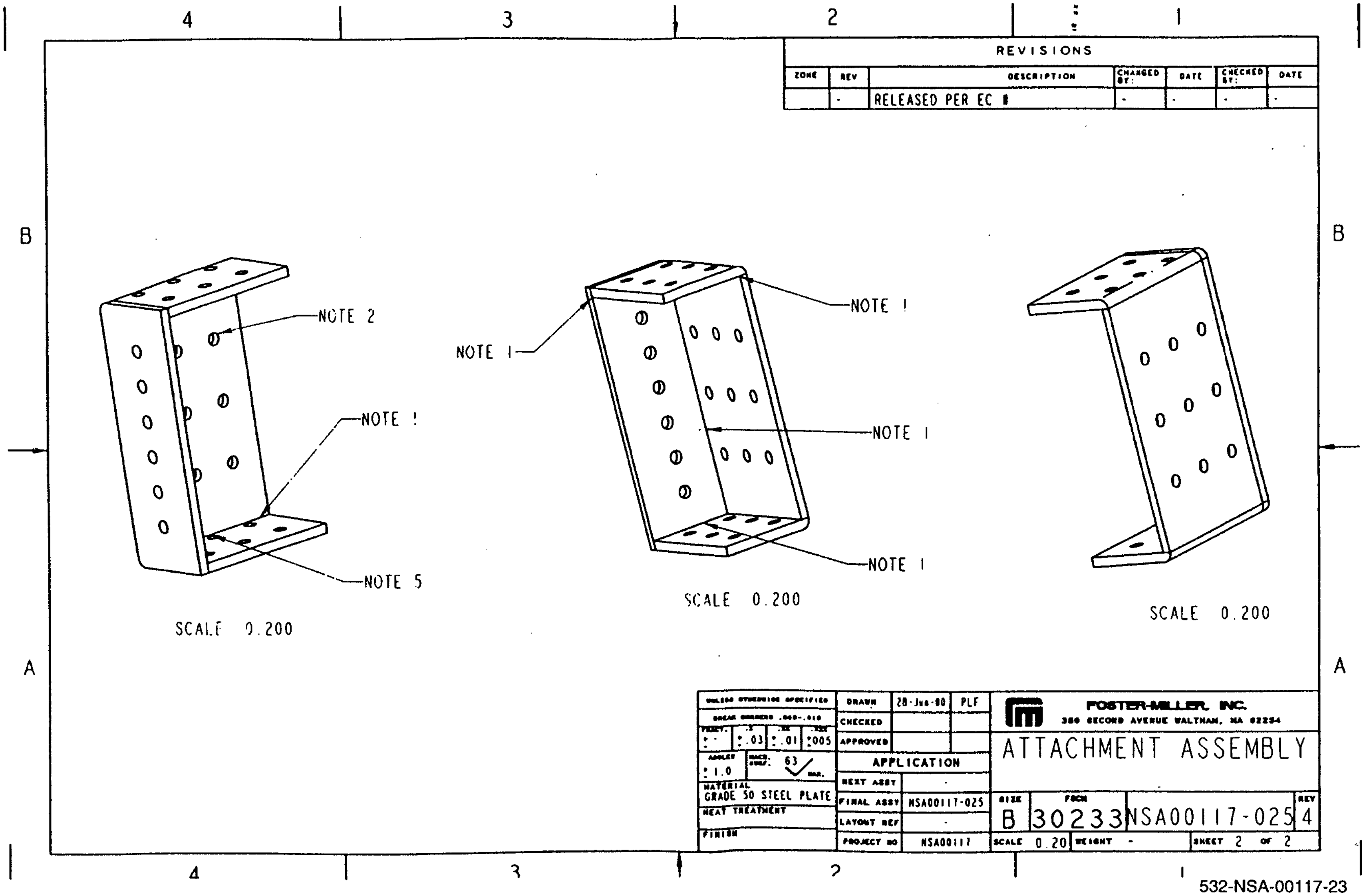


Figure 7b. Attachment assembly (continued)

4.1 Flange Bearing Stress Analysis

Diameter of flange bolt holes:

$D_{bhf} := 0.75 \cdot in$

Load sharing among three (3) rows of bolts in the flange assuming 3/4 in. UNC bolts with end, row and column spacing of $3D_{bhf}$, $4D_{bhf}$ and $4D_{bhf}$, respectively, results in a maximum per bolt load of P_{bmax}. A higher factor of safety is used to compensate for uncertainties in load sharing with loose location hole tolerances. A sketch of the flange bolt hole spacing is shown in Figure 8.

Desired factor of safety for the flange in bearing:

$FS_{CF} := 4.5$

Load per bolt in row with the highest percentage of the total load:

$P_{bmax} := 2200 \cdot lbf$

Maximum allowable compressive strength of composite beam:

$\sigma_c := 65 \cdot 10^3 \cdot psi$

65 ksi is the approximate compressive strength for the composite cantilever support beam as calculated by classic laminate analysis.

Minimum required composite beam flange thickness due to bearing stress. For the analysis, one of the bolts in the row with the highest percentage of the total load was selected.

$$t_{fmin} := \frac{4 \cdot P_{bmax} \cdot FS_{CF}}{\pi \cdot D_{bhf} \cdot \sigma_c}$$

$t_{fmin} = 0.259 in$

For the bearing stress in the flange to be acceptable, the minimum required flange thickness must be slightly larger than 1/4 in.

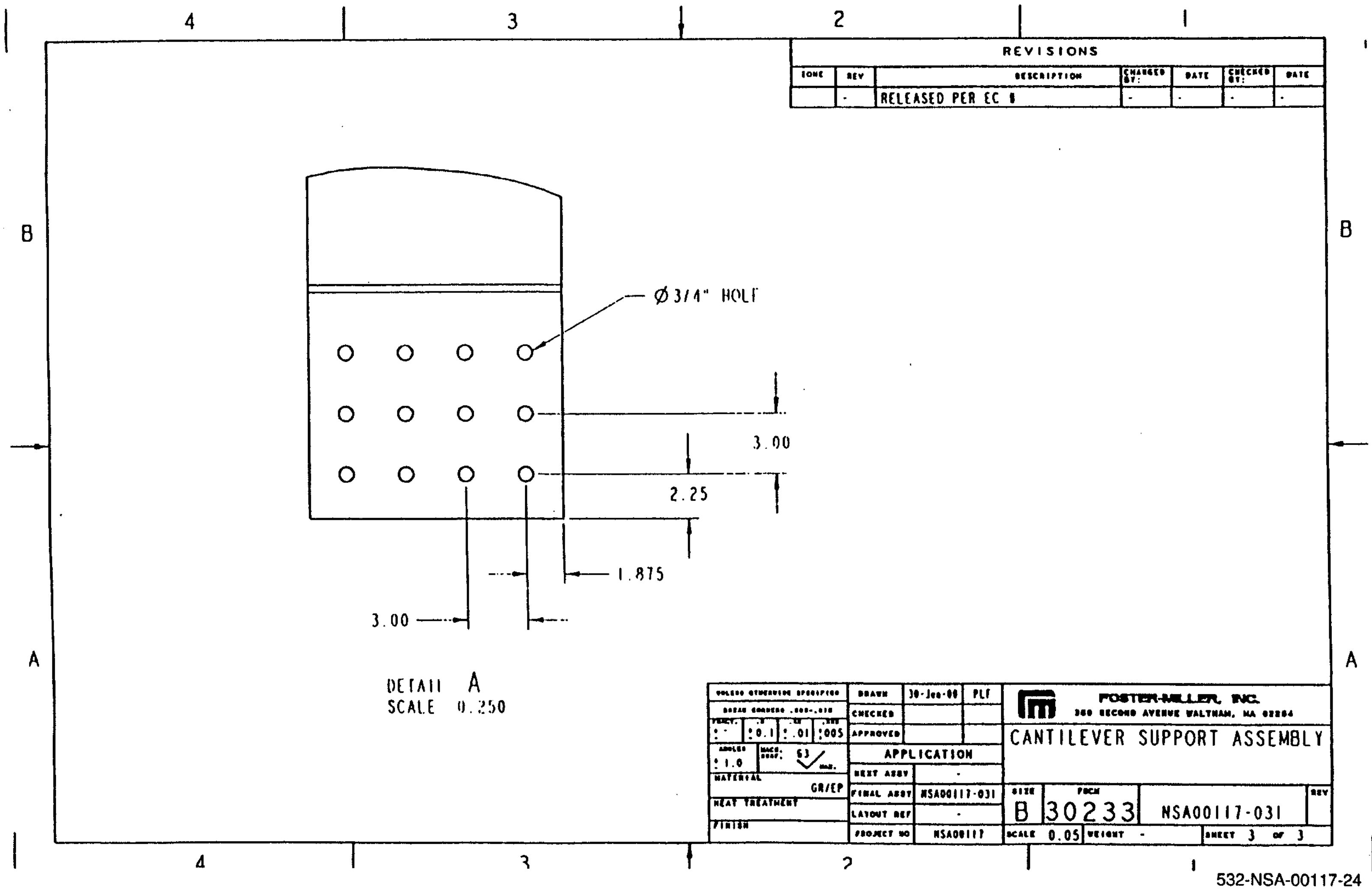
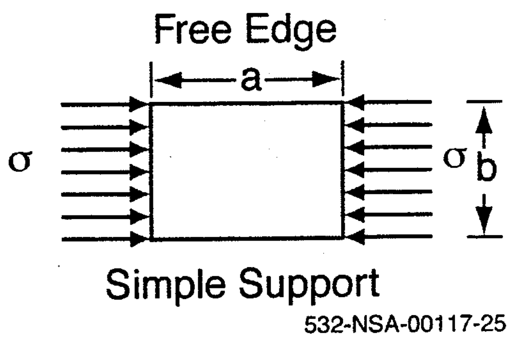


Figure 8. Cantilever support assembly

4.2 Flange Crippling Analysis

A rectangular plate under equal uniform compression on two opposite edges will simulate one-half of the flange.



Width:

$a := 36 \cdot in$

Height:

$b := 6.375 in$

Thickness:

$t := 0.45 in$

Modulus of elasticity (as per laminate analysis):

$$E := 6.63 \cdot 10^6 \cdot \frac{lbf}{in^2}$$

Poisson's ratio (as per laminate analysis):

$\nu := 0.19$

The ratio b/t must be greater than 10 for this procedure to be valid.

$$\frac{b}{t} = 14.167$$

The following variable K is used to calculate the critical unit compressive stress. The values are a function of the ratio a/b. In order to provide the user with a more complete range of K values, the following interpolation is defined:

$ratio_{ab} := (.5\ \ 1\ \ 1.2\ \ 1.4\ \ 1.6\ \ 1.8\ \ 2.0\ \ 2.5\ \ 3\ \ 4\ \ 5)$

$K := (3.62\ \ 1.18\ \ .934\ \ .784\ \ .687\ \ .622\ \ .574\ \ .502\ \ .464\ \ .425\ \ .416)$

$ratio := ratio_{ab}^{T}$

$K := K^{T}$

$KK(x) := \text{linterp}(ratio, K, x)$

$KK\left(\frac{a}{b}\right) = 0.41$

Determine critical unit compressive stress:

$x := \frac{a}{b}$

$\sigma'(x) := KK(x) \cdot \frac{E}{1 - \nu^2} \cdot \left(\frac{t}{b}\right)^2$

$\sigma'(x) = 1.406 \times 10^4 \frac{lbf}{in^2}$

Maximum axial stress in the beam (Loading was 185 psf over a 5 ft x 6 ft area):

$\sigma_{max} := 3709 psi$

Factor of safety for "Flange Crippling":

$FS_{fc} := \frac{\sigma'(x)}{\sigma_{max}}$

$FS_{fc} = 3.79$

The minimum desired factor of safety is 3, so the current flange design is sufficient. Therefore, according to the flange crippling analysis, the minimum required flange thickness is approximately 1/2 in.

4.3 Flange Shear-Out Analysis

Diameter of flange bolt holes:

$D := 0.75 \cdot in$

Thickness of the composite beam flange:

$t_f := 0.45 \cdot in$

Load per bolt in row with the highest percentage of the total load:

$P_{b3} := 2200 \cdot lbf$

Load per bolt in the row nearest the end of the composite beam (i.e., lowest percentage of the total load):

$P_{b1} := 1600 \cdot lbf$

Calculate shear-out stress in the composite flange for the applied load. For the analysis, select one of the bolts in the row closest to the end of the composite beam and also select one of the bolts in the row with the highest percentage of the total load.

Shear area between the first row of bolt holes in the flange and the end of the beam:

$A_{el} := 2 \cdot (3 \cdot D \cdot t_f)$

$A_{el} = 2.025 in^2$

Shear area between the second and third row of bolt holes in the flange:

$A_{23} := 2 \cdot (4 \cdot D \cdot t_f)$

$A_{23} = 2.7 in^2$

Shear-out stress between the first row of bolts and the end of the beam:

$$\tau_{el} := \frac{P_{b1}}{A_{el}}$$

$\tau_{el} = 790.123 psi$

Shear-out stress between the second and third row of bolts:

$$\tau_{23} := \frac{P_{b3}}{A_{23}}$$

$$\tau_{23} = 814.815\text{psi}$$

24 ksi is the approximate in-plane shear strength of composite beam:

$$\tau_{xy} := 24 \cdot 10^3 \cdot \text{psi}$$

Factor of safety for shear-out stress between the end of the beam and the first row of bolts:

$$FS_{e1} := \frac{\tau_{xy}}{\tau_{e1}}$$

$$FS_{e1} = 30.375$$

Factor of safety for shear-out stress between the second and third row of bolts:

$$FS_{23} := \frac{\tau_{xy}}{\tau_{23}}$$

$$FS_{23} = 29.455$$

It is apparent from this analysis that shear-out is not driving the design of the composite flange.

4.4 Flange Local Compression Analysis

Local compressive stress in composite due to pre-load (i.e., torque) applied to the flange bolts.

Initial torque on bolt:

$$T := 30 \cdot \text{ft} \cdot \text{lbf}$$

Standard lubrication constant for bolt:

$$k := 0.2$$

Tension "pre-load" resulting from the applied torque:

$$P_{TT} := \frac{T}{k \cdot D}$$

$$P_{TT} = 2.4 \times 10^{3}\,\mathrm{lbf}$$

Local compression area (i.e., area of 3/4 in. ID washer):

$$A_{lc} := 2.24\,\mathrm{in}^2$$

Local compressive stress:

$$\sigma_{lc} := \frac{P_{TT}}{A_{lc}}$$

$$\sigma_{lc} = 1.071 \times 10^{3}\,\mathrm{psi}$$

$$\sigma_c := 15 \cdot 10^{3} \cdot \mathrm{psi}$$

15 ksi is the approximate "thru-the-thickness" compressive strength for the composite cantilever support beam.

Factor of safety for local compressive stress in flange:

$$FS_{lc} := \frac{\sigma_c}{\sigma_{lc}}$$

$$FS_{lc} = 14$$

The desired factor of safety is 3, so the local compressive stress in the flange is acceptable.

4.5 Flange Tension (with stress concentration from bolt holes) Analysis

Diameter of flange bolt holes:

$$D_{bhf} := 0.75\,\mathrm{in}$$

Width of the flange:

$w_f := 12.75 \cdot in$

Thickness of the flange:

$t_f := 0.45 \cdot in$

Number of composite beam flange bolts:

$N_{bCF} := 12$

Number of rows of bolts in the flange:

$N_r := 3$

Number of bolts per row in the flange:

$$N_{br} := \frac{N_{bCF}}{N_r}$$

$N_{br} = 4$

Maximum resultant moment of beam:

$M_{max} := 33300 \, lbf \cdot ft$

The maximum resultant moment value is calculated by multiplying the 85 psf live load plus 100 psf static load over the 6 ft wide by 5 ft long section of the walkway.

Overall depth of beam

$h := 18 \cdot in$

Axial load induced in the beam from the resultant moment:

$$P_m := \frac{M_{max}}{h}$$

$P_m = 2.22 \times 10^4 \, lbf$

The axial load above was calculated by reducing the moment into a couple consisting of the overall depth of the beam, h, and a force, P_m. This is a conservative method to determine P_m because it assumes that the flange of the beam carries the entire load. In reality, the web of the beam will carry a finite amount of load, which reduces the burden on the flange of the beam.

Maximum allowable tension strength of composite beam:

$$\sigma_T := 65 \cdot 10^3 \cdot psi$$

The approximate longitudinal tensile strength of the cantilever support beam is approximately 65 ksi.

Net stress intensity factor:

$$K_{tn} := 2.82$$

The stress intensity factor (for isotropic materials) for multiple rows of holes in a thin, semi-infinite plate with the holes perpendicular to the loading direction was taken from Peterson's, "Stress Concentration Factors". Then the layup of the flange was considered and the stress intensity factor was adjusted accordingly.

Net cross sectional area of the flange:

$$A_n := w_f \cdot t_f - N_{br} \cdot D_{bhf} \cdot t_f$$

$$A_n = 4.387 in^2$$

Calculate maximum stress:

$$\sigma_{max} := K_{tn} \cdot \frac{P_m}{A_n}$$

$$\sigma_{max} = 1.427 \times 10^4 \, psi$$

Factor of safety:

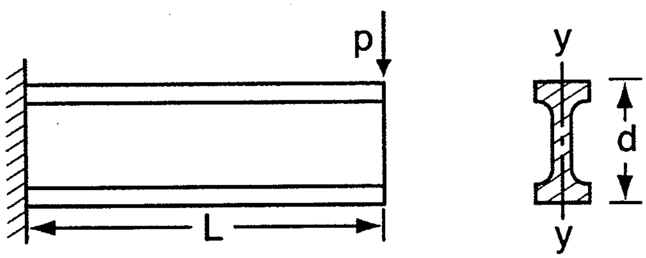
$$FS := \frac{\sigma_T}{\sigma_{max}}$$

$$FS = 4.555$$

The desired factor of safety is 3, so the tension stress in the flange due stress concentrations from the bolt hole is acceptable.

4.6 "Twist-Bend" Buckling Stability

For this analysis, the load is applied to the free end of the beam. For the actual cantilever support beam, the applied load acts on a point 6 ft from the fixed end. Therefore, the results of this "twist-bend" buckling analysis are conservative.



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Moment of inertia of the cross section about its vertical axis of symmetry:

$I_y := 155 \cdot in^4$

Flange Width:

$w := 12.75 \cdot in$

Flange Thickness:

$t_f := 0.45 \cdot in$

Moment of inertia of one flange about this axis of symmetry:

$I_f := \frac{1}{12} \cdot t_f \cdot w^3$

$I_f = 77.725 in^4$

Modulus of elasticity:

$E := 6.63 \cdot 10^6 \cdot \frac{lbf}{in^2}$

Modulus of rigidity:

$G := 2.57 \cdot 10^6 \cdot psi$

Torsional Stiffness:

$K := 0.924 in^4$

Depth, center to center of flange:

$d := 17 \cdot in + t_f$

$d = 17.45 in$

Beam length:

$L := 11 \cdot ft$

The critical load is calculated:

With m approximated by:

$$m := 4.01 + 11.7 \cdot \sqrt{\frac{I_f \cdot E \cdot d^2}{2 \cdot K \cdot G \cdot L^2}}$$

$m = 20.121$

The critical load is given by:

$$P' := \frac{m \cdot \sqrt{E \cdot I_y \cdot K \cdot G}}{L^2}$$

$P' = 5.705 \times 10^4 lbf$

The applied load on the composite beam is 5,550 lbf, which is calculated by multiplying 185 psf (live and snow loads combined) over the 6 ft wide by 5 ft long section of walkway.

Maximum load on the beam:

$P_{max} := 5550 lbf$

Factor of safety for "Twist-Bend Buckling":

$$FS_{tbb} := \frac{P'}{P_{max}}$$

$$FS_{tbb} = 10.279$$

The desired factor of safety is 3, so the "twist-bend" buckling critical load for this beam design is acceptable.

4.7 Web Bearing Stress Analysis

The bolts in the web of the cantilever support beam are loaded eccentrically and the result is that the shear loading on each web bolt is not the same. Therefore, the minimum required web thickness (for bearing) will be sized based on the maximum combined shear load in the bolt pattern. In addition, the web only reacts approximately 12 percent of the applied load. The majority of the applied load is reacted by the flanges of the cantilever support beam.

Diameter of web bolt holes:

$$D_{bhw} := 0.75 \cdot in$$

Maximum combined shear load on an individual bolt in the web:

$$P_W := 2.618 \cdot 10^3 \cdot lbf$$

Desired factor of safety:

$$FS_{LB} := 3.0$$

Maximum allowable compressive strength of composite beam:

$$\sigma_c := 65 \cdot 10^3 \cdot psi$$

Minimum required web thickness

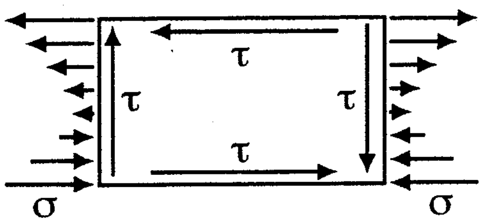
$$t_{wmin} := \frac{4 \cdot P_W \cdot FS_{LB}}{\pi \cdot D_{bhw} \cdot \sigma_c}$$

$$t_{wmin} = 0.205 in$$

For the bearing stress in the web to be acceptable, the minimum required web thickness must be approximately 1/4 in.

