Appendix E—FRP Trail Bridges in the United States

(Courtesy of the American Composites Association in 2000.)

FRP Trail Bridges in the United States					
Bridge name	**Location**	**Year built**	**Length (feet)**	**Width (feet)**	**System provider or FRP manufacturer**
Will Rogers State Park	Temescal Canyon Pacific Palisades, CA	1994	20	4	E.T. Techtonics, Inc.
San Luis Obispo (1)	San Luis Obispo, CA	1994	25	4	E.T. Techtonics, Inc.
San Luis Obispo (2)	San Luis Obispo, CA	1994	30	4	E.T. Techtonics, Inc.
San Luis Obispo (3)	San Luis Obispo, CA	1994	30	4	E.T. Techtonics, Inc.
San Luis Obispo (4)	San Luis Obispo, CA	1994	35	4	E.T. Techtonics, Inc.
San Luis Obispo (5)	San Luis Obispo, CA	1994	35	4	E.T. Techtonics, Inc.
San Luis Obispo (6)	San Luis Obispo, CA	1994	40	4	E.T. Techtonics, Inc.
Sierra Madre	Sierra Madre, CA	1994	40	4	E.T. Techtonics, Inc.
Malibu Creek State Park (1)	Malibu, CA	1994	40	5	E.T. Techtonics, Inc.
Malibu Creek State Park (2)	Malibu, CA	1994	20	5	E.T. Techtonics, Inc.
Tahoe National Forest Bridge	Grass Valley, CA	1994	20	5	E.T. Techtonics, Inc.
Deukmejain Wilderness Park (1)	Glendale, CA	1994	15	4	E.T. Techtonics, Inc.
Deukmejain Wilderness Park (2)	Glendale, CA	1994	20	4	E.T. Techtonics, Inc.
Deukmejain Wilderness Park (3)	Glendale, CA	1994	25	4	E.T. Techtonics, Inc.
Deukmejain Wilderness Park (4)	Glendale, CA	1994	25	4	E.T. Techtonics, Inc.
Will Rogers State Park	Malibu, CA	1994	40	5	E.T. Techtonics, Inc.
Point Bonita Lighthouse (1)	San Francisco, CA	1995	35	4	E.T. Techtonics, Inc.
Point Bonita Lighthouse (2)	San Francisco, CA	1995	70	4	E.T. Techtonics, Inc.
Pardee Dam Bridge	Valley Springs, CA	1995	25	5	E.T. Techtonics, Inc.
San Dieguito River Park	San Diego, CA	1996	70	8	E.T. Techtonics, Inc.
City of Glendora Bridge (1)	Glendora, CA	1996	18	6	E.T. Techtonics, Inc.
City of Glendora Bridge (2)	Glendora, CA	1996	22	6	E.T. Techtonics, Inc.
Grant Cty Park Bridge (1)	San Jose, CA	1997	20	5	E.T. Techtonics, Inc.
Grant Cty Park Bridge (2)	San Jose, CA	1997	35	5	E.T. Techtonics, Inc.
Grant Cty Park Bridge (3)	San Jose, CA	1997	40	5	E.T. Techtonics, Inc.
Grant Cty Park Bridge (4)	San Jose, CA	1997	40	5	E.T. Techtonics, Inc.
Grant Cty Park Bridge (5)	San Jose, CA	1997	50	5	E.T. Techtonics, Inc.
Santa Monica National Park	Calabasas, CA	1998	40	5	E.T. Techtonics, Inc.
Redwoods Natl Park (1)	Orick, CA	1999	80	5	E.T. Techtonics, Inc.
Redwoods Natl Park (2)	Orick, CA	1999	80	5	E.T. Techtonics, Inc.
Muir Beach Bridge (1)	Muir Beach, CA	1999	50	4	E.T. Techtonics, Inc.
Muir Beach Bridge (2)	Muir Beach, CA	1999	70	5	E.T. Techtonics, Inc.
Audubon Canyon Ranch Nature Preserve	Marshall, CA	1999	96	6	E.T. Techtonics, Inc.
City of Glendora Bridge	Glendora, CA	1999	28	6	E.T. Techtonics, Inc.
Santa Monica Bridge	Topanga, CA	2000	60	6	E.T. Techtonics, Inc.
Prairie Creek Redwoods State Park Bridge	Orick, CA	2000	46	5	E.T. Techtonics, Inc.
Santa Monica Bridge (1)	Calabasas, CA	2000	30	6	E.T. Techtonics, Inc.
Santa Monica Bridge (2)	Calabasas, CA	2000	75	6	E.T. Techtonics, Inc.
Rodeo Beach Pier	Sausalito, CA	2000	180	5	E.T. Techtonics, Inc.

Continued

FRP Trail Bridges in the United States

Bridge name	Location	Year built	Length (feet)	Width (feet)	System provider or FRP manufacturer
Alameda County Bridge	Castro Valley, CA	2000	18	4	E.T. Techtonics, Inc.
Humboldt State Park Bridge	Weott, CA	2000	40	4	E.T. Techtonics, Inc.
Golden Gate National Recreation Area (1)	Sausalito, CA	2001	25	5	E.T. Techtonics, Inc.
Golden Gate National Recreation Area (2)	Sausalito, CA	2001	25	5	E.T. Techtonics, Inc.
Topanga Canyon Bridge	Topanga, CA	2002	18	6	E.T. Techtonics, Inc.
Petaluma Bridge	Petaluma, CA	2002	40	6	E.T. Techtonics, Inc.
Boulder County Bridge	Boulder, CO	1994	35	6	E.T. Techtonics, Inc.
Heil Ranch Bridge	Boulder, CO	2000	45	6	E.T. Techtonics, Inc.
O'Fallon Park Bridge (1)	Denver, CO	2002	100	22	Strongwell
O'Fallon Park Bridge (2)	Denver, CO	2002	42	19	Strongwell
Sachem Yacht Club	Guilford, CN	2001	54	6	E.T. Techtonics, Inc.
Greensbranch - Pedestrian	Smyrna, DE	1999	32	6	Hardcore Composites
Catholic University Access Bridge	Washington, DC	1995	35	4	E.T. Techtonics, Inc.
Haleakala National Park (1)	Maui, HI	1995	40	4	E.T. Techtonics, Inc.
Haleakala National Park (2)	Maui, HI	1995	80	4	E.T. Techtonics, Inc.
Sealife Park Dolphin Bridge	Oahu, HI	2001	36	3	Strongwell
LaSalle Street Pedestrian Walkway	Chicago, IL	1995	220	12	Strongwell
Antioch Composite Pedestrian Bridge	Antioch, IL	1995	45	10	E.T. Techtonics, Inc.
Clear Creek Bridge (Daniel Boone National Forest)	Bath County, KY	1996	60	6	Strongwell
Levisa Fork of the Big Sandy River Footbridge	Johnson County, KY	1999	420	4	Strongwell
Bar Harbor Yacht Club Pier	Bar Harbor, ME	1995	124	5	Strongwell
Montgomery Cty Dept. of Park & Planning (1)	Silver Spring, MD	2000	23	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (2)	Silver Spring, MD	2000	26	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (3)	Silver Spring, MD	2000	30	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (4)	Silver Spring, MD	2000	32	6	E.T. Techtonic
Montgomery Cty Dept. of Park & Planning (5)	Silver Spring, MD	2000	32	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (6)	Silver Spring, MD	2000	40	6	E.T. Techtonics, Inc.
Becca Lily Park Bridge	Takoma Park, MD	2000	30	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (1)	Clarksburg , MD	2002	20	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (2)	Clarksburg, MD	2002	40	6	E.T. Techtonics, Inc.