4.8 Web Buckling Stability Analysis

A rectangular plate under uniform shear on all edges and bending stresses on the "b" edges will simulate the web of the composite beam.



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width:

$a := 36 \cdot in$

height:

$b := 18 \cdot in$

thickness:

$t := 0.25 \cdot in$

Modulus of elasticity:

$E := 6.63 \cdot 10^{6} \cdot \frac{lbf}{in^{2}}$

Poisson's ratio:

$\nu := 0.19$

Maximum shear stress in the beam (Loading was 185 psf over a 5 ft x 6 ft area):

$\tau := 1364 \frac{lbf}{in^{2}}$

Solve for the critical bending stress by first finding the critical shear stress that would buckle the plate if acting alone and using that to find the critical bending stress:

$$\tau_{critical} = K_\tau \cdot \left(\frac{E}{1-\nu^2}\right)\cdot\left(\frac{t}{b}\right)^2$$

Where K_τ is solved for below:

$$ratio_{ab} := (1.0\ \ 1.2\ \ 1.4\ \ 1.5\ \ 1.6\ \ 1.8\ \ 2.0\ \ 2.5\ \ 3.0\ \ \infty)$$

$$K_\tau := (7.75\ \ 6.58\ \ 6.00\ \ 5.84\ \ 5.76\ \ 5.59\ \ 5.43\ \ 5.18\ \ 5.02\ \ 4.40)$$

$$ratio := ratio_{ab}{}^T$$

$$K_\tau := K_\tau{}^T$$

$$KK_\tau(x) := \mathrm{linterp}(ratio, K_\tau, x)$$

$$KK_\tau\left(\frac{a}{b}\right) = 5.43$$

$$\tau_{critical} := KK_\tau\left(\frac{a}{b}\right)\cdot\left(\frac{E}{1-\nu^2}\right)\cdot\left(\frac{t}{b}\right)^2$$

$$\tau_{critical} = 7.205\times 10^3\ \frac{lbf}{in^2}$$

Knowing $t_{critical}$, the critical bending stress is found by

$$\sigma_{critical} = K\cdot\left(\frac{E}{1-\nu^2}\right)\cdot\left(\frac{t}{b}\right)^2$$

Where K is found below ($ratio_\tau$ is the ratio of actual shear stress to shear stress that acting alone would be critical):

$ratio_{\tau} := (0\ \ .2\ \ .3\ \ .4\ \ .5\ \ .6\ \ .7\ \ .8\ \ .9\ \ 1)$

$K := (21.1\ \ 20.4\ \ 19.6\ \ 18.5\ \ 17.7\ \ 16.0\ \ 14.0\ \ 11.9\ \ 8.20\ \ 0.0)$

$ratio := ratio_{\tau}^{T}$

$K := K^{T}$

$KK(x) := \text{linterp}(ratio, K, x)$

$KK\left(\frac{\tau}{\tau_{critical}}\right) = 20.437$

$\sigma_{critical} := KK\left(\frac{\tau}{\tau_{critical}}\right)\cdot\left(\frac{E}{1-\nu^2}\right)\cdot\left(\frac{t}{b}\right)^2$

$\sigma_{critical} = 2.712\times 10^4 \frac{lbf}{in^2}$

Maximum axial stress in the beam (Loading was 185 psf over a 5 ft x 6 ft area):

$\sigma_{max} := 3709\,psi$

Factor of safety for web buckling:

$FS_{wb} := \frac{\sigma_{critical}}{\sigma_{max}}$

$FS_{wb} = 7.311$

The desired factor of safety is 3, so the web buckling critical load for this beam design is acceptable.

Once the cantilever support beam was sized for strength, the resulting maximum deflection under the specified loading was examined. According to the AASHTO specification for pedestrian bridges, the deflection requirement of the cantilever support beam is L/300 for live loads, where L is the length of the support. The predicted maximum deflection of the beam under live load (i.e., 85 psf) approximately 0.16 in. or L/825. This predicted maximum deflection of the composite beam is well within the limit set forth in the aforementioned AASHTO specification.

Sizing the cantilever support beam for deflection (i.e., L/300 at live load), rather than ultimate strength, and adding extra plies to the bolted region of the beam was explored but it was determined that the resultant design was unacceptable. To satisfy the AASHTO deflection requirement, the minimum required flange thickness was less than 1/8 in. However, the minimum required flange thickness for "net" tension with stress concentration near the bolts holes was approximately 1/2 in. This is too large of a discrepancy between the flange thickness in the bolted region and the nominal flange thickness of the beam.

5. Bolt Sizing

In addition to sizing the composite beam, the hardware (i.e., bolts) was sized for the same loading (i.e., 185 psf over a 5 ft x 6 ft area) with a factor of safety (FS) of 3. The hardware used in the flange, web and the region where the cantilever support beam is bolted to the fascia beam of the existing bridge will be 3/4 in. UNC, Grade 8, steel bolts.

5.1 Fascia Beam Bolt Sizing

Bolt (pitch) diameter:

$$D_{bFB} := 0.652 \cdot in$$

The pitch diameter for a 3/4 in. UNC bolt is 0.652 in.

Number of fascia beam bolts:

$$N_{bFB} := 12$$

Length of bolt:

$$L_b := 3 \cdot in$$

Maximum applied moment:

$$M := 33300 \, lbf \cdot ft$$

Total shear load on fascia beam bolts:

$$V := 5550 \, lbf$$

Factor of safety on the design load:

$$FS := 3.0$$

Length of the composite beam:

$$L_B := 11 \cdot ft$$

Total height of the attachment:

$h_a := 17.0 \cdot in$

The equation for P_T assumes a 12-hole bolt pattern, evenly spaced in the height direction (i.e., the bolt row spacing is $h_a/7$, where h_a is the height of the attachment fitting).

Maximum tensile load on a single bolt:

$$P_T := \frac{3 \cdot M}{13 \cdot h_a}$$

$$P_T = 5.424 \times 10^3 \, lbf$$

Maximum tensile stress in the fascia beam bolts:

$$\sigma_T := \frac{P_T}{\frac{\pi}{4} \cdot D_{bFB}{}^2}$$

$$\sigma_T = 1.625 \times 10^4 \, psi$$

The maximum bending stress on a single bolt in the fascia beam was calculated using superposition. The bending stress was calculated using the bolt's pitch diameter. It was assumed that only half of the fascia beam bolts take the shear load, *V*.

Maximum bending stress in one (1) fascia beam bolt (analysis assumed fixed-guided end conditions):

$\sigma_b := 11.1 \cdot 10^3 \cdot psi$

The additional tension stress on a fascia beam bolt, which results when applying an initial torque or "pre-load" was calculated.

Initial torque on a fascia beam bolt:

$T := 30 \cdot ft \cdot lbf$

Standard lubrication constant for bolt:

$k := 0.2$

Tension "pre-load" resulting from the applied torque:

$$P_{TT} := \frac{T}{k \cdot D_{bFB}}$$

$$P_{TT} = 2.761 \times 10^{3}\,\text{lbf}$$

Tension stress in bolt resulting from applied torque:

$$\sigma_{TT} := \frac{P_{TT}}{\frac{\pi}{4} \cdot D_{bFB}^{\,2}}$$

$$\sigma_{TT} = 8.269 \times 10^{3}\,\text{psi}$$

Material Properties of Grade 8, High-Strength, Medium Carbon Steel bolts

Nominal ultimate tensile strength:

$$F_{Tu} := 150 \cdot 10^{3} \cdot \text{psi}$$

Nominal ultimate shear strength (threads excluded from the shear plane):

$$F_{Su} := 75 \cdot 10^{3} \cdot \text{psi}$$

Nominal yield tensile strength:

$$F_{Ty} := F_{Tu} \cdot 0.8$$

$$F_{Ty} = 1.2 \times 10^{5}\,\text{psi}$$

Nominal yield shear strength (threads excluded from the shear plane):

$$F_{Sy} := F_{Su} \cdot 0.8$$

$$F_{Sy} = 6 \times 10^{4}\,\text{psi}$$

Young's Modulus for bolt material:

$$E_{b} := 30 \cdot 10^{6} \cdot \text{psi}$$

Calculate bolt "Spring Constant" (calculation below for a 3 in. long bolt):

$$K_b := \frac{\frac{\pi}{4} \cdot D_{bFB}^{2} \cdot E_b}{L_b}$$

$$K_b = 3.339\times 10^{6} \frac{lbf}{in}$$

Maximum elongation of fascia beam bolts: (This will occur in the top row of bolts)

$$\delta_b := \frac{P_T}{K_b}$$

$$\delta_b = 1.625\times 10^{-3} in$$

Amount of rotation of the composite beam due to elongation of the fascia beam bolts:

$$\alpha := \mathrm{atan}\left(\frac{\delta_b}{h_a}\right)$$

$$\alpha = 5.476\times 10^{-3} deg$$

Component of composite beam tip deflection due to elongation of fascia beam bolts:

$$y_{tip\alpha} := \tan(\alpha) \cdot L_B$$

$$y_{tip\alpha} = 0.013 in$$

Component of composite beam tip deflection due to live load (85 psf x 5 ft x 6 ft):

$$y_{tipL} := 0.14 in$$

Total tip deflection of composite beam:

$$y_{tip} := y_{tipL} + y_{tip\alpha}$$

$$y_{tip} = 0.153 in$$

Ratio of tip deflection to total length of composite beam:

$$yr := \frac{L_B}{y_{tip}}$$

$$yr = 865$$

This ratio must be at least 300.

Relation for combined loading of fascia beam bolt under both tension and shear loading (at yield):

$$AM_{FB} := \frac{\left(\frac{FS \cdot V}{\frac{N_{bFB}}{2}}\right)^3}{\left(F_{Sy} \cdot \frac{\pi}{4} \cdot D_{bFB}^{\ 2}\right)^3} + \frac{\left\lfloor FS \cdot \left(\sigma_T + \sigma_{TT}\right)\right\rfloor^2}{F_{Ty}^{\ 2}}$$

$$AM_{FB} = 0.378$$

The above ratio must be less than 1 at the specified factor of safety.

Factor of safety of the fascia beam bolts in tension (at yield):

$$FS_{FBb} := \frac{F_{Ty}}{\sigma_b + \sigma_T + \sigma_{TT}}$$

$$FS_{FBb} = 3.369$$

The desired factor of safety is 3, so the spacing and size of the fascia beam bolts are acceptable.

5.2 Flange Bolt Sizing

Bolt (pitch) diameter:

$$D_{bCF} := 0.652 \cdot in$$

The pitch diameter for a 3/4 in. UNC bolt is 0.652 in.

Number of composite beam flange bolts:

$$N_{bCF} := 12$$

Load sharing among three (3) rows of bolts in the flange assuming 3/4 in. UNC bolts with end, row and column spacing of $3D_{bCF}$, $4D_{bCF}$ and $4D_{bCF}$, respectively, results in a maximum per bolt load of P_{bmax}.

Load per bolt in row with the highest percentage of the total load:

$$P_{bmax} := 2200 \cdot lbf$$

The maximum bending stress on a single bolt in the composite flange was calculated using superposition. For the analysis, select one of the bolts in the row with the highest percentage of the total load.

Maximum bending stress in one composite flange bolt (analysis assumed fixed-guided end conditions):

$$\sigma_b := 27.2 \cdot 10^3 \cdot psi$$

The additional tension stress on a composite flange bolt resulting from an initial torque or "pre-load" was calculated.

Initial torque on a flange bolt:

$$T := 10 \cdot ft \cdot lbf$$

Standard lubrication constant for bolt:

$$k := 0.2$$

Tension "pre-load" resulting from the applied torque:

$$P_{TT} := \frac{T}{k \cdot D_{bCF}}$$

$$P_{TT} = 920.245 lbf$$

Tension stress in bolt resulting from applied torque:

$$\sigma_{TT} := \frac{P_{TT}}{\frac{\pi}{4} \cdot D_{bCF}^{\ 2}}$$

$$\sigma_{TT} = 2.756 \times 10^3 psi$$

Material Properties of Grade 8, High-Strength, Medium Carbon Steel bolts

Nominal ultimate tensile strength:

$$F_{Tu} := 150 \cdot 10^{3} \cdot psi$$

Nominal ultimate shear strength (threads excluded from the shear plane):

$$F_{Su} := 75 \cdot 10^{3} \cdot psi$$

Nominal yield tensile strength:

$$F_{Ty} := F_{Tu} \cdot 0.8$$

$$F_{Ty} = 1.2 \times 10^{5} psi$$

Nominal yield shear strength (threads excluded from the shear plane):

$$F_{Sy} := F_{Su} \cdot 0.8$$

$$F_{Sy} = 6 \times 10^{4} psi$$

Factor of safety of the composite flange bolts in tension (at yield):

$$FS_{Fb} := \frac{F_{Ty}}{\sigma_b + \sigma_{TT}}$$

$$FS_{Fb} = 4.006$$

Factor of safety of the composite flange bolts in shear (at yield):

$$FS_{Fs} := \frac{F_{Sy} \cdot \left(\frac{\pi}{4} \cdot D_{bCF}^{2} \right)}{P_{bmax}}$$

$$FS_{Fs} = 9.106$$

The desired factor of safety is 3, so the spacing and size of the flange bolts are acceptable.

5.3 Web Bolt Sizing

As mentioned previously, the bolts in the web of the cantilever support beam are loaded eccentrically and the result is that the shear loading on each web bolt is not the same. Therefore, the size of the web bolts will be sized based on the maximum combined shear load in the bolt pattern. Also, the web bolts are loaded in a "double-clevis" configuration, which serves to double the shear area of the bolt.

Bolt (pitch) diameter:

$D_{bCW} := 0.652 \cdot in$

The pitch diameter for a 3/4 in. UNC bolt is 0.652 in.

Number of web bolts:

$N_{bCW} := 9$

Material Properties of ASTM A490 Steel bolts

Nominal ultimate tensile strength:

$F_{Tu} := 113 \cdot 10^3 \cdot psi$

Nominal ultimate shear strength (threads excluded from the shear plane):

$F_{Su} := 75 \cdot 10^3 \cdot psi$

Nominal yield tensile strength:

$F_{Ty} := F_{Tu} \cdot 0.8$

$F_{Ty} = 9.04 \times 10^4 \, psi$

Nominal yield shear strength (threads excluded from the shear plane):

$F_{Sy} := F_{Su} \cdot 0.8$

$F_{Sy} = 6 \times 10^4 \, psi$

The maximum bending stress on a single bolt in the composite web was calculated.

Thickness of attachment fitting where it is in contact with the composite beam web:

$t_1 := 0.5 \cdot in$

Gap distance due to tolerance issues between the composite beam web and the attachment fitting:

$g := 0.01 \cdot in$

Thickness of the composite beam web:

$t_2 := 0.44 \cdot in$

Moment arm for a single bolt if there is "slop" in the web bolt hole:

$$b := \frac{t_1}{2} + g + \frac{t_2}{4}$$

$b = 0.37in$

Maximum combined shear load on one bolt in the web:

$P := 2.618 \cdot 10^3 \cdot lbf$

Moment on a single bolt due to "slop" in the composite web bolt hole:

$$M_b := \frac{P}{2} \cdot b$$

$M_b = 484.33 lbf \cdot in$

Tensile stress on a single bolt in composite web due to bending:

$$\sigma_b := \frac{M_b \cdot \frac{D_{bCW}}{2}}{\frac{\pi}{4} \cdot \left(\frac{D_{bCW}}{2}\right)^4}$$

$\sigma_b = 1.78 \times 10^4 psi$

The additional tension stress on a composite web bolt, which results when applying an initial torque or "pre-load", was calculated.

Initial torque on a web bolt:

$T := 10 \cdot ft \cdot lbf$

Standard lubrication constant for bolt:

$k := 0.2$

Tension "pre-load" resulting from the applied torque:

$$P_{TT} := \frac{T}{k \cdot D_{bCW}}$$

$P_{TT} = 920.245 lbf$

Tension stress in bolt resulting from applied torque:

$$\sigma_{TT} := \frac{P_{TT}}{\frac{\pi}{4} \cdot D_{bCW}^{2}}$$

$\sigma_{TT} = 2.756 \times 10^{3} psi$

Factor of safety of the composite web bolts in bending (at yield):

$$FS_{Wb} := \frac{F_{Ty}}{\sigma_b + \sigma_{TT}}$$

$FS_{Wb} = 4.398$

Factor of safety of the composite web bolts in shear (at yield):

$$FS_{Ws} := \frac{F_{Sy} \cdot \left(\frac{\pi}{4} \cdot D_{bCW}^{2} \right)}{\frac{P}{2}}$$

$FS_{Ws} = 15.304$

The desired factor of safety is 3, so the spacing and size of the web bolts are acceptable.

6. Walkway Design

The walkway portion of the composite sidewalk system was designed solely by E.T. Techtonics. All schematics and background analysis, as supplied by E.T. Techtonics, are presented in Appendix B.

7. Cantilever Support Fabrication

The cantilever support beams were manufactured by Acme Fiberglass, Inc. of Hayward, CA. The beams consist of woven carbon fabric and epoxy resin with a fiber volume fraction of approximately 40 percent. The fabric consists of Grafil G34-700, 12K carbon fiber tows oriented in a 2 x 2 twill weave with an areal weight of 20 oz/yd^2. The fabric is supplied by Textile Products, Inc. of Anaheim, CA. The epoxy resin system used was Epoxical 2124A Resin/9283B Hardener.

The lay-up procedure for the cantilever support beams began with placing four layers of carbon fabric (135 in. x 31 in.) on to a mold alternating with four layers of carbon fabric (12 in. x 31 in.), which was cut at a ±45 deg angle. The ±45 deg layers were placed at the attachment (i.e., thicker) end of the beam. Each layer of carbon fabric was wet out with epoxy resin. Vacuum pressure was applied to the lay-up until the resin began to gel. The above procedure was repeated using a second mold. The vacuum bags were removed from each component and all surfaces were sanded in preparation for bonding the two components together. A wet out layer of carbon fabric was used to bond the two components together. Both molds were clamped together and held for 8 to 10 hr. After the clamps were removed, the lengthwise grooves were sanded and filled with unidirectional carbon fiber and epoxy resin. The next step in the fabrication process involved laying up 11 layers of carbon fabric on to both sides of the part. Also, two additional layers of ±45 deg carbon were added to each side. As mentioned before, all layers of carbon fabric were wet out with epoxy resin. The part was post cured for 8 hr at 150°F. After the part had cured, it was removed from the molds and trimmed to specification. Lastly, all surfaces were coated with one layer of resin. The above process was repeated until the desired number of beams was produced. Please refer to Figures 9 to 13 for schematics of the fabrication process. Each cantilever support beam weighed approximately 125 lb. The steel attachment fittings, which consisted of welded 5/8 in. thick steel plate, were fabricated by Mills Machine Works in Lawrence, MA. Each steel attachment fitting weighed about 50 lb. Each beam required two attachment fittings, one on either side of the web.

8. Cantilever Support Testing

All testing was conducted at the University of New Hampshire (UNH) by Dr. Robert Steffen of the Department of Civil Engineering. The cantilever support beams and steel attachment fittings were delivered to the UNH testing facility and assembled on site. The testing included a static test of a single cantilever support beam followed by a 6 month creep test, also using a single beam. For both tests the cantilever support beam was cantilevered from a 3 ft wide x 2 ft-6 in. tall x 6 ft-7 in. long, "fixed" concrete support, which was created specifically for these tests. Figure 14 shows an elevation view of the support. The support is fixed to the ground by two

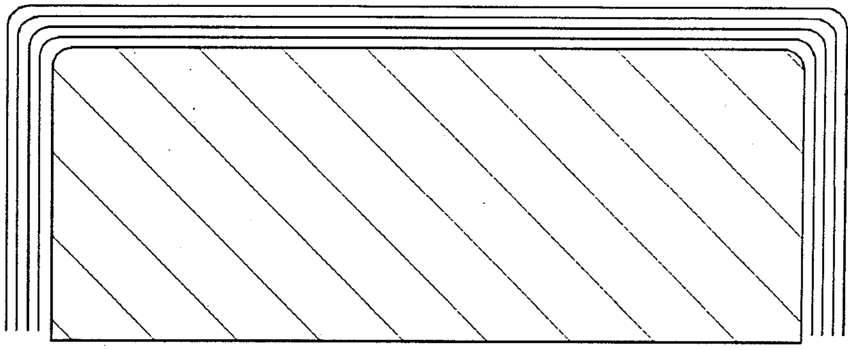

Figure 9. Lay-up procedure - full length plies

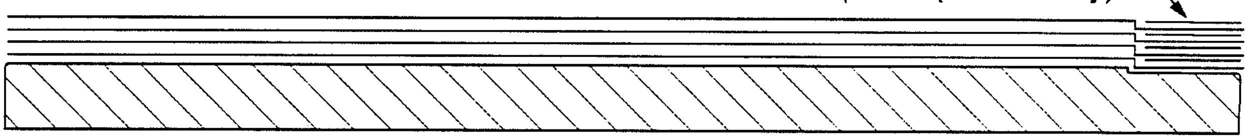

Figure 10. Lay-up procedure - ±45 deg pad-up plies

1-1/4 in. threaded rods, which run vertically through the support, and tie into the strong floor of the UNH structural laboratory. A 14 in. x 18 in. x 1 in. thick steel plate was placed on the concrete face shown in Figure 14 to create a level surface for the steel attachment fittings. Each beam has two attachment fittings, one on either side of the web. The attachment fittings are bolted to the flanges and web of the cantilever support beam. There are twelve 3/4 in. bolt holes in both the top and bottom flanges and there are nine bolt holes in the web of the beam. Steel shims were used between the steel plate and the support to level the specimen. Twelve 3/4 in. diameter threaded steel rods, which run longitudinally through the support, were used to attach the beam to the support. A torque of 150 ft-lb was applied to the nuts that connected the steel attachment fittings to the concrete support, and 50 ft-lb of torque was applied to the nuts that connected the steel attachment fittings to the cantilever support beam.