Continued

FRP Trail Bridges in the United States					
Bridge name	**Location**	**Year built**	**Length (feet)**	**Width (feet)**	**System provider or FRP manufacturer**
Montgomery Cty Dept. of Park & Planning (3)	Clarksburg, MD	2002	50	6	E.T. Techtonics, Inc.
Montgomery Cty Dept. of Park & Planning (4)	Clarksburg, MD	2002	60	6	E.T. Techtonics, Inc.
Tanner Creek/Weco Beach Bridge	Bridgman, MI	1999	33	6	E.T. Techtonics, Inc.
Aurora Pedestrian Bridge	Aurora, NE	2001	100	10	Kansas Structural
Composites, Inc.					
Homestead Bridge	Los Alamos, NM	1997	54	4	E.T. Techtonics, Inc.
City of Los Alamos (1)	Los Alamos, NM	1999	50	4	E.T. Techtonics, Inc.
City of Los Alamos (2)	Los Alamos, NM	1999	25	6	E.T. Techtonics, Inc.
City of Los Alamos (3)	Los Alamos, NM	1999	12	6	E.T. Techtonics, Inc.
Los Alamos National Laboratory Bridge (1)	Los Alamos, NM	2001	40	3	E.T. Techtonics, Inc.
Los Alamos National Laboratory Bridge (2)	Los Alamos, NM	2001	60	3	E.T. Techtonics, Inc.
City of Los Alamos (1)	Los Alamos, NM	2001	16	4	E.T. Techtonics, Inc.
City of Los Alamos (2)	Los Alamos, NM	2001	35	4	E.T. Techtonics, Inc.
City of Los Alamos (3)	Los Alamos, NM	2001	12	4	E.T. Techtonics, Inc.
Tiffany Street Pier	Bronx, NY	1998	410	49	Seaward International
Lemon Creek Park Bridge	New York, NY	1998	85	5	Seaward International
Barclay Avenue Bridge	Staten Island, NY	2001	32	6	E.T. Techtonics, Inc.
Scenic Hudson Bridge	Tuxedo, NY	2002	35	4	E.T. Techtonics, Inc.
Popolopen Creek Bridge	New York, NY	2003	N/A	N/A	Strongwell
Powell Park Bridge	Raleigh, NC	1997	15	4	E.T. Techtonics, Inc.
Blue Ridge Parkway Bridge	Spruce Pine, NC	2001	30	4	E.T. Techtonics, Inc.
Mt. Hood National Forest Bridge (1)	Sandy, OR	1997	30	3	E.T. Techtonics, Inc.
Mt. Hood National Forest Bridge (2)	Sandy, OR	1997	30	3	E.T. Techtonics, Inc.
Peavine Creek Bridge	Wallowa-Whitman National Forest, OR	1998	22	6	E.T. Techtonics, Inc.
Devil's Pool / Fairmount Park (1)	Philadelphia, PA	1991	20	4	E.T. Techtonics, Inc.
Devil's Pool / Fairmount Park (2)	Philadelphia, PA	1991	32	4	E.T. Techtonics, Inc.
Devil's Pool / Fairmount Park	Philadelphia, PA	1992	50	5	E.T. Techtonics, Inc.
Philadelphia Zoo	Philadelphia, PA	1994	100	10	Creative Pultrusion, Inc.
Dingman Falls Bridge (1)	Bushkill, PA	1996	70	6	E.T. Techtonics, Inc.
Dingman Falls Bridge (2)	Bushkill, PA	1996	80	6	E.T. Techtonics, Inc.
McDade Trail Bridge (1)	Bushkill, PA	2002	25	6	E.T. Techtonics, Inc.
McDade Trail Bridge (2)	Bushkill, PA	2002	40	6	E.T. Techtonics, Inc.
McDade Trail Bridge (3)	Bushkill, Pennsylvania	2002	40	6	E.T. Techtonics, Inc.
Clemson Experimental Trail Bridge	Clemson, SC	2001	30	6	E.T. Techtonics, Inc.
Francis Marion National Forest	McClellanville, SC	2002	60	6	E.T. Techtonics, Inc.
Las Rusias Military Highway	Texas	1997	45	4	Hughes Bros., Inc.
Lake Jackson Bridge	Lake Jackson, TX	2003	90	6	N/A

Continued

FRP Trail Bridges in the United States

Bridge name	Location	Year built	Length (feet)	Width (feet)	System provider or FRP manufacturer
Unknown	Charlottesville, VA	1978	16	7	N/A
Girl Scout Council of Colonial Coast Bridge	Chesapeake, VA	1999	50	8	E.T. Techtonics, Inc.
Blue Ridge Parkway Bridge (1)	Floyd, VA	1999	24	4	E.T. Techtonics, Inc.
Blue Ridge Parkway Bridge (2)	Floyd, VA	1999	34	4	E.T. Techtonics, Inc.
Blue Ridge Parkway Bridge (1)	Floyd, VA	2001	28	4	E.T. Techtonics, Inc.
Blue Ridge Parkway Bridge (2)	Floyd, VA	2001	34	4	E.T. Techtonics, Inc.
George Washington & Jefferson National Forest	Edinburg, VA	2001	35	6	E.T. Techtonics, Inc.
Staircase Rapids (1) (Hoodsport)	Olympic National Park, WA	1994	40	4	E.T. Techtonics, Inc.
Staircase Rapids (2) (Hoodsport)	Olympic National Park, WA	1994	50	4	E.T. Techtonics, Inc.
Staircase Rapids (3) (Hoodsport)	Olympic National Park, WA	1994	80	4	E.T. Techtonics, Inc.
Bovee Meadows Trail Bridge	Lake Crescent, WA	1995	75	6	E.T. Techtonics, Inc.
Falls Creek Trail Bridge	Gifford Pinchot National Forest, WA	1997	45	3	Creative Pultrusion, Inc.
Ohio River Bridge	Wheeling, WV	1999	1000	4	Hardcore Composites
Medicine Bow National Forest	Medicine Bow, WY	1995	20	5	E.T. Techtonics, Inc.

Appendix F—Web Sites

FRP Bridge Inspections

AEA Technology
Engineering Solutions—CPD4D Project Number AH9/124 Non-Destructive Evaluation of Composite Components (CPD4D) Web site: *http://www.aeat.co.uk/ndt/cpd4d/cpd4dsum.html*

Identification of Fiber Breakage in Fiber Reinforced Plastic by Low-Amplitude Filtering of Acoustic Emission Data. Web site: *www.kluweronline.com/article.asp?PIPS=491177&PDF=1*

Long-Term In-Service Evaluation of Two Bridges Designed with Fiber-Reinforced Polymer Girders. Bernard Leonard Kassner. Web site: *http://scholar.lib.vt.edu/theses/available/etd-09062004-152133/unrestricted/Kassner_Thesis.pdf*

Thermal Infrared Inspection of FRP Bridge Decks for Health Monitoring. Marybeth Miceli, Lucius Pitkin, Inc. (USA); John C. Duke and Michael Horne, Virginia Polytechnic Institute and State University (USA). Web site: *http://spiedl.aip.org/getabs/servlet/GetabsServlet?prog=normal&id=PSISDG005073000001000328000001&idtype=cvips&gifs=yes*

Transportation Research Board—NCHRP Project 10/64 Panel on Field Inspection of In-Service FRP Bridge Decks. Web site: *http://trb.org/directory/comm_detail.asp?id= 2879*

University of Delaware—Nondestructive Inspection of FRP Composite Bridge Using Vibration Techniques Web site: *http://www.ccm.udel.edu/Pubs/posters02/P_posters/P167.pdf*

General Information

Composites in Construction Pultruded Profiles. Reference and Bibliography Database. Compiler: Dr J.T. Mottram. Web site: *http://www.eng.warwick.ac.uk/staff/jtm/pfrp_latest.pdf*

Composites World. Web site: *http://www.compositesworld.com/*

Polymer Composites III 2004. Transportation Infrastructure, Defense and Novel Applications of Composites. Proceedings, March 30–April 1, 2004. West Virginia University, Morgantown, WV. Editors: Robert C. Creese and Hota GangaRao. Web site: *http://www.destechpub.com/pageview.asp?pageid=15104*

United States Department of Transportation Federal Highway Administration—FRP Library. Web site: *http://www.fhwa.dot.gov/bridge/frp/frppaper.htm*

Pedestrian Bridges

Antioch Composite Pedestrian Bridge, Antioch, IL (1996). Web site: *http://www.iti.northwestern.edu/research/completed/composites/antioch.html*

Homestead Bridge, Los Alamos, NM (1997). Web site: *http://composite.about.com/library/weekly/aa102797.htm*

LaSalle St. Composite Pedestrian Walkway (1994). Web site: *http://www.iti.northwestern.edu/research/completed/composites/lasalle.html*

Preliminary Design and Analysis of a Pedestrian FRP Bridge Deck. Lulea University of Technology, licentiate thesis by Patrice Godonou. Web site: *http://epubl.luth.se/1402-1757/2002/18/index-en.html*

Appendix G—FRP Suppliers, Designers, and Associations

American Composites Manufacturers Association
1010 North Glebe Rd.
Arlington, VA 22201
Phone: 703-525-0511
Fax: 703-525-0743
E-mail: *info@acmanet.org*
Web site: *http://www.mdacomposites.org/*

Bedford Reinforced Plastics, Inc.
R.D. 2, Box 225
Bedford, PA 15522
Phone: 814-623-8125, 800-FRP-3280
Fax: 814-623-6032
Web site: *http://www.bedfordplastics.com*

Creative Pultrusions, Inc.
214 Industrial Lane
Alum Bank, PA 15521
Phone: 814-839-4186
Fax: 814-839-4276
Web site: *http://www.pultrude.com/*

E.T.Techtonics, Inc.
P.O. Box 40060
Philadelphia, PA 19106
Phone: 215-592-7620
Fax: 215-592-7620
E-mail: *info@ettechtonics.com*
Web site: *http://www.ettechtonics.com/*

Fibergrate Composite Structures, Inc.
5151 Beltline Rd., Suite 700
Dallas, TX 75254
Phone: 972-250-1633
Fax: 972-250-1530
Web site: *http://www.fibergrate.com*

Hardcore Composites
618 Lambsons Lane
New Castle, DE 19720
Phone: 302-442-5900
Fax: 302-442-5901
E-mail: *sales@hardcorecomposites.com*
Web site: *http://www.compositesworld.com*

Infrastructure Composites International, Inc.
7550 Trade St.
San Diego, CA 92121
Phone: 858-537-0715
Fax: 858-537-3465, 858-537-3465
Web site: *http://www.infracomp.com*

Liberty Pultrusions East & West
1575 Lebanon School Rd.
Pittsburgh, PA 15122
Phone: 412-466-8611
Fax: 412-466-8640
Web site: *http://www.libertypultrusions.com*

Kansas Structural Composites, Inc.
553 S. Front St.
Russell, KS 67665
Phone: 785-483-2589
Fax: 785-483-5321
E-mail: *ksci@ksci.com*
Web site: *http://www.ksci.com*

Peabody Engineering
13465 Estelle St.
Corona, CA 92879
Phone: 800-473-2263
Fax: 310-324-7247
Web site: *http://www.etanks.com*

San Diego Plastics, Inc.
2220 McKinley Ave.
National City, CA 91950
Phone: 800-925-4855, 619-477-4855
Fax: 619-477-4874
Web site: *http://www.sdplastics.com/*

Seasafe, Inc.
209 Glaser
Lafayette, LA 70508
Phone: 800-326-8842
Fax: 337-406-8880
Web site: *http://www.seasafe.com*

Seaward International, Inc.
3470 Martinsburg Pike
Clearbrook, VA 22624
Phone: 540-667-5191
Fax: 540-667-7987
Web site: *http://www.seaward.com/*