The first specimen was tested under a statically applied load up to the design load of 16,650 lb (i.e., 185 psf over a 5 ft x 6 ft area). The configuration of the test is depicted in Figure 15. Additional images of the static test setup are presented in Figures 16 to 18. Strain gages were used to measure strain at various locations in the section, and linear varying displacement

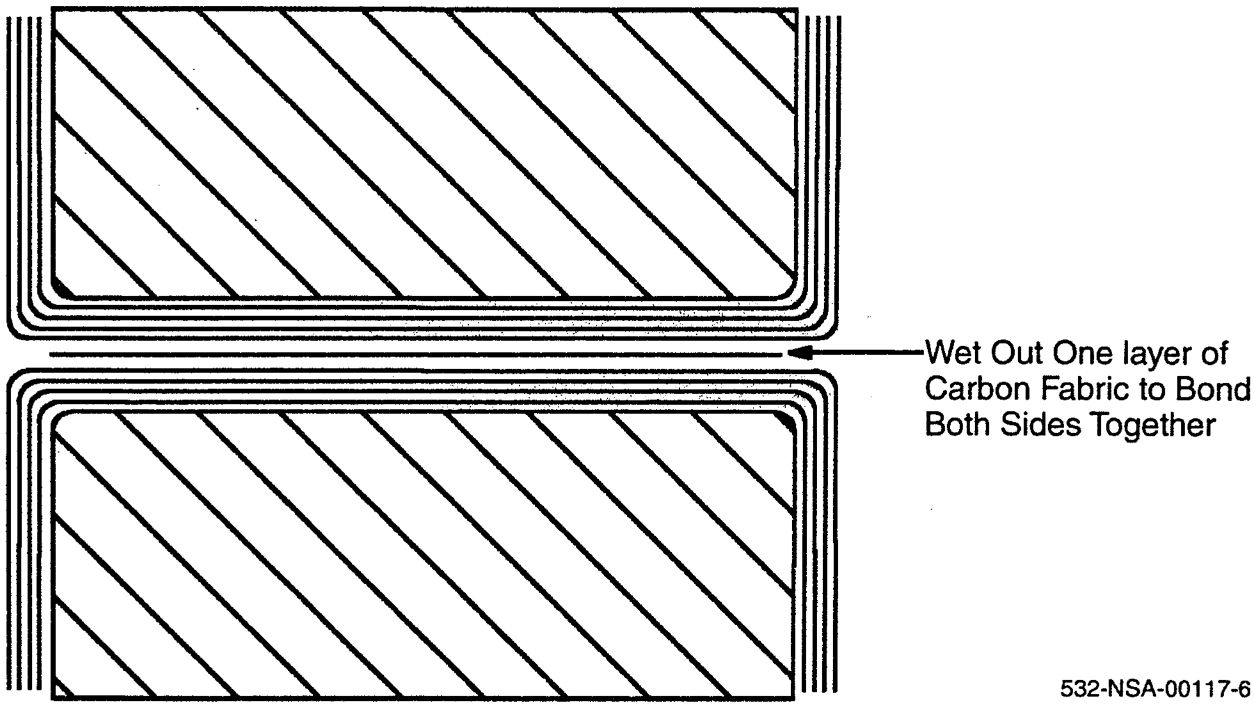

Figure 11. Lay-up procedure - forming I-beam

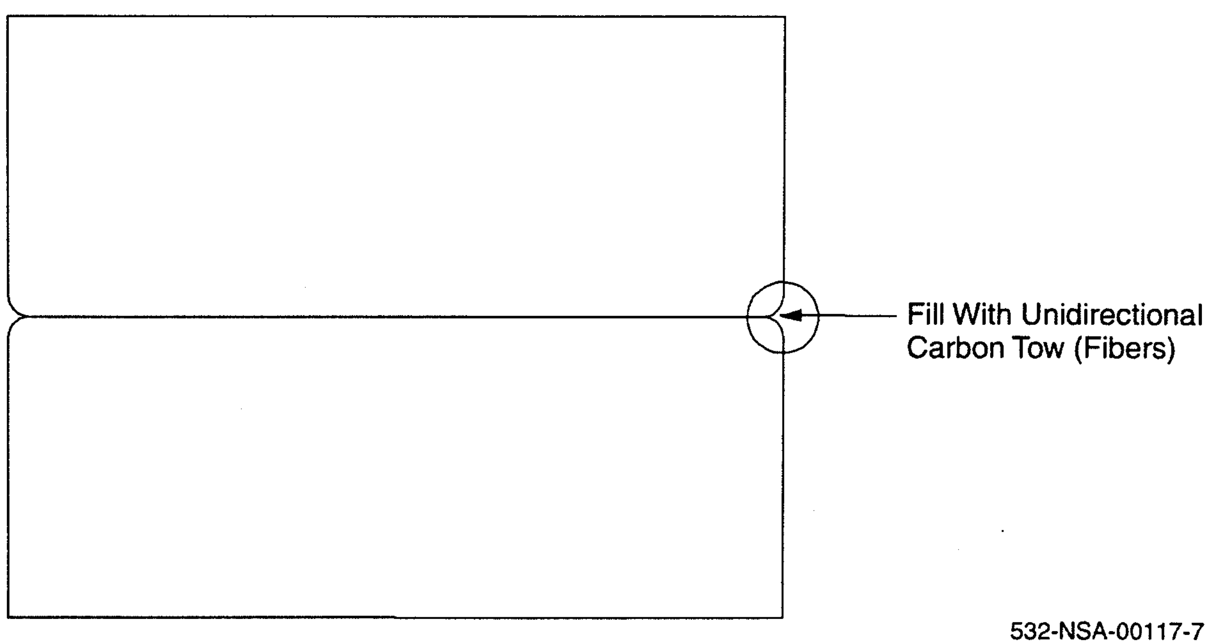

Figure 12. Lay-up procedure - filling gap

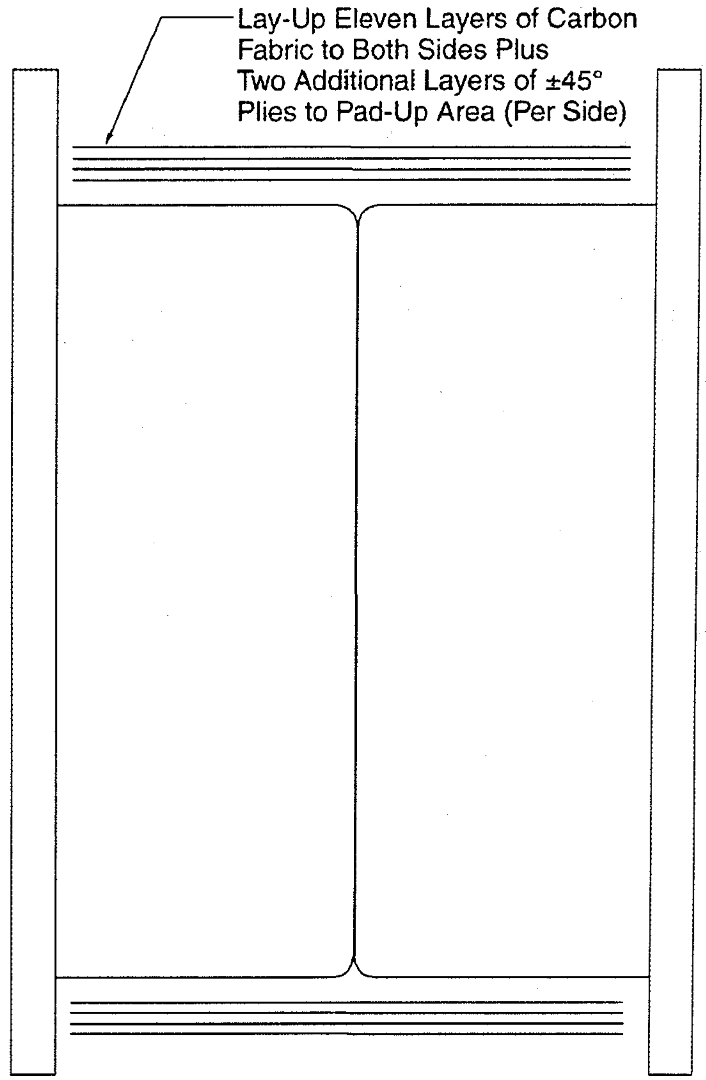

Figure 13. Lay-up procedure - building flanges

transducers (LVDTs) were used to measure deflections. One LVDT was placed at the centerline of loading, one was placed 3 ft from the free end of the beam (the outermost point of the actual pedestrian walkway), and one was placed on each side of the beam at the free end. All LVDTs measured the vertical deflection of the bottom flange of the beam. Load was applied by means of a manually controlled hydraulic ram at a rate of approximately 2000 lb/min and recorded by means of a calibrated load cell placed under the hydraulic ram. Load, strain, and deflections were recorded by an Optum® electronics data acquisition system at a rate of 1 scan/sec for the duration of the test.

A plot of applied load versus end of the walkway deflection is presented in Figure 19. Approximately 20 strain gages were used to gather data during the test. A schematic of all the strain gage locations is shown in Figure 20. Figure 21 shows a plot of stress versus strain for gage No. 2, which is located near one of the bolt holes in the top flange. In the preceding figure,

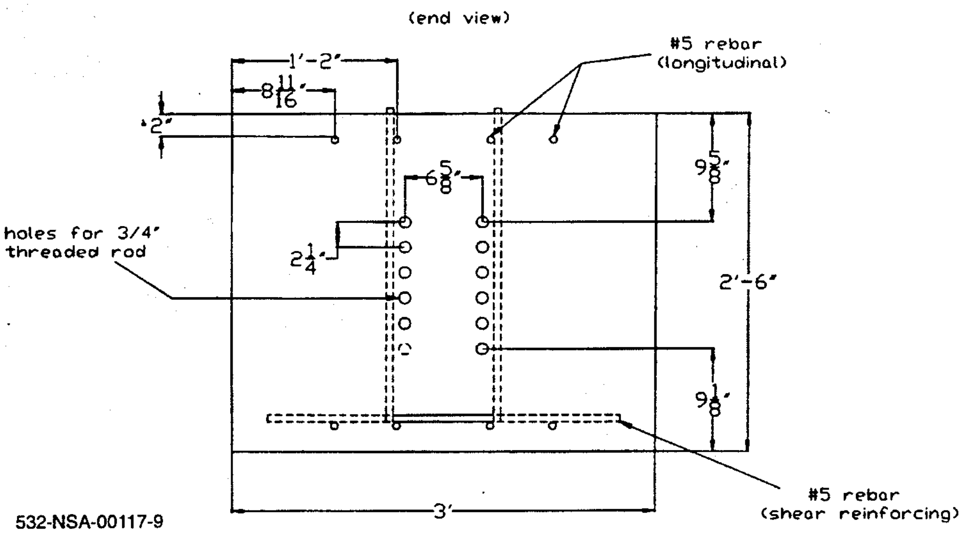

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Figure 14. Concrete fixed support (elevation)

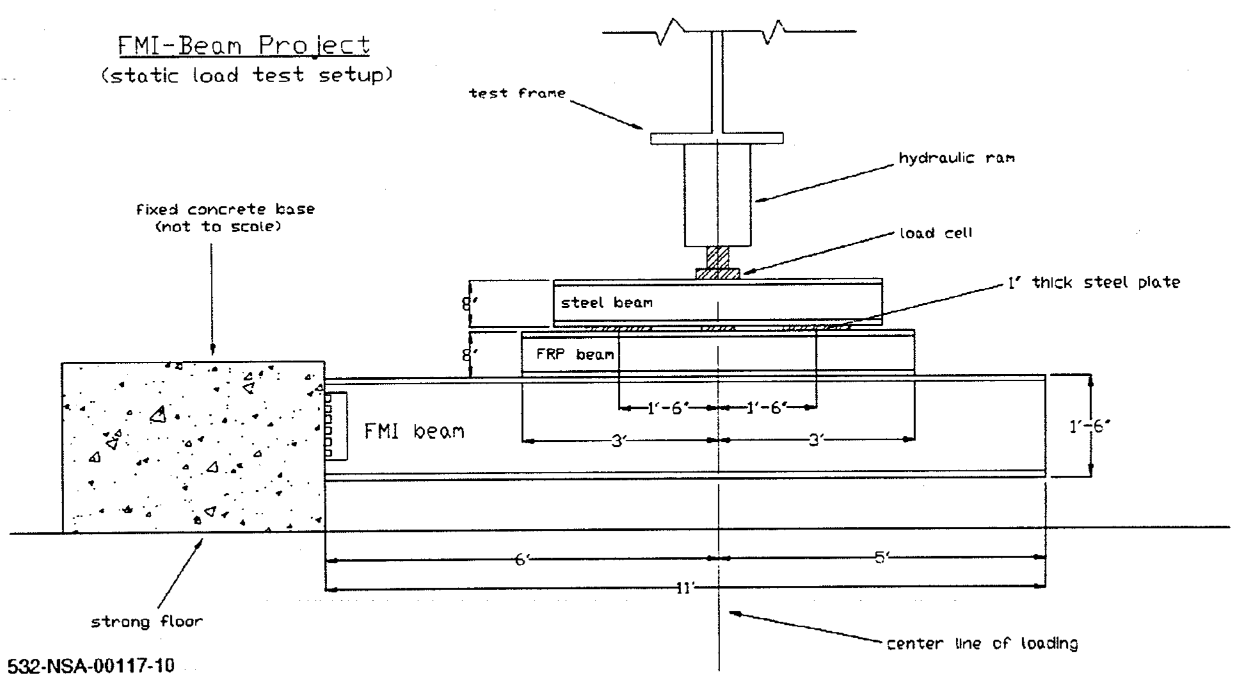

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Figure 15. Static test configuration

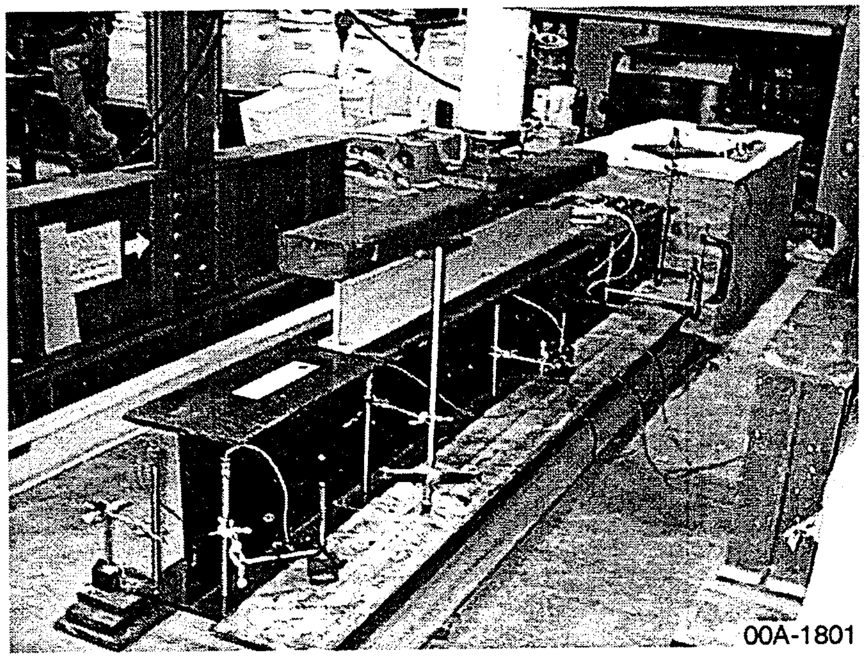


Figure 16. Static test setup

there is a "trend line" that is also plotted with the test data. This "trend line" was determined by first calculating the modulus of the test specimen and then using this value along with Hooke's Law (i.e., $\sigma = E\varepsilon$) to generate a stress-strain curve. The modulus was determined using the applied load data (i.e., applied moment) to calculate stress at a specific location and then the stress data was divided by the experimental strain data. Figure 22 shows a plot of stress versus strain for gage No. 4, which is located near the tapered region of the top flange. Figures 23 and 24 show a plot of stress versus strain for gages No. 5 and No. 6, which are further outboard of the tapered region on the top flange of the beam. It is shown in Figures 21 to 24 that the stresses in the beam decrease as the distance from the bridge attachment location increases. The largest stress observed upon inspection of the test data was approximately 10 ksi. This is far lower than the predicted tensile strength of the composite. The predicted strengths for this composite material were taken from MIL-HDBK-17 for an all-woven carbon/epoxy laminate. According to this reference, the predicted tensile, compressive and shear strengths were approximately 70 ksi, 73 ksi and 21 ksi, respectively. An average modulus value of 8.0 Msi was calculated from the experimental data. A modulus of about 7.0 Msi was predicted.

It was predicted prior to testing that the most likely location for failure in the test specimen would be the region near the flange bolt holes due to stress concentrations. There were no audible or visual signs of failure during the static test up to and including the design load of 16,650 lb. The deflection requirement of the cantilever support beam was L/300 for live loads, where L is the length of the support. For this test, which assumed a beam spacing of 5 ft on center, the live load (i.e., 85 psf) was approximately 2,500 lb. The allowable deflection was about 0.44 in. The observed deflection at 2,500 lb during the test was approximately 0.25 in., which satisfies the deflection requirement.

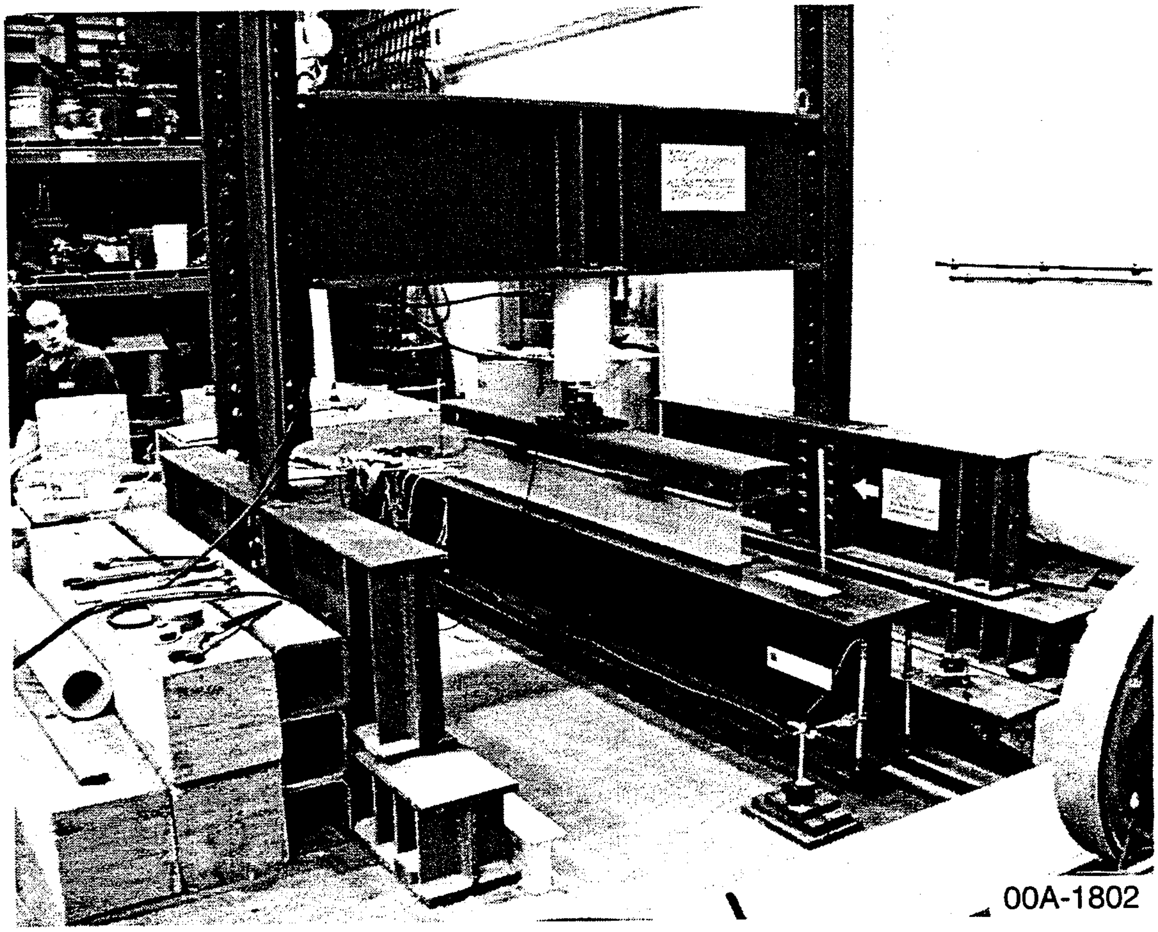


Figure 17. Static test setup

The creep test is currently underway at UNH. The test was designed to monitor the strains and deflections of the beam over a period of six months. The configuration of the strain gages and LVDTs is identical to the static test configuration. Three rectangular concrete beams, 1 ft wide x 1 ft high x 6 ft-3 in. long, were stacked on the test beam as shown in Figure 25. An additional image of the creep test setup is presented in Figures 26. The concrete beams simulate a uniform load over a 6 ft-3 in. long section. The total approximate weight of all three concrete beams is 2800 lb. Data was acquired at a rate of 1 scan/sec for the first 3 min after the total load was applied. For the next 30 min, a rate of 1 scan/min was used. For the remainder of the test, a rate of 1 scan every 8 min will be used.