Structural Fiberglass, Inc.
4766 Business Route 220 North
Bedford, PA 15522
Phone: 814-623-0458
Fax: 814-623-0978
Web site: *http://www.structuralfiberglass.com*

Strongwell
400 Commonwealth Ave.; P.O. Box 580
Bristol, VA 24203-0580
Phone: 276-645-8000
Fax: 276-645-8132
E-mail: *webmaster@strongwell.com*
Web site: *http://www.strongwell.com/*

Appendix H—Design of the Falls Creek Trail Bridge

DESIGN OF THE FALLS CREEK TRAIL BRIDGE
A Fiber Reinforced Polymer Composite Bridge

Scott Wallace, P.E.
Eastern Federal Lands Highway Division
Federal Highway Administration

INTRODUCTION

The design of the Falls Creek Trail Bridge, a 13.9-m- (45-ft 6-in-) long single-span, fiber-reinforced composite (FRP) bridge, was borne out of an old need and new technology. Lightweight, low maintenance structures that can be hauled into remote locations have been needed for a long time. However, applying fiber reinforced polymer (FRP) composites to such needs is a recent development driven by efforts of FRP composite manufacturers to enter the bridge industry. The Bridge Design office in the Eastern Federal Lands Highway Division (EFLHD) of the Federal Highway Administration (FHWA) became interested in developing a design approach for FRP bridges after seeing a presentation given by E.T. Techtonics, Inc., which highlighted the potential of the material. One of EFLHD's primary clients, the USDA Forest Service, had a large need for lightweight, low maintenance bridges for their trail system, and FRP bridges appeared to be an ideal solution.

In May, 1997, EFLHD met with representatives of the Forest Service, E.T. Techtonics, Inc., and GHL, Inc. The objective of the meeting was to bring together one of EFLHD's client agencies (Forest Service) with experts in the FRP composite industry to explore the possibility of making a lightweight, low maintenance bridge. E.T. Techtonics, Inc., one of the leading experts in the country on the use of FRP composites in pedestrian bridges and GHL, Inc., were working to increase the use of FRP composites in government projects.

EFLHD wanted to acquire the ability to design, specify, and produce plans for FRP composite pedestrian bridges. The Forest Service wanted a bridge that could be "packed" into remote locations and easily constructed onsite. The FRP industry wanted to expand the application of their products to include the bridge industry. All three parties also wanted to test the finished bridge extensively and disseminate the results to other agencies.

GENERAL FEATURES

A Pratt truss was chosen for this bridge (see figures 1 and 2), based on many of its intrinsic characteristics that fit well with characteristics of FRP composite structural shapes. These same characteristics are ideal for pedestrian bridges.

A truss is really a deep beam with unnecessary portions of the web removed. It optimizes the placement of the structural sections in order to get the most advantage out of them. The result is a large top and bottom chord with a minimal web in between them. It also places the individual sections such that they carry uniaxial loads along their length.

FRP composite sections are well suited for this type of use. Because of their fiber orientation, they are much stronger along their longitudinal axis than transverse to it. They are also readily available in structural shapes, such as tubes and channels, that have been traditionally used in trusses, making assembly easier.

The combination of a structural type that minimizes the amount of material needed and an extremely lightweight material provides an excellent structure for pedestrian bridge applications. Using the Pratt truss approach also provides a ready-made pedestrian rail on each side of the bridge with the top chord of each truss serving as the handrail.

The Forest Service needed a bridge that was not only lightweight and required little maintenance, but one that could carry considerable loads as well. In recent years they had experienced some very extreme snowfalls in the Pacific Northwest. Some of their pedestrian bridges which were designed for a 7.182 kPa (150 psf) snow load failed due to the weight of the snow. Because of this and the unfamiliarity with the FRP composite material, they requested that a design snow load of 11.97 kPa (250 psf) be used. This is equivalent to a wall of wet snow piled over 6 m (20 ft) high. The loading actually models a bank of snow that “mushrooms” out over the handrails, thus significantly increasing the load per unit surface area of the deck. The bridge superstructure was also designed to resist a design wind load based on 45 m/s (100 mph) winds.

Along with lightweight, low maintenance characteristics, and the ability to carry these extreme loads, the Forest Service wanted a bridge made of readily available components with a repeatable design so that it could be duplicated. FRP composites seemed to have the potential to meet all of their criteria.

MATERIALS

FRP composites are composed of a resin matrix binder that has been reinforced with fibers. The fibers provide tensile strength along their length and may be oriented in more than one direction. The resin binder holds the fibers together and in the proper orientation while transferring loads between fibers. It also provides all of the interlaminar shear strength for the member. Together, they combine in a working relationship much like that between reinforcing steel and concrete.

The structural sections making up the trusses on the Falls Creek Trail Bridge are manufactured by Strongwell and came from their EXTREN line (1). They contain glass fibers embedded in an isophthalic polyester resin. The fibers consist of continuous strand roving composed of thousands of fiber filaments running along the length of the member and continuous strand mat composed of long intertwined glass fibers running in different directions. The roving provides the strength along the longitudinal axis of each member and the mat provides the multidirectional strength properties. Each member also includes a surfacing veil composed of polyester nonwoven fabric and resin on the outside of the section to provide ultraviolet and corrosion protection.

The decking is also a Strongwell product and includes a 6-mm (1⁄4-in) EXTREN sheet with a gritted surface on top of DURAGRID I-7000 25-mm (1-in) grating. The grating is similar in composition to the structural shapes except that it contained a vinyl ester resin binder.

All of the FRP composite sections were manufactured using a pultrusion process. The process involves pulling continuous lengths of glass mat and roving through a resin bath and then into a heated die. The heat initiates the gelation (or hardening) of the resin and the cured profile is formed matching the shape of the die.

Only two other materials were used in the superstructure of this bridge. The sections were connected with galvanized bolts conforming to ASTM A307. And the superstructure was attached to the foundations by steel anchor bolt clip angles conforming to ASTM A36.

DESIGN

The design of the Falls Creek Trail bridge was performed in accordance with the American Association of State Highway and Transportation Officials’ (AASHTO) *Standard Specifications for Highway Bridges* (2) and *Guide Specifications for Design of Pedestrian Bridges* (3). Both specifications were needed in that while the standard specification provided good general bridge design guidance, the guide specification provided specific guidance relating to the unique characteristics of pedestrian bridges, which tend to be smaller, lighter, more flexible structures than standard highway bridges.

Neither specification, however, deals with FRP composites. Therefore, additional guidance and design techniques were developed from sources in the FRP composite industry. The *Design Manual for EXTREN Fiberglass Structural Shapes* (1) developed by Strongwell was a good source of information relating to the individual structural shapes of which the bridge was comprised. In addition, E.T. Techtonics, Inc., provided assistance in interpreting and modifying existing information; provided test data pertaining to connection capacity and other details; and reviewed the final design and details.