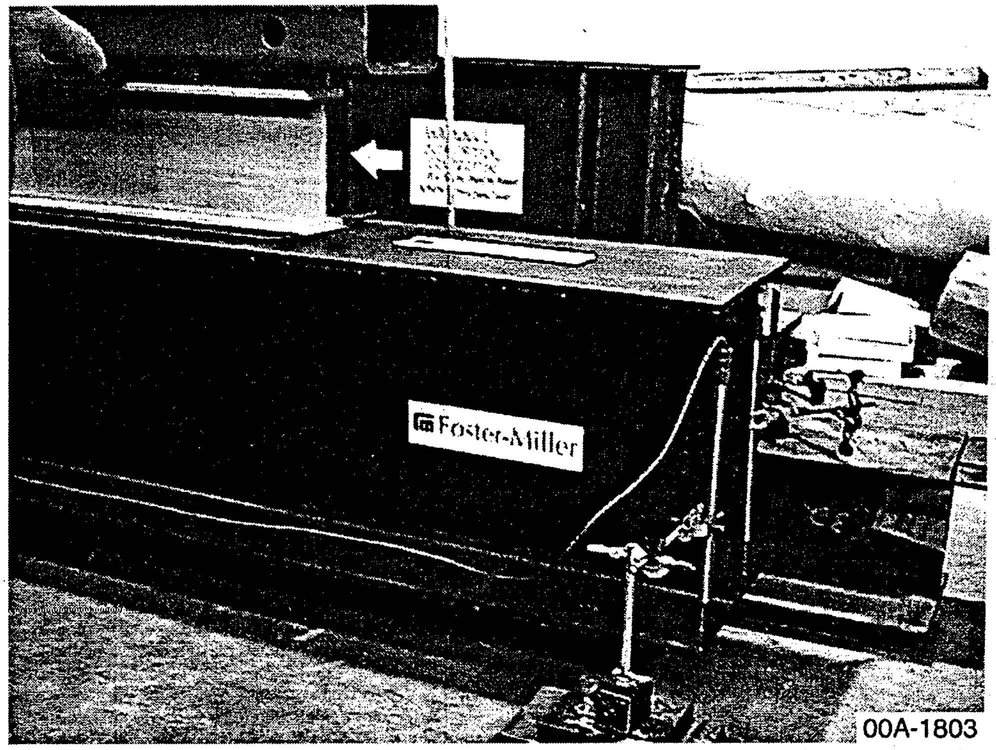


Figure 18. Static test setup

9. Conclusions

The Foster-Miller team has successfully designed and demonstrated an all composite sidewalk system. Structural tests of the support beams, as well as initial weight and cost estimates, indicate that the system will meet all performance requirements. A site has been selected by the Vermont Agency of Transportation for the first field installation of this system within the next year.

The objective of this effort was to design and develop a lightweight, corrosion free, cantilevered composite sidewalk system for roadway bridges. The composite sidewalk system designed by Foster-Miller, with assistance from E.T. Techtonics, consists entirely of fiber composite materials. Key specifications include the following:

- The cantilever support beam, which was designed exclusively by Foster-Miller, is a constant cross section, carbon fiber/epoxy 18 in. deep I-beam.
- The weight of each 11 ft long I-beam is approximately 125 lb.
- In the static test, the beam supported a 125,000 ft-lb moment at the connection, exceeding the safety factor of 3 on the design loading condition (85 psf live plus 100 psf snow load).

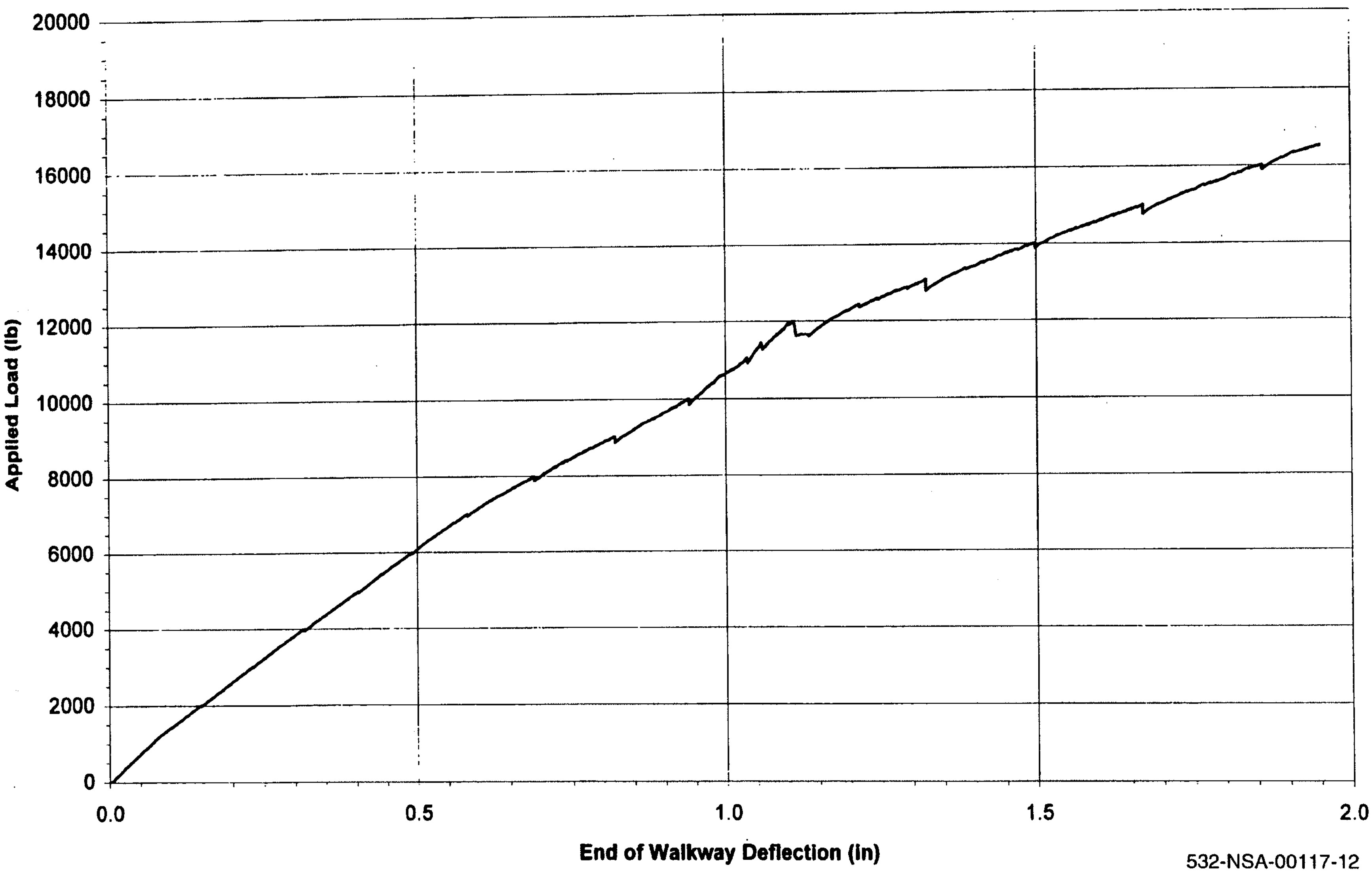


Figure 19. Applied loading versus end of walkway deflection (end of walkway is 9 ft from the bridge attachment location)

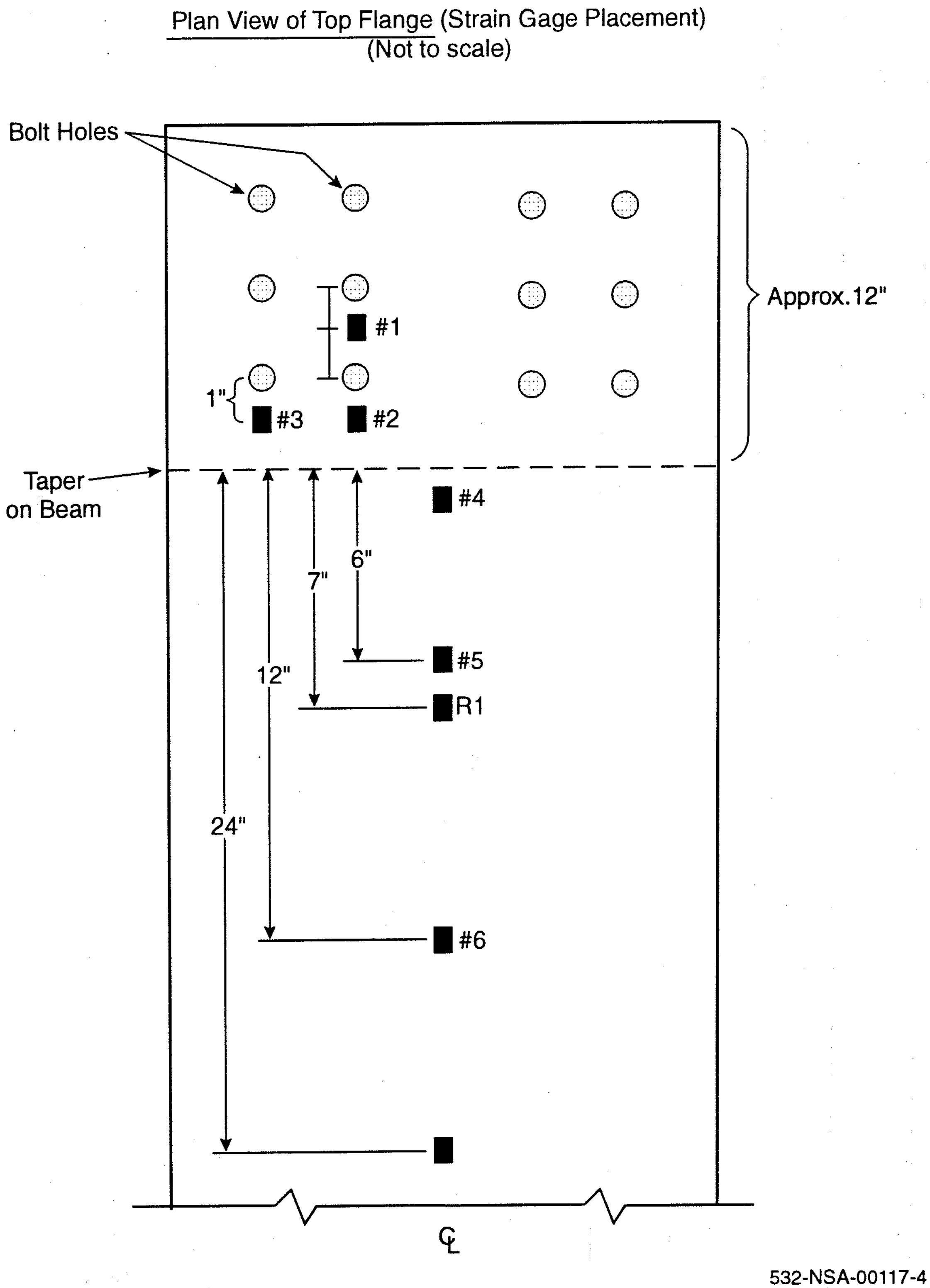
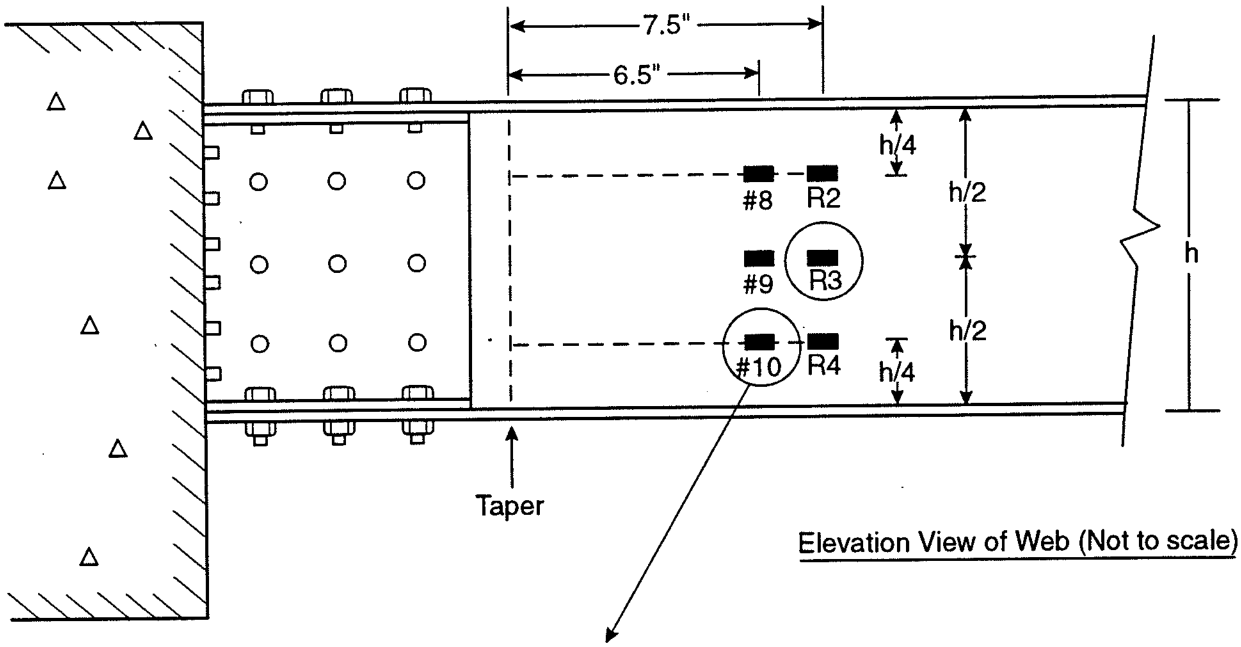


Figure 20. Schematic of all strain gage locations



*Note: Strain gage #15 is located on opposite side of web, back-to-back with strain gage #10

*Note: Rosette gage "R5" is located on opposite side of web, back-to-back with rosette gage "R3"

Plan View of Bottom Flange (Not to scale)

*Note: Strain gage #14 is located on opposite side of web, back-to-back with strain gage #13

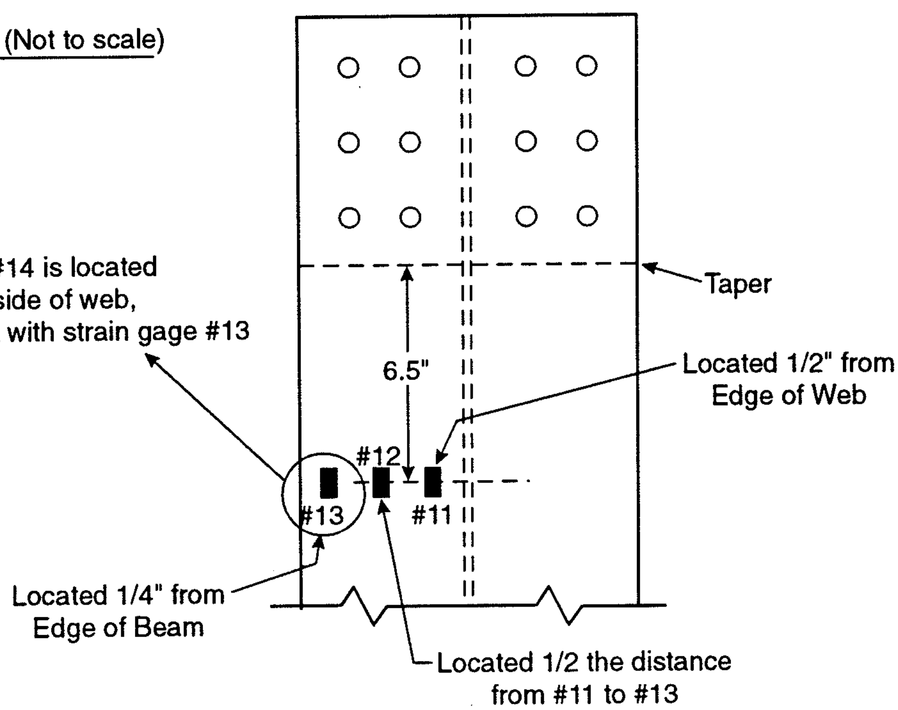


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Figure 20. Schematic of all strain gage locations (continued)

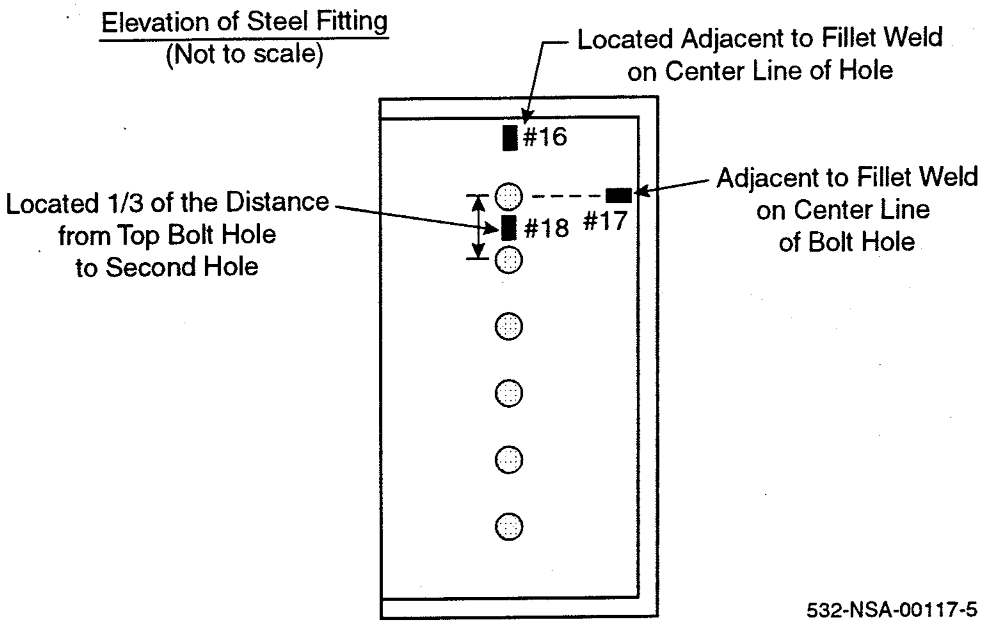

Figure 20. Schematic of all strain gage locations (continued)

- The beam also outperformed the L/300 design deflection test requirement from the AASHTO specification, "Guide Specifications for Design of Pedestrian Bridges."

- The sidewalk system superstructure, which was designed by E.T. Techtonics, is a 6 ft wide truss structure that is comprised of pultruded glass/polyester beams. It is rated for the same loading conditions as the beams.

- The total system is estimated to weigh 100 lb per running foot, approximately one-third of the estimated weight of a comparable steel and concrete system.

Directly due to the successful completion of this program, the Foster-Miller all-composite sidewalk system is slated for its first installation on Blakely Road in Colchester, VT. The Foster-Miller team, working closely with VAOT, is pursuing funding for final design, installation, testing and long-term monitoring of this system.

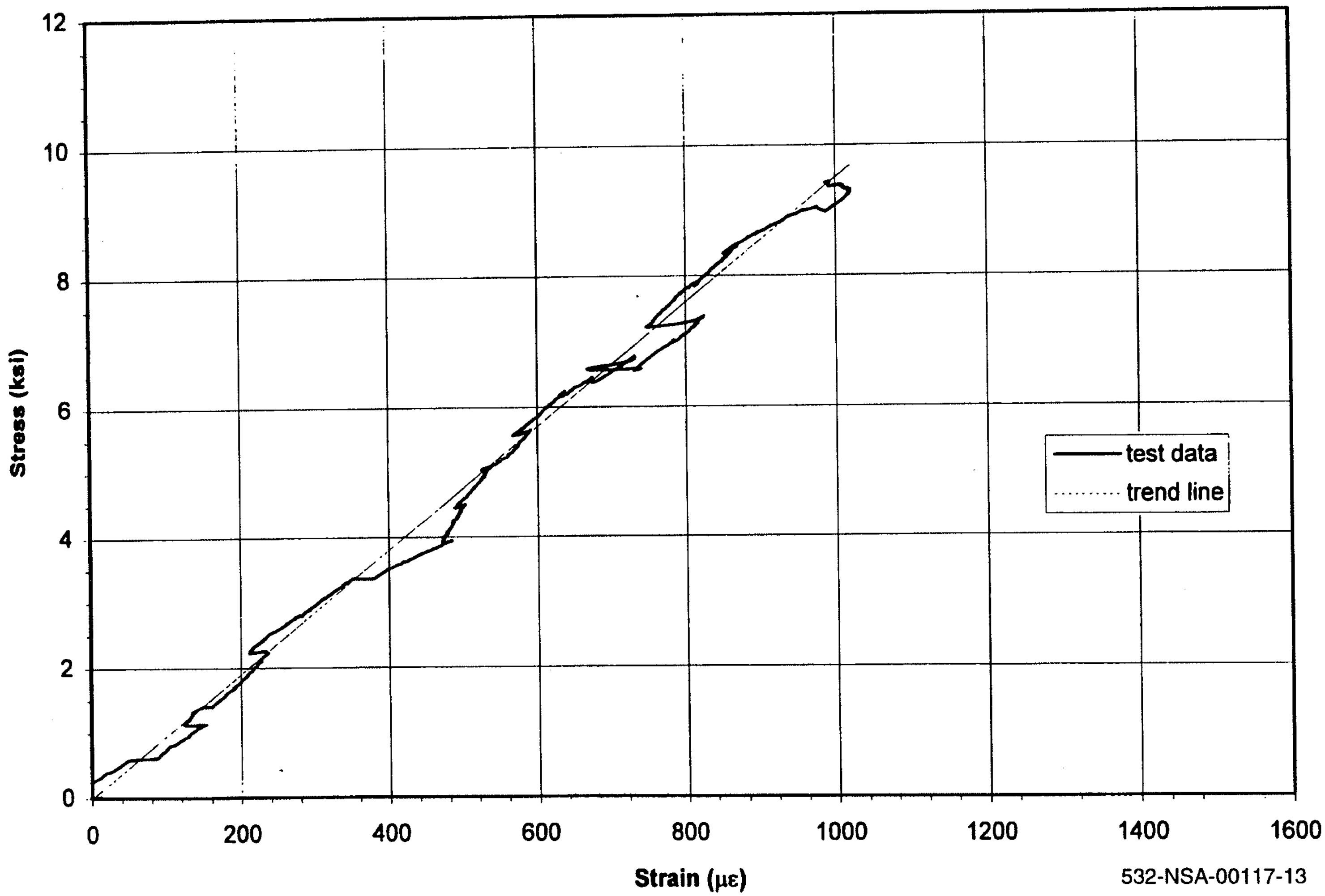

Figure 21. Stress versus strain data for composite cantilever support during static test (strain gage location No. 2)

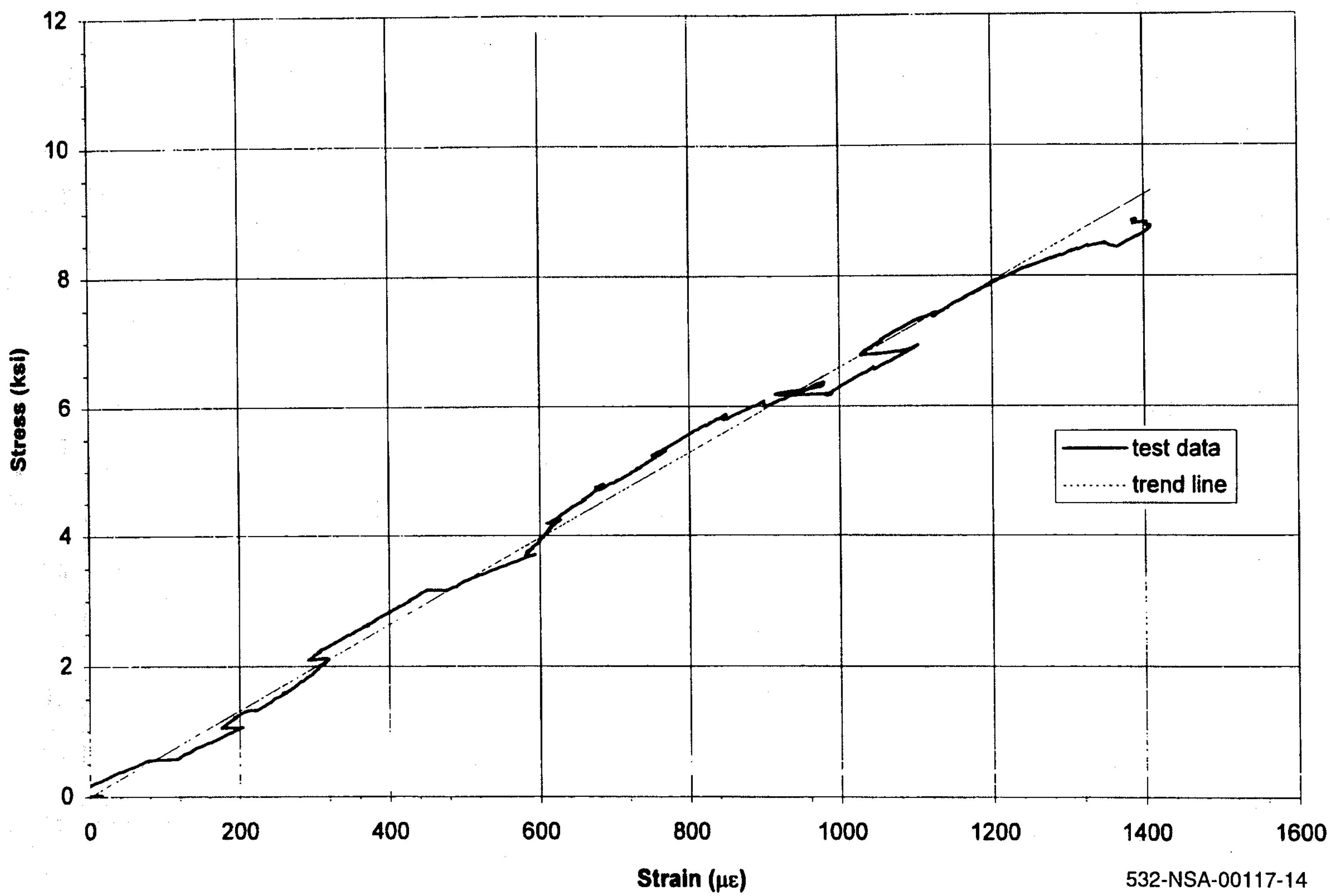

Figure 22. Stress versus strain for composite cantilever support during static test (strain gage location No. 4)

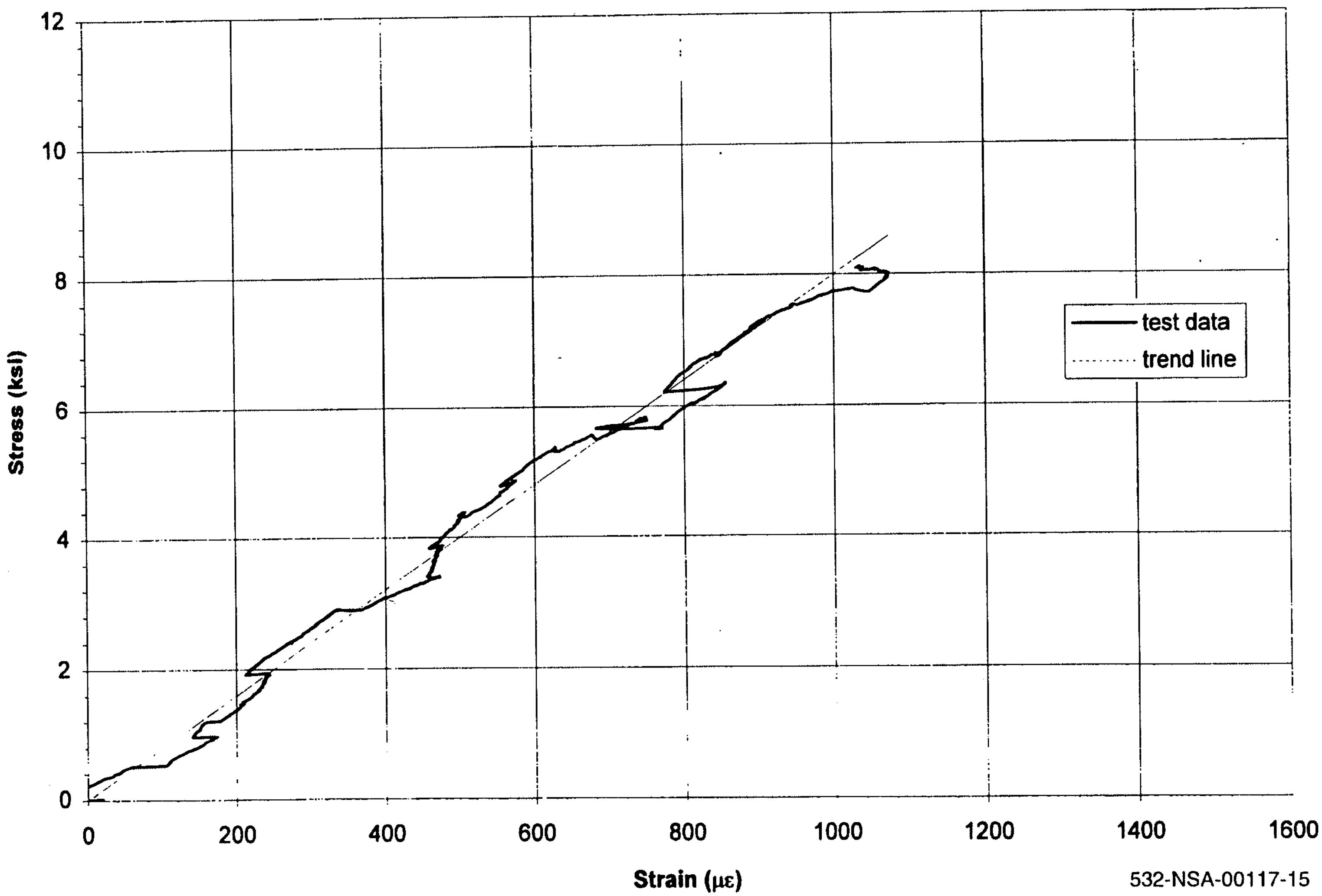

Figure 23. Stress versus strain for composite cantilever support during static test (strain gage location No. 5)

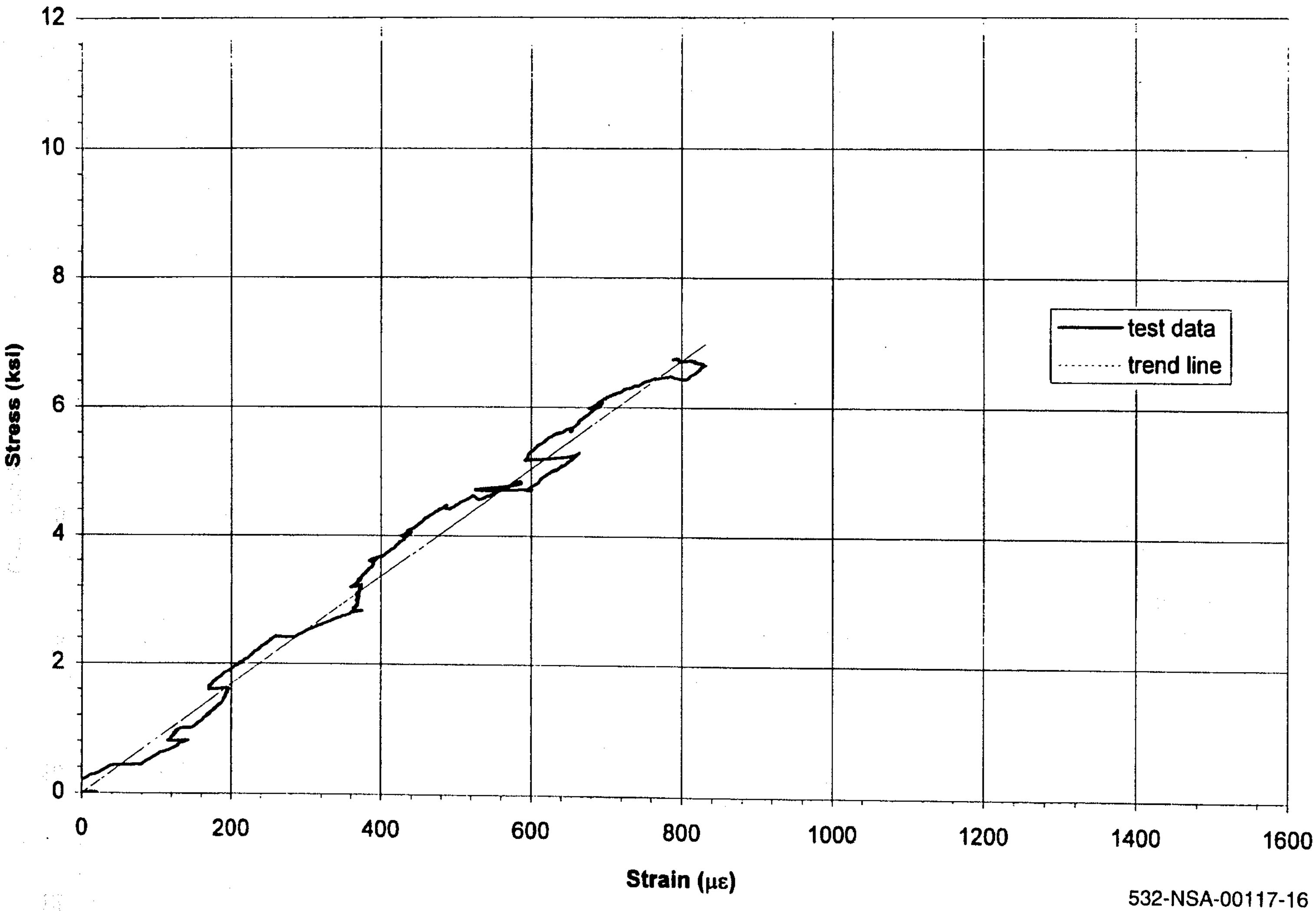

Figure 24. Stress versus strain for composite cantilever support during static test (strain gage location No. 6)

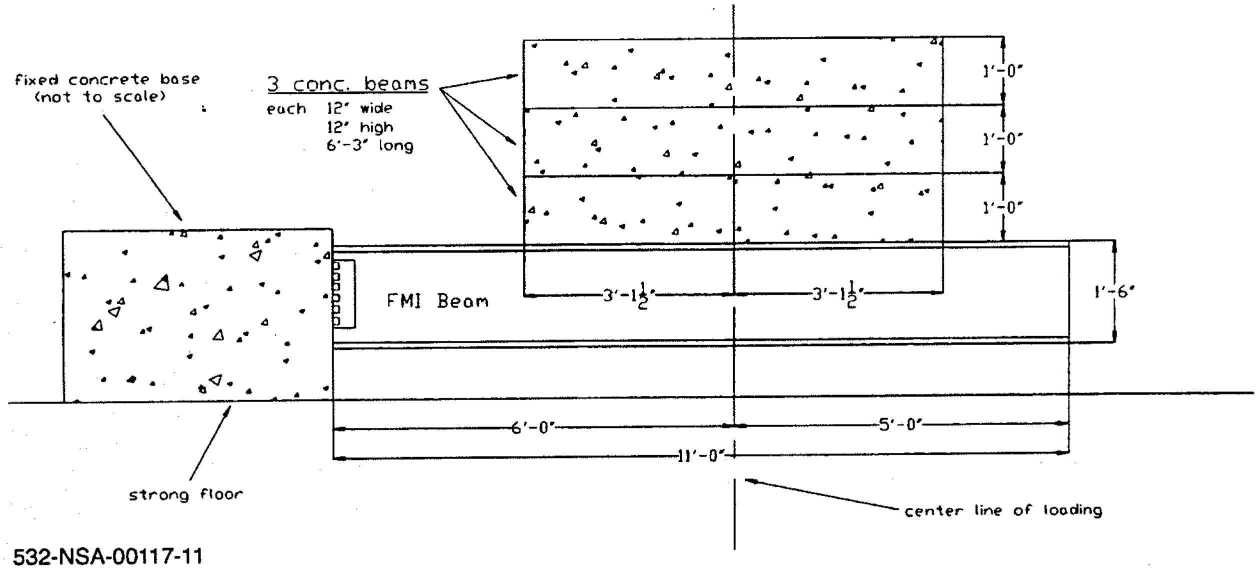

Figure 25. Creep test configuration

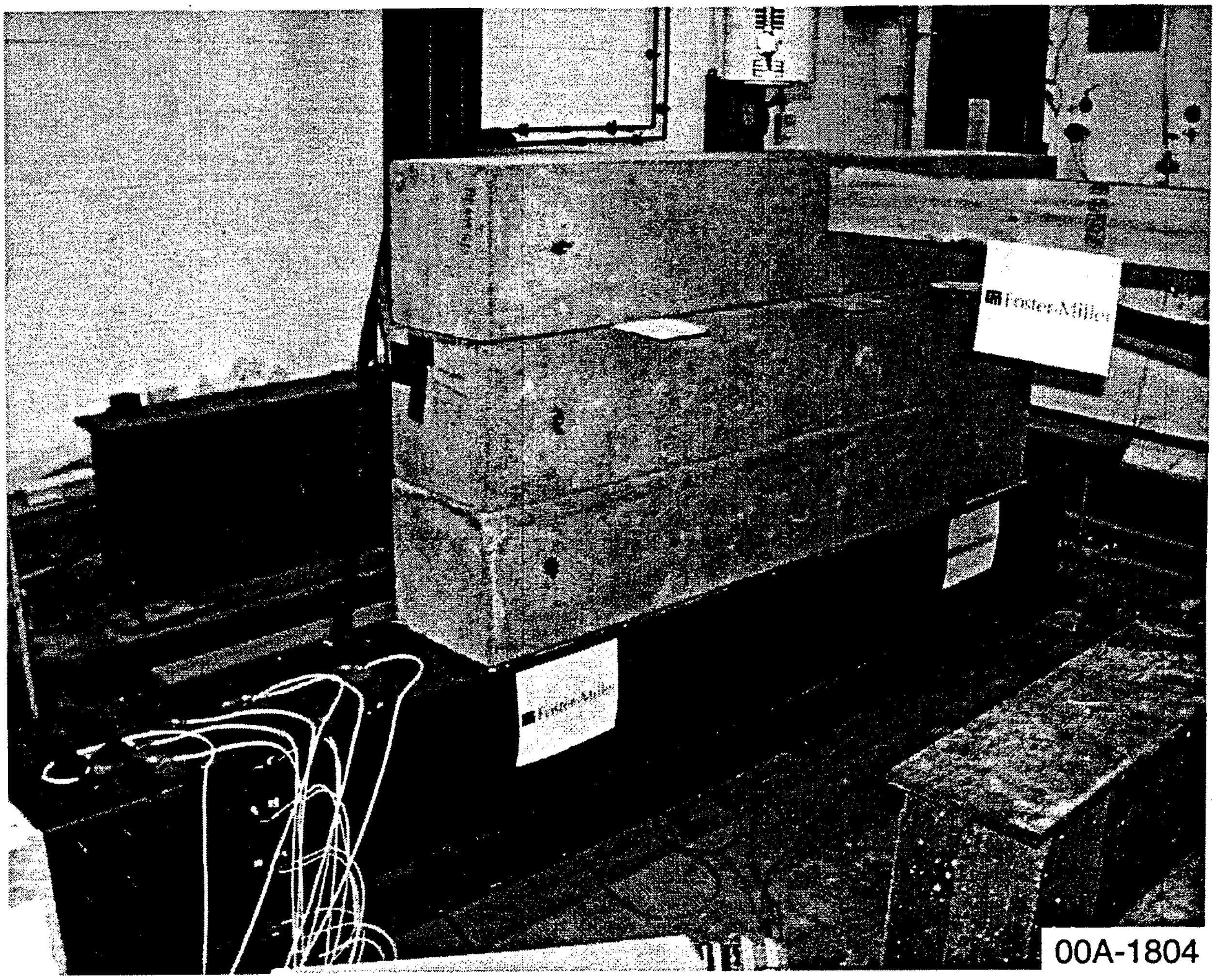

Figure 26. Creep test setup

APPENDIX A

GUIDE SPECIFICATIONS FOR DESIGN OF PEDESTRIAN BRIDGES

GUIDE SPECIFICATIONS

FOR

DESIGN OF

PEDESTRIAN

BRIDGES

Prepared by the Subcommittee on Bridges and Structures
of the Standing Committee on Highways

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American Association
of State Highway and
Transportation Officials

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Appendix A

GUIDE SPECIFICATIONS FOR DESIGN OF PEDESTRIAN BRIDGES

1.1 General

These Guide Specifications shall apply to bridges intended to carry primarily pedestrian and/or bicycle traffic. Unless amended herein, the existing provisions of the AASHTO *Standard Specifications for Highway Bridges*, 16th Edition, shall apply when using these Guide Specifications. Either the Service Load Design or Strength Design (Load Factor Design) methods may be used.