Because of the FRP composite sections being patterned after shapes common to the steel industry, some guidance and design techniques were developed based on the *Manual of Steel Construction* from the American Institute of Steel Construction (AISC) (4) as well.

It was necessary to design each structural member of the bridge with respect to allowable tension, allowable compression, allowable bending stresses, combined stresses due to axial forces and moments acting together, and shear. The design forces and moments used were the maximum values generated by an analysis of the structure with fixed joints, one pinned support, and one roller support.

Whenever a member was exposed to a bending moment in conjunction with an axial compression force in excess of 15 percent of the allowable axial compression, it was assumed that a secondary moment was generated. To account for this, a secondary moment amplification factor was employed. It was unnecessary to apply the same design approach to tensile members (4). This will be discussed further in the *Combined Axial Load and Bending* portion of this section of the report.

The bridge is loaded primarily with dead load (self-weight and snow) and wind load. By observation, it was determined that the most conservative AASHTO load group designation was load group II (2). Members designed with this design load group are permitted a 25-percent increase in allowable unit stresses. Similarly, AISC allows a 33-percent increase in allowable stresses based on Euler's equation if the wind load causes a stress increase of over 33 percent in all members (4), which occurred on this bridge. Therefore, since the critical design loads were caused by wind load and dead load, a 25-percent increase in allowable stresses and allowable Euler stresses was incorporated into the design. However, due to unfamiliarity with the equations from the Strongwell design manual, no allowable stress increase was applied to them.

Tension Members

Designing an FRP composite section to carry tensile loads is a very straightforward process. The allowable tensile stress for the sections used in the Falls Creek Trail Bridge is simply the ultimate tensile stress divided by a factor of safety regardless of the structural shape being designed.

In this bridge, the bottom chord, interior vertical posts, diagonal tension members, and horizontal bracing all experienced some tension. However, none of them were stressed to more than 40 percent of their allowable tensile stress.

Compression Members

As should be expected, designing an FRP composite section to resist compressive loads is more complex. The allowable compressive stress is a function of local, member, and Euler buckling characteristics, as well as structural shape and end conditions.

The structural channels and tubes that made up this bridge were all comprised of plate elements such as flanges and webs. These elements may develop wave formations when they are compressed; this is called *local buckling.* The stress at which local buckling occurs is a function of many factors. In typical structural members the primary factors are element slenderness (width/thickness ratio), aspect ratio (length/width ratio) and edge support conditions.

A constant (k) is used to adjust the calculated critical stress at which local buckling occurs to account for differing edge conditions. When both unloaded edges are fixed, as in the case of webs, $k = 7$. When one unloaded edge is fixed and one is free, as in the case of channel flanges, $k = 1.33$. The Strongwell column equations take this into account. For W and I shapes the equations are based on local buckling of the flange because their sections are proportioned such that the flanges will buckle before the webs. Therefore, in order to extend the use of these formulas to channels, shapes for which they do not provide column equations, it was necessary to examine local buckling in both the web and the flanges. The element that had a lower critical stress at which local buckling occurred, and therefore a higher width/thickness ratio, controlled the design. However, the web width/thickness ratio had to first be modified to allow for its edge conditions being different than those on which the formulas were based. Simply put, the width/thickness ratio for the web was replaced with an adjusted web width/thickness ratio equivalent to 1.33/7 times its actual ratio. The larger of the flange or adjusted web width/thickness ratios for each compressive member was then used in the appropriate Strongwell equation (Equations 1 or 2) to determine the short column mode ultimate compressive stress based on local buckling.

For square and rectangular structural tubes, the equations were applied without adjustments. The empirically derived Strongwell equations follow; ultimate compressive stress column equations, short column mode:

W and I shapes:

$$F'_u = \frac{0.5E}{(\frac{b}{t})^{1.5}} \tag{1}$$

Square and rectangular tubes:

$$F'_u = \frac{E}{16(\frac{b}{t})^{.85}} \tag{2}$$

where

F'_u = ultimate compressive stress (kPa)
b = element width (mm)
E = modulus of elasticity (kPa)
t= element thickness (mm)

Even if a compression member does not fail due to local buckling of one of its elements, the entire member could fail due to member buckling. This type of failure is a function of modulus of elasticity, end conditions, and member slenderness ratio. In order to design for member buckling, two equations were applied to each member. The appropriate Strongwell equation (Equations 4 or 5) for long column mode failures in W and I shapes or in tubes was first applied. These formulas, along with the short column formulas (Equations 1 and 2), are based on Strongwell's extensive testing of fiberglass shapes and are pertinent only to their EXTREN products. The general column formula developed in 1744 by Swiss mathematician Leonard Euler (5) was also applied to both the channels and the tubes. The more conservative results were used for determining the ultimate compressive stress based on member failure. In every member of this bridge, the Euler equation proved to be more restrictive. However, in some cases when the 25-percent increase in allowable Euler stress was taken into consideration the Strongwell equations controlled. Following are the ultimate compressive stress column equations, long column mode:

W and I shape:

$$F'_u = \frac{4.9E}{(\frac{Kl}{r})^{1.7}} \tag{3}$$

Square and rectangular tubes:

$$F'_u = \frac{1.3E}{(\frac{Kl}{r})^{1.3}} \tag{4}$$

Euler equation:

$$F'_u = \frac{\pi^2 E}{(\frac{Kl}{r})^2} 1.25 \tag{5}$$

where

E = modulus of elasticity (kPa)
l = column length (m)
K = effective length factor
r = radius of gyration (m)

As the primary compressive load carrying member on this bridge, the top chord presented some interesting problems. It was sufficiently restrained in the vertical direction by the posts to reduce it to a column braced at intervals equal to the distance between posts, 1.5 m (5 ft) when designing against buckling in the vertical plane. The posts also provided restraint against buckling in the horizontal plane. However, the degree of restraint provided was dependent upon the stiffness of the transverse U-shaped frame composed of two posts and their interconnecting crosspiece. For this condition, the top chord was modeled as a column braced at intervals equal to the post spacing by elastic springs whose spring constants correspond to the stiffness of the transverse U-shaped frames restraining it (4) as shown in figure 3.