1.2 Design loads

1.2.1 Live Loads

1.2.1.1 Pedestrian Live Load

Main Members: Main supporting members, including girders, trusses, and arches, shall be designed for a pedestrian live load of 85 pounds per square foot (psf) of bridge walkway area. The pedestrian live load shall be applied to those areas of the walkway so as to produce maximum stress in the member being designed.

If the bridge walkway area to which the pedestrian live load is applied (deck influence area) exceeds 400 square feet, the pedestrian live load may be reduced by the following equation:

$$w = 85\,(0.25 + (15/\sqrt{A_1}))$$

where w is the design pedestrian load (psf) and A_1 is the deck influence area (sq. ft.), which is that deck area over which the influence surface for structural effects is different from zero.

However, in no case shall the pedestrian live load be less than 65 pounds per square foot.

Secondary Members: Bridge decks and supporting floor systems, including secondary stringers, floorbeams, and their connections to main supporting members, shall be designed for a live load of 85 psf, with no reduction allowed.

1.2.1.2 Vehicle Load

Pedestrian/bicycle bridges should be designed for an occasional single maintenance vehicle load provided vehicular access is not physically prevented. A specified vehicle configuration determined by the Operating Agency may be used for this design vehicle.

If an Agency design vehicle is not specified, the following loads conforming to the AASHTO Standard H-Truck shall be used. In all cases, a single truck positioned to produce the maximum load effect shall be used:

Clear deck width from 6 ft. to 10 ft.:	10,000 lb. (H-5 Truck)
Clear deck width over 10 ft.:	20,000 lb. (H-10 Truck)

Deck widths of less than 6 ft. need not be designed for a maintenance vehicle load.

The maintenance vehicle live load shall not be placed in combination with the pedestrian live load.

A vehicle impact allowance is not required.

1.2.2 Wind Loads

A wind load of the following intensity shall be applied horizontally at right angles to the longitudinal axis of the structure. The wind load shall be applied to the projected vertical area of all superstructure elements, including exposed truss members on the leeward truss.

For Trusses and Arches:	75 pounds per square foot
For Girders and Beams:	50 pounds per square foot

For open truss bridges, where wind can readily pass through the trusses, bridges may be designed for a minimum horizontal load of 35 pounds per square foot on the full vertical projected area of the bridge, as if enclosed.

A wind overturning force shall be applied according to Article 3.15.3 of the *Standard Specifications for Highway Bridges*.

1.2.3 Combination of Loads

The load combinations, i.e., allowable stress percentages for service load design and load factors for load factor design as specified in Table 3.22.1A of the *Standard Specifications for Highway Bridges*, shall be used with the following modifications:

Wind on Live Load, WL, shall equal zero.
Longitudinal Force, LF, shall equal zero.

1.3 Design details

1.3.1 Deflection

Members should be designed so that the deflection due to the service pedestrian live load does not exceed 1/500 of the length of the span.

* The deflection of cantilever arms due to the service pedestrian live load should be limited to 1/300 of the cantilever arm.

The horizontal deflection due to lateral wind load shall not exceed 1/500 of the length of the span.

1.3.2 Vibrations

The fundamental frequency of the pedestrian bridge without live load should be greater than 3.0 hertz (Hz) to avoid the first harmonic. If the fundamental frequency cannot satisfy this limitation, or if the second harmonic is a concern, a dynamic performance evaluation should be made.

In lieu of such evaluation the bridge may be proportioned so that the fundamental frequency shall be greater than

$$f \geq 2.86 \ln (180/W)$$

where ln is the natural log and W is the weight (kips) of the supported structure, including dead load and an allowance for actual pedestrian live load. Alternatively, the minimum supported structure weight (W) shall be greater than

$$W \geq 180\, e^{(-0.35f)}$$

where f is the fundamental frequency (Hz).

1.3.3 Allowable Fatigue Stress

Allowable fatigue stress ranges for steel members shall be determined from Article 10.3 of the *Standard Specifications for Highway Bridges*, except that the allowable fatigue stress ranges for Redundant Load Path structures may be used, regardless of the actual degree of member redundancy. Fatigue provisions need not apply to pedestrian live load stresses for cases where heavy pedestrian loads are infrequent, but shall be considered for wind loads.

1.3.4 Minimum Thickness of Metal

The provisions of Article 10.8 of the *Standard Specifications for Highway Bridges* shall apply, except that the minimum thickness of closed structural tubular members shall be ¼ inch.

1.3.5 Welded Tubular Connections

Welded tubular connections shall be designed in accordance with the Structural Welding Code—Steel ANSI/AWS D1.1.

1.3.6 Half-Through Truss Spans

1.3.6.1 The vertical truss members and the floorbeams and their connections in half-through truss spans shall be proportioned to resist a lateral force applied at the top of the truss verticals that is not less than 0.01/K times the average design compressive force in the two adjacent top chord members where K is the design effective length factor for the individual top chord members supported between the truss verticals. In no case shall the value for 0.01/K be less than 0.003 when determining the minimum lateral force, regardless of the K-value used to determine the compressive capacity of the top chord. This lateral force shall be applied concurrently with these members' primary forces.

End posts shall be designed as a simple cantilever to carry its applied axial load combined with a lateral load of 1.0% of the axial load, applied at the upper end.

1.3.6.2 The top chord shall be considered as a column with elastic lateral supports at the panel points. The critical buckling force of the column so determined shall be based on using not less than 2.0 times the maximum design group loading in any panel in the top chord.*

---

* For a discussion of half-through truss designs, refer to Galambos, T.V., *Guide to Stability Design Criteria for Metal Structures*, 4th ed., 1988, New York: John Wiley and Sons, Inc., pp. 515–529.

GUIDE SPECIFICATIONS FOR DESIGN OF PEDESTRIAN BRIDGES

COMMENTARY

1.1 General

This Guide Specification is intended to apply to pedestrian and pedestrian/bicycle bridges that are part of highway facilities, and thus provide realistic standards that ensure structural safety and durability comparable to highway bridges designed in conformance with the AASHTO *Standard Specifications for Highway Bridges*. This specification should apply equally to all bridge types and construction materials, including steel, concrete, and timber.

The term "primarily pedestrian and/or bicycle traffic" implies that the bridge does not carry a public highway or vehicular roadway. A bridge designed by these specifications could allow the passage of an occasional maintenance or service vehicle.

This specification allows the use of the Service Load Design or Strength Design (Load Factor Design) methods as provided by the AASHTO *Standard Specifications for Highway Bridges*. It is not presently intended for use in conjunction with the AASHTO Load and Resistance Factor Design (LRFD) Specifications.

1.2 Design loads

1.2.1 Live Loads

1.2.1.1 Pedestrian Live Load

The 85 pounds per square foot pedestrian load, which represents an average person occupying 2 square feet of bridge deck area, is considered a reasonably conservative service live load that is difficult to exceed with pedestrian traffic. When applied with AASHTO service load allowable stresses or Group 1 load factors for Load Factor Design, an ample overload capacity is provided.

Reduction of live loads for deck influence areas exceeding 400 square feet is consistent with the provisions of ASCE 7-95, "Minimum Design Loads for Buildings and Other Structures," and is intended to account for the reduced probability of large influence areas being simultaneously maximum loaded. For typical bridges, a single design live load value may be computed based on the full deck influence area and applied to all main member sub-components.

The 65 pounds per square foot minimum load limit is used to provide a measure of strength consistency with the LRFD specifications, which specify 85 pounds per square foot combined with a lesser load factor than used under the Load Factor Design specifications.

Requiring an 85 psf live load for decks and secondary members recognizes the higher probability of attaining maximum loads on small influence areas. Designing decks also for a small concentrated load, for example 1 kip, may be considered where the bridge may be subject to equestrian use or snowmobiles.

1.2.1.2 Vehicle Load

The proposed AASHTO vehicle loads are intended as default values in cases where the Operating Agency does not specify a design vehicle. H-Truck configurations are used for design simplicity and to conservatively represent the specified weights.

1.2.2 Wind Loads

The AASHTO wind pressure on the superstructure elements is specified, except that the AASHTO minimum wind load per foot of superstructure is omitted. The 35 pounds per square foot value applied to the vertical projected area of an open truss bridge is offered for design simplicity, in lieu of computing forces on the individual truss members. The specified wind pressures are for a base wind velocity of 100 miles per hour and may be modified based on a maximum probable site-specific wind velocity in accordance with AASHTO Article 3.15.

1.2.3 Combination of Loads

The AASHTO wind on live load force seems unrealistic to apply to pedestrian loads and is also excessive to apply to the occasional maintenance vehicle, which is typically smaller than a design highway vehicle. The longitudinal braking force for pedestrians is also neglected as being unrealistic.

The AASHTO Group Loadings are retained to be consistent with applying the AASHTO Service Load and Load Factor design methods without modifications.

1.3 Design details

1.3.1 Deflection

The specified deflection values are more liberal than the AASHTO highway bridge values, recognizing that, unlike highway vehicle loads, the actual live load needed to approach or achieve the maximum deflection will be infrequent. Pedestrian loads are also applied much more gradually than vehicular loads. The AASHTO value of span/1000 is intended for deflections caused by highway traffic on bridges that also carry pedestrians.

1.3.2 Vibrations

Pedestrian bridges have on occasion exhibited unacceptable performance due to vibration caused by people walking or running on them. The potential for significant response due to dynamic action of walking or running has been recognized by several analyses of problem bridges and is provided for in other design codes, such as the Ontario Bridge Code. Research into this phenomenon has resulted in the conclusion that, in addition to stiffness, damping and mass are key considerations in the dynamic response of a pedestrian bridge to ensure acceptable design. The range of the first through the third harmonic of people walking/running across pedestrian bridges is 2 to 8 Hertz (Hz), with the fundamental frequency being from 1.6 to 2.4 Hz. Therefore, bridges with fundamental frequencies below 3 Hz should be avoided.

For pedestrian bridges with low stiffness damping and mass, such as bridges with shallow depth, light weight, etc., and in areas where running and jumping are expected to occur on the bridges, the design should be tuned to have a minimum fundamental frequency of 5 Hz to avoid the second harmonic. If the structural frequencies cannot be economically shifted, stiffening handrails, vibration absorbers, or dampers could be used effectively to reduce vibration problems.

When a pedestrian bridge is expected to have frequencies in the range of possible resonance with people walking and/or running, the acceleration levels should be designed to limit dynamic stresses and deflections. The basic intrinsic damping available in pedestrian bridges is low and fairly narrow in range, with 1 percent damping being representative of most pedestrian bridges. The design limits given in these Guide Specifications for either the minimum weight or the minimum fundamental frequency for pedestrian bridges are based on D. E. Allen and T. M. Murray, "Design Criterion for Vibrations Due to Walking," *ASCE Journal*, fourth quarter, 1993. Additional information is contained in H. Bachmann, "Case Studies of Structures with Man-Induced Vibrations," *ASCE Journal of Structural Engineering*, Vol. 118, No. 3, March 1992.

1.3.3 Allowable Fatigue Stress

The nominal fatigue provision specified for all steel pedestrian bridges is consistent with AASHTO's provision that fatigue should apply to any load case that includes live load or wind load but recognizes that pedestrian loads near maximum design levels can be infrequent. AASHTO specifies using 100,000 cycles for wind load conditions, and this should be sufficient for any pedestrian live load condition also. Use of the Redundant Load Path stress range values, even for nonredundant girders and trusses, recognizes that repetitive live loads approaching the design load are infrequent. Maintaining a minimum-level fatigue check will help ensure that fatigue-prone details are avoided.

1.3.4 Minimum Thickness of Metal

The $\frac{1}{4}$ in. minimum thickness value for tubular members modifies the $\frac{5}{16}$ in. minimum value for steel members in AASHTO Article 10.8, and it approximates the 0.23 in. value allowed for rolled beam and channel webs. The interior of closed tubular shapes is protected from the elements, and their clean exterior shape reduces their ability to trap and accumulate dirt and corrosive materials. Tubular members can be subject to corrosion through internal condensation.

1.3.5 Welded Tubular Connections

The design provisions of ANSI/AWS D.1 are noted to supplement AASHTO, which does not specifically address welded tubular connection design.

1.3.6 Half-Through Truss Spans

This article modifies the provisions of AASHTO Article 10.16.12.1 by replacing the 300 pounds per linear foot design requirement for truss verticals with provisions based on research by Holt and others. These provisions establish the minimum lateral strength of the verticals based on the degree of elastic lateral support necessary for the top chord to resist the maximum design compressive force.

The use of 2.0 times the maximum top chord design load to determine the critical buckling force in the top chord is in recognition that under maximum uniform loads, maximum compressive stresses in the top chord may occur simultaneously over many consecutive panels. For a discussion on this, refer to T.V. Galambos' *Guide to Stability Design Criteria for Metal Structures*.

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APPENDIX B

DESCRIPTION OF THE PROPOSED WALKWAY STRUCTURE

B.1 Description of the Proposed Walkway Structure

A picture of a typical E.T. Techtonics truss bridge (22 ft-0 in. span) is presented in Figure B-1. It was determined that the walkway portion of the composite sidewalk system would be designed using the standard fiber-reinforced-plastic (FRP) Warren truss system developed by E.T. Techtonics, Inc. (See Figures B-2 and B-3.) The FRP truss system offers strength/stiffness advantages as well as cost benefits over conventional FRP beam/deck systems currently available on the market. The Warren truss approach (alternating diagonals) was used rather than a standard Pratt or Howe truss configuration as this geometry allows one to conveniently span different lengths between support brackets.

The standard truss system uses a double beam (C6 x 1-11/16 in. x 3/8 in.) top and bottom chord connected with vertical posts (2 in. x 2 in. x 1/4 in. sq. tube) and diagonal members (2 in. x 2 in. x 1/4 in. sq. tube). The trusses are connected side to side with lateral cross pieces (2) (C6 x 1-11/16 in. x 3/8 in.) and horizontal bracing members (2 in. x 2 in. x 1/4 in. sq. tube). Safety mid-rails (C3 x 7/8 in. x 1/4 in.) are spaced 3-3/8 in. on center to satisfy code requirements (maximum opening less than 4 in.). FRP deckboard (1 in. pultruded grating + 1/4 in. non-skid gritted plate) spans between the intermediate crosspieces (1) (C6 x 1-11/16 in. x 3/8 in.). A307 galvanized steel bolts with 2 flat washers, lockwasher and nut are used to connect the structural members. Standard steel clip angles are used to connect the bridge system to the support brackets or foundation. FRP angles can be used in areas where high maintenance is a concern.