The transverse frame spring constant (C) upon which the effective length factor is based can be calculated according to the following formula:

Transverse frame spring constant:

$$C = \frac{E_{chord}}{h^2 [(\frac{h}{3 I_p}) + (\frac{b}{2 I_c})]} \qquad (6)$$

E_{chord} = modulus of elasticity of top chord
I_p = moment of inertia of vertical posts
h = effective height of vertical posts
I_c = moment of inertia of crosspiece
b = span of crosspiece between trusses

AASHTO provides an appendix to their pedestrian bridge guide specification (3) that includes a table for relating the transverse frame spring constant to an effective length factor for trusses with different numbers of panels. Neglecting the outriggers, the Falls Creek Trail Bridge had a transverse frame spring constant: C = 0.423. Based on this and taking into account its nine panels, the resultant effective length factor was K = 2.8.

If the top chord of this bridge was supported such that K = 2.8 it would only be able to carry approximately 3.5 kips of compression. Therefore, it was necessary to employ outriggers at every interior post. The outriggers sufficiently stiffen the transverse frame such that the effective length factor becomes K = 1. By increasing the stiffness of the transverse frame through the use of outriggers, and thereby increasing the stiffness of the elastic spring supports, the top chord's compression carrying capability was increased approximately 800 percent.

Having established the support conditions for the top chord it was important to then determine how the top chord would carry the axial compression applied to it. Because it is composed of two channels the top chord will function as two separate compression members acting individually between points where the two channels are attached to each other. If the channels were attached to each other only at the post connections, each would function as a compression member across a length of 1.5 m (5 ft). However, by fastening them together at the midpoint between the posts, their slenderness ratios were reduced by 75 percent and their ability to carry compressive forces individually was increased 400 percent. If the Strongwell long-column mode equation had controlled the design instead of the Euler equation, their allowable load would have increased 325 percent instead of 400 percent. Due to this significant increase in load carrying capability, the channels were bolted together with spacer blocks made of 51- by 102-mm (2- by 4-in) FRP composite tubes placed between them at the midpoint between the vertical posts.

The top chord will also try to carry the compressive loads as a single member with both channels working together. In an effort to maximize the load carrying capability of the top chord, the channels were placed four inches apart from each other. This was accomplished by using 51- by 102-mm (2- by 4-in) structural tubes as vertical posts and attaching the channels to the outside of the posts. By doing this the section modulus was increased substantially resulting in a much more laterally rigid member. This stiffer member carried compressive loads across an unsupported length equal to the distance between the posts. The Strongwell long column mode formula (Equation 3) and the Euler equation (Equation 5) were again employed, but the entire member was taken into consideration rather than just the individual channel.

It should be noted that when designing the top chord, AASHTO requires that the design load used for the determination of the critical buckling force should not be less than two times the maximum design load that any panel would experience. This requirement is in recognition of the fact that under uniform loading the maximum compressive stresses may occur simultaneously over consecutive panels (3). The use of what is basically a minimum factor of safety (FS) of two, seems wise in that there are a number of secondary factors and uncertainties involved in the analysis of top chord compression members that at present have not been quantified into an easily performed design procedure. These include torsional stiffness of the chord, lateral support contributed by the diagonals, initial crookedness of the chord, eccentricity of the axial load and uneven displacement of the posts as a moving load crosses the bridge. A factor of safety of three was employed for the design of all members of the Falls Creek Trail Bridge, thereby requiring no adjustment to meet this criteria.

Structural tubing also served as compression members on this bridge. The vertical end posts in particular carried a considerable amount of compression. By examining the Strongwell and Euler equations it can readily be noted that under axial compression loads the 51- by 102-mm (2- by 4-in) tubes, whose walls measure 51 by 6 mm (2 by $^{1}/_{4}$ in) and 102 by 3 mm (4 by $^{1}/_{8}$ in) respectively, tend to buckle in the plane of the truss. Both the width/thickness ratio and the slenderness ratio are higher in this direction, thereby causing the stress levels at which local buckling, member buckling, and Euler buckling take place to be lower. Although using the larger, rectangular tubes in place of 51- by 51-mm (2- by 2-in) square tubes (which have been used on other bridges) did not improve the buckling characteristics of the end posts, it did provide other advantages. As mentioned previously, the larger posts further separated the two channels comprising the top chord and resulted in an approximately 250-percent increase in member buckling resistance capacity in the horizontal direction for the top chord. They also provided increased lateral support to the top chord at each post and increased the overall lateral stiffness of the bridge. In addition, enough room was provided for the diagonals to cross between posts without intersecting each other. That is, if the vertical posts were made from 51- by 51-mm (2- by 2-in) tubes the diagonals would intersect each other, creating connection and stiffness difficulties.

Two diagonals were incorporated into each panel of the bridge trusses. As is common in Pratt trusses, one of the diagonals slopes upward toward the center of the span and is in compression while the other slopes downward toward the center of the span and is in tension. The exception to this occurred in the center panel where both diagonals experienced a small amount of tension. The tension diagonals were made of 51- by 51-mm (2- by 2-in) FRP composite structural tubes. The ends were filled with 44- by 44-mm (1$^{3}/_{4}$- by 1$^{3}/_{4}$-in) FRP composite solids to improve the connections. The compression diagonals were also made of 51- by 51-mm (2- by 2-in) FRP composite structural tubes but were filled from end to end with the solids in order to improve their compression carrying capability. The same local (Equation 2), member (Equation 4), and Euler (Equation 5) buckling equations mentioned previously were applied to the compression diagonals. Because the diagonals are connected at their centers they are assumed to be supported there and their unsupported length is equal to 50 percent of their actual length. The compression diagonals in the outside panels experienced the greatest loads and were stressed to approximately 35 percent of their allowable limit.

Bending

For Pratt truss bridges similar in size to the Falls Creek Trail Bridge, bending stresses generally will not control the design of the members. The multiple members attaching to each connection tend to adequately distribute the moment such that no single member experiences a large moment. However, two situations merit mentioning:

- If the supports are fixed, the moment in the bottom chord increases considerably

- By applying a lateral force equivalent to 0.01/K times the average design compressive force in the two adjacent top chord members to the top of the vertical posts, as specified in the AASHTO guide specification (3), a large moment is generated in the posts.

Although the supports on the Falls Creek Trail Bridge were not designed as fixed, they did possess some degree of fixity. It was therefore important to examine the effects on the structure of fixing the supports. An analysis was performed under two loading conditions. One condition included full loading, while the other removed the snow load but included a 38-degrees Celsius (100-degrees Fahrenheit) temperature rise. The results revealed that the bottom chord was transformed from a tensile member with small bending moments to a compression member with much larger bending moments in the plane of the truss near the supports. In this region the bottom chord experienced approximately 89 kN (20 k) of compression while subject to a 8 kN-m (70 k-in) bending moment. Because the bottom chord is identical in section to the top chord but better supported laterally by the crosspieces, it was able to resist buckling at stress levels that were about 50 percent of the allowable compressive stress and 30 percent of the allowable bending stress.