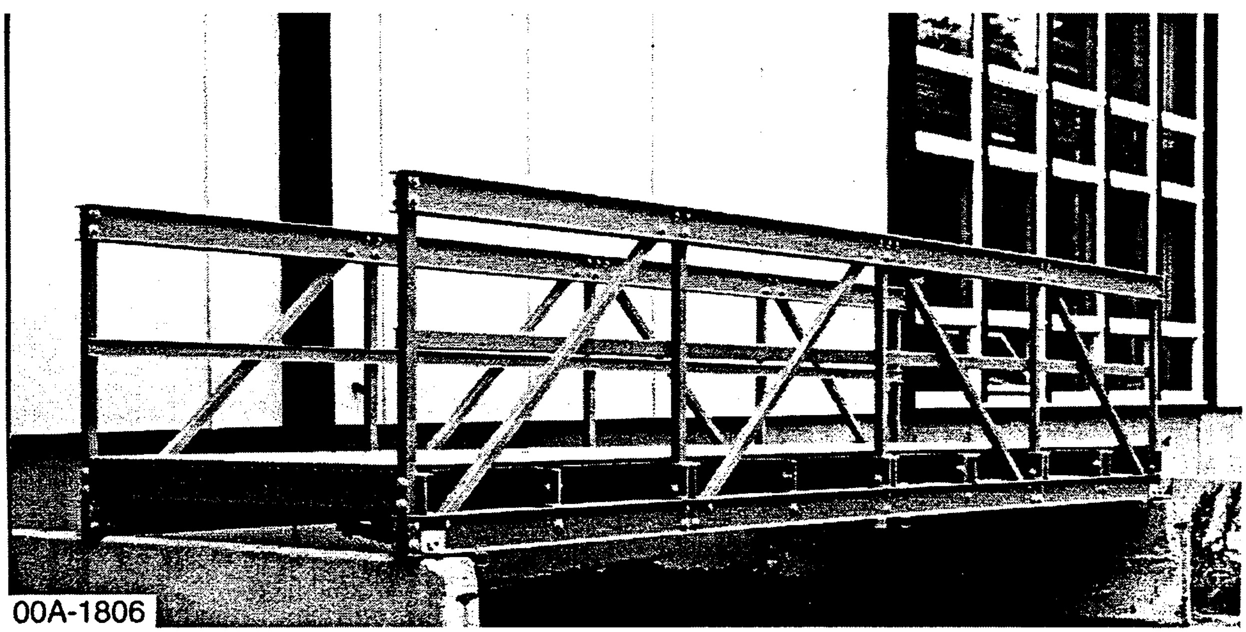

Figure B-1. E.T. Techtonics truss bridge

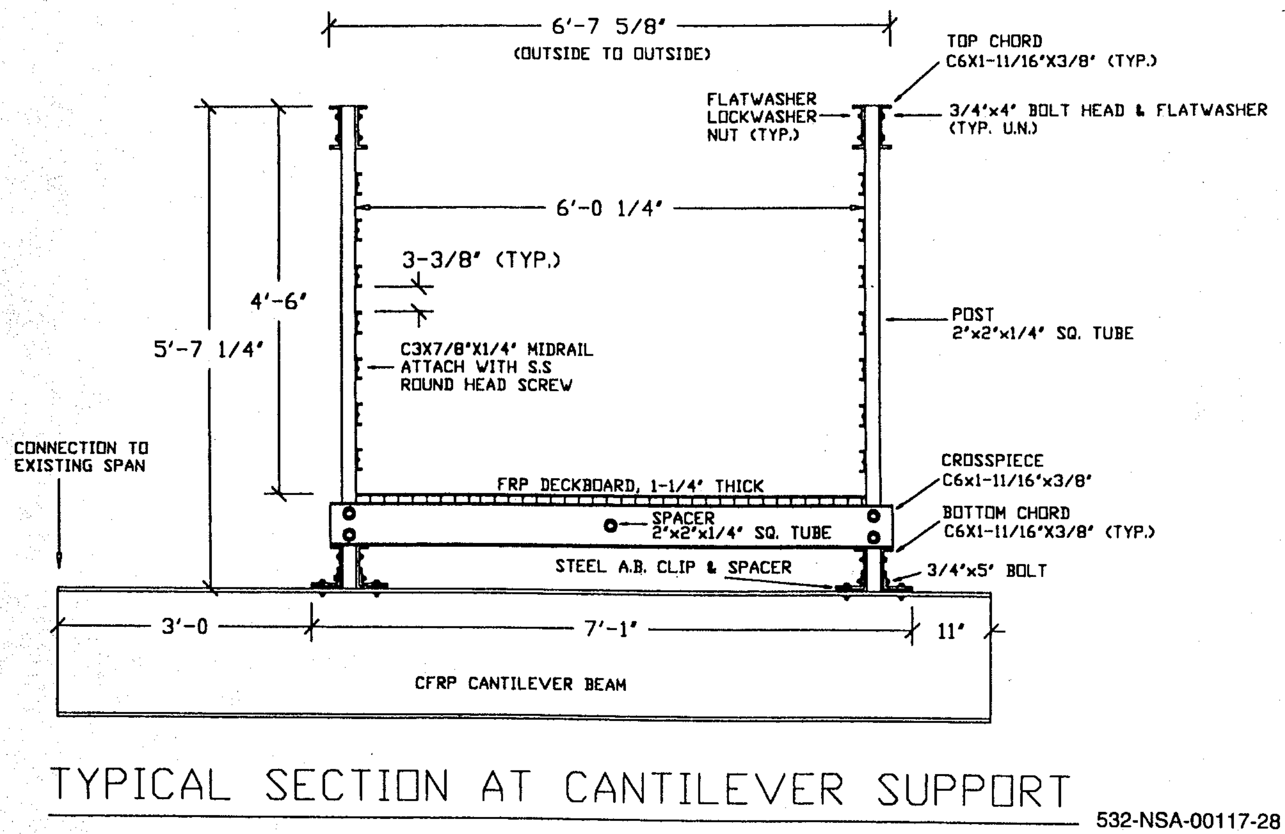

Figure B-2. Typical section at cantilever support

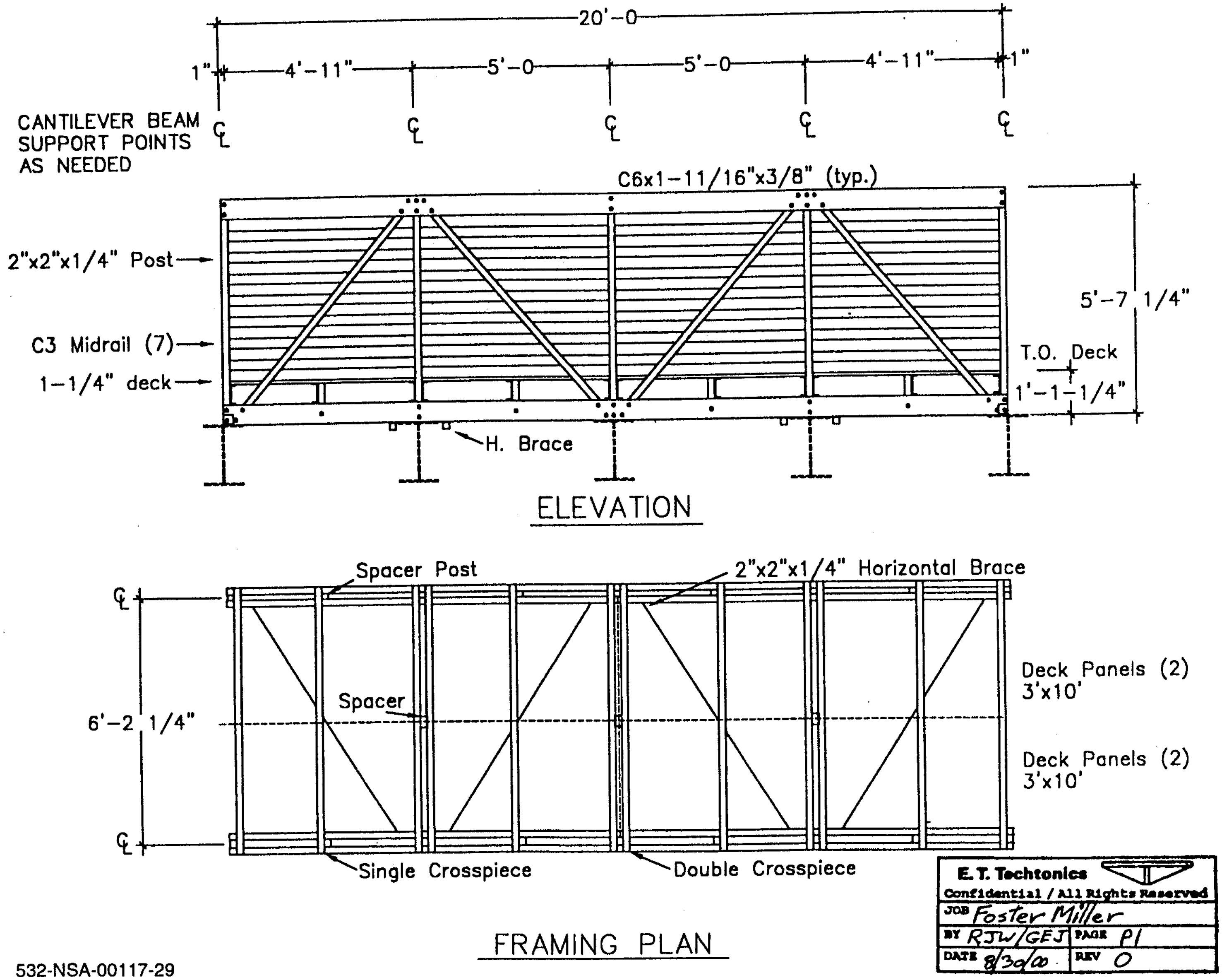

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Figure B-3. Framing plan

Computer models of the proposed design (maximum span 20 ft-0 between brackets) has been developed using STAAD/Pro. The bridge has been evaluated for the following conditions:

Live load	=	85 psf
Snow load	=	100 psf
Wind load	=	100 mph
Deflection	=	L/300
Frequency	=	Minimum 5 cycles/sec
Temperature	=	100°F variation

The following loading conditions were evaluated:

1. Dead load only
2. Dead load + live load
3. Dead load + snow load
4. Dead load + wind load
5. Dead load + 50% live load + seismic load
6. Dead load + live load + temperature

The system was analyzed for pin/roller and pin/pin end conditions. The FRP material properties used in the analysis can be found in the design specifications following this section. The system was also evaluated for a sustained snow load of 100 psf for 2 months. The STAAD/Pro analysis indicated that the proposed bridge system satisfied the above criteria.

Depending on site considerations, bridge spans will either be installed by assembly off-site and erection via crane truck, or assembly in place. In assembly off-site and erection via crane truck, spans will be brought onto the existing bridge fully assembled on a flatbed trailer and will be lifted onto support brackets with a small crane. As each span is placed on the cantilever support beams, a two man erection crew will attach the spans to the support brackets with mounting clips. It is anticipated that spans can be continuously placed and connected as quickly as the crane operator can rig and lift each section.

In assembly in place, the spans can be assembled directly on the cantilever support beams. Each truss side can be assembled by a 3 man crew using the preceding span deck as a staging area. Construction planking over the cantilever support beams will be used to gain temporary access out to the appropriate bracket location. A simple hoist frame can be used to pull each truss section into position. With trusses attached to the support beam, the assembly team will attach cross-braces, horizontal braces, mid-rails and deck panels. It is anticipated that a 20 ft-0 in. section of bridge can be assembled in position in 2 hr.

B.2 Pedestrian and Bicycle Bridge Specification

B.2.1 Design

Mounting Device

The manufacturer shall provide separate mounting clips as required by the design. These clips will be fabricated from galvanized A36 steel or FRP to enhance corrosion resistance. Clips will be securely attached to the support beams as required to provide adequate vertical and horizontal load transfer.

Railing Height

Railing height shall be 54 in. for pedestrian/bicycle type bridges.

Railing Openings

Minimum railing openings shall be less than 4 in. as specified in the above codes.

Diagonals

Bridges shall be provided with a minimum of one diagonal per panel. Two diagonals shall be provided as required for heavier load conditions. Bridges in excess of 20 ft-0 in. may be spliced for shipment.

B.2.2 Engineering

Uniform Live Load

All bridge components will be designed for 85 psf, which is standard AASHTO live load specifications.

Wind Load

Wind loading shall be taken as 20 psf on an open frame type structure unless otherwise specified by the governing code.

Snow Load

Snow load issues as determined by codes will be evaluated for sustained load considerations.

Seismic Load

Seismic loading shall be taken with 50 percent of the live load unless otherwise specified.

Allowable Design Stresses/Serviceability Criteria

E.T. Techtonics uses an Allowable Stress Design (ASD) Approach in the design of FRP bridges. Factors of safety used by E.T. Techtonics in the analysis of these bridges are as follows: (Based on Ultimate Strength of the FRP material)

Tension	- 3.0
Compression	- 3.0
Shear	- 3.0
Bending	- 3.0
End Bearing	- 3.0
Connections	- 3.0

Serviceability Criteria for live load deflection and natural frequency are as follows:

Deflection: Live Load	= L/300
Natural Frequency (minimum vertical direction)	= 5 cycles/sec

B.2.3 Materials

Walkway portion of the composite sidewalk system will be fabricated from high strength E-glass and isophthalic polyester resin unless otherwise specified. Weathering and ultraviolet light protection shall be provided by addition of a veil to the laminate construction. The color of the bridges will be olive green unless otherwise specified.

Minimum manufacturer's material specifications (ultimate strength and Young's Modulus) are as follows:

Tension	-	30,000 psi
Compression	-	30,000 psi
Shear	-	4,500 psi
Bending	-	30,000 psi
Young's Modulus (E)	-	2,500,000 psi

Minimum material specifications for bolt bearing connections have also been determined by E.T. Techtonics. This information has been included in Appendix B for connection design in Tables B-1 to B-4 (hollow tubes only) and Tables B-5 to B-8 (filled tubes). Note the manufacturers as follows; Strongwell, CP (Creative Pultrusions) and BRP (Bedford Reinforced Plastics).

Decking

Fiberglass decking will be used unless other materials (i.e., pressure treatedwood or concrete) are specified. The FRP deck will have a gritted surface unless otherwise determined. All

Table B-1. Ultimate load - one hole compression tests (2 in. x 2 in. x 1/4 in. square tube)

Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)	
Polyester Resin				
#1	9800	7900	7800	
#2	8400	9300	8100	
#3	8500	7500	7400	
Avg.	8900	8233	7767	
Polyester Resin(Grey) (Fire Retardant)				
#1	9300	NA	10100	
#2	8800	NA	9200	
#3	9040	NA	10500	
Avg.	9047	NA	9933	
Polyester Resin(Yellow) (Fire Retardant)				
#1	NA	8500	7900	
#2	NA	6600	7800	
#3	NA	7000	8600	
#4	NA	6300	-	
#5	NA	8100	-	
#6	NA	7200	-	
Avg.	NA	7283	8100	
Vinylester Resin (Fire Retardant)				
#1	15000	10600	9500	10500**
#2	12100	10000	7600	12100**
#3	12900	9600	7400	9400**
Avg.	13333	10067	8167	10667**

*CP Shapes are all yellow in color.
**Resubmission/2nd test with new shapes

Table B-2. Ultimate load - one hole tension tests (2 in. x 2 in. x 1/4 in. square tube).

Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)
Polyester Resin			
#1	9000	8000	8100
#2	8800	7800	8400
#3	7600	8300	9200
Avg.	8467	8033	8567
Polyester Resin(Grey) (Fire Retardant)			
#1	8300	NA	8500
#2	8080	NA	8600
#3	7460	NA	7000
Avg.	7947	NA	8033
Polyester Resin(Yellow) (Fire Retardant)			
#1	NA	9300	7900
#2	NA	9200	7600
#3	NA	8200	8000
#4	NA	6900	-
#5	NA	7300	-
#6	NA	6800	-
Avg.	NA	7950	7833
Vinylester Resin (Fire Retardant)			
#1	9960	8800	10100
#2	9840	6500	10300
#3	10960	11400	7800
Avg.	10253	8900	9400

*CP Shapes are all yellow in color.

Table B-3. Ultimate load - two hole compression tests (2 in. x 2 in. x 1/4 in. square tube)

Test #	STRONGWELL (lb)	CP* (lb) (Grey)	BRP (lb)
Polyester Resin			
#1	13600	14200	15100
#2	14000	15900	16500
#3	14400	16000	14800
Avg.	14000	15367	15466
Polyester Resin(Grey) (Fire Retardant)			
#1	16800	NA	15800
#2	15000	NA	16300
#3	16300	NA	17800
Avg.	16033	NA	16633
Polyester Resin(Yellow) (Fire Retardant)			
#1	NA	18000	12300
#2	NA	19400	13900
#3	NA	20300	13600
#4	NA	19000	-
#5	NA	19300	-
#6	NA	19500	-
Avg.	NA	19250	13266
Vinylester Resin (Fire Retardant)			
#1	20800	16000	13400
#2	23300	13900	14700
#3	21500	18200	14200
Avg.	21867	16033	14100

*CP Shapes are all yellow in color except polyester resin (grey)

Table B-4. Ultimate load - two hole tension tests (2 in. x 2 in. x 1/4 in. square tube)

Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)
Polyester Resin			
#1	13900	13500	16100
#2	14000	14100	16000
#3	17600	10500	14900
Avg.	15166	12700	15666
Polyester Resin(Grey) (Fire Retardant)			
#1	17500	NA	16000
#2	15000	NA	15800
#3	16000	NA	13700
Avg.	16167	NA	15167
Polyester Resin(Yellow) (Fire Retardant)			
#1	NA	14000	12800
#2	NA	17700	13900
#3	NA	16000	13800
#4	NA	14000	-
#5	NA	16800	-
#6	NA	15800	-
Avg.	NA	15717	13500
Vinylester Resin (Fire Retardant)			
#1	17000	15900	16300
#2	23400	18000	14100
#3	22700	16000	14700
Avg.	21033	16633	15033

*CP Shapes are all yellow in color.

Table B-5. Ultimate load - one hole tests (2 in. x 2 in. x 1/4 in. square tube w/1-1/2 in. solid insert)

Compression	Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)
Polyester Resin				
Unglued	#1	40100	36000	32800
Unglued	#2	41200	37000	31100
Glued	#1	39000	45800	31000
Glued	#2	40300	47100	31200
Polyester Resin(Grey/FR)				
Unglued	#1	44800	NA	27200
Unglued	#2	44000	NA	31100
Glued	#1	41400	NA	29900
Glued	#2	44500	NA	30600
Polyester Resin(Y/FR)				
Unglued	#1	NA	36700	32700
Unglued	#2	NA	34700	31000
Glued	#1	NA	43800	29500
Glued	#2	NA	47000	30900
Vinylester Resin(FR)				
Unglued	#1	42300	37800	29100
Unglued	#2	41200	38500	30300
Glued	#1	47400	46400	29800
Glued	#2	46800	46800	30200

*CP Shapes are all yellow in color

Table B-6. Ultimate load - one hole tests (2 in. x 2 in. x 1/4 in. square tube w/1-1/2 in. solid insert)

Tension	Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)
Polyester Resin				
Unglued	#1	18200	20100	17200
Unglued	#2	22100	20300	17500
Glued	#1	18600	15300	18500
Glued	#2	18300	16800	18400
Polyester Resin(Gr/FR)				
Unglued	#1	22400	NA	15800
Unglued	#2	20900	NA	15200
Glued	#1	20800	NA	15900
Glued	#2	20200	NA	17000
Polyester Resin(Y/FR)				
Unglued	#1	NA	20200	16300
Unglued	#2	NA	14500	16600
Glued	#1	NA	18800	17800
Glued	#2	NA	20500	17700
Vinylester Resin(FR)				
Unglued	#1	25900	18200	20000
Unglued	#2	25200	15100	19900
Glued	#1	23600	17900	20500
Glued	#2	24000	19100	18300

*CP Shapes are all yellow in color.

Table B-7. Ultimate load - two hole tests (2 in. x 2 in. x 1/4 in. square tube w/1-1/2 in. solid insert)

Compression	Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)
Polyester Resin				
Unglued	#1	57800	60500	53300
Unglued	#2	63000	51500	51800
Glued	#1	60500	52600	50300
Glued	#2	61500	48800	45100
Polyester Resin(Grey/FR)				
Unglued	#1	64000	NA	50600
Unglued	#2	66000	NA	49400
Glued	#1	67500	NA	52500
Glued	#2	72000	NA	52000
Polyester Resin(Y/FR)				
Unglued	#1	NA	52000	51000
Unglued	#2	NA	63500	53800
Glued	#1	NA	54000	50800
Glued	#2	NA	28000	52400
Vinylester Resin(FR)				
Unglued	#1	80000	49000	56800
Unglued	#2	74500	54000	58700
Glued	#1	82000	52000	57100
Glued	#2	81500	77500	55900

*CP Shapes are all yellow in color.

Table B-8. Ultimate load - two hole tests (2 in. x 2 in. x 1/4 in. square tube w/1-1/2 in. solid insert)

Tension	Test #	STRONGWELL (lb)	CP* (lb)	BRP (lb)
Polyester Resin				
Unglued	#1	48000	26000	35400
Unglued	#2	45100	19100	30000
Glued	#1	48200	22500	27200
Glued	#2	47900	22600	30800
Polyester Resin(Grey/FR)				
Unglued	#1	48900	NA	31200
Unglued	#2	49200	NA	33000
Glued	#1	47500	NA	33800
Glued	#2	49000	NA	34300
Polyester Resin(Y/FR)				
Unglued	#1	NA	28200	46100
Unglued	#2	NA	26000	43000
Glued	#1	NA	31100	30800
Glued	#2	NA	22500	29500
Vinylester Resin(FR)				
Unglued	#1	46500	19200	20000
Unglued	#2	43700	26100	43000
Glued	#1	54800	21100	20500
Glued	#2	51300	24100	18300

*CP Shapes are all yellow in color.

decking will be designed to meet the aboveloading, deflection and frequency requirements. Decking will be either screwed or bolted to the FRP superstructure.

Hardware

All hardware will be hot dipped galvanized A307 steel unless otherwise specified. All bolts will be a full body type with 2 flat washers, 1 lock washer, and 1 nut. In corrosive or high maintenance environments (marine applications) the use of 316SS steel is recommended.

B.2.4 Assembly

Due to the lightweight nature of the components of the bridge system (heaviest member weighs approximately 65 lb) and the bolted connections, the structural system can be shipped unassembled. Assembly will be the responsibility of the client. The bridge system can also be shop assembled and shipped to the site.

B.3 Testing/Conclusions (based on past E.T. Techtonics work)

Testing of a similar walkway system has been conducted at Lehigh University's ATLSS Center at Bethlehem, PA, the Royal Military College of Canada at Kingston, Ontario and the U.S. Forest Service Testing Laboratory at Madison, WI. Components of the system were initially tested at the ATLSS Center from 1988 to 1993. This test program conclusively demonstrated the excellent strength/stiffness characteristics of the system and its lightweight nature. The project was funded by the National Science Foundation's SBIR Program through a series of three awards received by E.T. Techtonics. Further testing of connections was conducted in 1997 to 1998, again with support from the National Science Foundation.

In other past work done by E.T. Techtonics, a 20 ft-0 in. x 10 ft-0 in. Pratt truss vehicular bridge was load tested to failure by the Royal Military College of Canada (1993 to 1995) with ultimate load occurring at approximately 35 tons. Failure occurred in the end diagonal tension members at the bolt holes. Subsequent connection tests have shown that filling the diagonals with 1-1/2 in. solid bar would increase the load carrying capabilities of the system to over 50 tons. From this load test it is apparent that the system can carry large loads. The project was funded by the Canadian Military. A copy of the research paper discussing this work is presented in Appendix C.

In other past work, the U.S. Forest Service conducted a sustained load test (125 psf) on a 22 ft-0 in. x 6 ft-0 in. Howe truss bridge for 6 months (1998 to 1999) in Madison, WI. Measured deflection increased approximately 30 percent from the initial measured deflection. Unloading of the system resulted in total recovery of the system. Results from this test indicated that FRP truss systems can carry significant sustained snow loads. Funding for this project was received from the Federal Highway Administration.

APPENDIX C

EXPERIMENTAL BEHAVIOR OF A REINFORCED PLASTIC VEHICLE BRIDGE

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Experimental Behavior of a Reinforced Plastic Vehicle Bridge

ABSTRACT

There is a growing interest in all-fiber-reinforced plastic (all-RP) bridges, because such structures may have potential as lightweight bridges for military operations and temporary bridges for low volume roadways. The main obstacles to using RP for vehicle bridges are material cost and lack of understanding of the mechanical properties of RP. Material costs can be offset by innovative structural systems and lower construction costs arising primarily from the light weight of RP. This paper discusses static and dynamic testing of an all-glass RP prototype vehicle bridge, with a span of 6 m (20 ft.) and a width of 3 m (10 ft.), designed to carry a 100 kN (11.2 tons) four-wheel vehicle. The structure is a pony-truss bridge, built using standard pultruded RP structural channels and hollow tube shapes, mechanically connected with bolted joints. The outer trusses may be fitted with prestressed cables in a queen-post arrangement below the trusses to increase the stiffness and ultimately the strength of the structure. Results of static and dynamic testing of the bridge with and without a transversely post-tensioned longitudinally laminated timber deck are presented.

INTRODUCTION

Although RP for bridges is being used mainly as reinforcement for concrete in conventional designs, some bridges have been designed with RP beams or decks, in conjunction with reinforced concrete or steel structural members. A few all-RP vehicle and pedestrian bridges have been constructed. They have been built mainly in China, where steel costs are high, and in North America, for specialty applications or to showcase the use of RP for civil engineering structures. Meier (1991), Head (1992), Head (1994), Johansen et al. (1992), and Johansen et al. (1994) review some of these bridges. Many all-RP bridges have been built using glass RP, although an impressive all-carbon-RP vehicle bridge is being designed in San Diego (FRP International 1993). The interest in all-RP bridges is growing, because these bridges are seen as possible solutions to particular bridging requirements, such as the need for lightweight bridges for military operations and temporary bridges for low volume roadways. The main obstacles to using RP for vehicle bridges are primarily material cost and lack of understanding of the mechanical properties of RP.

While RP has the advantages of having high specific strength and stiffness, and of being noncorroding, the cost of the material can be many times greater than that of steel or reinforced concrete. Nevertheless, savings can be realized through innovative material and structural designs. Material properties of RP are anisotropic and are a function of fiber orientation and lay-up of the material. Designers have the choice of either using standard pultruded members, with fixed fiber lay-up configurations, or specifying an optimized lay-up for a particular application. Designers, if they so wish, also have more freedom to depart from conventional structural member shapes, since manufacturers can modify their pultrusion dies to accommodate special requirements. A good understanding of RP properties can allow designers to be more creative with structural forms for their structures. In addition, there are savings from lower transportation and construction costs associated with the material's light weight. Prefabrication of RP structural units can also reduce the cost of construction. All of these make RP structures more competitive than the material costs alone would indicate.

Since glass fiber is much more economical than carbon fiber, glass fiber or glass RP has been favored in the construction of all-RP bridges. However, glass RP creeps more under sustained loading than steel or concrete, and glass RP components may have lower fatigue lives than their steel counterparts. A particular concern is the high stress concentration at bolt holes for bolted glass RP members, which can be as high as eight times the average net section stress.

This paper discusses static and dynamic testing of an all-glass RP prototype vehicle bridge, designed by E.T. Techtonics, and based on previous successes with all-glass RP prestressed pedestrian bridges (Johansen and Roll 1990; Johansen et al. 1992; Johansen et al. 1994). Two different deck types are being considered for the bridge, namely a lightweight reinforced concrete deck and a longitudinally laminated transversely post-tensioned timber deck (Taylor and Ritter 1990; OHBDC 1991). Results of static and dynamic testing of the bridge with and without the longitudinally laminated transversely post-tensioned timber deck are presented. The results of this investigation will be used to modify this single lane, short span, RP prototype bridge for two lanes of traffic and for longer spans.

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DESCRIPTION OF THE BRIDGE

The structure is a pony-truss bridge designed to carry a 100 kN (11.2 tons) four-wheel vehicle, having 70% of its total weight on its back wheel axle. This load corresponds to a light truck or the maintenance vehicle described by the Ontario Highway Bridge Design Code (OHBDC 1991). The OHBDC vehicle has a width of 2.2 m (7.2 ft.) and a length of 2 m (6.6 ft.).

The bridge is built using standard pultruded glass RP structural channel and hollow tube shapes manufactured by Creative Pultrusions Incorporated (1994). The glass RP consists of alternating layers of randomly oriented glass fibers and unidirectional fibers in a vinyl ester matrix. The fiber volume is approximately 60%.

The structure has an overall length of 6 m (20 ft.) and width of 3 m (10 ft.). Figure 1 shows the dimensions of the structure. As well, the figure shows the grid point labeling used to identify locations on the bridge for the static and dynamic testing and analysis.

The three longitudinal beams and seven transverse beams of the structure consist of back-to-back double channel sections, 203 × 55.6 × 9.5 mm (8 × 2-3/16 × 3/8 in.), spaced with square tube sections, 51 × 51 × 6.4 mm (2 × 2 × 1/4 in.), that transmit the deflections and rotations from the longitudinal beams to the transverse beam system below the deck. The bracing system attached to the bottom flanges of the transverse beams consists of equal leg angles 102 × 102 × 9.5 mm (4 × 4 × 3/8 in.). The outer trusses are made up of six panels, and have a height of 1.32 m (52 in.). The top and bottom chords of the trusses use the same double channels as the beams spaced by the same square tube sections which also function as the vertical and diagonal members of the trusses.

The third and fifth vertical members of each truss are extended below the bottom truss chords by 457 mm (18 in.) to serve as posts in a queen-post system for cables joining the two ends of the bridge. The upward thrusts at these posts and the pre-compression of the bottom chords of the trusses, provided by the tensioned cables, stiffen the structure and counteract creep deformations. Although only the outer trusses have been designed for the cable system, the corresponding pairs of hollow tube sections joining the longitudinal and transverse beams could equally be extended below the bridge to act as posts for additional cables. This capability simplifies widening and lengthening the structure, while increasing its stiffness and load-carrying capacity.

The components of the structure are connected throughout by 19 mm (3/4 in.) diameter bolted joints. The bolts may be either steel bolts or glass RP threaded rods, and the cables may be either steel twisted wire strands or aramid ropes. Aramid ropes have already been used for many pedestrian all-RP bridges (Head 1992; Johansen et al. 1992; Johansen et al. 1994). In applications where lightness of the structure is necessary, to facilitate transportation and erection, or where corrosion is a concern, the glass RP threaded rods and aramid ropes would be favored over equivalent steel components. The only parts of the structure that have been designed using steel are the inserts at the ends of the two bottom chords of the trusses, where the cables are attached. Steel turnbuckles and end fixtures are also used with the aramid ropes, but could be replaced by RP components. The weight of the structure connected with steel bolts and without decking is approximately 17.6 kN (2 tons).

This paper presents the test results for the bridge without and with a timber deck. A longitudinally laminated transversely post-tensioned timber deck was constructed for this study. This deck type is an Ontario innovation (OHBDC 1991), and was intended for use with wood bridges or for steel-wood composite bridges (Tharmabala and Bakht 1986). This construction was used for the deck, because it is easy to construct in the field, does not require heavy construction equipment, and is lighter, while being durable, compared to a concrete deck. According to the OHBDC (1991), the typical depth of the laminations is 184 to 286 mm (7.25 to 11.25 in.), but for this prototype bridge designed for lighter vehicles 38 by 89 mm (2 by 4 in.) spruce lumber was used. To construct the deck the laminations are placed longitudinally, in the direction of the flow of traffic. The laminations are prestressed using Dywidag bars spaced at 300 to 600 mm (1 to 2 ft.). Dywidag bars, 16 mm (5/8 in.) diameters, were used to transversely prestress the deck. The jacking sequence ensured the following prestressing levels in the deck: initially to 1 MPa, released after three hours to 0.5 MPa, with all testing done with a minimum of 0.35 MPa. Static tests were conducted with the deck simply placed on the longitudinal beams, then repeated with the deck attached to the longitudinal beams to try to achieve composite action between the deck and the RP bridge. The attachments of the deck to the longitudinal beams consisted of steel hollow sections bolted into the hollow sections used to space the longitudinal beams. The steel hollow sections extended into the deck through 100 mm (4 in.) holes drilled into the timber deck. These holes were then backfilled with expansive grout to secure the composite action between the steel hollow sections and the timber deck.

TEST SET UP

Static Testing

The structure with and without decking was subjected to concentrated point loading to study load distribution within the structure and to assess its stiffness. Additionally, tests were conducted for the structure with and without pretensioned steel cables. Figure 2 shows a photograph of the test setup without decking. The bridge was simply supported on a roller support along one of its ends (C1S1 to C5S1, Figure 1) and on a pin support along its other end (C1S7 to C5S7, Figure 1). The supports were mounted on steel frames at each end of the structure. A loading frame straddled the width of the structure, and load was applied using a hydraulic actuator. The actuator was placed to load the bridge at the center of the span on the longitudinal centerline (G3S4, Figure 1) and at a quarter of the width offset from the centerline (G2S4, Figure 1). The loading frame was then moved to load the structure at a third of the span length on longitudinal centerline and at a quarter of the width offset from the centerline (G3 and G2, respectively, Figure 1).

The actuator load was applied as a point load over an area of 150 by 250 mm (6 by 10 in). The maximum load level chosen for tests without decking was approximately half of a single rear axle wheel load, namely 17.3 kN (approximately 2 tons). With the longitudinally laminated transversely post-tensioned timber deck, the maximum applied load was approximately 40 kN (4.5 tons).

Thirteen linear variable displacement transformers (LVDT) were used to measure the vertical displacements along the bottom chord of one of the trusses and at key intersections between the longitudinal and transverse beams. Thirty-two electrical resistance strain gauges were placed on the top and bottom chords and diagonal members of one of the trusses, and on the longitudinal and transverse beams to confirm the behavior of the system. Data was collected by an IBM-compatible P.C.

Dynamic Testing

For dynamic testing, the bridge was instrumented with four accelerometers and two LVDT. The data-logging facilities consist of an IBM compatible 386 computer with a DAS 1400 data-logging card and an AAF1 anti-aliasing filter board.

The bridge without decking and with the transversely prestressed laminated timber decking, was excited at different locations on the top flange of the longitudinal beams or on the deck surfaces with an instrumented hammer designed to simulate a single wheel load. The impulsive force generated by the hammer was measured with a force transducer mounted on the face of the hammer. The linearity of the bridge responses was confirmed by dropping the hammer from three different heights. The repeatability of the tests was checked by dropping the hammer from the same height three times at each location.

Transfer functions between each hammer position and each accelerometer and LVDT location were calculated by dividing the Fast Fourier Transform (FFT) of the bridge response by the FFT of the hammer force. Modal analysis techniques were applied to the resulting transfer functions to extract the natural frequencies, damping ratios, and mode shapes for the bridge without and with the timber deck. Only the dynamic test results for tests on the composite longitudinally laminated transversely post-tensioned timber deck are discussed herein.

RESULTS

Static Tests

The load versus displacement for all points instrumented on the bridge confirmed that the behavior was linear for all static testing conducted, namely point loading without and with the timber deck, and with and without cables. Figure 3(a) shows a typical graph of the load versus vertical displacement behavior at eight different points on the bridge for point loading on the longitudinal centerline at the mid-span of the structure (G3S4, Figure 1) with the non-composite timber deck. The behavior is linear from the start of loading up to the maximum load applied to the structure, approximately 40 kN (4.5 tons). The maximum deflection at 40 kN is nearly 14 mm (0.55 in.). It is reasonable to expect that the behavior up to the design loads would remain linear, and would then approach maximum deflections of 1 in 380, which fall within the acceptable limits prescribed by the OHBDC (1991, Clause 2-6). Figure 3(b) shows the vertical deflections along the length of one of the trusses (C1S1 to C1S7, Figure 1) for a loading of 17.3 kN (2 tons) and 35 kN (4 tons) applied at the mid-span of the bridge (G3S4, Figure 1) with the non-composite timber deck and steel cables tensioned to 27 kN (approximately 6,000 lb.). It can be seen that deflections were small. The non-composite action of the timber deck with the RP bridge resulted in somewhat non-symmetric displacements along the bottom of the truss.

The load versus vertical displacement behavior for point loading on the longitudinal centerline at the mid-span of the structure (G3S4, Figure 1) with the composite timber deck is nearly identical to that shown in Figure 3(a). However, there were some differences in the vertical deflections along the length of one of the trusses. Figure 4 shows the vertical deflections along the length of one of the trusses (C1S1 to C1S7, Figure 1) for a loading of 17.3 kN (2 tons) and 35 kN (4 tons) applied at the mid-span of the bridge (G3S4, Figure 1) with the composite timber deck and steel cables tensioned to 27 kN (approximately 6,000 lb.). Deflections are small, and composite action of the timber deck with the RP bridge resulted in symmetric displacements along the bottom of the truss.

Dynamic Tests

The maximum impulse force of the instrumented hammer applied to the composite longitudinally laminated transversely post-tensioned timber deck was approximately 32 kN (7.2 kips), which was close to the maximum static load. The typical acceleration response died out in about 0.3 seconds, because the bridge structure is heavily damped. The natural frequencies for the structure without the prestressing cables were as follows: first mode at 12 Hz, second mode at 16 Hz, and third mode at 24 Hz. The natural frequency for the first mode of the structure with the cables prestressed to 27 kN (~ 6,000 lb.) was slightly higher, namely 13 Hz, but the natural frequencies of the second and third mode shapes were the same as for the structure without cables.

The modal parameters were measured with the half-power bandwidth method. The damping values approached 10% of critical; therefore, the structure is heavily damped, likely caused mainly by the heavy timber deck. It was observed that the prestressed cables tended to increase the natural frequencies of the structure, because the cables stiffen the structure.

SUMMARY AND CONCLUSIONS

This paper discusses static and dynamic testing and analysis of a prototype all-glass RP vehicle bridge, with a span of 6 m (20 ft.) and a width of 3 m (10 ft.), designed to carry a 100 kN (11.2 tons) four-wheel vehicle. The structure is a pony-truss bridge, built using standard pultruded glass RP structural channels and hollow tube shapes, mechanically connected with bolted joints. The outer trusses were fitted with optional prestressed cables in a queen-post arrangement below the trusses to increase the stiffness. Results of static testing of the bridge without and with timber decking are presented. Dynamic tests are outlined, and the results of modal analysis are discussed. The response of the bridge was found to be very stiff. Deflections under maximum applied loads were small, and did not exceed 14 mm. The natural frequencies of the structure for the principal bending modes were 12 Hz and greater.

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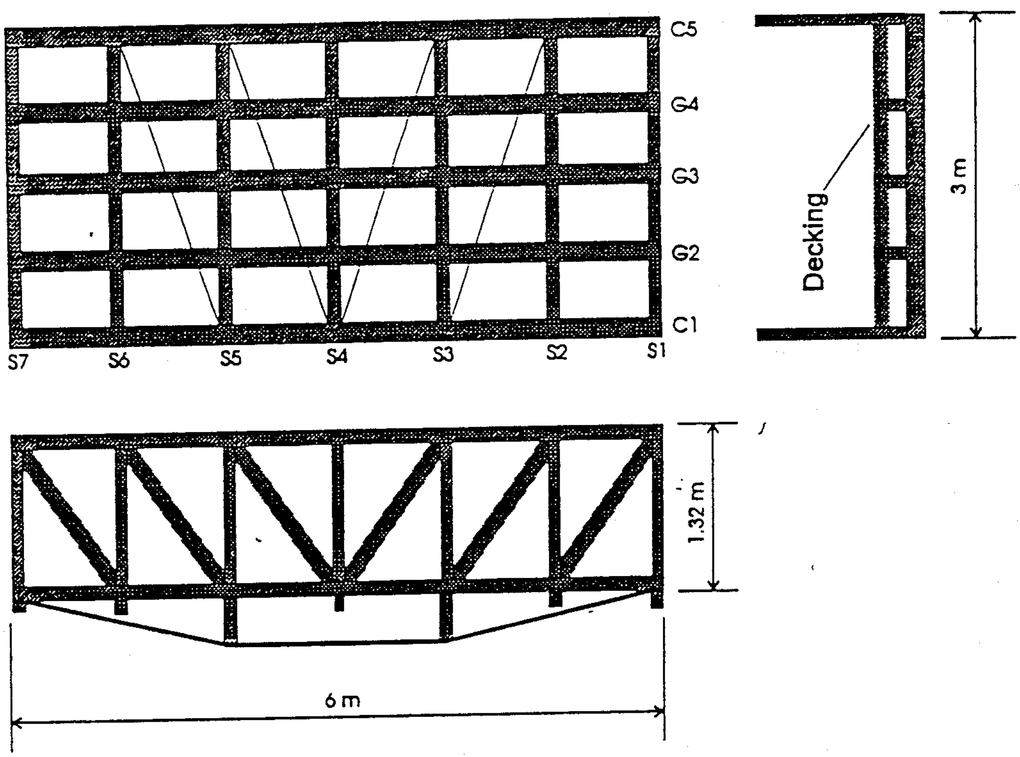

FIGURE 1. RP vehicle bridge.

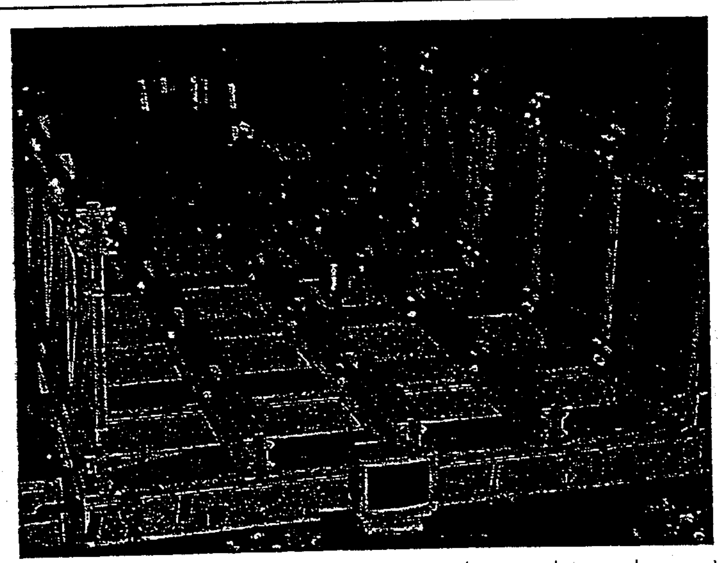

FIGURE 2. Bridge structure without decking (showing dynamic hammer).

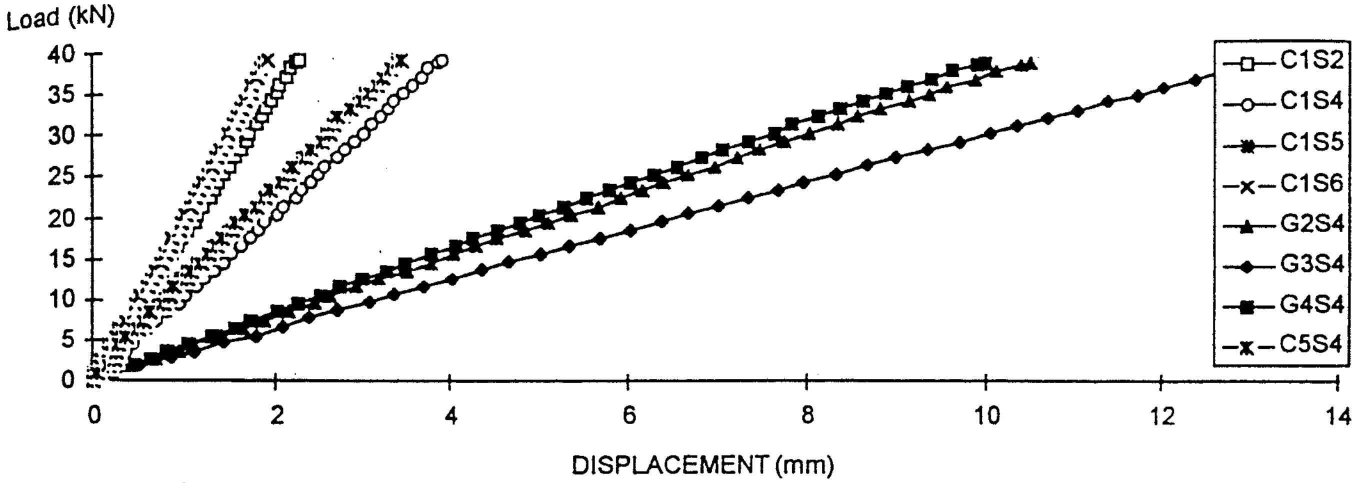

FIGURE 3(a). Vertical bridge displacements with non-composite timber deck, steel cables tensioned to 27 kN (approx. 6,000 lb.) - point load at G3S4.

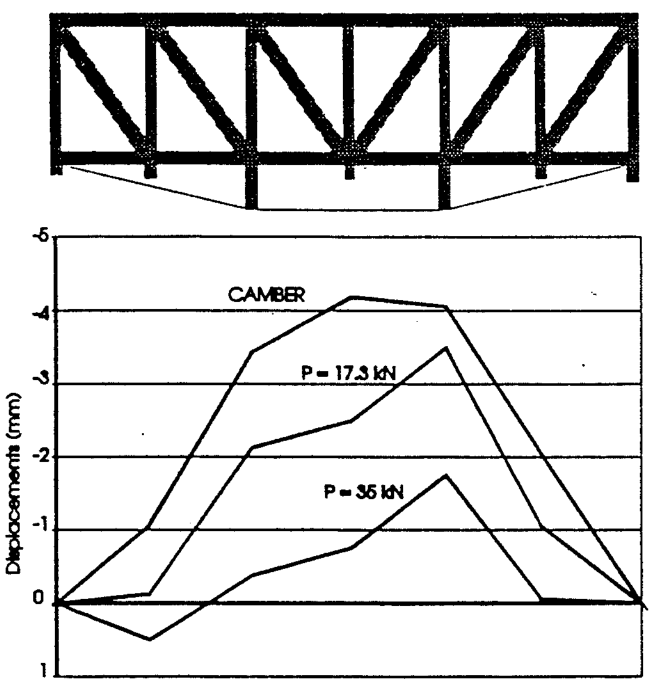

FIGURE 3(b). Vertical truss displacements with non-composite timber deck, steel cables tensioned to 27 kN (approx. 6,000 lb.) - point load (P) at G3S4.

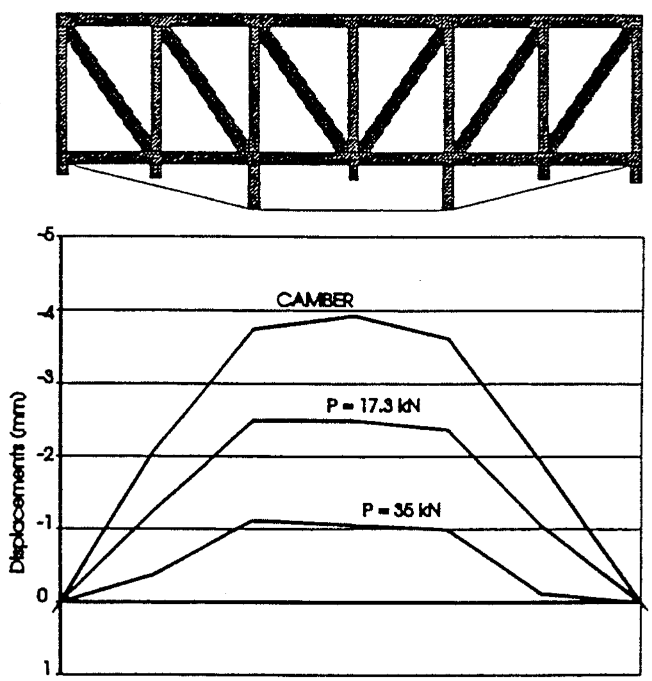

FIGURE 4. Vertical truss displacements with composite timber deck, steel cables tensioned to 27 kN (approx. 6,000 lb.) - point load (P) at G3S4.