The AASHTO guide specification takes a new approach to designing vertical posts. Instead of applying a minimum 4.378 kN/m (300 plf) force to the tops of the posts as required by the standard specification, it establishes a minimum lateral strength based on the degree of elastic lateral support provided by the post necessary for the top chord to resist its maximum design compressive force. It requires that a lateral force equivalent to 0.01/K times the average design compressive forces in the two adjacent top chord members be applied to the top of the verticals concurrently with all other design loads. Applying this design criteria effectively increased the design lateral bending stress in the interior vertical posts of this bridge by approximately 450 percent over that which the analysis produced. However, the bending stress level was approximately 65 percent of that which was allowed.

No member of the Falls Creek Trail Bridge was stressed beyond 65 percent of its allowable bending stress. However, each member also had to be proportioned to resist the combined effects of axial load and bending moment acting together. In order to consider these combined effects, the AISC combined stress equations were employed (4).

Combined Axial Load and Bending

Whenever a bending moment is applied to an axially loaded member, a secondary moment equal to the product of the eccentricity caused by the moment and the applied axial load is generated. Because any secondary moment caused by axial tension is opposite in sense to the primary, applied moment, the secondary moment will diminish rather than amplify the effects of the primary moment. Also, when the axial compression force is not in excess of 15 percent of the allowable axial compression, the effects of any secondary moment caused by the axial load are minor enough to be neglected. Therefore, when a member is exposed to either of these conditions, the secondary moment can be ignored. However, whenever a member is exposed to a bending moment in conjunction with an axial compression force in excess of 15 percent of the allowable axial compression, it should be assumed that a secondary moment is generated and its effects should be considered. To take the effects of the secondary moment into consideration, a secondary moment amplification factor is applied to the bending stress portion of the general combined stress equation.

For each member, the applicable following equations (7 to 10) must be satisfied. They are based on equations used by the steel industry (4) and are used as a check to assure that the combined effects of axial and bending stresses do not go beyond acceptable limits.

Axial tension and bending:

$$\frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1 \qquad (7)$$

Axial compression and bending ($f_a / F_a < 0.15$):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1 \qquad (8)$$

Axial compression and bending (Equation 1):

$$\frac{f_a}{F_a} + \left[\frac{C_{mx}}{(1 - \frac{f_a}{F'_{ex}})}\right]\frac{f_{bx}}{F_{bx}} + \left[\frac{C_{my}}{(1 - \frac{f_a}{F'_{ey}})}\right]\frac{f_{by}}{F_{by}} \leq 1 \qquad (9)$$

Axial compression and bending (Equation 2):

$$\frac{f_a}{0.6\,F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1 \qquad (10)$$

Secondary moment amplification factor:

$$\left[\frac{C_{mx}}{(1 - \frac{f_a}{F'_e})}\right] \qquad (11)$$

Euler stress (divided by FS):

$$F'_e = \frac{\pi^2 E}{3(\frac{Kl}{r})^2} 1.25 \qquad (12)$$

where

E = modulus of elasticity (kPa)
F'_e = Euler stress divided by a FS (kPa)
C_m = Secondary moment reduction factor
F_y = Specified minimum yield stress (kPa)
F_a = Allowable axial stress (kPa)
f_a = Computed axial stress (kPa)
F_b = Allowable bending stress (kPa)
f_b = Computed bending stress (kPa)

Shear

The FRP composite structural shapes are fabricated in such a manner that they have an inherent resistance to shear. Because the roving fibers are primarily oriented such that they run longitudinally through each member, they are strategically located to resist the shear. The crosspieces in the Falls Creek Trail Bridge were the only members that were subjected to substantial shear forces. As they transfer the loads from the deck to the trusses they develop their highest shear stresses at the point at which they connect to the vertical posts. Unfortunately, this is also the point at which holes were drilled in the webs of the crosspieces to attach them to the posts. The result of the applied loads and the reduced web section were stress levels of approximately 40 percent of the allowable shear stress for channels.

Bearing

All of the members of the Falls Creek Trail Bridge were bolted together. Even though the crosspieces rest on the top flange of the bottom chord, they are fastened to the vertical posts such that they do not transfer their loads to the trusses through bearing. Only two areas of the bridge transfer loads by means of bearing on another member. The FRP composite deck bears directly on the top flange of the crosspieces and the bottom chord bears directly on the grade beams at both ends of the bridge. In the case of the top flange of the crosspieces, the deck transfers its load through eighteen bearing bars which sufficiently spread the load along the crosspiece such that bearing is not an issue. It is only the last ten inches of the bottom flange of the channels making up the bottom chord that needed to be investigated.

Testing by E.T. Techtonics, Inc., has shown that a 3-in length of Strongwell's EXTREN C203 by 56 by 10 mm (C8 by $2^3/_{16}$ by $^3/_8$ in) can carry 35.586 kN (8 k) in bearing. Based on this data it was determined that the ultimate bearing capacity of the bottom chords was 222.411 kN (50 k) per chord, on each end of the bridge. The maximum reaction occurred on the leeward side of the bridge when fully loaded, and only amounted to approximately 62 kN (14 k). Therefore, a maximum bearing stress level of less than 30 percent was reached.

Connections

Approximately 2.25 kN (500 lb) of ASTM A307 galvanized bolts, nuts, and washers were used to connect all of the members together. The primary load carrying connections consisted of two 19-mm- ($^3/_4$-in-) diameter bolts spaced 100 mm (4 in) apart, with a 50-mm (2-in) edge distance at the end of the member. Tests have shown that the EXTREN structural tubes used in this bridge can carry ultimate tensile or compressive loads in excess of 62 kN (14 k) when connected in this manner. The configuration of the bolts also meets the general guidelines given in *Composites for Infrastructure, A Guide for Civil Engineers* (6). When filled, the compression diagonals have an ultimate compressive load capacity of over 220 kN (50 k). It is interesting to note that these same tests have shown that the ultimate capacity of these connections varies greatly depending on resin type and manufacturer. It is also interesting to note that the filled 102- by 102-mm (2- by 2-in) structural tubes used for compression diagonals gained very little tensile capacity by being filled. Evidently, the fiber orientation of the solids used to fill the tubes is such that it provides little additional tensile strength.

Other less critical connections used 13-mm- ($^1/_2$-in-) diameter bolts. All connections consisted of at least one bolt with a standard washer under its head and a nut with a standard washer and lock washer under it. It is important to include the standard washers in order to spread the forces coming from the bolt over a larger area of the member. The lock washer performs two important functions. It prevents the nut from working itself loose due to vibrations and shifting of the members, and also serves as a direct tension indicator Each nut was tightened until its corresponding lock washer compressed to a flat position.

Vibrations

The potential for significant responses due to the dynamic action of walking or running can be a problem on pedestrian bridges, especially those bridges that have low stiffness, little damping, and little mass. The Falls Creek Trail Bridge is just such a bridge. Studies have shown that the range of the first through third harmonic of people walking or running across a pedestrian bridge is 2 to 8 Hz, with the fundamental frequency occurring between 1.6 and 2.4 Hz. Therefore, AASHTO recommends that bridges such as this one be tuned to have a fundamental frequency larger than 5 Hz (3). They also provide guidance for estimating the fundamental frequency and checking that the bridge is properly proportioned to avoid excessive excitation:

$$f = 0.18\sqrt{g/\Delta_{DL}} \qquad (13)$$

where
f = estimated fundamental frequency (Hz)
g = acceleration due to gravity (m/s2)
Δ_{DL} = deflection due to dead load (m)

AASHTO recommends first estimating the fundamental frequency by considering the truss as a simply supported uniform beam. The calculation is based on the stiffness of the truss. For this bridge the estimated fundamental frequency produced by the AASHTO equation (Equation 13) was 11.8 Hz. The SAP90 analysis of the same structure produced a fundamental frequency of 11.6 Hz. Therefore, the estimate proved to be an excellent one for the given bridge. If the fundamental frequency cannot satisfy the minimum fundamental frequency criteria, or if the second harmonic is a concern, the guide specification provides a check of the proportioning of the superstructure to ensure that a minimum superstructure weight with respect to the fundamental frequency is present. Theoretically, the fundamental frequency can be increased by increasing the stiffness of the superstructure or decreasing its weight. The minimum allowable weight of the superstructure can be established using the following equation:

$$W = 800\, e^{-.35f} \qquad (14)$$

where
W = minimum allowable weight of superstructure (kN)
e = natural log base
f = estimated fundamental frequency (Hz)

This check, in effect, is a prohibition against overly reducing the weight of the superstructure. The Falls Creek Trail Bridge superstructure weighed in at approximately 18 kN (4 k), which was 25 percent heavier than the calculated minimum.

TESTING

In June 1998, the bridge was assembled at the USDA Forest Products Laboratory in Madison, WI. Later, it was instrumented with sixteen strain gauges and four devices for measuring deflections. In September 1998, it was subjected to a 12 kPa (250 psf) loading and left exposed to the Wisconsin weather. The monitoring began and is expected to continue for up to a year. Data will be continuously gathered by Forest Service personnel concerning deflection, strain, and temperature. A close study of the connections will also be performed. The points at which the vertical posts and diagonals all attach to the chords present an eccentrically loaded connection that will be closely examined.

The initial load testing data show that the actual deflections at the center of the span are approximately 30 mm (1.16 in). The amount of deflection recorded corresponds very closely with that which was anticipated. Design calculations predicted an initial deflection of 32 mm ($1\frac{1}{4}$ in).

During the same period of time another bridge is being tested next to this one. It is a 6.50-m- (21-ft 6-in-) long, 1.83-m (6-ft-) wide FRP composite truss bridge designed to carry pack stock and a snow load of 6 kPa (125 psf). Because it will be used by pack animals it will be closely monitored for deflection and lateral stability characteristics.

CONSTRUCTION

The bridge is scheduled to be constructed over a 2-day period in June 1999. It will be packed into the backcountry near Mt. Hood and installed on the Falls Creek Trail in the Gifford Pinchot National Forest. It will be constructed by Forest Service personnel with the assistance of FHWA. No heavy equipment or power tools will be required.

CONCLUSIONS

Many benefits of using FRP composites to construct a trail bridge were uncovered through the work on the Falls Creek Trail Bridge. The bridge is lightweight with its heaviest component weighing approximately 0.67 kN (150 lb). The assembled bridge weighs approximately 1.4 kPa (30 psf), based on area of deck, for a total of approximately 18 kN (2 tons). Yet, it still has a very high load carrying capacity. It can easily be constructed in just a few days using general maintenance personnel and without the aid of heavy equipment. It is also composed completely of off-the-shelf fiberglass structural shapes that are readily available from fabricators. When constructed it is virtually maintenance-free and looks identical to a small steel truss bridge. Also, the design is flexible and can easily be adjusted for bridges of different lengths up to spans of 18.29 m (60 ft). Depending on the loading conditions, the length can be adjusted in 1.524-m (5-ft) increments by adding or removing panels. Ultimately, however, the testing and inservice performance will largely determine the long-range viability of the Falls Creek Trail Bridge and others like it.

Currently, research and development efforts in the bridge building industry seem to be focusing on material testing. Because of the unfamiliarity of FRP composites in this industry, a great deal of work needs to be done to develop means to adequately test these materials. This information can then be used to develop much needed material specifications and will likely lead to new and improved design methods and procedures. At the same time, other barriers must be overcome including the high initial cost of the material, the lack of design codes and inspection methods for FRP composites, and the lack of proven inservice durability data.

In some ways, overcoming these barriers is made even more difficult by the manufacturers. Because FRP composites are engineered materials, meaning that the composition of the material is adjusted to produce particular performance characteristics, each manufacturer sells an entirely different product. These products are proprietary and are protected by their owners, who are currently unwilling to make their specific fiber architecture (precise material proportions and fiber orientation) available. This makes producing standard tests, general design procedures, and specifications extremely difficult. The industry may have to loosen their hold on this type of information if they desire a market in the bridge industry.

The results of the initial load testing suggest that the analysis methods used to model the load carrying capacity of this bridge were very accurate. When the actual performance of the bridge to date is considered as well, the design procedures described in this report appear to provide a good basis for a thorough, reliable design of an FRP composite truss bridge. However, the procedures represent the latest scholarship in a growing and changing field and will need to be adapted as materials and our understanding of their behavior advance. Also, some of the procedures shown here apply only to bridges made out of components from Strongwell's EXTREN line. They would need to be modified in order to be used to design with other products.

FRP composite bridges are not currently a practical solution for most bridge needs. Further study and testing are needed to gain a better understanding of the material and its uses. However, they do appear to have the potential to uniquely meet an important need for lightweight, strong, low maintenance, attractive trail bridges in remote locations.

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